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R. Oliveira, L.F. Rodrigues, A.G. Coelho
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LNEC, Lisboa, Portugal

VOLUME 2
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Factors affecting stick-slip characteristics of rock, and implications for earthquakes
Facteurs influant sur les caractéristiques d’adhérence-glissement des roches, et implications pour les tremblements de terre

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ABSTRACT: It has been observed that stick-slip on faults is a source of earthquakes. For the purpose of exploring characteristics of this phenomenon an experimental investigation was carried out on cylindrical specimens of granite containing a single discontinuity. Specimens were loaded triaxially up to 70 MPa confining pressure using a 5 MN servo-controlled stiff testing machine.

Important factors influencing stick-slip characteristics were examined, including effects of surface roughness, strain rate and moisture content of the sliding surfaces.

Stick-slip has taken place in granite specimens with various degrees of surface roughness, but at different confining pressure levels. Strain rate affected the static coefficient of sliding friction. Increase in strain rate decreased the static coefficient of friction. Presence of a very little water through the sliding surfaces affected sliding behaviour of the rocks significantly. In wet sliding surfaces occasional stick-slip only at high level of confining pressure was observed.

RÉSUMÉ: C’est bien connu qu’un glissement au niveau des failles peut être la source d’un tremblement de terre. Pour reconnaitre les caractéristiques de ce phénomène, une série de recherches expérimentales a été achevée sur cylindres de granite ayant une seule fissure. Utilisant une machine équipée par un servo-control de 5MN, les spécimens ont été chargés triaxiallement jusqu’à pression maximum de 70 MPa.

Par conséquent, les différents facteurs ayant une influence importante sur les caractéristiques du glissement des rochers ont été observés y compris la rugosité de la surface, le taux de déformation et la présence de l’humidité à la surface du glissement.

Le glissement a eu lieu sans exception dans tous les spécimens mais pour différentes pressions latérales. L’expérience a aussi montré que le taux de déformation affecte le coefficient du frottement statique: un taux plus élevé implique une diminution de ce coefficient. Enfin, il a été observé que la présence de la moindre eau à surface de glissement a des influences importantes sur le comportement des masses rocheuses. Le glissement a eu lieu parfois sur les surfaces humides mais seulement sous pressions latérales élevées.

INTRODUCTION

It has been observed that motion on fault surfaces can take place suddenly to produce an earthquake. During sudden slip shear stress is relieved, and the fault surfaces may then remain locked together until at some later time slip takes place suddenly again. Such sudden intermittent motion on pre-existing faults in the earth is similar to the
sudden intermittent motion that has been observed during frictional sliding between rock surfaces in laboratory experiments. In the laboratory experiments, this jerky type of motion is described as stick-slip (Byerlee, 1970). Most of the experimental investigations performed on the stick-slip phenomenon have not been conducted in favourable testing conditions. Specimen size, particularly undesirable testing systems have led to opposite, and perhaps misleading observations and conclusions. It was shown (Fahimifar 1993) that the type and configuration of seat and platen in the cell specimen system have significant effects on the sliding characteristics of the joint surface. Among several configurations, including a spherical seat at the top and a platen in the bottom which is often the case, the most satisfactory end-specimen condition was a configuration, in which pairs of hardened steel discs were polished and lubricated with a molybdenum disulphide grease and placed at the top and bottom of specimens. On the basis of this modification experimental research was performed to investigate the sliding and frictional characteristics of rock types containing a single plane of weakness under triaxial testing condition.

There are many factors affecting the stick-slip characteristics and behaviour, and most of them have been investigated before. On the basis of the modified cell-specimen system significant factors investigated in this experiment.

EXPERIMENTAL PROCEDURES

The experiment was carried out by a 5 MN servo-controlled stiff testing machine in the same way as described by Fahimifar (1993). The procedure used for the preparation of specimen was in accordance with the ISRM suggested methods (Brow, 1986). Granite specimens were cored to a nominal diameter of 75 mm and length of 150 mm, and were cut into two pieces with a diamond saw. The cut was perfectly plane at angle of 60° with respect to the short specimen axis.

A multichannel analogue data acquisition unit capable of interfacing with a microcomputer and with the servo-controlled system was used. The triaxial cell has sufficient internal space to allow large lateral displacement when sliding takes place along the joint surface. It has been designed for confining pressures up to 70 MPa. Confining pressure was applied hydraulically by a continuously operating electric pumping unit.

DISCUSSION OF RESULTS

From the examination and observation of the plots the following aspects are discussed.

EFFECT OF SURFACE ROUGHNESS

Three types of sliding surfaces were used in this work: saw cut, ground and cleaned-used surfaces. Cleaned-used surfaces mean those specimens that have been used in the previous tests after cleaning the sliding surfaces from gouge. Stick-slip was observed in granite in all types of the surfaces used, but in different confining pressures for the three different surfaces. In the saw cut surfaces stick-slip was observed in the range of 10-70 MPa confining pressures (figure 1). However, on the ground and cleaned-used surfaces below 30 MPa confining pressures stick-slip disappeared (figures 2 and 3). This behaviour may be attributed to the fact that in a saw cut sliding surface, because of further irregularities with respect to a ground surface, contact between two surfaces is limited to only a portion of the sliding area (the tips of asperities are in contact), and therefore the applied confining pressures in the range of 10-30 MPa and the resulting deviatoric stress is high enough to interlock the asperities to
Fig.1: Stress-strain plots for granite specimens at different strain rates. Confining pressures, 10, 30, 70 and 70 MPa.

Fig.2: Stress-strain plot for a granite specimen with ground sliding surfaces obtained in a multi-stage test in steps of 30, 70, then 30 and 10 MPa Confining pressures.

Fig.3: Stress-strain plot for a granite specimen with cleaned-used sliding surfaces, confining pressures in steps of 10, 30, 50 and 70 MPa.
the required level to cause stick-slip. In a ground sliding surface, however, because of a further real contact area between two surfaces (relative to the saw cut joint) the differential stress level at which stick-slip can possibly occur will increase. In reused sliding surfaces, although the sliding surfaces have been cleaned there is a considerable filling material (gouge) on the sliding surfaces which means a higher differential stress is required to interlock the asperities for the occurrence of stick-slip.

Figure 1 shows the stress-strain plots for a saw cut granite in 10, 30, and 70 MPa confining pressures. In 10 MPa the stick-slip amplitudes was very small with very low stress drop in each cycle. Figure 2 shows that stick-slip in the ground surfaces was diminished below 30 MPa confining pressure, however, at this level of applied pressure, after about 3% axial strain stick-slip was occurred with very low amplitudes which are very similar to the stick-slip on saw cut surfaces at 10 MPa confining pressure (figure 1 lower plot). By increasing confining pressure to 70 MPa (figure 2), the occurrence of stick-slip is very clear with much higher amplitudes. When confining pressure decreased to 30 and then to 10 MPa, stick-slip disappeared again and stable sliding continued. Presumably, this is due to the development of a layer of gouge over the sliding surfaces, which prevents entire contact and interlocking between asperities. In cleaned-used surfaces as in figure 3 the same behaviour as ground surfaces was observed. Up to 50 MPa confining pressure stick-slip was not occurred; in 50 and 70 MPa confining pressures, however, stick-slip was clearly present.

**STRAIN RATE EFFECTS**

A number of tests were carried out at strain rates of 2.08 x 10^{-5}/s (0.0125% / min), 4.17 x 10^{-3}/s (0.25% / min), 8.33 x 10^{-4}/s (0.5% / min), and 4.17 x 10^{-3}/s (25% / min). Figure 1 illustrates the typical plots obtained. For 30 MPa confining pressure loading began at a strain rate of 4.17 x 10^{-5}/s (0.25% / min). After a number of stick-slip movement (figure 1 plot 2, the first portion) the strain rate was increased to 4.17 x 10^{-3}/s (second portion of the plot). For 70 MPa confining pressure (figure 1 first part of the top plot) loading began at a strain rate of 4.17 x 10^{-5}/s (0.25% / min). After 10 stick-slip movements, strain rate was reduced to 2.08 x 10^{-5}/s (second part of the top plot in figure 1). Finally after three stick-slip occurred the specimen was unloaded, and then immediately loaded at a fast strain rate of 8.33 x 10^{-4}/s (5% / min) as in figure 1 plot 3.

In general, two significant events are observed among these runs.

(i) As strain rate decreased to a slower rate, the differential stress increased to a higher value in each stick-slip amplitude.

(ii) Increase in strain rate resulted in decrease in stress drop in each cycle of stick-slip.

Analysis of shear and normal stresses over the sliding surfaces reveals that an increase in strain rate results in a decrease in the coefficient of static friction. The coefficient of static friction, \( \mu_s \), is defined as \( \tau / \sigma \) where \( \tau \) is the shear stress at which a stick-slip occurs.

Shear and normal stresses can be obtained as follows:

\[
\sigma = (\sigma_1 + \sigma_3) / 2 - (\sigma_1 - \sigma_3) / 2 \cos \beta \quad (1)
\]

\[
\tau = (\sigma_1 - \sigma_3) / 2 \sin 2\beta \quad (2)
\]

In which, \( \beta \) is the joint orientation angle relative to specimen axis. In this experiment as \( \beta = 30^\circ \), \( \tau \) and \( \sigma \) may be given by:

\[
\sigma = (\sigma_1 + \sigma_3) / 2 - (\sigma_1 + \sigma_3) / 4 \quad (3)
\]
\[ \tau = 0.433(\sigma_1 - \sigma_3) \]  
(4)

using \( \mu_s = \tau / \sigma \) the following equation will be obtained.

\[ \mu_s = 1.732(\sigma_1 - \sigma_3)/(\sigma_1 + 3\sigma_3) \]  
(5)

Equation (5) shows that \( \mu_s \) is proportional to the \( (\sigma_1 - \sigma_3) \), i.e. the deviatoric stress; it may be concluded that increase in strain rate leads to decrease in deviatoric stress or in fact reduction in the static coefficient of friction.

Figure 1 (plot 2 the second part of the curve from about 1.2% axial strain) illustrates the significance of a very fast strain rate. As can be seen, the level of stress dropped significantly with respect to the first portion of the curve and the stick-slip amplitudes are very irregular. It implies that at a much faster strain rate, stick-slip may be disappeared throughout sliding.

The dependency of the static coefficient of friction to time seems to be a logarithmic function, since an order of magnitude change in strain rate produces a few percent change in \( \mu_s \). This dependency can be seen in figure 4 for 70 MPa confining pressure and strain rates of \( 2.08 \times 10^{-5}/\text{sec} \) (0.0125%/min), \( 4.17 \times 10^{-5}/\text{sec} \) (0.25%/min), and \( 8.33 \times 10^{-4}/\text{sec} \) (5%/min). As is observed the relationship is not linear and if the curve is extrapolated from its two ends interesting findings are resulted. The lower end of the curve intersects the X axis (strain rate). This implies that at a very fast strain rate \( \mu_s \) decreases dramatically and therefore stick-slip diminishes through sliding. On the other hand, the upper end of the curve continues asymptotically (parallel to the X axis) and it implies that at a very slow strain rate the static coefficient of friction tends to a constant value.

Time dependency in the stick-slip phenomenon was reported by other workers. Byerlee and Brace (1968) in their experiment did not observed any significant effect, however, Deterich (1970), Scholz et al (1972) and Engelder et al (1975) showed that strain rate affects stick-slip characteristics.

FRICIONAL SLIDING IN WET SURFACES

In order to assess the stick-slip characteristics in wet surfaces during sliding a series of tests were performed on granite with sliding surfaces of variable moisture content. The amount of water added to moisten the sliding surfaces of the granite specimens for each series of test were: 0.615, 0.2171, 0.0362 mg/g respectively. In the first run, the surfaces of two halves of the specimen were left in contact with water for 15 minutes. As soon as the sliding surfaces became quite wet the test was performed immediately. In the second run, the same procedure was repeated but the amount of water was decreased to 0.2171 mg/g on the sliding surfaces. In the third run, moisture content was decreased significantly so that there was no observable water on the surfaces (0.0362 mg/g). Figures 5 and 6 illustrate the stress-strain plots in a multi-stage test procedure for the three runs. As figure 5 shows at all levels of confining pressures, stable sliding is observed with a low positive slope in each step. Plot one in figure 6 shows the curve for the second run. The same behaviour as the first run occurred in all stages.

Plot two (figure 6) illustrates the sliding behaviour on wet surfaces during the run three. As is observed stable sliding has continued in all stages up to 70 MPa confining pressure. At this level of stress irregular stick-slip has occurred which are quite different from those of dry sliding surfaces (for instance, figure 1). On
Fig. 4: Static Coefficient of Friction vs Log Strain Rate.

Fig. 5: Stress-strain plot for a granite specimen with wet sliding surface (0.615 mg per unit weight). Multi-stage test in steps of 10, 30 and 50 MPa.

Fig. 6: Stress-strain curves for granite specimens with wet sliding surfaces, lower plot 0.2171, upper plot 0.0362 mg water. Confining pressures, 10, 30, 50 and 70 MPa.
examination of the specimen after each run a little mixture of water with gouge was observed over sliding surfaces in the first and second run. In the third run the surfaces were apparently dry, but some near to yellow spectrums were observable throughout the sliding surfaces which were an indication of moisture on those parts. In fact presence of very little moisture over sliding surfaces makes the rock soft. This may be attributed to the fact that when water surrounds the crystals or grains, interlocking of asperities decreases significantly. Consequently, stable sliding and disappearance of stick-slip is expected to occur.

The susceptibility of stick-slip to the presence of water has probably a significant role in the faulted areas in which stick-slip may occur. Depending upon the amount of moisture content through sliding surfaces it might be possible to predict the fault behaviour during sliding.

SUMMARY AND CONCLUSIONS

For the purpose of evaluating factors affecting stick-slip characteristics of rocks, a series of triaxial tests were performed on granite specimens. The most important conclusions are:

1. Stick-slip may occur through sliding of surfaces with different degrees of roughness, but not at the same level of confining pressures or deviatoric stresses.

2. Strain rate variation influenced stick-slip characteristics. Increase in strain rate resulted in decrease in both the stress drops and static coefficient of friction.

3. The occurrence of stick-slip was very susceptible to the presence of water. Presence of a very little moisture over the sliding surfaces caused stick-slip to disappear or occasional stick-slip to occur.

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Some problems on parameter statistics in rock mechanics
Quelques problèmes de statistique paramétrique en mécanique des roches

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ABSTRACT: Rock body is an engineering material with strong variability. Because of the change of ingredients and structure of rock which induce heterogeneity and anisotropy in rock mechanical properties, the rock mechanical parameters derived from test values of parts of samples are random variables with great uncertainty. It is necessary to perform probability analysis. The article analysis the origin of variability of rock mechanical parameters, optimum samples content, statistical methods, application of Bayes values and process of fetching values of rock mechanical parameters.

RÉSUMÉ: Les roches sont des matériaux avec une très forte variabilité. À cause des changements des minéraux et de la structure de la roche, qui induisent l'hétérogénéité et l'anisotropie mécanique, les paramètres de comportement mécanique dérivés des essais sur des éprouvettes sont des variables aléatoires avec une forte incertitude. C'est donc nécessaire d'entreprendre des analyses statistiques. La communication s'occupe de l'origine de la variabilité des propriétés mécaniques des roches, d'éprouvettes optimales, de méthodes statistiques, de l'utilisation des valeurs de Bayes et des processus d'obtention des paramètres mécanique des roches.

1 Introduction

Rock mechanical parameters in rock and soil engineering design, which are often derived from simple statistical analysis of test data and refer experiential data are a little arbitrary. Because of heterogeneity and anisotropy in mechanical properties of rock and external factors, the sample data are uncertain and with great scope. It is unreasonable to determine parameters in terms of average values. Concerning about the distribution types, discreet of test data and data process, however, the probability statistical method can be put forward.

2 Variability of physical mechanical properties of rock

Rock body is an engineering material with great variability, which results from ingredients and structure of rock. Evolutional process of rock from construction to transform is very sophisticated. Sedimentary rocks, in process of sedimentation, because of external material ingredients in vertical orientation, which are rhythmic, develop "Bedding" which are thick and thin or form difference of structure strength; vibration of earth's crust or change in water environment result in distribution of crisscross strata and lens body; difference of phase in magmatic rock makes differentiation among crystalline grain and mill ingredients. Constructional movement makes rock strata consolidation, fold and crack which make displacement and full of crack in rock body. After a circularly evolutionary process of denudation and weathering changing into soil, transportation and re-sedimentation, geological section and mechanical properties of rock body have obvious heterogeneity and anisotropy.

The uncertainty of parameters of rock me-
mechanics, in addition to uncertainty of rock properties, also comes from work method and knowledge, including:
1. Uncertainty of boundary of geological strata.
2. Anisotropy at same strata, involving sedimentation effect and uncertainty of stress.
3. The deviation from insufficient samples.
4. Test method in scene or room, disturbance of samples, accuracy of instrument or observe deviation.
5. Uncertainty of computational model.

The total anisotropy of rock properties is

\[ \delta_R = \sqrt{\delta_D^2 + \delta_s^2 + \delta_k^2 + \delta_t^2 + \delta_o^2} \]  

where \( \delta_R \) is an expression of total uncertainty between test values and actual values. However, by means of statistical extent variability of the second, the values of samples. It include to some forth and the fifth we can apply below formula to revise variability coefficient of statistical values.

\[ \delta_R = \delta_x / \psi \]  

\[ \psi = 1 - \sqrt{\delta_D^2 + \delta_s^2} \]  

where \( \delta_x \) is variable coefficient and \( \psi \) is revised coefficient of variables.

The uncertainty of geological boundary is related with rationality of distribution of investigation net, the number of investigation points and accuracy of geological works, especially how to deduced stratum between two holes, which can apply trend surface method, Markov model and Monte-Carlo simulate for approximation, make probability sections of strata.

3 Optimum samples content

The number of samples, concerning about heterogeneity of rock body, level of confidence and requirement for accuracy, are determined by the formula:

\[ N \geq \left( \delta_x / t_\alpha / P_x \right)^2 \]  

Where \( N \) is minimum samples content, \( \delta_x \) is variability coefficient of guarantee rate, and \( P_x \) is index of accuracy or relative deviation rate. Generally in rock engineering, when confidence level are required \( \alpha = 0.05 \), \( t_\alpha = 1.645 \). Relative deviation rate is about \( P_x = 10\% \), and variability coefficient of rock mechanical properties, \( \delta_x \) is between 0.2 and 0.3. The number of test groups of each body is between 10 and 20. If \( \delta_x = 0.15 \), \( N = 6 \) is the best samples content. According to research of M.S. Yucemen, the variability of rock resulted from insufficient samples which is related with the number of samples, is approximated \( \delta_x \sqrt{N} \). When \( N = 10 \sim 20 \), the uncertainty value is between 0.05 and 0.1, when variability coefficient resulted from test distribution, dimension reaction and stress is 0.24 and variable coefficient resulted from the uncertainty of computational model is about 0.1, the general variability is about 0.30 \( \sim \) 0.35.

4 The statistical process of rock mechanical parameters

The statistical process of rock mechanical parameters is on the basis of efficient observation and data collective by which the characteristic value (Average value and standard deviation) and experimental distribution, by which the statistical characters of parent population are replaced, can be solved, point estimate and interval estimate can be determined in terms of statistical deduction, and the tests of distribution type can be performed. \( \chi^2 \), K-S and A-D have been generally used in test. A-D method has strong identifying for small number of samples, so it is proper for rock engineering. Most random variables of rock parameters comply with normal distribution, logarithmic normal distribution or extremum distribution.

The statistical process of rock mechanical parameters is listed in Figure 1.
Monte-Carlo method of random sample is applied in rock engineering. After obtaining theoretical probability distribution model and statistical characteristic values, we applied computer to simulate random number which comply with uniform distribution in [0,1] interval, by certain transform, get simulate value of certain problems.

For restriction of number of rock test in singer engineering, Bayes method is often used to estimate parameters of rock mechanic. Rock mechanics of parameters in some areas depend on experimental data. Experimental data should be a comprehension of great scale of experiences, however, not be limited in local experience, so a number of these must be performed for a special engineering. Because of limited number of test, reliability is lower. However, we can combine experience with test, and the Bayes method is a good way for that. The prior probability in Bayes formula express probability distribution of experience, which

Figure 1 - The flow diagram of statistical process about parameters of rock and soil.
came from large parent population (National or Local), the great number of sample promote reliability of data. Test data in an engineering area are stem from same parent population. On basis of known experiential data, therefore, test probability is conditional probability. Prior probability must be re-estimated after performing test, and is calculated in terms of conditional probability after test. Applied full probability formula:

$$P(B_1|A) = \frac{P(B_1) \cdot P(A|B_1)}{\sum_{j=1}^{k} P(B_j) \cdot P(A|B_j)}$$

(5)

where $P(B_i)$ is prior probability. $P(B_1|A)$ is posterior probability. Denominator is full probability. In terms of posterior probability, rock engineering parameters with big probability can be solved called Bayes parameters values:

$$\mu_b = \sum_{i=1}^{k} P(B_i|A) \cdot f_i$$

(6)

where $\mu_b$ is Bayes values, $f_i$ is interval value of frequent and $K$ is the number of segments.

5 Transcend probability of engineering parameters

After characteristic values of a physical-mechanical character is solved, the probability when it is greater than or equal to characteristic values can be solved by below formula:

$$P(X \geq \mu_b) = \psi(\infty) - \psi\left(\frac{\mu_b - \mu}{\sigma}\right) = 1 - e^{-\left(\frac{\mu_b - \mu}{\sigma}\right)}$$

(7)

When random variables comply with normative distribution:

$$P(X \geq \mu_b) = 1 - \psi\left(\frac{\ln \mu_b - \ln \mu}{\delta_{ln}}\right)$$

(8)

When random variables comply with extremum distribution:

$$P(X \geq \mu_b) = 1 - \exp\left[-\exp\left(Y\right)\right]$$

(9)

$$Y = -1.283 \left[\frac{\mu_b - \mu}{\sigma} + 0.45\right]$$

where $P(X \geq \nu_b)$ is transcend probability, $\mu$ and $\sigma$ are average value and standard deviation of parent population. $\mu_b$ is a statistical value.

Where $\mu_{x,x}$, $\sigma_{x,x}$ and $\delta_{ln}$ are average value, standard deviation and variability coefficient. Because of difference between element and parent population, the variability of element should be considered and $\mu$ and $\sigma$ must be revised.

When strength variables comply with normative distribution, the unit distribution function comply with $\Gamma$ distribution. Applied Bayes method to compute posterior distribution parameter:

$$\mu = \frac{n_1\mu_1 + n_{o}\mu_o}{n_1 + n_o}$$

(10)

$$\sigma = S \cdot \sqrt{\frac{n_2 - 1}{2}} \cdot \frac{\Gamma((n_2 - 2)/2)}{\Gamma((n_2 - 1)/2)}$$

(11)

$$S = \frac{1}{n_2 - 1} \left[ n_o\mu_o^2 + (n_o - 1)S_o \right] + \left[ (n_1 - 1) + n_1\mu_1^2 \right] - n_o\mu^2$$

(12)

where $\mu_o$, $S_o$, $n_o$ are average value, variance, and the number of samples of the parent population; $\mu_1$, $S_1$ and $n_1$ are average value, variance and the number of samples of element respectively. $\mu$, $S$, $\sigma$ and $n$ are average value, variance, standard deviation and the number of samples,

$$n = n_o + n_1$$

(13)
6 Fetching parameters of rock mechanics

Data fetch process is listed in Figure 2. After fetching base values by test, we sort and merge them according to element of engineering geology, input them into computer to perform statistics, fetch arbitrary item from Monte-Carlo and Bayes values after they are solved and then calculate standard value by fetching probability fractile. Standard value is changed into design value only when compensatory coefficient is considered.

![Diagram](image)

Figure 2 - The process of fetching parameter values of rock and soil engineering.

The formulas are listed below:

1. Standard value

\[ R_b = \mu_R - 1.645 \sigma_R \]  

(14)

2. Design value

\[ R = \psi \cdot R_b \]  

(15)

where \( \mu_R \) is statistical value; \( \sigma_R \) is standard deviation; \( \psi \) is compensatory coefficient. For freshly and slightly decayed rock it is assigned \( 0.1 \sim 0.23 \) and for weakly decayed rock it is assigned \( 0.17 \sim 0.25 \). The standard value is assigned probability fractile 0.05 if construct factors and time effect is not concerned about.

(2) The shear strength of rock dam base

1. Standard value

When random variables \( (f,c) \) comply with normative distribution:

\[ f_k = \mu_f + K_2 \sigma_f \]  

(16)

When random variables \( (f,c) \) comply with logarithmetic normative distribution:

\[ f_k = \frac{\mu_f}{\sqrt{1 + \sigma_f^2}} \exp \left[ K_2 \sqrt{\ln (1 + \sigma_f^2)} \right] \]  

(17)

where \( K_2 \) is coefficient relate with probability, when probability is assigned fractile 0.05, \( K_2=1.645 \), when probability is assigned fractile 0.2, \( K_2=-0.84 \) when probability is assigned fractile 0.5, \( K_2=0 \), \( \mu_f \) is average value, \( \sigma_f \) is standard value and \( \delta_f \) is variability coefficient.

2. Design value

\[ f_k = f_k / \Gamma_f \]  

(18)

normative distribution:

\[ \Gamma_f = \frac{1 + K_2 \delta_f}{1 + K_1 \delta_f} \]  

(19)

logarithmic normative distribution:

\[ \Gamma_f = \frac{1}{\exp \left[ (K_1 - K_2) \delta_f \right]} \]  

(20)

where \( f_d \) is design value; \( f_k \) is standard value; \( \Gamma_f \)
is compensatory coefficient \( K_1 = \psi^{-1}(\alpha f_1) \);
\( K_2 = \psi^{-1}(\alpha f_2) \); \( (\alpha f_1) \) is fractile of design value of
parameters in distribution. When \( (\alpha f_1) = 0.02275 \),
\( K_1 = -2 \). \( (\alpha f_2) = 0.05 \) is fractile of standard value
of parameters in its distribution. When
\( (\alpha f_2) = 0.05 \), \( K_2 = -1.645 \) and when \( (\alpha f) = 0.2 \),
\( K_2 = -0.84 \).

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Engineering geological classification of weak argillaceous rocks in some Egyptian deserts
Classification de géologie de l'ingénieur de roches argileuses faibles dans quelques déserts égyptiens

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ABSTRACT: In general, the Egyptian desert formations consist of sand and gravel interbeded by lenses of weakly argillaceous rocks which is locally called "TAFILA". However, theses formations can be classified in the geological literature into a wide range of clayey shales to sandy mudstones. Also they have wide variations from the point of view of engineering behaviour. Ten Egyptian desert regions including 29 locations are taken to apply this study. The engineering geological classification of these deposits are based on the determinations of the natural dry density (yd), the predominant particle size; the unconfined compressive strength before wetting (oc) and after wetting (oc'); the swelling and collapsing percentages under 0.2 MPa and the slaking durability percentage (s.d.). The study indicated that these weak rocks could be classified into two main groups each containing three subgroups. The three subgroups of the first main group are named clayey TAFILA; silty TAFILA and sandy TAFILA. They have a relative high value of dry density, a high percentage of clay content, a weak value of compressive strength before wetting, an ability to swelling of the second main group are named TAFFY sil and TAFFY sand having a relative low value of dry density, a very to extremely weak value of compressive strength before wetting, an ability to collapsing more than swelling and a very low slaking durability percentage. However all these weak rocks have an extremely weak compressive strength after complete wetting.

RÉSUMÉ: En général les formations déserts Egyptiens sont constituées des matériaux sableux et/ou gravieux présentant souvent des niveaux siltos-argileux, sous forme de lentilles ou de barres discontinuées. Ces niveaux sont appelés sur place "TAFILA". Cependant, ces matériaux se classent en littérature géologique sur un grand choix de schistes argileux et sableux. D’ailleurs ils démontrent des variations du point de vue du comportement d’ingénieur Dix régions désertes Egyptiennes comportant 29 endroits échantillonnés sont prises pour appliquer cet étude. La classification d’ingénieur géologue de ces gisements est basée sur la détermination des propriétés suivantes: Densité sèche naturelle; Granulométrie dominante; Resistance en compression simple avant et après mouillage; Pourcentage de gonflement et d'affaissement sous 0.2 MPa et Pourcentage de durabilité en étalement. Ces roches faibles peuvent être classifiées en deux groupes principaux. Chacun se compose de trois sousgroupes. Le premier groupe est appelé TAFILA argileux; TAFILA silteux et TAFILA sableux qui ont des valeurs relatives bien augmentées en densité sèche et en pourcentage d'argile et diminuées en résistance en compression simple avant mouillage, pourcentage de durabilité en étalement et le gonflement est plus fréquent que l'affaissement. Le deuxième groupe est divisé en argile TAFLEUX; silt TAFLEUX et sable TAFLEUX qui ont des comportements opposés au premier groupe.

1 INTRODUCTION
The deposits in many Egyptian deserts consist principally of sands or/and gravel. Some weak argillaceous rocks or cemented soils have been encountered as lenses or thinner layers between theses formations.

These weak rocks are generally unstable, especially when they are exposed to wetting. They have caused considerable damages to roads, dams and building foundations constructed with or on it. The basic characteristics of these weak rocks are that they have various colors such
as grey, brown, yellowish, reddish brown, etc... They contain some clay materials and are partially saturated materials. They have a potential for swelling or collapsing with change in moisture content, to a degree governed by the environmental changes to which these rocks are subjected. The magnitude and direction of volume change will depend on many factors, including the mineralogy of the clay mineral present, the relative proportion of active clay particles to non-clay particles, the initial moisture content, density, particle structure and stress conditions of the rock, the new environmental conditions imposed on the rock and the time available for response by the clay (Dudley, 1970; Gromko, 1974; Peck et al., 1987; Azcue & Valenti, 1985 and Singer et al., 1989). They are also characterized by having high relative strength at their natural moisture content and a drastic decrease in strength has been observed after wetting.

The geological classification of the argillaceous cemented materials such as shales and mudstones proposed by Nead, 1936; Underwood, 1967 and Pettijohn, 1975 are usually not satisfactory for engineering purposes.

An attempt is made herein to develop a classification for these weakly rocks based on their mechanical properties such as unconfined compressive strength before and after wetting conditions, volume changes behavior (swelling & collapsing) and slaking durability percentages.

For this purpose 29 samples are collected from 10 Egyptian desert regions. The calcareous and the very highly indurated formations are not included in this study because there are not enough samples for these types. A further study could be make to classify these formation types.

2 THE PROBLEM AND CAUSES OF PHENOMENON

According to Fookes et al., 1971; Popescu, 1986; Gidgasu, 1987; Chandler & Apted, 1986 and Varley, 1990, the commonest known problem for engineering rocks and soils in the world are

1. Basic rocks subject to rapid physical - chemical weathering.
2. Carbonate soft rocks.
3. Various types of pedogenic materials (e.g. lateritic rocks, calcretes).
4. Swelling rocks (e.g. weathered shales, fissured stiff clay, mudstones).
5. Collapsing rocks (e.g. loess, residual and wind deposited).
7. Salt bearing rocks.

8. Organic and compressible soils.

In Egypt the most predominant of these problematic rocks are two types, the swelling and the collapsing rocks. Therefore, they were encountered in the studied regions and enough samples were available for this investigation.

A typical situation associated with a swelling or collapsing foundation rock is illustrated in figure 1. With certain conditions in the rock foundation materials, structures which may have stood satisfactorily for some time suddenly or gradually experience a settlement or a heave has been done, with a highly reduction of their shear strength. The inevitable break down of water supply and sewage systems, in addition to surface drainage and irrigation will cause the formation of perched water table and temporary water horizons which will alter the natural moisture equilibrium and change the degree of saturation of the foundation rock.

An appreciable swelling or collapsing of a rock requires the following three conditions (Popescu, 1986; El-Sohby et al., 1986 and Abduljawad & Al-Gassous, 1991):

1. A potential unstable partially saturated rock structure.
2. A critical increase of rock moisture content.
3. A value of the applied stress which must be low enough for swelling rocks and high enough for collapsing rocks to develop a metastable condition.

The swelling rocks are characterized by a high relative value of natural dry density a low values of void ratio and a high percentage of clay content. Where as the collapsing rocks are characterized by a low relative values of natural dry density ; high values of void ratio and a low percentage of clay content.

Swelling is a result of the complicated processes taking place in the course of the interaction between water and the lattice structure of a clay rock particle. There are two basic mechanisms involved in swelling phenomena (Gillott, 1986):

1. Interparticle or intercrystalline swelling, shown diagrammatically in figure 2 which is effective for all kinds of clay minerals. In a nearly dry clay deposit relict water holds the particles together under tension from capillary forces. On wetting, the capillary tension are relaxed and the clay expands. The effect is the same whether the clay has the form of particles as shown in the upper part of the figure or of crystals as shown in the middle part. The short dashes in the figure which link the layers of the clay crystals imply that the layers are strongly bonded by molecular forces.
2. Intracrystalline swelling which is chiefly a characteristic of montmorillonite group of minerals. The layers that make up the individual single crystals of montmorillonite are weakly bonded, mainly by water in combination with exchangeable cations. On wetting, water enters not only between the single crystals, but also between the individual layers that make up the crystals.

However, the structure of collapsing rocks is typical open and contains many void spaces. They have their grains held in place by some bonding materials which are susceptible to removal or reduction by the arrival of additional water. When the support is removed, the grains are able to slide on one to another moving into the vacant spaces and the material suffer structural collapses.

Clay coatings have been suggested by Popescu, 1986 as a major bonding agent between the bulk sand or silt grains (figure 3). Cementing agents such as calcium carbonate, iron oxide or welding of the grain contacts provide temporary strength of many collapsing rocks. In cases where the rock consists of sand with a fine silt binder, the temporary strength is due to the capillary tension at the air water interface in capillary voids (Barnes et al., 1973). Whatever the physical basis of the bond strength, all collapsing rocks are weakened by the addition of water. Collapse of rock structure is more immediate in the case where the grains are held together by capillary suction, slow in the case of chemical cementing, and much slower in the case of clay bonding (Clemen and Finbeir, 1981).

The reduction of natural temporary strength of the swelling and collapsing rocks due to wetting (saturation) can be related to rock composition, including cementing agents, percentage of various textures fabric and the capillary forces between particles, etc. (Clemence and Finbeir, 1981 and Abduljauwad, 1991. Upon wetting the bonding forces between the grains of the weak rocks are lost or softened and the particles relax, causing reduction of their strength.

3 MATERIAL TESTED

A total 29 undisturbed samples are taken from nine Egyptian desert regions (figure 4). The detailed locations and a brief geological description are shown in table 1. The cementation of weak rocks and the absence of water table furnished suitable conditions for obtaining block samples from
open pits. Block of rocks were cut by hand trowel from the sides of test pits, sealed in plastic bags to preserve the natural moisture content, and transported with a minimum delay and minimum disturbance to the laboratory.

4 SAMPLE PREPARATION AND TEST PROGRAM:

Sampling and trimming procedures of weakly rocks required great care in order to avoid disturbance. The trimming itself was a challenge. First, the samples were hand saved down to a size of about 30% above the final dimensions required. Next, the sample was slowly shaved by a variety of fine steel blades and sand papers until the proper dimensions were reached. Finally, each sample was encased carefully with aluminum foil, kept in a plastic bag and preserved until the time of performing the required tests.

From each location 3 undisturbed samples and one representative disturbed sample were prepared to determine their physical and mechanical properties. The physical properties contain the natural moisture content (mo); the natural dry density (d); the degree of saturation (Sr); and the percentages of clay, silt and sand (CL%, ML%, & SM% respectively). The mechanical properties include the slaking durability percentage (s.d.) the unconfined compressive strength at natural condition (UCS); the unconfined compressive strength after wetting (UCS'); and the volume changes after saturation (Av).

For this purpose the following test program was executed:

1. The disturbed sample is used to determine the percentages of clay, silt and sand contents in the whole sample according to ASTM, 1985.

2. The first undisturbed sample was trimmed to any size. This sample is required to determine the natural moisture content and the natural dry density according to ASTM, 1985. and the slaking durability percentage after getting it out from the oven (according to Fookes et al., 1971 and Taylor, 1968).

3. The second undisturbed sample was trimmed to 35 mm in its diameter and 50 mm in its height. This sample was provided to measure the unconfined compressive strength at the natural condition (according to Hawkes & Mellor, 1970 and Barton et al., 1974).

4. The third undisturbed sample was trimmed to the same size as in case 3. This sample is prepared to determine the volume change percentages after its saturation under 0.2 Mpa (according to Barren et al., 1963 and El-Sobhy & Ellaboudy, 1987). This percentage is positive for the swelling.
Table 1: Typical and mechanical properties of studied rock samples

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Particle Size</th>
<th>Unconfined Compressive Strength</th>
<th>Unconfined Shear Strength</th>
<th>Elasticity Modulus</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartzite</td>
<td>1-2 mm</td>
<td>200 MPa</td>
<td>20 MPa</td>
<td>70 GPa</td>
<td>0.25</td>
</tr>
<tr>
<td>Sandstone</td>
<td>0.1-1 mm</td>
<td>150 MPa</td>
<td>15 MPa</td>
<td>60 GPa</td>
<td>0.30</td>
</tr>
<tr>
<td>Limestone</td>
<td>0.01-0.1 mm</td>
<td>100 MPa</td>
<td>10 MPa</td>
<td>50 GPa</td>
<td>0.35</td>
</tr>
<tr>
<td>shale</td>
<td>1-2 mm</td>
<td>120 MPa</td>
<td>12 MPa</td>
<td>55 GPa</td>
<td>0.28</td>
</tr>
</tbody>
</table>

To apply the obtained results, the mechanical properties of the rock sample are determined. The mechanical properties include the unconfined compressive strength, unconfined shear strength, and elasticity modulus. The Poisson's ratio is also calculated for each sample. The particle size distribution and the mechanical properties are the key factors in determining the behavior of the rock in various engineering applications.
Fig. 5 Geological Classification of the Weakly Argillaceous Rocks

Fig. 6 Mechanical Properties of Studied Rocks in Function of the Geological Classification
Fig. 7 A Proposed Schematic for Engineering Geological Classification of Studied Weakly Rocks.

- The natural dry density ranges between 16.5 and 17 KN/m³.
- The percentage of clay contents ranges between 10 and 15% (table 2).

7 CONCLUSIONS

The results presented in this paper lead to the following conclusions:

1. Argillaceous weak rocks prevailing in Egypt have a weak compressive strength in the natural state (Gc = 1 - 3 Mpa) and a very weak to extremely weak in the wetting state (Gc' = 0 - 0.9 Mpa).

2. By introducing the local name "TAFLA", these argillaceous weak rocks could be classified from the point of view of engineering geology into two main groups. Each group contains three subgroups.

   - The first group is named clayey TAFLA; silty TAFLA and sandy TAFLA.
   - The second group are TAFLY clay; TAFLY silt and TAFLY sand.

3. The first group "TAFLA" have the following characteristics:
   - The natural dry density is more than (17.5 - 18 KN/m³).
   - The clay percentages are more than (20% - 25%).
   - The unconfined compressive strength at the natural conditions is between 1.5 and 3 Mpa.
   - The ability of swell is more than the ability to collapse.
   - The percentage of slaking durability ranges between low and very low (10% - 25%).

4. The second group "TAFLY" have the following characteristics:
   - The natural dry density is less than (16.5 - 17 KN/m³).
   - The clay percentages are less than (10% - 15%).
   - The unconfined compressive strength at the natural conditions is between 1 and 1.4 Mpa.
   - The ability to collapse is more than the ability to swell.
   - The percentage of slaking durability is very low (0% - 5%).

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Effect of swelling percentage on the shear strength of a compacted clay liner material

Effet du pourcentage de gonflement sur la résistance au cisaillement d'un matériau de revêtement d'argile compactée

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ABSTRACT: A clay liner material composed of a mixture of silty clay and bentonite was compacted at a moisture content 1% less than its optimum moisture content. An experimental testing program was carried out on this liner material to study the effect of different relative swelling percentages (0, 20, 25, 50 and 100%) on the shear strength parameters of this material using direct shear apparatus. Three different methods were used to achieve these swelling percentages. The results of the experiments indicate that the angle of internal friction and cohesion were both different with each method, even though they were measured at the same swelling percentage.

RÉSUMÉ: Une barrière argileuse composée d'un mélange silteux et de bentonite a été compactée à teneur en eau de 1% moins que la teneur en eau optimum. Un programme d'essais expérimentaux a été exécuté sur ce matériau pour étudier l'effet de différentes pourcentages de gonflement (0, 20, 25, 50 et 100 %) sur les paramètres de résistance au cisaillement utilisant la boîte de cisaillement direct. Trois méthodes différentes ont été utilisées pour arriver aux pourcentages de gonflement demandés. Les résultats expérimentaux indiquent que l'angle de frottement interne et la cohésion étaient différents avec chaque méthode même quand ils sont mesurés au même pourcentage de gonflement.

1 INTRODUCTION

In recent times, bentonite based materials have been used increasingly as liner materials in engineering design to inhibit the migration of contaminants from a disposal site to the surrounding environment. Thus, a study of the effect of swelling percentage on the shear strength of liner materials is very useful as it may undergo different moisture conditions starting from an initial partially saturated state to a state of complete saturation.

Previous research in this field dealt with only the relation between swelling characteristics and shear strength of soils after complete saturation, i.e. 100 % relative swelling. Gibbs et al., 1960; El-Sohby et al., 1987; Katti et al., 1969; Mersi and Olson 1970; Graham et al., 1989; Handy and Turgut 1987; Borgesson et al., 1990; Friedli, 1984 and O'Hawari and Schetelig 1991 studied the effect of amount of swelling strain on the shear strength of clays but for different initial swelling strains. They soaked the samples until full saturation and sheared them, hence they only dealt with shearing after complete saturation.

In the present work, shearing the samples at 0%, 20%, 50% and 100% relative swelling, having started with the same initial conditions. Direct shear tests were conducted in a consolidated undrained mode. Three different ways to arrive at these swelling percentages were used, namely:

1. Complete saturation with axial free swell allowed, i.e. the sample is completely saturated under the standard load of 6.9 kPa and then reloaded back to the desired degree of relative swelling.

2. Complete saturation with controlled swell, i.e. the sample is completely saturated under an applied load corresponding to the desired degree of relative swelling.

3. Controlled degree of saturation, i.e. the sample is not completely saturated under a very light load to the desired degree of relative swelling.
2 SAMPLE PREPARATION

The liner material tested was a mixture of a natural silty clay and bentonite. The silty clay came from the Cincinnati area, Ohio, U.S.A., having the following properties:

- Liquid Limit ($W_l$): 30 to 31.9 %
- Plasticity Index (PI): 10 to 10.3 %
- Optimum water content ($m_{opt}$): 13 %
- Maximum dry density ($\gamma_{d,max}$): 17.7 kN/m$^3$

The bentonite used was from Wyoming, U.S.A., having the following properties:

\[ W_b = 520\% \text{ ; } PI = 458\% \]

\[ m_{opt} = 37\% \text{ ; } \gamma_{d,max} = 13 \text{ kN/m}^3 \]

The percentage of bentonite mixed with the silty clay was based on the desire of obtaining an appreciable amount of free swell. Standard Proctor tests (ASTM, D698) and free-swell tests (Holtz and Gibbins, 1956) for different percentages of bentonite were conducted. Figure 1 shows the obtained results. From this figure, it can be observed that the favorable percentage of bentonite with regards to maximum dry density and optimum water content is between 15 to 20%. However, a mix with 20% of bentonite was better considering free-swell. Hence, an expansive soil mix comprising of 80% silty clay and 20% bentonite was chosen. For obtaining a higher value of dry density, Modified Proctor test (ASTM, D1557) was performed on this material. It gave a maximum dry density of 18.8 kN/m$^3$ at an optimum water content of 13%.

The initial test sample conditions were as follows:

- Initial dry density ($\gamma_{d}$): 18 kN/m$^3$
- Optimum water content ($m_p$): 12% = $m_{opt} - 1$
- Diameter of the sample: 62.7 mm
- Height of the sample: 12.7 mm

The following procedure was used to prepare the sample:

1. Take enough weight of oven dried powdered soil material to make three samples of height and diameter as specified above.
2. Mix 20% of bentonite with 80% of silty clay in a mechanical mixer.
3. Add water slowly, using a burette, to obtain an initial water content of 12%.
4. Continue the mixing operation till a homogenous color is obtained.
5. Wrap the soil mix in three layers of aluminium foil and store it in the moisture chamber inside a sealed container for at least 48 hours for curing the soil mix at the 12% water content.
6. Knowing the desired weight, height and diameter of the sample, compact it statically in the oedometer directly, using an universal loading apparatus, with a constant speed of 2 mm/min. The static compaction pressure required was 238 MPa to arrive at the sample height of 12.7 mm.
7. Swell the sample to the desired degree of relative swelling in the oedometer according to ASTM, D3077.
8. After the sample has stabilized with the required degree of relative swelling, conduct a direct shear test according to ASTM, D3080.

3 TEST PROGRAM

Case 1: complete saturation under a very light normal stress

Procedure:

1.1 Three samples were allowed to swell under 6.9 kPa normal stress to attain stabilized swell conditions (in about 1 week). The samples were then consolidated and sheared under different normal stresses (69, 138, 207 kPa) in the shear box.
1.2 Three samples were allowed to swell under 6.9 kPa normal stress until swelling had stabilized. The samples were then consolidated until their height reached that of 50% of the final swelling (50% relative swelling, $R_s$). The samples were then sheared...
as in step 1.1.
1.3 Same as in 1.2, but the samples were consolidated until their height reached that of 20% of the final swelling.
1.4 Same as in 1.2, but the samples were consolidated until 0%, i.e. to the initial height of sample.

Case 2: complete saturation under various normal stresses

Procedure:

2.1 Three samples were allowed to swell under 24 kPa normal stress which corresponds to about 50% of relative swelling, $R_i$, to attain stabilized swell conditions. Then the samples were consolidated and sheared.
2.2 Same as in 2.1, but swell took place under 34.5 kPa normal stress which corresponds to about 25% of the relative swelling.
2.3 Same as in 2.1, but swell was accomplished under 138 kPa normal stress which corresponds to 0% of relative swelling.

Case 3: incomplete saturation under a very light normal stress

Procedure:

3.1 Three samples were allowed to swell under 6.9 kPa normal stress to attain 50% of relative swelling. They were then consolidated and sheared.
3.2 Same as in 3.1, but only 20% of relative swelling attained.
3.3 Three samples were consolidated and sheared in the shear box under the initial conditions, i.e. 0% of relative swelling.

4 EXPERIMENTAL RESULTS

The following is a description of results obtained, as shown in figs. 2, 3, 4, 5 and 6. For the sake of clarity, the discussion is divided into three phases:

Phase A [at zero percentage of relative swelling, $R_i=0\%$]

A.1 This is the case 1.4 which The sample was completely saturated and was allowed to swell freely (under 6.9 kPa) and then was brought back to 0% relative swelling by reloading with a normal load of 490 kPa. It can be seen that the cohesion, $C$ is reduced to a lower value of 67.6 kPa from the initial value of 207 kPa, while the angle of internal friction, $\phi$ increased to an average value of 21° from the initial value of 4°.
A.2 Complete saturation case with no swelling allowed to take place, i.e. the initial stress was 172.5 kPa rather than 6.9 kPa, to attain 0% of relative swelling (Case 2.3). Here $C$ is brought down to a much lower value (only 51.8 kPa) while $\phi$ increases significantly to 30°.
A.3 Incomplete saturation, i.e. compaction followed by shear of sample, corresponding to case (3.3). It is seen that the shear strength parameter C has a much bigger value relative to the shear strength parameter $\phi$ and plays a governing role. The observed values are $C = 207$ kPa and $\phi = 4°$.

Phase B [from 0 to around 50% of relative swelling]

B.1 Complete saturation with free swell allowed, corresponding to cases (1.3) and (1.2). It is noticed that $C$ decreases slightly to 48.3 kPa, but $\phi$ decreases significantly to 9°.
B.2 Complete saturation with no free swell allowed to take place corresponding to cases (2.2) and (2.1). Here $C$ shows a slight decrease to 34.5 kPa while $\phi$ decreases but is still bigger than case 1 (19°).
B.3 Incomplete saturation under 6.9 kPa corresponding to cases (3.2) and (3.1). $C$ decreases to a minimum of 12.4 kPa while $\phi$ increases to a maximum of 30°.

Phase C [from around 50% to 100% of relative swelling]

C.1 Completely saturated with free swell allowed corresponding to cases (1.2) and (1.1), $C$ does not show significant change as it approaches case (1.1), the change being only from 41.4 to 41.4 kPa but $\phi$ decreases significantly from 9° to 3°.
C.2 Complete saturation case with no free swell allowed corresponding to cases (2.1) and (1.1), $C$ shows the same behaviour as case 2 above. However $\phi$ decreases significantly from 19° to 3°.
C.3 Incomplete saturation corresponding to cases (3.1) and (1.1), it is normal to expect $C$ to rise to the value of complete saturation, 100% relative swelling corresponding to case (1.1). Hence $C$ increases from 12.4 to 41.4 kPa but $\phi$ decreases from a peak maximum value to that corresponding to case (1.1), i.e. from 30° to 3°.

5 PRELIMINARY OBSERVATIONS

The swelling behaviour of bentonite based materials originates from the special microstructure of the montmorillonite minerals. Each montmorillonite mineral consists of several unit molecular integrations, between which are Na⁺ cations which have a high energy of hydration. When the water is introduced to the montmorillonite mineral, the water molecules interact with montmorillonite's basal surfaces and with the interlayer cations in a dynamic manner (Sposito, 1984, Newman, 1987 and Guven, 1990). Water's molecular motion (rotational, vibrational and translational) and its orientation are thereby perturbed. The hydration energy of the Na⁺ cations of montmorillonite is for example around 3.97 Kcal/mole (Garrels and Fardy, 1982). This dynamic interaction of water molecules within the interlayer space is expected to have significant effect on the structure of interlamellar water and hence on swelling and the shearing strength of bentonite.

By the dynamic energy, the swelling phenomenon starts and the particles become delaminated as flocculated or honeycombed structures. This may increase the interlocking between the particles and hence the friction component of shear strength of the swelling material increases too. This statement is in agreement with Stepkowska, 1990 who mentioned that contacts between structural elements of montmorillonite is in principle due to from exchangeable cations between unit layer of one particle or between a unit layer forming a "bridge" and two neighboring clay particles. The energy of these bonds amounts to about 20 Kcal/mole. Thus they participate most probably in the deformation process of montmorillonite during their swelling. This energy results in particle disturbance and disarrangement or delamination due to "dragging". Hence the friction component ($\phi$) is due to particle disarrangement.
6 DISCUSSION OF RESULTS

6.1 States of incomplete saturation

The compacted soil specimen before allowing to swell (case 2.3) has particles sticking to each other, while there is no movement yet of soil particles, hence a bigger value of C is observed while \( \phi \) is insignificant.

When water is introduced to this expansive soil mix, cases 3.2 and 3.1, it acts as a lubricant between the particles which decreases the sticking, hence C decreases to a minimum value and then rises slightly to the state corresponding to complete saturation.

Simultaneously, the movement and delamination between particles increase and hence \( \phi \) rises and continues to rise until it reaches maximum potential energy of particle movement and then it decreases to the state corresponding to complete saturation (no movement at all). This is, because when swelling starts the swelling energy is distributed in all directions of the sample, which is laterally confined. Hence the delamination and interlocking between the particles increases more in the lateral direction than in the vertical direction, hence we have an increase in \( \phi \) during shearing at the horizontal plane. After around 50% of relative swelling, the remaining energy of swelling is directed to the vertical direction mostly which results in a decrease of \( \phi \) in the horizontal plane.

The fact that C rises slightly as the near saturation state is approached (after dipping to a minimum), can be explained by the fact that when movement between particles stabilizes, a slight cohesion is built up.

6.2 States of complete saturation

It is seen that at 0% swelling (without allowing any free swell, case 2.3) the particle structure is conserved and movement takes place between the soil particles in the internal structure and it gives a relatively large value of \( \phi \). As swelling was allowed to take place, the value of \( \phi \) reduces and this reduction can be explained by the decrease of internal interlocking.

However considering case 1.4, the expansive soil mix was allowed to swell freely until complete saturation, on reloading to reach the same initial height, the internal particle structure was disturbed, the clay did not gain the original interlocking as in case 2.3, which
has the same degree of relative swelling (0%).
Considering the C values, there was no difference found, because it is not affected by the particle movement or interlocking. It only represents the adhesion between particles, which is not considerably affected in the complete saturation case with or without free swell.

7 CONCLUSION

1. Overall, it was observed that the shear strength parameters C and Ø will change considerably during swelling phenomenon approaching a saturated state.

2. It was noticed that during swelling, Ø increased to a maximum at around 50% relative swelling and then decreased as R_1=100% was approached. Conversely, C decreased to a minimum value again at around R_1=50% and increased slightly when R_1=100% or complete saturation was reached.

3. In the completely saturated state, in general, Ø decreased significantly as a function of the relative degree of swelling, while C decreased slightly.

4. The increase of Ø with decrease in relative degree of swelling was more prominent for the case of complete saturation with no free swell allowed, than for the case of complete saturation with free swell allowed and followed by reloading to the desired degree of relative swelling.

5. The strength parameter Ø is more sensitive to the degree of relative swelling than compared to C, which does not change much in the completely saturated cases.

6. In the case of incomplete saturation, i.e., short term or end of construction case, an increase was observed in the shear strength parameter Ø, while C decreased significantly. In the long term or completely saturated case, both C and Ø decreased with increasing degree of relative swelling.

7. For any desired relative degree of swelling, i.e., long-term or completely saturated cases, the expansive soil tested in this research had a much higher Ø when let to swell to that degree, than to let it swell freely to an R_1=100% and then to compress it back to desired R_1.

8 ACKNOWLEDGEMENT

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Influence du fluide hydratant sur le gonflement des argiles pures
Influence of the hydrating fluid on the swelling of pure clays

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RESUMÉ : Par une investigation expérimentale en laboratoire, nous mettons en évidence l'influence qu'exerce la composition chimique du fluide hydratant sur les paramètres de gonflement de 2 argiles pures : une montmorillonite grasse et une kaolinite. L'hydratation de ces 2 argiles par des solutions de chlorure de potassium (KCl), sodium (NaCl) et calcium (CaCl₂) préparées à différentes concentrations a permis d'étudier les effets séparés de la nature du cation (sa valence et sa taille) et de la concentration des solutions sur les paramètres de gonflement. Les résultats trouvés ont révélé que paramètres de consistance et potentiel de gonflement libre varient, pour toutes les solutions testées, suivant des schémas identiques. Ces résultats ont montré en particulier que le potentiel de gonflement de la montmorillonite chute considérablement pour toutes les solutions à faible concentration (0.25 mole/l). L'augmentation de la concentration des solutions au delà de 0.25 mole/l ne semble pas affecter sérieusement le potentiel de gonflement. La pression de gonflement ne diminue nettement qu'à de fortes concentrations.

ABSTRACT : Through a laboratory study, we show the control of the saturating water chemistry upon the swelling characteristics of 2 pure clays : a fat montmorillonite and a kaolinite. Saturating these clays by solutions of sodium, potassium and calcium chlorides prepared at different concentrations allowed to study the separate effect of the type of cation present in the solution (size and valency) and that of the fluid concentration on the swelling parameters (swelling potential and swelling pressure). The results revealed that consistency limits and free swelling potential values vary, for all the tested solutions, following the same trends. The results showed in particular, the swelling potential of the montmorillonite drops tremendously at low concentrations of cations (0.25 mole/l), when concentrations higher than 0.25 mole/l do not appear to affect considerably the swelling potential. The swelling pressure, on the other hand, vary distinctively only at high concentrations.

INTRODUCTION

L'amplitude du gonflement et les pressions qu'il peut engendrer quand il est contrarié, dépendent d'un grand nombre de facteurs tels que la composition minéralogique du sol, sa structure, la nature des cations adsorbés et de ceux présents dans la phase liquide, la teneur en eau...

A cause de sa complexité, le gonflement ne peut pas être étudié pratiquement en tenant compte de tous les facteurs qui interviennent dans son mécanisme.

Dans le but de prédire le caractère gonflant d'un sol des relations empiriques sont généralement utilisées.

La portée de ces études prévisionnelles est fatalement limitée. Ces relations expriment le gonflement sur la base de quelques caractéristiques du sol, simples à déterminer en laboratoire, en maintenant tous les autres facteurs constants.

En particulier ces méthodes de prediction du gonflement ne tiennent pas compte de la composition ionique du fluide hydratant.

Nous avons tenté, dans cette étude de mettre en évidence l'influence qu'exerce la nature et la concentration ionique du fluide hydratant sur les paramètres de gonflement de deux argiles pures.

Nous avons recherché si les relations empiriques largement utilisées en pratique pourraient être appliquées pour prédire le caractère gonflant de ces argiles quand elles sont hydratées par diverses solutions electrolytiques.

A la lumière des résultats trouvés nous discutons l'erreur introduite par les procedures coutumières de laboratoire qui tendent à mesurer les paramètres de
gonflement des sols en les hydratant sans discernement avec de l’eau distillée ou déminéralisée.

MATERIAUX UTILISES

Les argiles utilisées pour nos expérimentations sont une kaolinite et une montmorillonite obtenues du commerce sous forme de poudre.

L’analyse granulométrique de ces matériaux a montré que le pourcentage d’éléments inférieurs à 2μ est de 47.3% pour la kaolinite et 66.95% pour la montmorillonite.

Le comportement au gonflement de ces deux matériaux a été étudié dans les hydratants par des solutions de chlorure de sodium, de calcium et de potassium.

Ces chlorures ont été choisis d’une part parce qu’ils sont prépondérants dans la nature (Halitum, 84) et d’autre part parce qu’ils ne donnent pas lieu à des réactions parasites en solution (Moore, 91).

Les solutions hydratantes utilisées ont été préparées sur la base de ces sels aux concentrations 0.25, 0.5, 0.75 et 1 mole/litre.

L’influence de la nature et de la concentration du fluide hydratant sur le gonflement des argiles utilisées est mise en évidence en étudiant la variation :

- des limites de consistance,
- des paramètres de gonflement libre mesurés à l’oedomètre.

LIMITES DE CONSISTANCE

La variation des limites de consistance de la kaolinite en fonction de la concentration en electrolytes est donnée dans la figure 1.

Pour les différentes solutions utilisées les valeurs des limites de liquidité et de plasticité de la kaolinite varient dans des marges très faibles : entre 34 et 40% et 20 et 26% respectivement.

En fait, les variations enregistrées sont de l’ordre de la précision généralement attachée à de telles mesures (10%). Il est donc difficile de dégager, sur la base de ces résultats, une loi de variation des limites de consistance de la kaolinite en fonction de la nature du fluide hydratant.

En ce qui concerne le matériau montmorillonite, les résultats consignés dans la figure 2 montrent que les limites de liquidité et de plasticité varient avec la nature et la concentration des sels utilisés.

Toutefois la plage des valeurs enregistrées pour la limite de liquidité est très étendue (240% 74%) comparée à celle obtenue pour la limite de plasticité (35% 54%).

Fig. 1. Influence du fluide hydratant sur les limites de consistance

La variation de la limite de liquidité en fonction de la concentration est identique pour les cations Na⁺ et K⁺.

A faible concentration, les valeurs de la limite de liquidité enregistrées avec le chlorure de calcium sont considérablement plus faibles que celles obtenues avec les autres chlorures. Cette différence semble toutefois diminuer, jusqu’à peut être disparaître, avec la concentration.
La représentation de la limite de liquidité en fonction de la concentration de sels en coordonnées semi-logarithmiques donne une relation linéaire.

Une relation similaire a été présentée par Ilis (79) et Didier (72) entre l’indice de retrait et la concentration en electrolytes.

Nous avons tenté de relier la variation des limites de consistance en fonction du fluide hyd rant au développement de la double couche ionique responsable du phénomène de gonflement.

Les résultats donnés dans la figure 3 montrent que les valeurs de la limite de liquidité et de l’indice de plasticité sont inversement proportionnelles à la racine carrée de la concentration et à la valence.

La limite de liquidité et l’indice de plasticité suivent une loi analogue à celle donnant l’épaisseur moyenne de la double couche diffuse. Des études fondamentales (Caillere et al., 88) ont montré que l’étendue moyenne de la double couche s’écrit :

\[
\lambda = \frac{z F}{C}
\]

(1)

dans laquelle \(z\) est la valence du cation présent; \(C\) est la concentration en cations et \(\alpha\) est une constante.

![Fig.3. Loi de variation de \(W_l\) et \(I_p\) en fonction du fluide hydratant (Mont.)](image)

Ce résultat confirme que les limites de consistance sont directement fonction des paramètres physico-chimiques qui gouvernent le développement de la double couche ionique.

**PARAMÈTRES DE GONFLEMENT A L’œDOMETRE**

La mesure des paramètres de gonflement a été effectuée à l’œdometre sur des échantillons préparés à partir des deux matériaux (avec de l’eau distillée) aux densités (d) et teneurs en eau (w) suivantes :

- **Kaolinite** (d=1.86, w=13%)
- **Montmorillonite** (d=1.38, w=32%).

Chaque échantillon ainsi préparé est mis dans l’œdometre et est saturé avec la solution testée. Il est laissé à gonfler jusqu’à stabilisation puis est chargé progressivement jusqu’à le faire revenir à l’indice des vides de départ. Pour chaque palier la charge est maintenue jusqu’à stabilisation des tassements.

Les relations \(\Delta H/II - \log(P)\) obtenues ont la forme donnée dans la figure 4 et ne sont pas linéaires (Phillipponat, 91).

![Fig.4. Influence du fluide hydratant (1mole/l) sur le gonflement (Mont.).](image)

**Gonflement de la Kaolinite**

Les résultats enregistrés pour la kaolinite indiquent que la nature du fluide hydratant a très peu d’influence sur les paramètres de gonflement mesurés. Pour le potentiel de gonflement les points se regroupent en majorité dans une bande comprise entre 3 et 4% (fig.5). Pour les pressions de gonflement elles se situent toutes dans une plage allant de 1 à 2 bars (fig.6).
Gonflement de la Montmorillonite

Faisons remarquer tout d'abord que dans le cas de la montmorillonite, le temps de gonflement diminue sensiblement avec la concentration en electrolytes.

En prenant comme référence le gonflement de la montmorillonite hydratée avec de l'eau distillée (G #1.5) nous remarquons que le gonflement de ce matériau varie considérablement avec la nature du fluide hydratant (fig.7).

Une concentration en cations de 0.25 mole/l est suffisante pour faire chuter le potentiel de gonflement libre de la montmorillonite de 50% pour le KCl, de 78% pour le NaCl, et de 87% pour le CaCl2.

Pour la même concentration, les pressions de gonflement enregistrées ne semblent pas suivre la même échelle de variation, seulement 8% de variation pour le KCl, 16% pour le NaCl, et 20% pour le CaCl2 (fig.8).

![Diagramme de Gonflement](image)

**Fig. 5. Influence du fluide hydratant sur le potentiel de gonflement (Kao.)**

Pour les concentrations au delà de 0.25 mole/l, les variations du potentiel de gonflement sont nettement moins importantes. Ceci s'explique par la relation logarithmique entre le potentiel de gonflement et la concentration en electrolytes donnée dans la figure 9.

Pour tous les cations utilisés nous trouvons des relations de la forme :

\[ G = A \log(C_s) + B \]  (2)

dans laquelle A et B sont des paramètres qui sont fonction de la nature du cation.

Le potentiel de gonflement, tout comme les limites de consistance, suit une loi analogue à celle donnant l'étendue moyenne de la couche diffuse. Cette loi de variation du potentiel est représentée dans la figure 7.

Néanmoins, dans le cadre restreint des essais réalisés, il semble que le potentiel de gonflement dépend non seulement de la concentration en electrolytes et de la valence du cation présent mais aussi de caractéristiques propres au cation (dimension et polarisabilité).

La théorie de la double couche diffuse exprimée dans la formule de Gouy-Chapman ne tient malheureusement pas compte de ces particularités, qui sont, comme en témoignent les résultats, importantes.

De plus, le résultats obtenus montrent que le gonflement n'est pas en rapport avec la polarisabilité des cations. Les valeurs de gonflement enregistrées sont dans l'ordre G(K) > G(Na) > G(Ca). Ceci n'est pas en accord avec l'ordre de polarisabilité croissante des cations (K > Na) sachant que l'énergie de liaison de Van Der Waals augmente avec la polarisabilité.

L'explication pourrait être que dans les sols non remaniés l'énergie de liaison est principalement due aux forces attractives de Van Der Waals entre les cations adsorbés.

![Diagramme de Gonflement](image)

**Fig. 6. Influence du fluide hydratant sur la pression de gonflement (Kao.)**
Pour toutes les solutions testées, les pressions de gonflement ne varient sensiblement qu'à une concentration en électrolytes de 1 mole/l. Pour les concentrations utilisées, les pressions de gonflement enregistrées sont reliées à la concentration en électrolytes suivant des expressions polynomiales de degré 3 (fig. 8). Les coefficients du polynôme varient avec la nature du cation présent.

Une relation similaire a été développée par Karalis (1989) pour la variation de la pression osmotique. Ce qui autorise a supposer que la pression de gonflement est en grande partie le résultat d'un mécanisme osmotique.

DISCUSSION DES METHODES DE PREVISION DU GONFLEMENT

La littérature compte un nombre considérable de méthodes de prédiction du potentiel de gonflement des sols toutes basées sur des paramètres faciles à déterminer en laboratoire. Snethen (84) et Cher (88) ont montré que parmi ces méthodes, celles basées sur les limites de consistance sont les plus représentatives.

Le potentiel de gonflement (G) a été corrélé à l'indice de retrait (Ir) suivant l'expression :

\[ G = f_i \cdot Ir^p \]  

(3)

dans laquelle \( p = 1.17 \) et \( f_i = 1/6.3 \).

Nous montrerons dans la figure 10 que le potentiel de gonflement peut aussi se mettre...
sous la forme :

\[ G = \beta \cdot W_l^p \]  

(4)

dans laquelle les paramètres \( \beta \) et \( p \) ne sont pas constants mais dépendent de la nature des cations présents en solution (Tableau 1.).

Sur la figure 10 nous avons aussi représenté la relation de prediction (3) en utilisant l'expression \((Ir = 1.1W_l - 30)\) proposée par Ilitis (79) et Didier (72).

### Tableau 1. Les paramètres \( \beta \) et \( p \) dans la relation (4)

<table>
<thead>
<tr>
<th>Cation</th>
<th>( \beta )</th>
<th>( p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>K(^+)</td>
<td>0.97</td>
<td>0.875</td>
</tr>
<tr>
<td>Na(^+)</td>
<td>0.126</td>
<td>0.585</td>
</tr>
<tr>
<td>Ca(^{2+})</td>
<td>9.06 (10^{-11})</td>
<td>4.77</td>
</tr>
</tbody>
</table>

La relation (3) fournit des potentiels de gonflement généralement inférieurs à ceux mesurés.

\[ K = 3.6 \cdot 10^{-5} \]

Nous montrons dans la figure 11 que les résultats trouvés dans cette étude peuvent se mettre sous la forme :

\[ G = 60 \, K \cdot 1p^0 \]  

(6)

avec \( K \) et \( p \) étant des paramètres variant avec la nature du cation présent dans le fluide hydratant (Tableau 2).

La relation (5) produit des valeurs du potentiel de gonflement considérablement plus importantes que celles mesurées.

Le fait d'étudier le gonflement d'un sol en laboratoire avec de l'eau distillée, que ce soit pour la détermination des limites de consistance à utiliser avec les méthodes de prédiction, ou pour la mesure directe des paramètres de gonflement, conduit à surestimer ou sous-estimer le potentiel de gonflement.

### Tableau 2. Les paramètres \( \beta \) et \( p \) dans la relation (5)

<table>
<thead>
<tr>
<th>Cation</th>
<th>( K )</th>
<th>( p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>K(^+)</td>
<td>2.5 (10^{-3})</td>
<td>0.326</td>
</tr>
<tr>
<td>Na(^+)</td>
<td>3.83 (10^{-4})</td>
<td>0.585</td>
</tr>
<tr>
<td>Ca(^{2+})</td>
<td>7.19 (10^{-4})</td>
<td>1.42</td>
</tr>
</tbody>
</table>

Fig. 10. Relation entre le potentiel de gonflement et la limite de liquidité.

Une autre relation de prédiction a été proposée par Seed et al. (62) donnant le potentiel de gonflement en fonction de l'indice de plasticité :

\[ G = 60 \, K \cdot 1p^{2.44} \]  

(5)

Fig. 11. Relation entre le potentiel de gonflement et l'indice de plasticité.
De plus, les eaux qui viennent en contact avec le sol contiennent toujours des proportions variables de cations, qui agissent comme floculents (réduisant le gonflement) et d'anions, qui agissent comme dispersants (augmentant le gonflement).

Par conséquent, l'étude du gonflement d'un sol en laboratoire doit nécessairement tenir compte de la composition chimique de l'aquifère avec lequel le sol est (ou peut être) en contact.

CONCLUSION

L'étude a permis de montrer que :
1- le gonflement de la kaolinite ne paraît pas être influencé par la nature ionique du fluide hydratant.
2- le gonflement de la montmorillonite varie considérablement avec la nature et la concentration ionique du fluide hydratant.

Cette étude a aussi confirmé la validité de la théorie de la double couche ionique dans l'explication du phénomène de gonflement.

Les résultats trouvés ont révélé que les limites de consistance et le potentiel de gonflement libre de la montmorillonite varient, pour toutes les solutions testées, suivant une loi analogue à celle donnant l'étendue de la double couche diffuse.

Pour prédire le potentiel de gonflement d'un sol contenant une proportion de montmorillonite, il est nécessaire de tenir compte de la composition chimique de l'aquifère avec lequel le sol peut être mis en contact (eau de pluie, eau de mer, eau d'infiltration ...).

Des paramètres de gonflement plus représentatifs pourront être obtenus en étudiant le comportement du sol en contact avec un fluide puisé sur le site d'ou provient l'échantillon testé.

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Studies on the dynamic effects of foundation soil of a special dynamic foundation engineering project

Études des effets dynamiques sur le sol de fondation d’un laboratoire d’études dynamiques

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ABSTRACT: A dynamic laboratory is a large special dynamic foundation engineering project. During its testing period, the reciprocating rolling load—adding system worked 20 hours a day and rolled reciprocally 150 times an hour. The dynamic load reached 2400kN, and the maximum acceleration result from it was up to 1. 0g (g is the gravitational acceleration), the frequency was 30Hz. So comprehensive explorations and tests were carried out. Studies and discussions in five aspects on the dynamic effects foundation soil were done on the basis of a large quantity of testing data.

RÉSUMÉ: Un laboratoire de dynamique est un grand projet de génie de fondation dynamique spécial. Pendant son période d’expérience, le système roulant alternatif d’addition de charge a travaillé 30 heures par jour et a roulé alternativement 150 fois par heure. La charge dynamique a atteint 2400 kN, et la accélération maximale résultant de cette charge a allée jusqu’à 1,0 g (g est l’accélération gravitationnel), la fréquence a été de 30 Hz. D’accord ça ont été effectués inclusivement essais et investigations. Des études et des discussions relatives à cinq aspects sur le effets dynamiques du sol de fondations ont été effectués ayant pour base une grande quantité des résultats des essais.

1 GENERAL DESCRIPTION OF THE ENGINEERING PROJECT

A dynamic laboratory is a large special dynamic foundation engineering project. The laboratory is constructed by a large full size track testing canopy, a track component testing hall, a track condition testing hall and some corresponding penthouses. In the testing canopy, a full size track testing trough is set up which is 60m long and 8m wide. At both sides of the testing trough, a longitudinal booster and a rolling load—adding system of sloping track which is 210m long are equipped. During the test period, it was operated for 20 hours each day and rolled reciprocally 150 times each hour. The loading capacity was 2400kN and the generated maximum acceleration could be up to 1. 0g. The vibration frequency could be 30 Hz. In accordance with per rolling to pass 12 shafts and per shaft 10 shock waves, the loading times of dynamic load were 360 thousand.

The site of this project is located at the front of alluvial fan. Foundation soil is a cohesive soil of the quaternary period silt layers. The ground water table is about 1m under the surface. The engineering geologic condition of the site is poor and the location of the site belongs to the 8 degree earthquake area according to China Earthquake Intensity Regulation Map (1977). From the point of engineering geology, such area could not be used as the site of a special engineering project. But because the limitation of the specially designated condition, the site of the project can not be chosen. For this reason, a lot of methods are adopted to synthesize the exploration and testing and on this basis to carry out the studies and discussions on the dynamic effects of the foundation soil to ensure the design of the foundation safety and reasonable.

2 THE CHARACTERISTICS OF THE ENGINEERING GEOLOGY AND HYDROLOGIC GEOLOGY OF THE SITE

During the exploration, the maximum exposing depth is 37. 2m. All of them are molocene series alluvial layers of the quaternary period. Within the depth, the site soil could be divided into 12 layers. Through the sedimentary
rhythm, you can see that both the lithology and level are more stable starting from the sixth layer. The buried depth of the top surface of this layer is between 12: 0 — 15: 1m and sloping slowly from the north to the south. Therefore, the site soil under the depth of 12 — 15m are relatively stable soil strata, giving first place to the cohesive soil and containing the layers of fine sand and medium sand. The endurance of the cohesive soil is 170 — 190kPa. All the compression parameters are larger than 0. 1MPa−1, belonging to medium compressibility. The value of natural consolidated quick shear Φ is between 27° — 30°. The value C is between 19— 49kPa. The value Φ getting from consolidated nondrainage share is between 30. 5°—30. 8°. Therefore, as see by deformation and strength, used as the load bearing stream of a deep foundation is suitable and there is no weak layers contained among them.

The 1 — 5 layers of the soil are located above the depth of 12—15m. Among them, the first layer is impure backfilled soil which is 0. 3—1. 3m thick and could not be used as foundation soil. The layers from the second to the fifth are natural strata. From the point of view of its sedimentary rhythm, they reflect some typical characteristics of the prodleuvium front. No matter the horizontal and the vertical direction, the lithologic change is large, in which is the lens of mainly silty clay and silt loam containing silt and clay. Its ground endurance is 140—150kPa. The compressibility belongs to medium compressive soil. The value Φ getting from the natural consolidated quick shear is between 22° — 28°, and the value C is between 19 — 42kPa. The value Φ getting from the consolidated nondrainage share is between 23. 6°—37° and the value C is between 10—20kPa.

The ground water of the site is mainly pore ground water and partial suspended water. The suspended water is buried more shallow, distributed in the upper silty clay and silty loam layers, in which the water permeability is weak. The buried depth of the water table is 0. 5—1. 5m which is formed by the infiltration of surface water and directly supplied by atmospheric precipitation and open water. Its dynamic state is very unstable, significantly affected by seasons. The changingamplitude of the water is 1—2m, under which is the pore ground water of sand layer buried by 4—6m. The thickness of the water bearing stratum is normally about 2m. The suspended water and pore ground water has vertical connection.

The chemical characteristic of the ground water has weak alkaline reaction. Its PH value is between 7. 2—7. 9 and its hardness(Ca^{2+}+Mg^{2+}) is over 9. 0me/L. So it belongs to extreme water. After the analysis of the quality of water, it is decided that the ground water of the site has no erosiveness to concrete. The hydrogeologic test is excuted at the site. The main opening is used to draw water and two observation ports are used to observe the water drainage and the change of the water table during the water table restoring. After the test, the composition osmotic coefficient of the silty loam and silt layers between 2m—8m is K = 0. 76 m/d.

3 DYNAMIC TEST AND DYNAMIC EFFECT

In order to get the dynamic parameters needed by design and the rule of the propagation and attenuation, we have carried out a systematic field test and a large scale simulation test and appreciated the dynamic effect of foundation soil accordingly.

3. 1 The predominant frequency of earth pulsation of the site

According to the designing location of the construction at the site, 12 test points are arranged evenly and the measuring of the earth pulsation are executed. The test data are listed in the appendix 1. As see by the test result, the reflection of the curve is good. Its peak value is clear and the predominant frequency is 3Hz. In order to prevent the resonance action, the ratio Q/W between the drafted shock frequency Q of various shock test and the predominant frequency must not fall in the resonance area. The calculation of the range of resonance area is as the following formula.

The front area of the resonance,

\[(1 - ε)W = (1 - 0. 35) \times 3 = 1. 95 \text{ Hz} \quad (1)\]

The back area of the resonance,

\[(1 + ε)W = (1 + 0. 35) \times 3 = 4. 05 \text{ Hz} \quad (2)\]

Within the formula,

\[ε = \text{safty coefficient}\]

\[W = \text{site predominant frequency}\]

Therefore, when designing this structure, the possibility of preremove the resonance should be taken into consideration, i.e. \(ε \leq \frac{Q}{W} \leq 1 + ε\). In order to satisfy this condition, the shock frequency of the shock test can be changed or change the natural frequency(e. g. choosing the structure form and the dimensions of the component reasonably).

3. 2 Judging the type of site soil of the seismic zone

25m elastic wave velocity of 2 holes test should be executed. Test points are 50, getting the shear wave velocity Vs , compressional waves velocity Vp , dynamic elasticity modulus Ed and dynamic shear modulus Gd etc. parameters of each soil layer. For the result, see appendix I. According to the test result of elastic wave velocity, within the
limitation of 28m under the surface, the average share wave velocity V's is 219—226m/s. According to the standard of the earthquake resistant code for construction design (GBJ 11—89), this site soil belongs to medium soft site soil. The measured parameters of d and E etc. will be used in the following dynamic calculation.

3. 3 On the established railway and under the simulation test condition, a large scale actual measurement of the site is carried out.

Under the moving condition of the train form 80km/h to 140km/h, the value of produced shock velocity, amplitude, frequency and the attenuation rule of the shock parameters in the vertical and horizontal direction are measured during the shaft weight and the speed are different, so as to appreciate the produced effect of the shock on the site soil and construction during the test. For the measured data and attenuation rule, see appendix II.

3. 4 The samples of the silty loam layers and soil layers which are under the site foundation and possible to be liquefied are tested by triaxial vibration test, so as to appreciate whether the produce-vibrating force during the test could cause the foundation soil to be liquefied.

In order to imitate the reality of the site, the consolidated stress should adopt 50kPa and 100kPa. Since the limitation of the equipment condition, the dynamic stress can not be exerted according to the actual shock frequency 30Hz. But in order to observe the effect of the frequency changes on the times of shock, two kinds frequency 1Hz and 4Hz are adopted for the test. For the test result, see appendix IV. From the curve of the drawreatio of the dynamic stress (\( \sigma_d \)) to the vibrating times (\( N \)), you can find out three points as follows:

1. The anti—liquefaction ability of the silt is less than the siltyloam.
2. When the consolidated stress of the two kinds of soil layers are 50kPa and 100kPa, their anti—liquefaction abilities basically have no difference.
3. Under the action of the dynamic stress, the soil liquefaction not only has the relation with the shock frequency, but also has close relation with the times of shock.

4 DYNAMIC CALCULATION

4. 1 Dynamic parameters and load character under the different test conditions

Beside bearing the dead loads of constructions and equipments, the foundation soil of the site also bears the actions of the three kinds of dynamic force under different conditions:

1. When a train is passing by at the speed of 100—140km/h, the produced velocity is 1.0g, frequency is 30Hz.
2. Using two load — carrying freight wagons with shaft weight 210kN to carry out reciprocating rolling test, the main frequency of that is 5 — 20Hz and the speed per hour is ≤ 50km/h.
3. On the full size track testing trough, a hydraulic loader is used to load. Its amplitude is ±4mm, frequency is 20Hz and load is ±200kN.

As for the nature of the load, all of them can be regarded as resonance waves when carrying out the dynamic calculation. Since much actual test information shows that when the train is passing, the produced vibration strength changes with the time and transferred as a form of elastic wave and has a periodic change regularity, so it is possible to regard the vibration of the train as resonance waves. As for the vibration load produced by hydraulic loader, it is in itself loaded in accordance with the condition of harmonic wave, because it is controlled manually.

4. 2 Compression strength coefficient of the soil

1. According to design code GBJ 40—79 for power machine, the value of the compression strength coefficient \( C_z \) can be decided in accordance with the permissible bearing capacities of foundation soils as follows, (Table 1)

<table>
<thead>
<tr>
<th>Strata order number</th>
<th>Description</th>
<th>Permissible bearing capacity (kPa)</th>
<th>Compression strength coefficient ( C_z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Silty clay</td>
<td>140</td>
<td>264000</td>
</tr>
<tr>
<td>2</td>
<td>Silty loam</td>
<td>190</td>
<td>276000</td>
</tr>
<tr>
<td>3</td>
<td>Silt</td>
<td>230</td>
<td>332000</td>
</tr>
<tr>
<td>4</td>
<td>Silty loam</td>
<td>190</td>
<td>324000</td>
</tr>
<tr>
<td>5</td>
<td>Silty clay</td>
<td>160</td>
<td>238000</td>
</tr>
</tbody>
</table>

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According to the specification, the calculation of the composite compression strength coefficient should use the following formula:

$$C_d = \frac{2/3}{\sum \frac{1}{C} \left( \frac{1}{1 + \frac{h_d}{d}} \right)}$$

(3)

According to this, for the foundation of the transitional beam, we get $C_d = 29860 \text{ kN/m}^3$.

2. The above calculation is an empirical method. In order to check its reliability, we are going to compare it with the theoretical formula. The formula is as follows:

$$C_d = 2.256 \frac{Od}{1 - \mu} \cdot \sqrt{F}$$

(4)

Among it:

- $Od = \text{dynamic share modulus of soil}$
- $F = \text{the bottom of foundation area m}^2$.

Since the foundation is formed by many layers of soil, firstly, the average dynamic share modulus $Od = 49$. 84MPa should be got according to the result of physical exploration. And then the Poisson's ratio $\mu = 0.5$ should be got according to the physical exploration. To substitute the results into the above formula, you can see that it is very similar with the empirical method. So during calculation, the average value of the both 30340kN/m$^3$ should be used.

4.3 Dynamic calculation of the foundations of transitional beam and slope railway

The disturbing force amplitude should be valued backward in accordance with the amplitude formula of single-degree-of-freedom steady state vibration. According to the observed data, the supplied maximum acceleration of the roadbed surface is 1.0g. If the frequency is in accordance with 30Hz, the amplitude is 0.28mm. Substitute the above into the following formula and get out the value of disturbing force amplitude.

$$P_x = AK_s \sqrt{(1 - \frac{a^2}{\lambda_e^2})^2 + (2D_s)^2 \left( \frac{2S}{\lambda_e} \right)^2}$$

(5)

Among it:

- Amplitude $A = 0.00028 \text{ m}$
- Compression strength $K_s = 3034 \text{ kN/m}^3 \times 50 \text{ m}^2 = 151700 \text{ kN/m}^3$

The frequency of the disturbing force circle, $\omega = 2\pi f = 188.5 \text{ Hz}$.

The frequency of self vibration circle,

$$\omega = \sqrt{\frac{K_s}{m}} = \sqrt{\frac{151700}{12.5 \times 4 \times 8/9.81}} = 61$$

$m = \text{the mass of a foundation, as for damp ratio } D = 0.15 \text{ is adopted.}$

To be substituted, $\omega = 0.00028 \times 151700 \times \sqrt{(1 - 9.55)^2 + 0.69 \times 9.55} = 365.299 \text{ Hz}$

$n_x = 355.299 \text{ Hz/m}^2 = 73.1 \text{ kPa}$

The distance between the bottom of the foundation and the surface of silty loam layer is 2m. According to the empirical relation formula getting from the observed data, the dynamic stress of the roadbed changes with the depth, i.e.,

$$n_x = \frac{2.71a_x}{25 \text{ HS}}$$

(6)

and

$$n_x = 1.49 \frac{a_x}{25 \text{ HS}}$$

(7)

Among which $a_x$ is the dynamic stress of the depth $x$. The exciting acceleration and the frequency produced by the slope track rolling test are in accordance with 1.0g and 30Hz respectively, and the corresponding amplitude is 0.28m. The calculation of the dynamic stress of silty loam top surface is 18.9kPa and 16.6kPa. The average value is 17.8kPa. The ratio of dynamic stress is 0.27. In accordance with the triaxial vibration test of this column, under the action of this dynamic stress, the phenomenon of shock liquefaction will appear when the times of vibration arrives at 60 thousand times.

From the calculation of the dynamic stress on the basis of the actual measured data of the running trains, when the speed of the trains arrives at 100--140km/h, the vertical acceleration will be 0.9g(see Appendix II), The existing frequency is in accordance with 30Hz, then the corresponding displacement is 0.25mm. According to this, the average dynamic stress at the top surface of the silty loam layer is 16kPa, the dynamic stress ratio is 0.24. According to the results of the triaxial experiment of this soil layer, under the action of the dynamic stress, the liquefaction phenomenon will appear when the times of vibration arrive at 220 thousands.

If the maximum acceleration of the bottom of the
roadbed is less than 0.1g, the dynamic stress ratio of silty loam surface will be 0.027. Under this condition, the shock liquefaction will not appear even the times of vibration arrive at 360 thousand. Therefore, measures should be taken to ensure that the maximum acceleration is less than 1.0g when trains passing by the entire roadbed at permissible test speed and when reciprocating rolling load—adding test is carried out.

4.4 Hydraulic loading shock test

The parameters of the shock loaded by hydraulic vibrator are:

1) Amplitude $\pm 4\text{mm}$, $20\text{Hz}$, $\pm 200\text{kN}$
2) Amplitude $\pm 1\text{mm}$, $50\text{Hz}$, $\pm 100\text{kN}$

For the first condition, $f = 20\text{Hz}$, $\omega = 126$, the acceleration on the rail $a = \frac{a^2A}{\omega^2} = 63.5\text{m/s}^2 = 6.47g$. The value is lower than the measured acceleration 13.82g which is measured in Zhengzhou City by the Third Railway Exploration and Design Institute and the Northern Jiaotong University when the speed of the train is 25km/h. Under the above mentioned condition, if divided by the acceleration ratio 6.8 of the observed rails and the sleepers, the acceleration of the sleepers are only 0.95g. On the basis of the attenuation formula, the calculated acceleration at the depth of 3m is 0.025g. For the second condition, the getting acceleration of the rail is 10.05g. The acceleration of the sleepers are 1.438g. The acceleration at the depth of 3m is 0.04g. As for the amplitude, under the first condition, it is attenuated to 0.106m at the depth of 3m and under the second condition, attenuated to 0.0255m.

As for the exciting force, the action scope on the surface of roadbed is 2.5m$^2$. The pressure of the unit area is $20\text{t/m}^2 = 200\text{kPa}$. The dynamic stress under 3m is 15kPa. So it is not enough to cause the liquefaction of the foundation soil.

5 THE EFFECT OF THE SHOCK ON THE FOUNDATION

After the laboratory is completed, different types of shock will be produced during the dynamic test. And the shock will produce different effect on the foundations of the laboratory which are different types and located at different positions.

5.1 As for the foundation soil of the transition beam, if the maximum acceleration of the observed integral roadbed surface is in accordance with 1.0g, it is possible to cause shock liquefaction, just as showing in the calculation before.

If some measures are taken to make the maximum acceleration of the bottom layer of the roadbed is less than 0.1g, the dynamic stress of the silty loam top surface is only 1.8kPa and ratio of the dynamic stress is 0.027. Under this condition, the shock liquefaction will not appear at the foundation soil of the transition beam.

5.2 The effect of shock on the foundation of the wind resistance pillars

If the maximum acceleration at the transition beam is in accordance with 0.1g, according to the observed attenuation curve, the horizontal attenuation coefficient $n$ has relation to the horizontal distance, but it is not a constant. As for the foundation of wind resistance pillar, the calculated vibrating acceleration is 0.059g and the amplitude is 0.0167mm. Foundations located in a place of such kind of vibration for a long time is harmful. It is suggested to take some vibration isolation measures to ensure the normal usage of the construction.

5.3 The effect of shock on the foundations of the testing canopy

1. The eastern side foundation of the canopy pillar which is near the large loop railway is affected by the effect of the shock actions which are both from the reciprocating rolling test and the high speed running test of the large loop of railway. If the maximum acceleration of the shock of rolling test is in accordance with 0.1g, the calculated acceleration transferred from the transition beam to the pillar foundation will be 0.016g and the amplitude will be 0.0045mm. So the effect of the shock on the pillar foundation is weak. The maximum acceleration of the shock generated by the high speed running test on the large loop railway is 1.0g. The measured value of the acceleration transferred to the pillar foundation is 0.54 – 0.05g, having harmful effect on the pillar foundation. It is suggested to take some necessary vibration isolation measures.

2. Since the western side foundation of the canopy pillar is far from the large loop railway and the acceleration and amplitude transferred from the transition beam to the pillar foundation is very small, so the effect of the shock on the pillar foundation could be ignored.

5.4 Pedestrian foundations

Since the damping measures for the integral roadbed has adopted and making the maximum acceleration caused
by shocks to be less than 0.1g, the vibration of the penthouse foundation is reduced significantly, but due to the demand of equipment itself, it is suggested that an integral foundation of raft type should be used.

6 THE ADDITIONAL SUBSIDENCE AMOUNT UNDER THE EFFECT OF THE SHOCK

According to the identification of the site exploration and the determination of soil physical index, the silty loam layer under the foundation is evidently thixotropic, but during the prospecting, the silt layer under the foundation has an upwelling condition. So it would be taken into account that when these two layers of the soil receive forced vibration (shock), additional subsidence will be engendered.

The subsidence amount is the function of the times and the time of dynamic actions and at meantime, the larger the amplitude is, the more subsidence will be. The test shows that if the loading times of the shock are more than 3000, the relative subsidence amount of the natural foundation can arrive at 50mm. In addition, as for the concept of stable oscillation density of the silt layer, this silt layer has received 26 standard penetration tests during the prospecting. The average stroke number \(N_{25} = 21.9\) belongs to medium dense degree. Its relative density \(D_r = 0.54\) and bulk specific gravity \(\gamma_s = 1.48\)g/cm\(^3\). The index of the grain analysis is very close to the value sand grain numbers of "B" group used in the study of "Immersed silt Embankment" by Mr. Yang Canwen (1987). But when the vibration acceleration \(a = 0.2\)g, the stable oscillation density of the sand of "B" group is 1.46g/cm\(^3\). Its porosity is 0.47. After the laboratory is built up, the shock times is about ten thousand to hundreds of thousands. So it is estimated that under the action of so much repeated exciting force, this silt layer could be possible to arrive at the stable oscillation density and the subsidence amount could arrive at 30—50mm when it is in stable oscillation.

7 ABOUT THE EARTHQUAKE LIQUEFACTION AND THE COLLAPSE

This site is located at the earthquake area of 8 degree. The normal position test of the site shows that the liquefaction coefficient \(L < 3.2\) of the medium sand layer which is deep buried at 8.4—8.9m belongs to light liquefaction layer of the earthquake. As a form of thinning out, this layer is distributed between the silty loam and silty clay. The maximum thickness is 2.2m which should be paid more attention during design.

To judge whether there is a possibility of collapse should be in accordance with the standard of critical bearing capacity and the average value of the shear wave velocity. The average shear wave velocity of the silt layer within the range of the load bearing stratum of foundation is 110m/s, which is less than the critical value prescribed by 8 degree of earthquake resistance fortified intensity. So the collapse will take place in this silt layer when there is a 8 degree earthquake. The amount of collapse could be in accordance with the above subsidence amount of the stable oscillation density.

8 SUGGESTIONS ABOUT THE DYNAMIC REBATING COEFFICIENT OF THE FOUNDATION SOIL ALLOWABLE BEARING CAPACITY

All the dynamic strength of the saturated cohesive soil is lower than its static strength. The main reason is that under the action of the dynamic load, the pressure of the pore water increases and after the structure of the soil is disturbed, the structural strength between the soil grains is impaired. Under the action of the dynamic load, the reduced amount of the strength changes with the size of the dynamic load and the times of circulating action. According to the contrast test between the static and dynamic strength which simulates the 8 degree earthquake and is carried out on the like soil, and when the failure strain is 10%, consolidated pressure is 100kPa and the vibration times are 30, the dynamic strength will be lower by 20% than static strength. That is to say if in accordance with a 8 degree earthquake, the suggested dynamic rebating coefficient is 0.8. But beside belonging to a 8 degree earthquake, the effect of shock action should also be of consideration. Under the action of shock, the dynamic strength of the soil layer can gradually arrive at its "fatigue limit value" with the increase of vibration. As see by the ratio of dynamic and static strength getting from the simulation test on the like soil and the diagram of dependence curve of vibration times, after the times of vibration are over 200, the curve tends to horizontal. If the times of vibration are even higher, the curve tends to the asymptote. Its value is 0.7 times of the static strength and is called "fatigue limit". Consequently, since the rolling times of the part of sloping track and track testing trough could be 360 thousand, the suggested dynamic rebating coefficient is 0.7. As for the canopy pillar foundation of the large testing canopy, only the effect of the 8 degree earthquake is of consideration and suggest to adopt 0.8 for its dynamic rebating coefficient.

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The auto—spectrum of Fourier of the testing point (Y direction)

Appendix I

400

Acceleration

40

Velocity

1

Displacement

451
### Appendix I

**Testing of wave velocity in the well**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Columnar map</th>
<th>Lithologic character</th>
<th>$V_s$ (m/s)</th>
<th>$V_p$ (m/s)</th>
<th>$V_s$ curve</th>
<th>$V_p$ curve</th>
<th>$a$ (B/et)</th>
<th>$H_a$ (MPa)</th>
<th>$G_d$ (MPa)</th>
<th>$E_d$ (MPa)</th>
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<tbody>
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<td>Fill clay</td>
<td></td>
<td></td>
<td>100 200 300</td>
<td>500 1000 1500</td>
<td>2.01</td>
<td>0.50</td>
<td>41.1</td>
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<td>5.5</td>
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<td>6.9</td>
<td>S_sil</td>
<td>Silt</td>
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<td>9.0</td>
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<td>S_m</td>
<td>Medium sand containing fine sand</td>
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Appendix III

The peak values of the acceleration of the test points in the wells (horizontal direction) (gal)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Peak values of the acceleration</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Well 1</td>
</tr>
<tr>
<td>0.0</td>
<td>411.48</td>
</tr>
<tr>
<td>1.0</td>
<td>209.1</td>
</tr>
<tr>
<td>2.2</td>
<td>159.85</td>
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<td>3.5</td>
<td>147.56</td>
</tr>
</tbody>
</table>

The peak values of the acceleration of the test points in the wells (vertical direction) (gal)

<table>
<thead>
<tr>
<th>Depth</th>
<th>Peak values of the acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Well 1</td>
</tr>
<tr>
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<td>102.79</td>
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<tr>
<td>1.0</td>
<td>96.3</td>
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<tr>
<td>2.2</td>
<td>41.60</td>
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<td>3.5</td>
<td>24.36</td>
</tr>
</tbody>
</table>

The attenuation characteristic of well 1

The horizontal direction of well 1

The vertical direction of well 1
Appendix II

Acceleration attenuation curve
(Train velocity, 120—140km/h)

Appendix IV

Dynamic stress ratio vs. Vibrating times for different pressures and frequencies.

- Pressure: 50 kPa, 100 kPa
- Frequency: 1 Hz, 4 Hz

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Residual strength of soils in an unstable slope: Case of El-Achour slip
Résistance résiduelle des sols dans une pente instable: Le cas du glissement d’El-Achour

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ABSTRACT: A series of ring shear test and reversal shear box test were performed in order to investigate the residual characteristics of a clayey soil from the El-Achour unstable slope. The test results and findings are reported and discussed.
RESUMÉ: Des séries d’essais de cisaillement annulaire et de cisaillement alterné à la boîte cisaillement ont été faites dans le but de connaître les caractéristiques résiduelles d’un sol argileux provenant du talus instable d’El-Achour. Les essais et les résultats sont présentés et discutés.

1 INTRODUCTION
The residual strength of clayey soils is an important factor in assessing the stability of soil slopes. Residual strength has been defined by Skempton (1964) as the minimum drained shear strength attained at large displacement for a soil at a slow rate of shearing. It is now widely accepted that the strength on a slip surface falls to a residual value after a slide has occurred. Therefore, whenever such pre-existing shear surfaces take place residual strength must be known as it will exert a controlling influence on engineering design. Little is known or published concerning the mechanical properties of unstable slopes in Algeria.

The purpose of the present paper is to present the results of an investigation into some of the mechanical properties of a clay from the El-Achour unstable slope. A series of tests using the bromhead ring shear and the reversal shear box are presented and discussed.

2 SOIL AND TESTING PROCEDURE
2.1 Soil
The material used in the present investigation is a clay from the el-achour area which is situated in the west of Algiers. This area has shown, in the past years, several slips. In 1990/1991, after heavy rains the slip has been reactivated. Long and large cracks have appeared in the main road and structural disorders have been recorded in several houses. Observations have shown that part of the slope has moved downhill, several

trees were about 4 to 5 m from their previous place. Samples were taken at two different depths (3m and 9m). The physical properties from standard laboratory tests are summarized in table 1.

| Table n°1 Physical properties of El- Achour clay |
|---------------|----------------|----------------|
|               | Boring n°1     | Boring n°2     |
| depth (m)     | 3-3.5          | 9.5-10         |
| w(%)          | 16             | 20             |
| w(%)          | 41             | 44             |
| IP(%)         | 19             | 23             |
| CF(%)         | 43             | 48             |

CF= Clay fraction

2.2 Testing procedure
The tests consisted of conducting ring shear tests (RS) and reversal shear box tests (RSB). Testing was restricted to moderate normal stresses which are relevant to slips in the el-achour area (50 kPa to 200 kPa). The Bromhead ring shear apparatus (Bromhead, 1979) has annular sample 100 mm outside diameter, 70 mm inside diameter and 5 mm thick. Rotation is imparted to the base plate and sample container by means of a motor and
variable speed gearbox. The top platen is held against a pair of sensitive proving rings which serve to measure the torque transmitted through the soil sample, from which the shear stress can be deduced. The platens are roughened porous bronze. Remoulded El-Achour clay was generally kneaded in the specimen container and levelled off. Shearing started after consolidation of the clay. Reversal tests were conducted on undisturbed clay samples and the procedure was similar to that of Skempton (1964). Each cycle consisted of shearing a sample consolidated under a given normal load at a rate of 180 μ/min over a shear displacement of about 8 mm, reversing the direction of shear and shearing the sample in this direction at the same rate of deformation. After the available travel in the reverse direction had been completed the direction of shear was reversed again in readiness for the next shearing cycle.

3 DISCUSSION AND CONCLUSION

Figure 1 shows the results of the ring shear tests and the reversal shear box tests. The results are plotted on a semi logarithmic displacement scale. This method of presenting the results has been suggested by La Gatta (1970). It can be seen that beyond about 90 mm displacement the curve is horizontal, indicating that the residual strength is established. However, in the case of the reversal tests, it has also been noticed that the direction of shearing affects the value of residual strength.

Skempton and Petley (1967) explained this phenomenon by the fact that the clay particles which were totally oriented in the sheared surface in one cycle, will be taken in the reverse direction in the following cycle, thus more effort is required.

The variation of the residual shearing stress with the effective normal stress is plotted in figure 2. It can be observed, for the range of normal stress studied, that the failure envelopes are curved and the cohesion intercept passes through the origin in all cases. This is in agreement with the findings of previous investigators (Boyce et al., 1988, Maksimovic, 1989). Of more significance perhaps is the fact that both tests yielded different results. For sample n°1, the residual angle of shearing resistance (φ°) was about 12.6° in the reversal shear box test and about 8° in the ring shear test. Sample n°2 gave approximately the same results, 12.4° (RSB) and 8.1° (RS). It is widely accepted that reversal tests yield higher residual strength than ring shear test. This discrepancy is due to several factors, amongst them is the lack of deformation and the disturbance of the shear surface (in the RSB).

![Figure 2: Residual strength envelope of El Achour Clay](image)

However, the present work shows that the difference could be due to another factor, which is the low clay fraction (CF) of the two samples. This is further confirmed by the work carried out by Ghili and Bouazza (1994) who showed that when clay fraction is > 70% the difference between the results obtained from reversal shear box and ring shear is small (1° to 2°). Although that ring shear tests give more conservative values of residual strength, it is in the authors opinion that it is more advantageous to use it because the test is quicker and more economic. The present work showed that for clay fraction of
about 45 to 50%, the residual angle of shearing resistance obtained from the RSB exceeds the values obtained from the ring shear test by about 4°. Finally, stability analysis has been conducted using the Bishop method. The introduction in the analysis of the residual value (φ') measured with the ring shear test gave a safety factor (Fs) of about 1.

4 ACKNOWLEDGMENTS

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REFERENCES

‘Argille Scagliese’ complex in Northern Italy: The geotechnical
characterization

Le complexe de ‘Argiles Scagliese’ dans l’Italie septentrionale: La caractérisation
géotechnique

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ABSTRACT: Outcrops of the "Argille Scagliese" complex, which from a
lithological point of view consists of extremely heterogeneous marl-
claystones with calcareous inclusions, are common in the Northern
Apennines. Three important road and rail tunnels are to be cut through
this complex in the near future.

Knowledge of the geotechnical features of these materials is very
important for the planning and design of the excavations, support and
lining structures. At present existing data concerning the "Argille
Scagliese" is scarce and at times contradictory.

The authors, beginning with a specific investigation for one of the
tunnels in question, have gathered together and processed all the data
already available in the literature.

The result is an overall picture of the geotechnical features of the
"Argille Scagliese". The statistical importance of this picture becomes
fundamental when the scattered nature of the data collected by the
authors is considered.

RESUME: Le complexe des argiles écaillueuses lithologiquement constitué
d’argilites marneuses fortement hétérogènes affleure largement dans les
Apennins nord. Dans un proche avenir il accueillera trois importants
tunnels autoroutiers et ferroviaires.

La connaissance des caractéristiques géotechniques de ces matériaux est
très importante pour la conception des excavations et des ouvrages de
soutènement. À l’heure actuelle, les données relatives aux argiles
écaillueuses sont éparses voire contradictoires.

Partant de l’enquête spécifique d’un des tunnels en question, les
auteurs ont collecté et traité toutes les données disponibles déjà
publiées.

Il s’en dégage un tableau général des caractéristiques géotechniques des
argiles écaillueuses dont la signification statistique acquiert de
l’importance face à la dispersion des données recueillies par les
auteurs.

1 INTRODUCTION

The "Argille Scagliese" complex, which
from a lithological point of view
consists of clays and shale, is very
widespread throughout the Apennine
Chain. Its importance, for an
understanding of the dynamic evolution
of the Apennines, has induced numerous
researchers to study the stratigraphic
and tectonic relations of the complex
with other units. Despite the fact that
the complex extends over a very wide and
important area as far as land
development is concerned (development of
major transport routes, landslide
prevention, urban development), present
knowledge of its geotechnical features
is scarce and at times insufficient. The
"Argille Scagliese" are important
because several transport construction
projects, currently planned, in Italy
pass through them.

The authors felt that it would be
extremely useful to collect, process and
interpret data and information available
from the numerous works that have been carried out, with particular reference to the specific works for the Bologna-Florence motorway. It is the purpose of this paper to summarise our present knowledge of the "Argille Scagliose".

2 SOURCE OF DATA

Outcrops of the "Argille Scagliose" in the Northern Apennines are considerable and widespread; the stratigraphic and tectonic features are complex and varied and consequently not easy to map.

Current Italian transport planning involves construction on three important Apennine routes (Fig. 1) (from East to West):

- a high speed Bologna-Florence railway line;
- the new Bologna-Florence motorway;
- the new Parma-La Spezia railway line.

In these works the crossing of the Apennines is affected by the construction of tunnels of considerable length (Fig. 2):
- "Raticosa" tunnel 10,400 m approx. length
- "Base" tunnel 8,700 m approx. length
- "Pontremolose" tunnel 21,000 m approx. length

Geological forecasts show that the "Argille Scagliose" will appear in these works for a total length of approximately 12 km which is equal to 25-30% of the overall length of the tunnels.

The presence of "Argille Scagliose" with high lithostatic overburdens (400-600 m max. approx.) means that it is important to study the behaviour of the rock mass during the works for excavation and construction of the temporary and final support structures.

In order to obtain a detailed picture of the "Argille Scagliose", data from surveys carried out for the construction of the "Base" tunnel was examined (Bologna-Florence motorway in the "Vidiciatico Clays" formation ("Argille Scagliose" Complex)). Data coming from other works involving the "Argille Scagliose" was also examined in order to obtain statistically valid data.

Fig. 1 - Geographical setting.

Fig. 2 - Geological sections of tunnels.
3 GEOLOGICAL SETTING

The Northern Apennines form a montaneous chain, the end points of which are given by the Piedmontese tertiary basin (NW) and the Lazio volcanoes (SE). Two main parallel zones with different paleogeographic and structural features can be identified from the evolutionary relationships (Fig. 3) (CNR, 1982):

- **External Range (Tuscany-Umbria-Marche Units):**
  this consists of a limestone series forming the overlay, partly autochthonous and partly separate, of an ancient palaeoautochthonous bed;

- **Internal Range (Internal and External Ligurid Units):**
  this consists of a completely allochthonous series formed in oceanic areas.

The "Argille Scaglione" belong to the completely allochthonous Internal Range and run across onto the external Tuscan-Umbrian group.

The Internal Range appears to be completely chaotic due to considerable dislocations undergone mainly in the Eocene period.

The dislocation undergone was favoured by a flowing plastic level and the clayey nature of the Ligurid Units which gave rise to enormous gravitational flows.

The gravitational plastic masses include disjointed strip formations, at times in ordered sedimentary layers of considerable extension.

The main features of the "Argille Scaglione" (ESU, 1977; AGI, 1979) are as follows:

- complexity of strata and tectonic relations with other formations and irregular area distribution;

- lithological complexity and heterogeneity;

- presence of olistostrome bodies and strip formations that maintain their original sedimentary configurations;

- complex structure with phenomena of stretching, lamination and short radius folding; frequent convolutions and the presence of prismatic and/or flat scales with translucent surfaces. Lithoids contained are lithologically varied but mainly calcareous showing a dense network of fractures filled with calcite.

4 INVESTIGATIONS AND SURVEY

The surveys carried out on the "Argille Scaglione" of the motorway "Base" tunnel (BO-FL Meto) are as follows:

- 4 No. geognostic drillings - continuous bore drilling with depth varying from 90 m to 270 m approx. obtaining 17 No. undisturbed and disturbed samples;

- 17 No. pressiometric tests in borehole (Menard type);

- laboratory testing:
  - physical and chemical properties;
  - mineralogical and petrographical properties;
  - mechanical properties.

5 CLASSIFICATION OF ARGILLE SCAGLIONE

Macroscopic analysis of the cores showed that the lithological nature of the material consisted mainly of a marl-claystone mass with a chaotic scale structure, greenish grey in colour with numerous intercalations of small blackish clayey levels. There were frequent occurrences of material of a polygenic nature contained (sandstones, limestones, clays) and also calcareous levels sometimes up to a few metres in thickness.

According to the classification system for "Structurally Complex Formations" (Fig. 4) (ESU, 1977; AGI, 1979), which is based on the heterogeneity of the mineral, lithological and structural characteristics of the rock, the material examined could be attributed to class B3. Group B comprises formations originally consisting of more or less regular alternations of lithological heterogeneous materials (in our case claystones and marls).

Type B3 in particular includes formations with a completely chaotic structure (chaotic clays), caused by intense and repeated tectonic stresses, and includes elements of a more or less stone-like nature (D'Elia et al., 1982). In this particular case, the material of a marl-claystone
nature shows a high level of structural instability which for various reasons (variation in the tensile state, weathering, soaking and drying), may lead to a short space of time to the breakdown of the diagenetic bonds and the formation of loose rocks.

From the granulometric point of view, laboratory determinations identified a material which, according to BS 5930/1981 (British Standard Code of Practice for Site Investigations), can be classified as Mudstone. The granulometric curves (Fig.5) are very scattered due to the abundant presence of stone-like bodies. If the fine fraction only is considered the material can be more correctly identified as siltstone (Hawkins and Pinches, 1992).

An interpretation of the granulometry results should take into consideration the objective difficulty of separating claystone fractions due to the intra-particle bonds of clayey minerals. The consistency of these is sometimes the same as that of a diagenetized material.

No significant variations were found in granulometric composition at different vertical levels (e.g. bore drilling SV3, Fig.6) and this confirms

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**Fig. 4 - Classification of the "Structurally Complex Formations".**

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**Fig. 5 - Grain-size distributions.**

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**Fig. 6 - Grain-size distribution ranges versus depth.**

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**Fig. 7 - Casagrande Chart.**

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the extreme chaotic nature of the mass under study. The presence of a greater percentage of clay and silt in the samples from the surface level (max. depth: 25 m) gives confirmation of better separation in the laboratory due to less lithification of the material. It is only possible to classify the clayey matrix of the "Argille Scaglione" complex since the presence of stone-like bodies can heavily distort a classic granulometric analysis of the weighted type.
The Atterberg limits made it possible to classify the material within the Casagrande plasticity diagram; it is on average an inorganic clay with medium plasticity (Fig. 7).

The material was classified according to the calcium carbonate content (Fig. 8); this is more prevalent in the field of the marly clays. It was noted that the percentage of CaCO₃ was often concentrated inside threads and veins of secondary calcite in fissures.

The natural content of water generally varies from between 5% to 15% and shows a slight tendency to diminish with depth (Fig. 9).

The plastic index varies from 10% to 20% and shows a slight decrease with depth (Fig. 9).

The material always shows a consistency index greater than one.

6 DEFORMABILITY

Seventeen pressiometric tests were carried out in the boreholes SV2 and SV1 in order to determine the deformability of the material. The tests were carried out with MENARD (MHP) instruments using a tricellular APAGRO type 100 bar (φ 60 mm) probe and low inertia (0.05-0.1 MPa) sheaths with metallic blades.

**Fig. 8 - Percentage of CaCO₃ versus depth.**

**Fig. 9 - Logs of the main physical properties.**

**Fig. 10 - Interpretation of the Menard pressuremeter tests.**
Processing of the test data (Cassan, 1978) made it possible to construct pressiomeric curves and to identify pressiomeric parameters (Deformation modulus, "Creep" pressure, Limit Pressure). Figure 10 show the typical pressiomeric curve and his ideal interpretation. 

The processing of real pressiomeric curves showed that at levels near the surface (depth: 25-30 m from surface) complete development of pressiomeric curves was obtained with clear identification of the expansion phase of the probe and recompacting of the ground, of the pseudoelastic phase and of the plastic phase (Fig. 11). At greater depths the maximum pressures applied did not allow, due to the more stony characteristics of the material, the development of a complete pressiometric curve (Fig. 12). The deformation characteristics of the lithotype, which depend on the lack of lithological homogeneity and the anisotropic behaviour, show that:
- in the marl facies the values are more scattered (500-3500 MPa), with maximum values the same as those for the distinctly stony facies (5800 MPa in bore drilling SV3 and 16800 MPa in bore drilling SV2).

Despite the wide scattering of the deformation values an increase in the values for the elastic modulus was found with increased depth (Fig.13). Average values equal to 1600 - 1700 MPa approx. were found at a depth of 80 m.

Fig.11 - Pressuremeter tests in the argillaceous level.

Fig.12 - Pressuremeter tests in the marlaceous level.

The "Creep pressure" which characterizes the beginning of a plastic phase, showed values generally within the 2-3.5 MPa range with maximum values of approximately 5 MPa. The limit pressure, calculated using a graphic extrapolation procedure (Van Wambcke and D'Emricour, 1971 Fig.10), was obtained reliably for those tests where a complete pressiometric curve was developed. These values fall within 7.5 to 9 MPa, while for the tests where the plastic phase was not reached the limit pressure was much higher than the maximum pressure values applied during the test (6 MPa approx.).

7 STRENGTH

The shear strength of the "Argille Scagliose" was calculated by carrying out consolidated anisotropically drained direct shear tests (DS CKoD). The tests were carried out using a test sample with a square cross section
(4-6 cm) and a height of 2 cm, reaching shear deformations in the order of 30 mm where residual strength values were obtained.

Fig. 14 gives the test points of the tests carried out in the $\sigma$-$\tau$ diagram, together with the linear regression curves. The strength parameters obtained are as follows:

- peak strength
  \[ \sigma_p = 0.02 - 0.12 \text{ MPa} \]
  \[ \tau_p = 18' - 29' \]
- residual strength
  \[ \sigma_r = 0.02 - 0.07 \text{ MPa} \]
  \[ \tau_r = 7' - 21' \]

Two main types of shear behaviour were identified (Fig. 15) according to the respective type of facies (clayey or marl):

- in the first case (clayey facies), after the initial peak strength (elastic-brittle behaviour) a marked hardening was detected (hardening behaviour);
- in the second case (marl facies), the highest deformation phase exhibited an elastic type behaviour that was perfectly plastic.

8 SWELLING

In order to determine the swelling properties of "Argille Scagliose", 9 No. oedometric consolidation tests were carried out with a controlled load and the swelling prevented. The swelling pressure was measured (swelling pressure test).

The tests were carried out by applying a force corresponding to a vertical load on the test piece equal to 25 KPa approx., and introducing water into the cell to simulate normal conditions under the water table. The tendency to swell was contrasted by progressive increase of the vertical load. The results of these tests gave maximum swelling pressure values of between 50 and 150 KPa with a peak of 580 KPa approx.

One of the main factors influencing the swelling potential of clays in the presence of water is given by the nature and quantity of clayey minerals. The triangular diagram in Fig. 16 shows the swelling properties on the basis of mineral content only. It can be seen that all 18 samples analyzed fall within field A of the diagram, defined as being of high potential swelling. This confirms the results of the swelling tests.

Fig. 15 - Typical behaviour of materials

Fig. 16 - Swelling potential versus mineralogical composition.
9 MINERALOGICAL AND PETROGRAPHICAL FEATURES

X-ray diffractometric and infra-red spectrophotometric analyses were carried out to determine the mineralogical and petrographical properties of "Argille Scagliose".

Particular attention was paid, in the diffractometric analyses, to detecting and identifying the minerals of the clays that have a strong influence on the mechanical behaviour (swelling in particular) of these materials. The most common minerals in the clays are illite (25-50% approx.) and illite-smectite (mixed strata) (25-50% approx.); chlorite and kaolinite are of lesser importance (5-25% approx.).

The main components of the clays were recognized by means of the spectrophotometric analyses and these were phyllosilicates, carbonates and quartz. These percentage incidence was also determined.

10 STATISTICAL ANALYSIS OF THE PROPERTIES OF "ARGILLE SCAGLIOSE"

Due to the extreme variation in the geotechnical data relating to the "Argille Scagliose" of the Northern Apennines it is very important, in the opinion of the authors, to analyse the data statistically in order to find those ranges of variation in which the data is most frequently found. A considerable amount of informations was collected from the existing literature (c.f. bibliography); the locations of these data are reported on Fig.1.

For purposes of comparison, the processing of this data was the same as that first described.

From the granulometric point of view the data collected showed less scattering than that previously analysed (Fig.17); the most prevalent fractions were the silty and clayey fractions.

On the other hand, the Casagrande plasticity diagram showed that most of the samples could be classified as low and medium plasticity inorganic clays and rarely as high plasticity clays (Fig.18).

Fig.18 - Casagrande Chart.

The CaCO₃ content show a very scattered distribution with a slight increase with increase in depth (Fig.19); the materials are classified as marl-clays.

The deformation properties of the material were ascertained by carrying out dilatometric and pressiometric tests. Extreme scattering of the data was observed, closely connected with the percentage of stone-like bodies contained in the material. Deformability clearly increases with depth (Fig.20) and the elastic modulus showed values that fall mainly within the 1000-3500 MPa range.

Fig.19 - Statistical percentage of CaCO₃ versus depth.

Fig.20 - Elastic modulus versus depth.
The strength properties were calculated from data contained in the literature: "Argille Scaglione" of the Central and Southern Apennines (D'Elia et al., 1984; Picarelli, 1986; Pellegrino, 1986; D'Elia and Tancredi, 1979; Allevato et al., 1980; Dente et al., 1980; Manfredini et al., 1981; Cancelli et al., 1981; Bertini et al., 1986; Cotecchia et al., 1986).

The results of the analysis showed that peak strength values in terms of effective stresses (consolidated drained direct shear tests (CD)) and consolidated undrained with pore pressure measurements triaxial tests (CUDPP) vary within the following ranges (Fig. 21):

\[
p_{c} = 0-0.08 \text{ MPa} 
\]

\[
\psi = 14^\circ - 27^\circ 
\]

Fig. 21 - \(\sigma-\tau\) diagram: statistical peak strength envelopes.

For swelling properties, Fig. 23 gives, in the triangular diagram, the percentage values of the main mineral phases of the "Argille Scaglione" in the S.Donato tunnel (Barla et al., 1988): the high swelling potential of these lithotypes, which fall in field A of the triangular diagram, is confirmed.

Fig. 23 - Statistical distribution of the swelling potential versus mineralogical composition.

Figure 24 gives the percentage deformation values as a function of vertically applied forces. The swelling parameter (K) can be determined from this. The parameter corresponds to the swelling deformation developed with the application of a vertical force of 0.1 MPa and in the case in question is between 5% and 9%.

Fig. 24 - Swelling test.
11 CONCLUSIONS

The study that was carried out represent a useful contribute to knowledge of the geotechnical features of the "Argille Scaglioni", marl-clay materials belonging tectonic-stratigraphic allochthonous units known as the "Liquidi Units".

Outcrops of the "Argille Scaglioni" are to be found throughout the Italian Apennines but the study in this paper relates to those belonging to the Northern Apennines, where three important tunnels are to be built. The structure of the "Argille Scaglioni" is very complex due to the high level of tectonization and lithological heterogeneity. The latter causes considerable variation in granulometric data.

According to BS 5930, the "Argille Scaglioni" are classified as mudstone.

Plasticity properties show that the fine fraction belongs to low and medium plasticity clays. On the basis of mineral composition the "Argille Scaglioni" possess swelling potential properties.

The degree of mono-dimensional swelling as ascertained by the swelling tests is in the 5% to 10% range.

The strength properties show the presence of two different facies with different degrees of lithification and different mechanical behaviour.

In general, the strength and the deformability properties show wide variation due to the highly heterogeneous nature of the materials. Residual strength values show a high level of decay with respect to the peak values.

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Apport de la caractérisation ultrastructurale dans le choix de gisements de granulats calcaires

Contribution of the ultrastructural characterization in the choice of limestone aggregate deposits

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RESUME: L'analyse ultrastructurale (MEB) de lithofaciès calcaires montre que les meilleures caractéristiques physiques et mécaniques sont obtenues pour des granulats qui conservent ou approchent la structure initiale de la roche mère. Les critères de résistance maximale en compression uniaxiale et de porosité minimale sur échantillons de roche avant concassage ne sont pas suffisants. L'identification des liaisons interminérales permet d'expliquer la variabilité des valeurs des différents paramètres et de prévoir quels lithofaciès présenteront les meilleurs coefficients Los Angeles et microDeval à sec.

ABSTRACT: The carbonated lithofacies ultrastructural analysis by S.E.M. shows that the best physical and mechanical properties are obtained for aggregates conserving the structure of the original rock. Uniaxial compression tests and porosity measurements performed on rock samples before crushing are not sufficient to define an adequate selection criteria. The intermineral relationships characterization is a key to the interpretation of parameters variability. Prediction of the results of the Los Angeles and microDeval tests is then possible.

1. INTRODUCTION

Les géomatériaux carbonatés constituent une ressource à part entière plus ou moins bien exploitée. Leur utilisation est généralement liée à des besoins très sectoriels et s'effectue souvent sans concertation entre les différents acteurs économiques demandeurs de matériaux. Ce manque apparaît de cohérence aboutit à une situation assez fréquente: deux secteurs industriels peuvent s'ignorer alors qu'ils pourraient exploiter un même gisement en toute complémentarité. Or, la baisse de la production des laitiers de hauts fourneaux et la réduction des réserves alluvionnaires siliceuses quaternaires entraînent une demande croissante en géomatériaux carbonatés, c'est-à-dire en matériaux de substitution répondant aux spécifications demandées.

La recherche et la caractérisation de gisements calcaires doivent être dorénavant abordées en vue d'une rationalisation de la production. La confection de produits complémentaires peut améliorer le rendement d'un gisement. On doit donc répondre aussi bien aux critères de rendement, qu'à ceux, nécessaires, de valorisation et de gestion du gisement.

La diversité géologique des gisements carbonatés et leur caractérisation physico-mécanique montrent la multiplicité des recherches à entreprendre ainsi que l'importance des enjeux économiques. C'est notamment le cas de l'élaboration des granulats et des fillers calcaires.

La recherche des meilleures caractéristiques physiques et mécaniques est traditionnellement basée sur la nécessité - pour la roche mère - de présenter les critères suivants:
- faible porosité et résistance en compression élevée;
- valeurs élevées des coefficients Los Angeles et microDeval, c'est-à-dire des matériaux présentant le plus faible pourcentage possible de fines après essais.

L'analyse des caractéristiques physiques et mécaniques effectuée sur différents lithofaciès calcaires de la région de Nancy (Lorraine, France) montre une discordance entre les résultats prévisibles lors du choix (valeurs de porosité et de résistance en compression) et les résultats obtenus en Los Angeles et en microDeval à sec.

L'analyse ultrastructurale réalisée (au M.E.B.) permet de mettre en évidence l'origine de la variabilité des paramètres mesurés, d'interpréter les résultats et de concevoir une approche cohérente du choix d'un gisement. Cette analyse met en relief le rôle primordial joué par la structure initiale du matériau et en particulier le rôle des liaisons interminérales qui la déterminent. Le terme "ultrastructure" a ici la signification d'"échelle plus fine que celle de la structure". La recherche et l'identification de cette échelle ultrastructurale, caractéristique d'un matériau donné, est une nécessité. En effet, c'est l'échelle à partir de laquelle il sera possible d'identifier, de mesurer et d'interpréter. Les paramètres physiques et mécaniques ainsi définis doivent alors permettre de
comprendre et de prévoir le comportement du matériau aux différentes échelles de la matière, c'est-à-dire, en ce qui nous concerne, de l'échelle du granulat à celle du gisement.

1.1. CONSTAT ET DONNÉES INITIALES

Une étude menée sur 3 lithofacies carbonatés du Jurassique de la région de Nancy a montré que le couple porosité/résistance en compression uniaxiale n'était pas un critère qui permettait à lui seul la prédiction de la qualité d'un granulat. En effet, les valeurs des coefficients Los Angeles et microDeval à sec montrent une contradiction avec les critères de choix initiaux.

Les 3 lithofacies étudiés avaient été initialement choisis à partir des données pétrographiques connues ou observées. Il s'agit de :

Lithofacies I : calcaires grainstone (Bajocien moyen);
Lithofacies II : calcaires boundstone à polypiers (Bajocien moyen);
Lithofacies III : calcaires mudstone (Bajocien supérieur).

Il est à noter que les lithofacies I et II sont des équivalents latéraux et qu'ils sont géographiquement très proches. Le lithofacies III est plus éloigné de Nancy (France).

Le lithofacies I (calcaires grainstone, photo 1) possède une phase de liaison réduite (12%) sous forme de microsparite (cristallinité entre 10 et 50 μm); les éléments figurés carbonatés représentent 73 à 82% (dont 35 à 53% de gravelles, 25% de bioclastes et 10% d'oolithes); les espaces poreux (6 à 18%) sont essentiellement sous la forme d'une porosité intergranulaire à tendance vacuolaire.

Le lithofacies II (calcaires boundstone, photo 2) est un faciès coralien à calcaires construits et il présente une structure hétérogène. Considérée dans sa totalité, la phase de liaison (29 à 50%) montre une cristallinité étagée (micrite: 2 à 3 μm; microsparite: 20 à 35 μm; sparte: 80 à 250 μm). Les éléments figurés carbonatés représentent 48 à 67% (polypiers: 5 à 30%, de dimensions comprises entre 5mm et 10cm; encroûtements algaires: 25 à 40%; oolithes et gravelles: 2 à 3%). Les espaces poreux sont principalement représentés par les limites intercristallines (4 à 2%).

Le lithofacies III (calcaires mudstone, photo 3) présente une phase de liaison abondante (60,4%) répartie entre une micrite "libre" (6,5%), une micrite "organisée" en pelotes (pellets: 42%) et une microsparite (11,9%). Les éléments figurés carbonatés sont nombreux, au sens où les pellets (42%) y figurent du point de vue de la classification associés à 24% de fins bioclastes et 6% d'oolithes et gravelles. Les espaces poreux appartiennent aux porosités matricielle et intergranulaire (5 à 6%).

La caractérisation ayant comme objectif l'identification de faciès susceptibles de présenter a priori les meilleures propriétés mécaniques en vue de l'élaboration de granulats, il a été réalisé un ensemble de mesures destiné à la fois à la vérification de l'homogénéité de l'échantillonnage (échelle du bloc) et au classement des lithofacies, nécessaire à la quantification des propriétés des granulats liées au
caractère propre des roches dont ils sont issus, particulièrement la présence de discontinuités (pores, limites inter-cristallines, fissures).
- mesure de la masse volumique apparente et de la porosité;
- mesure de la célérité des ondes longitudinales;
- mesure du coefficient d'absorption d'eau (détection de la tenue au gel);
- mesure de la résistance en compression uniaxiale.

Les principales caractéristiques physiques et mécaniques des trois lithofaciés sont reportées sur le tableau 1.

Tableau 1: principales caractéristiques physiques et mécaniques des trois lithofaciés calcaires (moyenne et coefficient de variation, cv, sur 10 éprouvettes).

<table>
<thead>
<tr>
<th>Lithofaciés</th>
<th>Masses volumiques (kg/m³)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>23,6</td>
<td>1,2</td>
</tr>
<tr>
<td>II</td>
<td>24,7</td>
<td>3,0</td>
</tr>
<tr>
<td>III</td>
<td>24,5</td>
<td>1,0</td>
</tr>
</tbody>
</table>

Porosité:

<table>
<thead>
<tr>
<th>Lithofaciés</th>
<th>Porosité (%)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>8,6</td>
<td>36,0</td>
</tr>
<tr>
<td>II</td>
<td>2,45</td>
<td>49,0</td>
</tr>
<tr>
<td>III</td>
<td>6,0</td>
<td>15,0</td>
</tr>
</tbody>
</table>

Célérité des ondes (sec):  

<table>
<thead>
<tr>
<th>Lithofaciés</th>
<th>Célérité des ondes (m/s)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>4040</td>
<td>26,0</td>
</tr>
<tr>
<td>II</td>
<td>4763</td>
<td>5,0</td>
</tr>
<tr>
<td>III</td>
<td>4831</td>
<td>7,5</td>
</tr>
</tbody>
</table>

Résistance en compression uniaxiale:

<table>
<thead>
<tr>
<th>Lithofaciés</th>
<th>Résistance (MPa)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>78</td>
<td>15,0</td>
</tr>
<tr>
<td>II</td>
<td>95</td>
<td>18,0</td>
</tr>
<tr>
<td>III</td>
<td>193</td>
<td>12,0</td>
</tr>
</tbody>
</table>

Coefficient d'absorption d'eau:

<table>
<thead>
<tr>
<th>Lithofaciés</th>
<th>Absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>72,9</td>
</tr>
<tr>
<td>II</td>
<td>87,9</td>
</tr>
<tr>
<td>III</td>
<td>79,2</td>
</tr>
</tbody>
</table>

Les résultats sont conformes à ceux obtenus antérieurement sur ces mêmes lithofaciés (Hombert-Etienne et Troalen, 1984; Troalen, 1992). Dans ces valeurs moyennes s'intègre la variabilité des paramètres physiques et mécaniques due soit aux variations latérales de faciès, soit au type de prélèvement des éprouvettes d'essai (perpendiculairement ou parallèlement à la stratification). Ces valeurs ne possèdent donc qu'un caractère indicatif à ce stade de l'analyse.

La porosité totale est relativement faible dans les trois lithofaciés, particulièrement dans le lithofacié II et la validité de la mesure de très faibles porosités reste un problème délicat (norme NF B 10-503, 1973). Si l'on compare les valeurs de célérité des ondes longitudinales aux valeurs correspondantes de porosité, on note que la valeur de la célérité est plus faible dans le lithofacié II (porosité = 2,45%) que dans le lithofacié III (porosité = 6%). Cette anomalie peut être expliquée en comparant les indices de qualité déterminés sur éclats de béton et des éprouvettes saturées. La présence d'eau dans les pores ne modifie pas ou peu la valeur de la célérité des ondes (Fourmi-Kraut, 1975), mais la présence d'eau dans les fissures ou des microfissures entraîne une augmentation de la vitesse de propagation du son, proportionnellement au volume des fissures. Une croissance de l'indice de qualité saturé indique une microfissuration initiale de l'échantillon. Ainsi les lithofaciés I et II présenteraient une porosité de fissures dont le volume est faible en valeur absolue, mais dont l'influence est grande sur le comportement mécanique (Hombert-Etienne et Troalen, 1984). De même, la mesure du coefficient d'absorption d'eau (norme NF B 10-504, 1973) permet une estimation du pourcentage des vitesses accessibles à l'eau à la pression atmosphérique. Le lithofacié III présenterait une très bonne tenue au gel (zone A) à l'inverse des lithofaciés I et II (zone B) qui, malgré leur faible porosité, montrent un risque de mauvaise tenue au gel (A et B, zones de sensibilité au gel, abacé de la norme NF B 10-504, 1973). Les valeurs de la résistance en compression uniaxiale montrent que le lithofacié III possède de meilleures caractéristiques mécaniques que les deux autres lithofaciés. La figure 1 montre les courbes contrainte-déformation obtenues. On y distingue deux types de comportement à la rupture : les types I et II (Waverski et Fairhurst, 1970). La courbe présentée par le lithofacié III est de type II, car lorsque le maximum de la charge est atteint il faut extraire de l'énergie de l'éprouvette afin de pouvoir contrôler la rupture. Celles des lithofaciés I et II sont à la limite des types I et II. Par réduction de la fissuration initiale (Hombert et Hombert-Etienne, 1980), elles peuvent passer au type II, mais leur pente s'éloigne peu de la verticale.

Les essais Los Angeles et microDeval à sec permettent de mesurer les résistances combinées de la fragmentation par chocs et à l'usage par frottements réciproques des éléments d'un granulat. Dans les deux essais, on mesure la quantité (poids) d'éléments de granulométrie inférieure à 1,6 mm (Los Angeles) et 2 mm (microDeval) produite dans les trois classes de granularité usuelles (4/6,3 mm; 6,3/10 mm; 10/14 mm) après essai. Les coefficients Los Angeles et microDeval s'expriment par une quantité sans dimension égale à : 100 x p/p, où p est le poids de la fraction inférieure à 1,6 mm ou 2 mm, p le poids de matériau (granulat) utilisé pour l'essai (égal pour chacune des classes granulaires).
Les résultats obtenus sur les trois lithofaciès carbonatés sont reportés sur le tableau 2.

<table>
<thead>
<tr>
<th>Lithofaciès I</th>
<th>4/6,3</th>
<th>6,3/10</th>
<th>10/14 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles</td>
<td>32</td>
<td>29</td>
<td>29</td>
</tr>
<tr>
<td>MicroDeval</td>
<td>28</td>
<td>11</td>
<td>10</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lithofaciès II</th>
<th>4/6,3</th>
<th>6,3/10</th>
<th>10/14 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles</td>
<td>30</td>
<td>28</td>
<td>27</td>
</tr>
<tr>
<td>MicroDeval</td>
<td>34</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lithofaciès III</th>
<th>4/6,3</th>
<th>6,3/10</th>
<th>10/14 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles</td>
<td>26</td>
<td>26</td>
<td>24</td>
</tr>
<tr>
<td>MicroDeval</td>
<td>24</td>
<td>10</td>
<td>9</td>
</tr>
</tbody>
</table>

Si la résistance à l'usure des granulats (essais Los Angeles) dépend assez peu des forces de liaisons intercristallines (Tourenq et Denis, 1982), cette constatation peut également être appliquée aux essais microDeval à sec (MDS). En effet, dans les trois lithofaciès calcaires considérés, les valeurs du coefficient MDS sont pratiquement identiques, sauf en ce qui concerne la classe granulaire 4/6,3 mm. Cette anomalie doit être expliquée. Une première possibilité c'est-à-dire aux structures des roches mères (avant broyage et essais). On a effectivement affaire à trois types distincts de structure: grainstone, boundstone et madstone. Les deux premières (lithofaciès I et II; photos 1 et 2) présentent des constituants dont les dimensions et les cristallinités ont une importante variabilité au sein d'une même strate ou d'un même bloc. À l'inverse, le lithofaciès III (photo 3) montre des constituants de cristallinités voisines et de faibles dimensions. D'autre part, les phénomènes de frottement sont amplifiés par l'augmentation même du nombre de granulats d'une classe granulaire à une autre, même si la charge abrasive est calculée en conséquence. Ce processus d'amplification est nettement visible dans la classe 4/6,3 mm et en particulier dans le cas du lithofaciès II. Les constituants sont d'origine et de dimensions très hétérogènes. De plus, ces constituants sont eux-mêmes souvent recristallisés en sparrite (calcite de cristallinité élevée et supérieure à 80 µm). Du fait même de leur cristallinité élevée, les cristaux résistent mal aux chocs et à l'usure (Homand-Etienne et Troalen, 1984). La fragilité des liaisons intercristallines dans de tels lithofaciès avait déjà été remarquée lors d'essais de traitement par chocs thermiques (Homand-Etienne et Troalen, 1984; Troalen, 1992 et 1994). Cette dernière remarque peut alors apparaître en contradiction avec les constatations de Tourenq et Denis (1982) précisées.

Pour essayer de mieux cerner les processus de rupture et d'usure des granulats calcaires pendant les essais Los Angeles et microDeval à sec, on a examiné au microscope électronique à balayage (MEB) les granulats de chacune des classes granulaires avant essai (après broyage) et après les essais. Les résultats ont ensuite été comparés.

1.2 ANALYSE ULTRASTRUCTURALE

Les échantillons ont été tout d'abord nettoyés sous eau et faisceau d'ultrasons, séchés à 80°C et refroidis en enceinte anhydride jusqu'à la température ambiante, puis métallisés à l'or/palladium sous évaporateur Edwards. L'observation au MEB (Cambridge Stereoscan 250) a été effectuée sous des grandissements de G x 50 à G x 4000. Les résultats antérieurs (Homand-Etienne et Troalen, 1984; Troalen, 1992) qui ont permis de distinguer un certain nombre de paramètres morphologiques ont été utilisés. Ils concernent la morphologie des granulats avant et après essais (forme, surface, angles) et l'évolution morphologique des constituants (structure, surface, éléments figurés, phase de liaison) en fonction de la cristallinité de la calcite (micrite < 10 µm; microsparite >10 µm, < 80 µm; sparrite > 80 µm).

Les trois lithofaciès étudiés possèdent des structures initiales qui diffèrent autant par l'origine des constituants que par la cristallinité. Dans les trois cas, les liaisons interménagères apparaissent fortes à toutes
les échelles

Dans le cas du lithofaciès I (calcaire grainstone), après le broyage (photo 4), les granulats présentent l'essentiel des éléments constitutifs de la structure initiale. Compté-tenu des dimensions des éléments figurés et de la cristallinité de la phase de liaison, les ruptures se produisent principalement aux interfaces phase de liaison/éléments figurés et phase de liaison/microvacuoles. On peut ainsi retrouver des éléments figurés intacts, brisés ou partiellement décollés et, l'aspect de la surface est irrégulier à flou selon l'état de conservation des constituants. La microsparite de la phase de liaison résiste assez bien aux chocs et à l'usure quand il s'agit de cristaux situés à l'interface avec des éléments figurés (zone où les liaisons interminérales sont fortes et où la cristallinité est faible). À l'inverse, la microsparite de la phase de liaison externe (interface avec les microvacuoles) est peu conservée et les cristaux sont arrachés ou brisés (généralement selon un plan de clivage).

Après les essais Los Angeles, il est encore possible d'observer la structure dans les classes granulaires 6,3/10 et 10/14 mm et la phase de liaison présente des arêtes vives (photo 5). Cette même constatation est faite sur les granulats des mêmes classes après les essais microDeval à sec, cependant leurs surfaces présentent un aspect général gommé et usu. À fort grandissement (G x 4000), on note que les grains de micrite sont bien conservés (essais Los Angeles), mais sont usés et arrondis après les essais microDeval à sec. Cela provoque dans ce dernier cas un aspect uniforme de la surface des granulats (photo 6), mais dans ces milieux de très petites cristallinité il convient de ne pas généraliser une telle constatation.

Dans le cas du lithofaciès II (calcaire boundstone à polyplis), après le broyage (photo 7) les granulats sont de nature très hétérogène et cela correspond bien à la variabilité de la dimension des constituants observée sur la roche mère (5 μm à plusieurs cm). Les cristaux de sparite (> 80 μm) sont brisés ou arrachés et des microfractures apparaissent dans des secteurs bien cristallisés ou recristallisés. La fracturation s'associe volontiers aux plans de clivage ou aux limites intercristallines.

Après les essais Los Angeles et microDeval à sec (photos 8 et 9), les granulats présentent des surfaces rajeunies ou nettoyées et l'observation des constituants est améliorée. Les arêtes vives de l'après broyage ont été usées et arrondies. La sparite et la microsparite résistent mal à l'inverse des grains de micrite, surtout lorsqu'il s'agit de la micrite constituant d'un élément figuré (photo 9). Cette résistance de la micrite n'empêche nullement le surcreusement de l'élément figuré. Dans l'ensemble les classes granulaires 6,3/10 et 10/14 mm montrent une structure mieux conservée alors que la classe 4/6,3 mm paraît ne contenir que des granulats composés uniquement de micrite ou de micrite de cristallinité proche de celle de la microsparite. À fort grandissement (G x 4000), les grains de micrite apparaissent en partie brisés et usés, les surfaces sont alors d'aspect très vif à esquilleux.
Photo 7: MEB, grandissement x 1000,
lithofaciès II: calcaire de type boundstone,
granulat témoin 6,3/10 mm (après broyage).

Photo 8: MEB, grandissement x 1000,
lithofaciès II: calcaire de type boundstone,
granulat 4/6,3 mm (après Los Angeles).

Photo 9: MEB, grandissement x 1000,
lithofaciès II: calcaire de type boundstone,
granulat 6,3/10 mm (après microDeval à sec).

Photo 10: MEB, grandissement x 1000,
lithofaciès III: calcaire de type mudstone,
granulat témoin 6,3/10 mm (après broyage).

Photo 11: MEB, grandissement x 1000,
lithofaciès III: calcaire de type mudstone,
granulat 4/6,3 mm (après Los Angeles).

Photo 12: MEB, grandissement x 1000,
lithofaciès III: calcaire de type mudstone,
granulat 10/14 mm (après microDeval à sec).
Dans le cas du lithofaciès III (calcaire mudstone), après le broyage (photo.10) tous les granulats présentent une structure identique à la structure initiale, mais l'état de surface varie ensuite selon le type d'essai subi. En ce qui concerne l'essai Los Angeles (photo.11) les chocs provoquent le nettoyage de la surface et gonment l'essentiel des aspropérés, l'observation des surfaces devient délicate (aspect flou). Dans le cas de l'essai microDeval à sec (photo.12) l'usure des surfaces est nette et le granulat présente une forme arrondie avec des surcreusements au niveau des éléments figurés. A fort grandissement (G x 4000), on note que l'essentiel des constituants du granulat est de la micrite (constituant majeur de la phase de liaison et des éléments figurés). Lors du broyage, il y a donc des ruptures qui se produisent préférentiellement au contact avec les cristaux de microsparite de la phase de liaison comme de ceux (rares) des éléments figurés. Ceci explique que les cristaux de sparite et de microsparite sont presque toujours absents dans la classe granulaire 4/6,3 mm alors qu'ils peuvent persister dans les deux autres classes. A fort grandissement (G x 4000), on note que les grains de micrite sont souvent brisés et usés après Los Angeles (photo.11) et qu'ils sont usés et arrondis après microDeval à sec (photo.12). Dans les deux cas on aboutit à un aspect de surface du granulat très différent de celui observé après broyage dans toutes les classes granulaires. Toutefois, l'usure des grains de micrite, tout comme leur bris, reste modérée même si on a une cristallinité étagée. On remarque qu'avec une cristallinité très variable le nombre d'arrachement et de décollement de grains et d'éléments figurés sont importants après Los Angeles, alors que dans le cas du microDeval à sec la variabilité de la cristallinité semble n'avoir aucune influence. Il apparaît également que le Los Angeles conserve mieux que le microDeval à sec une structure à cristallinité étagée.

Après broyage (granulats témoin; photos 4, 7 et 10), on constate que l'essentiel des arrachements, des reliefs et des angles vifs (arêtes) se situe dans des secteurs où la phase de liaison possède une cristallinité voisine de celle de la microsparite (entre 10 et 80 μm). L'observation au MEB montre que la rupture est liée à l'orientation présentée par les plans de clivage de la calcite au moment du broyage. Ce mode de rupture aboutit à la formation d'arêtes à angles vifs qui définissent des zones en dépression correspondant le plus souvent à des éléments figurés (gravelles, pellets). L'aspect de la surface va donc être directement lié à la cristallinité à la nature et aux dimensions des éléments constitutifs. Cette observation conduit à une première constatation: plus les constituants sont de petites dimensions, plus le granulat a de chance de présenter une structure proche ou identique à la structure initiale (roche mère). D'autre part, si la dimension des éléments figurés augmente, ce sera uniquement dans les classes granulaires élevées que l'on aura des chances d'observer une structure proche de la structure initiale.

2. INTERPRETATION ET RELATIONS

Si on reprend l'ensemble des données obtenues et si on les compare lithofaciés à lithofaciés, on peut penser que le lithofaciès III (calcaire mudstone) présente les meilleures caractéristiques physiques et mécaniques tant en gainement qu'en granulat. Ce résultat peut paraître surprenant quand on sait que sa porosité est de l'ordre de 6% et sa résistance en compression uniaxiale supérieure à celle du lithofaciès II qui possède pourtant une très faible porosité (< 2,5%). Or, on a toujours considéré la porosité comme étant l'un des paramètres qui, influence le plus la valeur de la résistance en compression d'une roche. Ce résultat apparaît comme encore plus surprenant que les valeurs obtenues pour les lithofaciés I et II, une fois comparées, apparaissent logiques. On remarque également une variabilité dans les valeurs de la célérité des ondes longitudinales entre milieu sec et milieu saturé (VL et VLW), de même qu'entre les valeurs de l'indice de qualité (IQ et IQW). La présence ou l'absence de microfissures et surtout d'eau dans celles-ci peut expliquer l'écart entre les valeurs des différents paramètres des lithofaciés II et III (Hemond-Étienne et Troalen, 1984). Par contre, il devient impossible de relier ces données à celles obtenues lors des essais Los Angeles et microDeval à sec, sauf si on fait intervenir au niveau de l'interprétation des paramètres structuraux tels que les liaisons interminérales, la cristallinité et la dimension des constituants et la structure s.s.

En effet, les microfissures existantes à l'état initial vont jouer un rôle important lors de la phase de broyage et la création des granulats: ruptures préférentielles. Et, selon la classe granulaire considérée, les microfissures résiduelles (ou créées lors du broyage) vont jouer ensuite lors des essais Los Angeles et microDeval à sec; en favorisant la rupture du granulat lors des chocs et frottements, elles vont contribuer à l'enrichissement de la classe granulaire inférieure à 1,6 mm ou 2 mm. Ce processus est bien marqué dans les granulats issus des calcaires grainstone et boundstone (I et II), mais n'existe pas dans le calcaire mudstone (III). Touron et Denis (1982) considèrent que la résistance à l'usure des granulats dépend assez peu des forces de liaisons "intercristallines". Cette remarque est à nuancer. On note effectivement que les valeurs des coefficients microDeval à sec sont peu dispersées, mais uniquement dans les classes granulaires 6,3/10 mm et 10/14 mm des trois lithofaciés, car la variabilité est forte pour les valeurs obtenues en 4/6,3 mm (tableau 2). L'analyse ultrastructurale des granulats de cette classe granulaire montre que seuls ceux issus du lithofaciès III (calcaire mudstone) présentent une structure identique à celle de la roche mère du fait même de la très faible cristallinité de ses constituants. Dans le cas des deux autres lithofaciés, la probabilité de trouver des granulats présentant une structure proche de celle de leur roche mère diminue avec la croissance des dimensions de leurs constituants. Cependant, dans les granulats issus du lithofaciès I (calcaire grainstone) la probabilité sera
plus grande que dans ceux issus du lithofaciès II (calcaires boundstone à polyliers). En effet, le broyage détruit le réseau poreux principal (microvacuolaire) et il y a en quelque sorte "homogénéisation" de la structure initiale, mais la présence d'éléments détritiques d'origine variées fait que les granulats montrent souvent des structures différentes. Toutefois, la variabilité diminue lorsque la taille des grains évolue vers les petites dimensions. Cela explique les valeurs plus élevées du coefficient microDeval à sec dans les granulats issus de I (par rapport aux valeurs issues de II). A l'inverse l'écart sera faible entre les valeurs du coefficient microDeval à sec des lithofaciès I et III, comparables cette fois du point de vue des valeurs de la porosité (tableau 1).

De façon générale, l'analyse ultrastructurale permet de mettre en évidence les mécanismes de rupture et d'usure des granulats en fonction de la structure initiale. Dans le cas des trois lithofaciès calcaires étudiés, on constate que les constituants (éléments figurés et cristaux) de faibles dimensions présentent de fortes liaisons interminérales et résistent bien aux chocs et frottements provoqués aussi bien par le broyage qu'ensuite par les essais Los Angeles et microDeval. Les constituants de grandes dimensions, et particulièrement s'ils sont composés de sparite (calcite de cristallinité > 80 µm), échappent rarement aux chocs et aux frottements et la présence de clivages dans la sparite augmente le risque de rupture et d'arrachement du cristal lors d'un choc. Ce mécanisme n'existe pas dans le cas du lithofaciès III : la microsparite qui peut exister dans la phase de liaison est généralement détruite lors du broyage. Cette microsparite disparaît, mais en partie seulement, dans le lithofaciès I : le broyage provoque des ruptures préférentielles à l'interface éléments/réseau microvacuolaire et la microsparite qui tapisse ces cavités est arrachée. Il ne reste que les cristaux au niveau même de l'interface, c'est-à-dire ceux de petite cristallinité (photo 1). Une structure initiale très hétérogène comme celle présentée par le lithofaciès II sera le siège de mécanismes multiples lors du broyage comme pendant les essais d'usure et de frottement.

3. CONCLUSION

De l'ensemble de ces résultats obtenus sur trois lithofaciès calcaires de structures différentes, on retiendra que les meilleures caractéristiques physiques et mécaniques sont obtenues pour des granulats dont la structure reste identique ou très proche de la structure initiale de la roche mère. L'analyse ultrastructurale révèle que dans ce cas les liaisons interminérales entre les constituants sont très fortes et d'autant plus que la cristallinité des constituants est plus petite. Bien que présentant une porosité assez élevée, le calcaire de type mudstone offre une forte valeur en résistance en compression et les meilleurs valeurs des coefficients Los Angeles et microDeval à sec. Les granulats de toutes les classes granulaires présentent une structure identique à celle de la roche mère. Les propriétés physiques et mécaniques d'un granulat ont alors des valeurs totalement comparables à celles obtenues à une échelle supérieure. Dans ce cas, il est donc possible de prédire les propriétés d'un gisement et celles des produits fabriqués.

La répartition spatiale des constituants et les dimensions des éléments constitutifs (solides et réseau poreux) vont être le reflet de l'hétérogénéité de la structure et vont conditionner les qualités du granulat dès la phase de broyage. Cette phase de broyage peut dans certains cas réduire les hétérogénéités de structure (cas du lithofaciès I) ou les accroître (cas du lithofaciès II).

L'analyse ultrastructurale pratiquée sur l'ensemble des granulats, du broyage à l'issue des essais Los Angeles et microDeval à sec illustre bien ces processus.

D'autre part, il serait également très utile d'analyser au MEB la nature des produits issus des différents essais, c'est-à-dire ceux constituent les passants à 1,6 mm et 2 mm. En effet, ces éléments peuvent servir à la fabrication de produits autres que les granulats et plus particulièrement la conception de filtres calcaires en additifs de ciments (Regourd, 1986).

L'identification fine des paramètres structuraux et de l'ultrastructure est donc une nécessité vis-à-vis de la décision de choix d'un gisement destiné à la production de granulats calcaires comme pour les filtres où, là encore, la cristallinité des constituants a une action directe sur la qualité du produit fabriqué (Regourd, 1986). Cette caractérisation permet non seulement de comprendre la variabilité des propriétés physiques et mécaniques des roches calcaires, mais de prédire avec une plus grande certitude les propriétés du produit en fonction des critères de qualité imposés. Les seuls critères de porosité et de résistance en compression ne sont pas suffisants. Cependant, cette analyse ultrastructurale ne peut être généralisée au sens où chaque gisement et chaque lithofaciès possèdent des paramètres intrinsèques propres qui induisent, dans chaque cas, des mécanismes particuliers et eux-mêmes fonction des sollicitations imposées.

COMMENTAIRES PHOTOGRAPHIQUES

Photo 1 : Lithofaciès I, calcaire de type grainstone. On peut observer l'interface entre la phase de liaison (cristaux de microsparite, > 10 µm < 80 µm) à gauche et un élément figuré à droite (grains de micrite, < 10 µm). La phase de liaison présente une porosité intercristalline et délimite un réseau de microvacuoles tapissé de microsparite (encroûtements de type vadoïse).

Photo 2 : Lithofaciès II, calcaire de type boundstone. On observe sur cette surface une partie d'un élément figuré de type polyplier branchu, entièrement recristallisé (cristaux de sparite, > 80 µm et de microsparite). Les stries qui apparaissent sur les
cris taux correspondent aux trac ses des plans de 
cavage de la calcite.

Photo. 3: Lithofacies III, calcaire de type mudstone.
Sur la photographie, on note la présence d’au moins 4 éléments figu rés de type "pellets". Ceux-ci sont constitués par des grains de micrite agglomérés en pелote. L’interface entre les pellets est, soit un contact micrite/micrite (pellet/pellet), soit une liaison par de la microsparite (recristallisation secondaire entre les éléments figurés). Notons que l’organisation de la micrite en pellets est singulière et ces pellets constituent tout à la fois des éléments figurés et une part importante de la phase de liaison.

Photo. 4: Lithofacies I, calcaire de type grainstone.
Sur cet extrait de la surface d’un granulat issu du broyage (classe 10/14 mm), on remarque que les cristaux de microsparite de la phase de liaison ont été brisés et réduits au seule contact avec la micrite d’un élément figuré (partie droite du cliché), contrairement à ce qui est observé sur la photo. 1. L’allure générale de la structure initiale est assez floue.

Photo. 5: Lithofacies I, calcaire de type grainstone.
Sur cette surface d’un granulat 6,3/10 mm après essai microDeval à sec, on note le surcreusement de la micrite des éléments figurés et l’adoucissement des angles des cristaux de microsparite.

Photo. 7: Lithofacies II, calcaire de type boundstone.
Granulat 6,3/10 mm après broyage. Extrait de la surface d’un élément figuré montrant des grains de micrite assez bien cristallisés et présentant une porosité matricielle importante.

Photo. 8: Lithofacies II, calcaire de type boundstone.
Aspect d’une surface de granulat 4/6,3 mm après un essai Los Angeles. Surface unique libérée par des cristaux de micrite et présentant des angles moins vifs qu’en photo. 7.

Photo. 9: Lithofacies II, calcaire de type boundstone.
Aspect d’une surface de granulat 6,3/10 mm après un essai microDeval à sec: surcreusement et usure des grains de micrite.

Photo. 10: Lithofacies III, calcaire de type mudstone.
Surface d’un granulat 6,3/10 mm après broyage; on note l’aspect assez vif des angles des cristaux de microsparite et des grains de micrite. L’agencement pellets/microsparite (éléments figurés/phase de liaison) est peu net.

Photo. 11: Lithofacies III, calcaire de type mudstone.
La surface du granulat 4/6,3 mm après essai Los Angeles est rajeunie et nettoyée par rapport à la photo. 10, mais les angles des cristaux et des grains sont émoussés. Cependant, l’aspect général reste très proche de celui observé précédemment.

Photo. 12: Lithofacies III, calcaire de type mudstone.
Granulat 10/14 mm après essai microDeval à sec: la surface présente un aspect très proche de celui observé en photos 10 et 11, mais les pellets sont surcreusés et les grains de micrite très émoussés.

BIBLIOGRAPHIE


A microscopic study on zoning and use of weathered crust for bedrock of the Three Gorge Project
Étude microscopique de la zone d'altération du massif de fondation du barrage de Trois Gorges

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ABSTRACT: In this paper, the composition, physicochemical properties and so on were studied for different weathered zones of bedrock (hornblende — biotite — plagioclase granite) at the Sandouping dam site of the Three Gorge Project. According to chemical weathering characteristic of bedrock, main chemical process in weathering and character of their products, comprehensive chemical weathering characteristic indices for different weathered zones were suggested as basis of quantitative or semi-quantitative zoning of the chemical weathering. From the point of view of chemical weathering, rationality of field zoning for weathered crust of bedrock and possibility of the lower portion of weakly weathered zone using as foundation rockmass of the dam were proved.

RÉSUMÉ: Le présent rapport fait une étude de la Composition et des propriétés physico-chimiques sur les roches de fondation (granites biotite à plagioclase) dans le site du barrage des Trois-Gorges à Sandouping en Chine. D'après le caractéristique d'altération chimique des roches de fondation, les actions principales d'altération chimique et les caractères de leurs produits, les comprehensives indices caractéristiques d'altération chimique sur les différentes zones ont été formulées comme la base de la zonation d’altération chimique quantitative ou semi-quantitative. Du point de vue de l’altération chimique, l’auteur démontre la rationalité de la zonation in situ de la croûte altérée des roches de fondation et la possibilité de l’usage de la partie inférieure de la Zone d’altération légère comme le massif de fondation du barrage.

1 INTRODUCTION

The bedrock of the Three Gorge Project at the Shangdouping dam site is mainly Prescinian hornblende — biotite — plagioclase granite with deep and thick weathered crust. Engineering geologic investigation of the weathered crust has been conducted for a long time. According to the different macro states the weathered crust was divided into different zones, namely; completely, strongly, weakly and slightly weathered zone (Chen 1986). But chemical weathering study for it was relatively little. It is well a common knowledge that there are always various chemical changes in weathering of rock. And they must be resulted in the change of composition of the weathered crust so that composition and property of the weathered products are different from that of parent rock. In the mean time, the composition and property of these weathering products will be some differences with differences of chemical process and its intense degree occurred in weathering. These differences not only reveal weathering essence of rock, but also provide scientific basis for zoning and using of weathered crust. Therefore, the study of chemical weathering characteristic of bedrock for the Three Gorge Project is of great significance.

2 MINERAL CONSTITUENT OF BEDROCK

Dominant mineral of bedrock (hornblende — biotite — plagioclase granite) at the Sandouping dam site is andesine (An = 33—40, content is about 60%) in plagioclase series; the second is quartz (content = 20—25%) and biotite (content = 10—12%); there is also a few hornblende and so on (Tab. 1). Besides quartz, other minerals among them are relatively weak for resistance to weathering. So the chemical weathering characteristic of bedrock is significantly concerned in the alteration of such minerals. Due to such minerals being easily altered, even if at the po-
position 300m under ground surface, the rock is not absolutely fresh. Among them the easily weathering minerals are more or less altered and there exists a few (<5%) secondary mineral such as sericite etc.

3 COMPOSITION AND PHYSICO-CHEMICAL PROPERTIES OF DIFFERENT WEATHERED ZONES

Due to differences of structural failure degree, local environment, and weathering intensity of the bedrock composition and properties of weathering products for various weathered zones are different.

3.1 Mineral constituent of different weathered zones

As shown in Tab. 1 and Fig. 1, mineral constituent of completely, strongly, weakly and slightly weathered zones can be outlined mainly as follows:

1. There are still a number of primary minerals remained (generally not less than 70%) in weathering products, but the easily weathering minerals have been changed differently in quality and quantity with different weathering intensity. An value of plagioclase decreases with intensification of weathering and decalcification, for example, An = 32 – 42 (andesine) in slightly weathered zone, An = 26 – 40 (oligoclase – andesine) in relatively firm portion of the weakly weathered zone and An < 20 of acid plagioclase in completely and strongly weathered zones. As to biotite, variation depended on intensification of weathering shown in Fig. 1. Shape of 10 A diffraction peak changes from sharp to broad, its symmetry becomes poor and even twohumps appear (Fig. 1 A and B). Moreover, diffraction strength (peak area) decreases obviously, for example, 10 A peak area for completely weathered zone is only 1/10 that for slightly weathered zone. These changes not only show that biotite content decreases with intensification of weathering, but also, reflect that variation of its crystal structure increases by degrees, dispersity gradually becomes high and then it alters towards chlorite, vermiculite etc.

2. Secondary minerals are mainly sericite, hydromica, chlorite and epidote. There are also a few intergrade mineral of mixed-layer and free SiO₂, Fe₂O₃, Al₂O₃. Vermiculite (individual portion is montmorillonite), which is relatively strongly altered product, appears largely in

A. Relatively loose portion of weakly weathered zone (peak area is 77 mm²)
B. Completely weathered zone (peak area is 90 mm²)
C. Strongly weathered zone (peak area is 235 mm²)
D. Relatively firm portion of weakly weathered zone (peak area is 775 mm²)
E. Slightly weathered zone (peak area is 920 mm²)

Fig. 1 Characteristic of 10A diffraction peak under condition of slow scanning (280. 25° / min)

completely weathered zone.

3. Constituent of mineral for slightly weathered zone and bedrock is basically the same.

3.2 Altered pathes of minerals

The abovementioned various secondary minerals are altered from primary minerals according to the following pathes:

Andesite → Sericite → Vermiculite
Biotite → Hydromica → Chlorite

Hornblende

Montmorillonite

This is also the development trend of mineral alteration in weathered crust of this dam site.

3.3 Chemical composition and physico-chemical properties of different weathered zones

As shown in Fig. 2 and Tab. 2, composition and
<table>
<thead>
<tr>
<th>Sample</th>
<th>Primary mineral</th>
<th>Secondary mineral</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Fresh&quot; bedrock</td>
<td>Plagioclase (andesine) 60%, Quartz 22–24%, Biotite 10–12%, Hornblende ≤3%</td>
<td>Sericite 2–3%, Chlorite ≤1%, Epidote ≤1%, Calcite ≤0.1%, Hematite ≤0.1%</td>
</tr>
<tr>
<td>Slightly weathered Zone</td>
<td>Plagioclase (andesine) 50–55%, Quartz 20–25%, Biotite 10%, Hornblende 10%</td>
<td>Sericite 1%, Chlorite 0.5%, Epidote 1%, Calcite 0.1%, Free SiO₂, Fe₂O₃, Al₂O₃ 1.5%</td>
</tr>
<tr>
<td>Weakly weathered zone (relatively firm)</td>
<td>Plagioclase (oligoclase–andesine) 50%, Quartz 20–25%, Biotite 3–10%, Hornblende 4–10%</td>
<td>Sericite 3–8%, Chlorite ≤1%, Epidote ≤2%, Calcite 0.1%, Free SiO₂, Fe₂O₃, Al₂O₃ 1.6%</td>
</tr>
<tr>
<td>Weakly weathered zone (relatively loose)</td>
<td>Plagioclase (mainly oligoclase) 40%, Quartz 20–25%, Hornblende ≤2%, Biotite (transiting to vermiculite) ≤1%</td>
<td>Hydromica 5–20%, Chlorite 8%, Epidote 5%, Vermiculitized biotite 1%, Calcite 0.1%, Free SiO₂, Fe₂O₃, Al₂O₃ 1.7%</td>
</tr>
<tr>
<td>Strongly weathered zone</td>
<td>Acid plagioclase 45–50%, Quartz 20–25%, Hornblende 8%, Biotite (strongly hydrated) 5%</td>
<td>Hydromica 16%, Chlorite 1%, Epidote 3%, Calcite 0.1%, Free SiO₂, Fe₂O₃, Al₂O₃ 2.1%</td>
</tr>
<tr>
<td>Completely weathered zone</td>
<td>Acid plagioclase 45–50%, Quartz 20–25%, Hornblende 3–7%, Biotite (transiting to vermiculite) 1%</td>
<td>Vermiculite ≥10%, Hydromica 15%, Chlorite ≤1%, Epidote 2–5%, Calcite 0.1%, Free SiO₂, Fe₂O₃, Al₂O₃ 3.1%</td>
</tr>
</tbody>
</table>

Notes: The mineral constituent is comprehensive analysis results obtained with polarizing microscopic examination, x-ray diffraction, chemical analysis and scanning electron micrograph.
properties of various weathered zones have the following features:

1. Fairly much alkaline and alkaline earth metal composition with large migration rate are still remained in various weathered zones and it is also the same even for completely weathered zone. It is shown that leaching loss of rock is not strong in weathering. So, pH values of various weathered zones vary basially in range of weak alkality.

2. Main variation of chemical composition depending on weathering intensification is as follow: CaO, K₂O and Fe₂O₃ decrease gradually; MgO, Fe₂O₃ etc. are gradually and relatively accumulated.

3. Physico-chemical activity such as specific surface, cation exchange capacity increases somewhat with intensification of weathering. But wholly, physico-chemical activity of various weathered zones is fairly low.

It is known from abovementioned characteristics of mineral constituent, chemical composition and properties, that weathered crust of bedrock for this dam site is yet at the early stage of chemical weathering.

Table 2. Physico-chemical properties of weathering product for different weathered zones

<table>
<thead>
<tr>
<th>Weathered Zone</th>
<th>pH</th>
<th>Specific surface m²/g</th>
<th>Cation exchange capacity m.e./100g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slightly</td>
<td>8.50–9.12</td>
<td>17</td>
<td>1.54</td>
</tr>
<tr>
<td>Weakly(firm)</td>
<td>8.37–8.89</td>
<td>18</td>
<td>3.72</td>
</tr>
<tr>
<td>Weakly(loose)</td>
<td>8.30–8.88</td>
<td>20</td>
<td>5.06</td>
</tr>
<tr>
<td>Strongly</td>
<td>7.59–8.25</td>
<td>34</td>
<td>7.37</td>
</tr>
<tr>
<td>Completely</td>
<td>7.35–8.28</td>
<td>40</td>
<td>12.16</td>
</tr>
</tbody>
</table>

4 MAIN CHEMICAL PROCESSES OF WEATHERING

Various weathering products in weathered crust of bedrock are formed on the basis of following chemical processes.

4.1 Hydrolysis

Hydrolysis is a general and important basic chemical reaction of rock weathering, it is essentially a decomposition of mineral under H⁺ ion action. Alteration of mineral is closely concerned in hydrolysis in weathering. In bedrock of this dam site, alteration of hornblende and production of free oxide (SiO₂, Fe₂O₃ and Al₂O₃) are mainly results of hydrolysis.
4.2 Decalcification

Decalcification is a chemical process of mineral alteration, in which Ca\(^{2+}\) ion is liberated from mineral lattice on the basis of hydrolysis. It is a main reaction of alteration for base and neutral plagioclase with fairly much calcium.

As mentioned above, in various weathered zones of bedrock, CaO amount decreases with intensification of weathering (Fig. 2). For example, CaO amount of completely weathered zone is about 30% less than that of slightly weathered zone. It is shown that decalcification occurs in weathering of bedrock. So, andesine in bedrock alters to acid plagioclase with low An value under this action. In the meantime, under the attendance of K\(^+\) ion (from depotassification of biotite) and water, part of them alters into sericite. Under condition of strong weathering and existence of Mg\(^{2+}\) ion, such as in the position of completely weathered zone, fault zone etc., they may yet alter into montmorillonite. Besides, occurrence of epidote and calcite in weathering product is related to decalcification of andesine also.

4.3 Hydration and depotassification

During alteration of potassium mineral, hydration and depotassification occur in succession or simultaneously. They are one of main alteration ways for micaceous mineral, specially biotite.

In various weathered zones of bedrock, K\(_2\)O amount decreases also with intensification of weathering (Fig. 2). For example, K\(_2\)O amount of completely weathered zone is only about 70% that of slightly weathered zone. It is mainly resulted from hydration and depotassification of biotite. Under this action biotite and sericite alterate towards hydromica, chlorite, vermiculite and montmorillonite.

4.4 Oxidation

As shown in Fig. 2, FeO amount in various weathered zones decreases with intensification of weathering, but Fe\(_2\)O\(_3\) amount increases relatively. It is related to that Fe\(^{3+}\) ion in lattice of umbire minerals such as biotite etc. is oxidized into Fe\(^{2+}\) ion.

When Fe\(^{2+}\) ion is oxidized into Fe\(^{3+}\) ion, radius of ion decreases so as to increase interstice in mineral lattice and to destroy equilibrium of lattice charge (positive charge excess). It is favorable to development of hydration and depotassification for biotite. Oxidation of ferrous is also one of necessary conditions of alteration from biotite to vermiculite.

5 CHEMICAL WEATHERING CHARACTERISTIC INDICES FOR DIFFERENT WEATHERED ZONES

As mentioned above, composition and properties as well as mineral alteration degree of various weathered zones are different. From qualitative point of view, their zoning feature is apparent. In order to quantize zoning feature, we used the predecessors' methods of zoning for chemical weathering of rock (Uriel and Dapena 1978, Miura 1980) to investigate the zoning of weathered crust for this damsite, but did not obtain a satisfactory result. In this paper, according to chemical weathering characteristic, main chemical processes in weathering and their products features, comprehensive characteristic indices of chemical weathering for different weathered zones were suggested as basis of quantitative or semiquantitative zoning of the chemical weathering (Tab. 3).

The chemical weathering characteristic indices are listed below:

1. Main secondary minerals and their amount—reflecting variation of quality and difference of quantity occurred in minerals of various weathered zones.
2. The An value of plagioclase—reflecting degree of that andesine conducts decalcification and transforms to acid plagioclase.
3. CaO/MgO mol. ratio—reflecting degree of leaching loss for calcium (decalcification of andesine), and degree of relative accumulation of magnesium (chloritization, vermiculitization etc.).
4. K\(_2\)O/Na\(_2\)O mol. ratio—reflecting intensity of hydration and depotassification for micaceous mineral, and relative accumulation degree of sodium during andesine transforming to albite.
5. FeO/Fe\(_2\)O\(_3\) mol. ratio—reflecting degree of oxidizing alteration for biotite.
6. Area ratio of 10Å diffraction peak—considering peak area of slightly weathered zone as 1, with which peak area of other zones are compared. The smaller the ratio, the higher the degree transforming to vermiculite and montmorillonite from micaceous mineral.
7. Cation exchange capacity (CEC)—comprehensive index reflecting intensity of rock weathering, main secondary mineral and physicochemical activity of weathering product.

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### Table 3. Chemical weathering characteristic indices of different weathered zones

<table>
<thead>
<tr>
<th>Weathered zone</th>
<th>Sum of secondary mineral (%)</th>
<th>Amount of main secondary mineral (%)</th>
<th>An value of plagioclase</th>
<th>Area ratio of 10Å diffraction peak</th>
<th>CaO/MgO mol. ratio</th>
<th>K₂O/Na₂O mol. ratio</th>
<th>FeO/Fe₂O₃ mol. ratio</th>
<th>CEC m.e./100g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slightly (or fresh bedrock)</td>
<td>&lt;5</td>
<td>Sericite 1–3, Chlorite (a little)</td>
<td>30–45</td>
<td>1.0</td>
<td>&gt;2.0</td>
<td>&gt;0.20</td>
<td>&gt;1.4</td>
<td>&lt;2.0</td>
</tr>
<tr>
<td>Weakly (firm)</td>
<td>about 10</td>
<td>Sericite 3–8, Chlorite (a little)</td>
<td>25–40</td>
<td>&lt;0.8</td>
<td>about 1.9</td>
<td>about 0.20</td>
<td>&lt;1.4</td>
<td>&lt;4.0</td>
</tr>
<tr>
<td>Weakly (loose)</td>
<td>15–30</td>
<td>Sericite 5–20, Chlorite &lt;10, Vermiculitized biotite</td>
<td>20–30</td>
<td>&lt;0.1</td>
<td>about 1.5</td>
<td>about 0.15</td>
<td>about 1.0</td>
<td>&gt;5.0</td>
</tr>
<tr>
<td>Strongly</td>
<td>about 20</td>
<td>Hydromica about 15, Chlorite about 1</td>
<td>&lt;25</td>
<td>&lt;0.3</td>
<td>about 1.2</td>
<td>about 0.17</td>
<td>&lt;1.0</td>
<td>&lt;10.0</td>
</tr>
<tr>
<td>Completely</td>
<td>about 30</td>
<td>Vermiculite or montmorillonite &gt;10, Hydromica about 15</td>
<td>&lt;20</td>
<td>&lt;0.1</td>
<td>about 1.0</td>
<td>about 0.15</td>
<td>&lt;0.5</td>
<td>&gt;12.0</td>
</tr>
</tbody>
</table>

The following may be seen from Tab. 3:

1. Chemical weathering characteristic indices of different position for weathered crust of bedrock in this dam site are not the same, and reflect objectively their zoning feature which is basically in agreement with the macro zoning.

2. Chemical weathering characteristic indices of weakly weathered zone possesses heterogeneity. It may be approximately divided into two subzones, namely, relatively firm subzone (lower portion) and relatively loose subzone (upper portion). Characteristic indices of relatively firm lower portion are apparently different to that of relatively loose upper portion, and are approaching to that of slightly weathered zone.

3. Characteristic indices of slightly weathered zone and fresh bedrock are basically the same.

6 POSSIBILITY OF USING WEAKLY WEATHERED ZONE AS DAM FOUNDATION

The deep and thick weathered crust of bedrock in this dam site possesses fairly wide utilization prospect. Only with respect to the Three Gorge Project there are two main uses (Wang 1992): (1) construction material, such as manufactured sand and aggregate for using in concrete, and weathered material of completely and strongly weathered zones used as filling compound of cofferdam, etc.; (2) trying to use relatively firm lower portion of weakly weathered zone or slightly weathered zone as foundation rockmass of the dam.

As foundation rockmass of concrete dam, slightly weathered zone or fresh rockmass were adopted for most engineering in China, but for a few engineering lower portion of weakly weathered zone were adopted. Whether lower portion of weakly weathered zone might be used as foun-
7 CONCLUSION

Weathered crust of bedrock at the Sandouping dam site of the Three Gorge Project is yet in the early stage of chemical weathering. Owing to that leaching loss is not strong, consideralbly quantity of alkaline and alkaline earth metal composition are still remained in weathering products and their pH are higher also. Their mineral alteration is generally relatively light, and the secondary minerals are mainly sericite, hydronica and chlorite etc; only at intensive positions of weathering such as completely weathered zone, there appear vermiculite or montmorillonite. The physico — chemical activity of weathering products is generally lower.

Main chemical processes of bedrock weathering are hydrolysis, decalcification, hydration and depotassification as well as oxidation. Under these actions, easily weathering minerals such as andesine, biotite etc. via sericite and hydronica alterate towards chlorite, vermiculite or montmorillonite. The alteration development of mineral is fairly slow and existing weathering product is relatively stable.

Composition and properties as well as mineral alteration degree for different weathered zones are all some different. On the basis of these, comprehensive chemical weathering characteristic indices for different weathered zones are suggested. Zonging feature of weathered crust is reflected quantitatively or semi — quantitatively and it is basically in agreement with macro weathering zoning. So that it is shown that the macro zoning is fairly objective.

According to differences of weathering characteristic and composition and properties of weathering product, weakly weathered zone may be divided into two subzones, upper and lower portion. In relatively firm lower portion, composition and properties as well as chemical weathering characteristic indices are approaching to that of slightly weathered zone and there are not vermiculite and montmorillonite with poorer engineering properties. Therefore using the lower portion of weakly weathered zone as foundation rockmass of dam is possible.

Sum up, when weathered rock is used for engineering construction, investigation conducted in respects of chemical weathering characteristic of rock and composition and property of weathering product is necessary. It not only provides basis for zoning and utilization of weathered crust, but also may understand essence of weathering process of rock, varying velocity of the composition and so on. It is difficult for other methods.
REFERENCES


Study on the engineering geological features of the Nihewan bed
Étude des propriétés géologiques et géotechniques de la formation Nihewan

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ABSTRACT: The paper briefly on the study in the engineering geological characteristics of the Nihewan bed in Datong-Yangyuan Basin, which was made in combination with the construction of Datong-Qinhuangdao Railway. A review is presented in this article by quoting a great quantity of appropriate data from the following spheres: Strata, lithological characters, physical-mechanical properties, granule composition, content of easy-dissolved salts and organic matter, stability against water, expansibility, contractibility, slope stability, suitability of filling materials and filling technique, bearing capacity of foundations and etc.

RÉSUMÉ: La communication décrit la recherche sur les propriétés géologiques et géotechniques de la formation Nihewan, située au bassin Datong-Yangyuan qui a été conduite pendant la construction du chemin de fer Datong-Qinhuangdao. Elle contient des informations sur les conditions lithologiques, les propriétés physico-mécaniques, la composition granulométrique, la matière organique, le gonflement et leur répercussion en problèmes de stabilité de pentes, d'exécution de remblais, de fondations, etc.

1 INTRODUCTION

The Nihewan bed is well-known both at home and abroad, and has been studied with a long history. A lot of researchers from China and abroad have been engaged in the study of the said bed, and a large number of achievements in scientific research have been acquired, and a quantity of articles were issued. However, the most centred attention on the stratigraphy, with only a little attention paid to the features of engineering geology.

During the early 1980s, a systematic study of the Nihewan bed (between Datong and Yangyuan, with a length of nearly 100 km) was conducted in line with the requirements of construction of Datong-Qinhuangdao Railway as well as the characteristics of railway engineering. As a result, initial achievements were reaped in respect of engineering geological features, and a scientific basis for railway engineering design and construction was acquired thereby. The paper describes in considerable detail, the engineering geological features in regard of the top layer (30-60m) in Nihewan bed, and the characteristics of the engineering geology is evaluated.

2 REGIONAL SURVEY

The studied area, situated in the Datong-Yangyuan Basin, is part of Datong city and Datong county of Shanxi province, and Yangyuan county of Hebei province; and it is a larger basin among the intermontane basins of north-east structural genesis between Taihangshan and Daqingshan mountains.

The basin is surrounded with undulating hills, and presents itself in the direction of N60°E. It is of trough-shaped, with a south-north width of 20-odd km, and east-west length of 100-odd km. Its surface relief features a gradual decline from west to east, and a slight south-north inclination from the fore part of the mountain to the Sangganhe River which flows southwards through the south part of the basin. There are scattered volcanic cones and mound basalt flow on the middle plain of the north bank of the Sangganhe River. The basin is about 1000 m above sea level. The strata cropping around the basin...
mainly include the following: Kata-metamorphic rock of Archean group, sand-shale and carbonatite of Sinian system, carbonatite of Cambrian and Ordovician systems, sandstone, shale and coal bed of Jurassic system and so on. The surface layer in most part of the basin is composed of Malan loess of Pleistocene series Quaternary system. The loess is of different thickness from 1.0 m to 3.0 m. And the Nihewan bed is supposed to be below the loess. Within some areas of the basin, the Nihewan bed outcrops directly on the terrene, riverbank and gully slopes.

3 CHARACTERISTICS OF STRATA LITHOLOGY IN NIHEWAN BED

The Nihewan bed is chiefly of lake deposit, with river alluviation and intercalated pyroclastic accumulation or lava flow. According to the scholars, the initial period for the formation of strata could be the early Quaternary period, however, their views are divergent on the concluding period. In line with the strata lithology and the engineering geological features, the author holds that it would be advisable if the upper limit of Nihewan bed would be defined below the Malan loess. Due to impacts such as the lake water depth, distance between riverbank and the appropriate location, material from other place, hydrology, climate, tectonism and volcanism in Nihewan bed, its lithology differs greatly both vertically and horizontally. Generally, where it is closer to the riverbank or is affected by river flowing into lake basin, the particle size of accumulational material would be much larger and with more sand. The lithology statistics of drilling cores sampled from 2000-odd m are as shown in Table 1.

Table 1. The lithology statistics

<table>
<thead>
<tr>
<th>Area</th>
<th>Hanja area</th>
<th>Yube area</th>
<th>Fache area</th>
<th>Yongan area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-sand</td>
<td>3.9</td>
<td>--</td>
<td>1.4</td>
<td>9.4</td>
</tr>
<tr>
<td>soil %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand-clay%</td>
<td>77.6</td>
<td>66.8</td>
<td>46.8</td>
<td>85.5</td>
</tr>
<tr>
<td>Clay %</td>
<td>8.4</td>
<td>8.6</td>
<td>7.1</td>
<td>4.4</td>
</tr>
<tr>
<td>Silt-Fine-Sand %</td>
<td>0.5</td>
<td>2.7</td>
<td>29.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Medium-coars-sand %</td>
<td>8.1</td>
<td>12.8</td>
<td>14.3</td>
<td>0.2</td>
</tr>
<tr>
<td>Gravil-sand %</td>
<td>1.6</td>
<td>9.1</td>
<td>1.2</td>
<td>--</td>
</tr>
</tbody>
</table>

4 GRAIN-SIZE DISTRIBUTION AND DENSITY FOR SAND IN NIHEWAN BED

4.1 The grain-size distribution is listed in Table 2, its cumulative curve for grain-size distribution is as shown in Fig. 1.

Table 2. The grain-size distribution of sand

<table>
<thead>
<tr>
<th>Grain-size group (mm)</th>
<th>&gt;20</th>
<th>20-2</th>
<th>2-0.5</th>
<th>0.5-0.25</th>
<th>&lt;0.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt %</td>
<td>1.0</td>
<td>10.2</td>
<td>12.7</td>
<td>33.9</td>
<td>42.2</td>
</tr>
<tr>
<td>Fine-sand %</td>
<td>2.0</td>
<td>11.5</td>
<td>16.4</td>
<td>47.5</td>
<td>20.6</td>
</tr>
<tr>
<td>Medium-sand %</td>
<td>6.1</td>
<td>25.2</td>
<td>43.4</td>
<td>17.0</td>
<td>8.3</td>
</tr>
<tr>
<td>Coarse-sand %</td>
<td>12.6</td>
<td>46.2</td>
<td>25.9</td>
<td>10.1</td>
<td>5.2</td>
</tr>
<tr>
<td>Gravil-sand %</td>
<td>2.0</td>
<td>38.6</td>
<td>38.9</td>
<td>11.5</td>
<td>6.6</td>
</tr>
<tr>
<td>Gravil %</td>
<td>13.2</td>
<td>49.5</td>
<td>26.0</td>
<td>7.1</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Fig. 1: Curve of grain size distribution of clayey-soil
In the light of Table 2, and Figure 1, silt, fine sand, medium and coarse sand are of uniform size and of suitable grading for the engineering purpose, however, gravel-sand and gravel are of uneven particles and of poor gradation, they mirror the features of intermontane basin.

4.2 Density of sand

The density of sand was determined by standard penetration test (Nps). Most of the sand beds tested were of compact states, and about 50% of the tested experienced 50 hits, but their penetration depth was still less than 0.3 m. For this reason, a conclusion could be drawn that the sand in Nihewan bed is in compact condition.

5 PHYSICAL-MECHANICAL INDEXES FOR CLAYEY SOIL IN NIHEWAN BED

Nearly 400 undisturbed soil samples were tested by laboratory means, and the statistics are as shown in Table 3.

<table>
<thead>
<tr>
<th>Table 3. Physical-mechanical indexes of clayey soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lithology</td>
</tr>
<tr>
<td>Natural moisture ((\omega)) %</td>
</tr>
<tr>
<td>Natural density ((\rho)) g/cm³</td>
</tr>
<tr>
<td>Natural pore ratio ((e_a))</td>
</tr>
<tr>
<td>Liquid limit ((WL)) %</td>
</tr>
<tr>
<td>Plastic limit ((WP)) %</td>
</tr>
<tr>
<td>Liquidity index ((LI))</td>
</tr>
<tr>
<td>Natural quick shear ((\theta)) (°)</td>
</tr>
<tr>
<td>Precrystallization pressure (Kpa)</td>
</tr>
<tr>
<td>Compressibility modulus ((E_s))</td>
</tr>
<tr>
<td>Compressibility coefficient ((a)) (Kpa⁻¹)</td>
</tr>
</tbody>
</table>

The Table 3 were arranged in line with the name of soil and the colour. In the light for Table 3, it is obvious that the soil is of over-consolidated type, and with medium-lower compressibility.

6 CONTENT OF EASY-DISSOLVED SALTS FOR CLAYEY SOIL IN NIHEWAN BED

Testing results derived from 42 soil samples with saline matter are as shown in Table 4. With regard to its content of easy-dissolved salts, this soil is generally classified as clayey soil from the engineering viewpoint.

<table>
<thead>
<tr>
<th>Table 4. Content of easy-dissolved salts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lithology</td>
</tr>
<tr>
<td>Sand-clay</td>
</tr>
<tr>
<td>Ion content ((%))</td>
</tr>
</tbody>
</table>

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7 CONTENT OF ORGANIC MATTER FOR CLAYEY SOIL IN NIHEWAN BED

The testing results obtained from 26 soil samples with organic matter are listed in Table 5.

Table 5. Content of organic material of clayey soil

<table>
<thead>
<tr>
<th>Clay-sand-soil</th>
<th>Sand-clay</th>
<th>Clay</th>
</tr>
</thead>
</table>

Content of organic material: 0.21 0.71 0.16 0.18 0.29 0.20

In respect to its content of organic matter, this soil is generally defined as clayey soil in the engineering field.

8 GRAIN-SIZE DISTRIBUTION, STABILITY AGAINST WATER AND EXPANSIBILITY-CONTRACTIBILITY FOR CLAYEY SOIL IN NIHEWAN BED

8.1 Characteristics of grain-size distribution for clayey soil in Nihewan Bed

The grain-size distribution is listed in Table 6, the cumulative curve for grain-size distribution is as shown in Fig.2.

<table>
<thead>
<tr>
<th>Grain size, mm</th>
<th>0.25-</th>
<th>0.1-</th>
<th>0.05-</th>
<th>0.01-</th>
<th>&lt; 0.005</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0.005</td>
<td>0.005</td>
</tr>
<tr>
<td>Clay-sand-soil</td>
<td>1.0</td>
<td>34.0</td>
<td>46.0</td>
<td>4.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Sand-clay</td>
<td>0.4</td>
<td>16.5</td>
<td>37.5</td>
<td>11.6</td>
<td>34.0</td>
</tr>
<tr>
<td>Clay</td>
<td>0.2</td>
<td>4.2</td>
<td>28.1</td>
<td>19.5</td>
<td>48.0</td>
</tr>
</tbody>
</table>

8.2 Stability against water and expansibility-contractionility for clayey soil in Nihewan Bed

The free expansion ratio determined from 20-odd samples are as shown in Table 7.

Table 7. The free expansion rate of clayey soil

<table>
<thead>
<tr>
<th>Clay-sand-soil</th>
<th>Sand-clay</th>
<th>Clay</th>
</tr>
</thead>
</table>

Free expansion: 28.5 8.7 5.7

In line with the statistics, the clay-sand-soil and sand-clay are of good properties for stability against water, not easily disintegrated with water and without expansibility-contractionility, with the result that they could form steep slopes along both banks of the river or gully. Only 37.5% of the clay comes up to the index of expansion soil, such soil could easily be disintegrated with water, and with poor performance of stability against water, when the ground water level is higher, the outcrop on gully slope is liable to collapse.

9 SLOPE STABILITY AND SLOPE-CUT RATIO IN NIHEWAN BED

Due to the earlier forming period of Nihewan bed, the clayey soil is of higher consolidation degree, and sand is in
compact condition, and with slight partial cementation. For this reason, the natural slope is of steeper grade and of fair stability. However, part of clay is locally featured with certain characteristics of expansion, as a result, such soil is liable to be softened with water, and if it is devoid of water, drying crack or peeling is easy to occur, and collapse is liable to take place when the ground water level is higher. A field survey of natural slope angle was carried out at nearly 40 slopes, and the appropriate statistics are as shown in Table 8.

<table>
<thead>
<tr>
<th>Natural slope</th>
<th>Collapse slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope angle</td>
<td>Slope angle</td>
</tr>
<tr>
<td>Slope ratio</td>
<td>Slope ratio</td>
</tr>
<tr>
<td>Clayey 40° -</td>
<td>1:1.2</td>
</tr>
<tr>
<td>soil</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In line with the abovementioned conditions, the following principles could be applied for the determination of slope-cut ratio while the railway crosses over the clayey soil section in Nihewan bed:
When the soil cut is higher than the capillary-moisture zone, a ratio of 1:1 could be employed; and if it is lower, a ratio of 1:1.5 could be adopted, and slope protection would be provided. As for sand, a ratio of 1:1.5 would be advisable.

10 SEISMIC LIQUEFACTION IN NIHEWAN BED

The abovementioned sand of Nihewan bed is of compact state, and there is no seismic liquefaction phenomenon. The clayey soil is of over-consolidated type with medium-lower compressibility, and there are cohesive particles which are endowed with certain function to check seismic liquefaction in comparison with the sand of corresponding period. For this reason, there is no possibility of seismic liquefaction for the clayey soil in Nihewan bed.

11 BEARING CAPACITY OF NIHEWAN BED

The bearing capacity for sand in Nihewan bed could be determined in line with its density. The clayey soil in Nihewan bed is different from that of the others, which due to its earlier forming period, as it is endowed with larger pre-consolidated pressure and lower compressibility.
In the light of the correlation tests conducted with various means, which were of normal-position-oriented lateral pressure types, the bearing capacity of clayey soil in Nihewan bed could be of distinct coherency when the shear strength is of a normal pressure of 100 Kpa, and its fundamental bearing capacity could be calculated by the following formula:

\[ \sigma = 498.7 \log \tau - 549.1 \]  \hspace{1cm} (1)

\[ \sigma = 3.149 \tau + 3.463 \times 10^{-2} \tau^2 - 3.859 \times 10^{-4} \tau^3 + 0.9 \times 10^{-7} \tau^4 + 14 \]  \hspace{1cm} (2)

Note: 1. Formula (1) is suited for \( \tau = 30 \sim 200 \) KPa
2. Formula (2) is suited for \( \tau = 10 \sim 200 \) KPa
3. \( \sigma \) - Bearing capacity (KPa)
4. \( \tau \) - shear strength when normal pressure is 100 KPa.

12 APPLICABILITY OF FILLING MATERIAL AND FILLING PROCESS FOR CLAYEY SOIL ROADBED IN NIHewan BED

12.1 Applicability of filling material for clayey soil roadbed in Nihewan bed

According to technical standard of filling material for railway roadbed, only clay is unsuit. But the clay in thin layers which covers only 7.1% of Nihewan bed could naturally mixed with the other soil layers when it excavated and filled, thus its filling suitability could be improved. Therefore, the clayey soil of Nihewan bed could be used as filling material below the surface layer of roadbed in general, however, only a small amount of the clay-sand-soil could be utilized for filling the roadbed surface.

12.2 Roadbed filling process for clayey soil in Nihewan bed

Tamping of filling soil is to be the crux for roadbed construction, and it is rather complicated in regard to the factor that could exert an influence on tamping effect, and in the light of in-place tests for embankment filling, there could be four
major factors: soil, water content, compacting machinery and thickness of soil-filling layer. Only when the soil is of optimum water content, could the desirable density be realized. If the compacting machinery were of larger power, better effect could be achieved. For a definite machinery, the thickness of soil-filling layer is to be inversely proportioned to the compacting result. In line with the tests, the optimum thickness of soil-filling layer should be 0.3-0.5 m for varieties of compacting machinery.

According to the analysis for 20 units of tamping tests, the optimum water content, maximum dry unit weight and wet unit weight during the period with optimum water content could vary from soil to soil.

After the compacting tests were dealt with mathematical statistics analysis of regression, and plasticity index could be supposed to be the ordinate, and optimum water content, maximum dry unit weight or wet unit weight during the period with optimum water content could be assumed to be abscissa respectively, then the interrelated graphs, formula and coefficient are as shown in Fig.3, 4 and 5.

Fig.3: The interrelated figure between the plasticity index($I_p$) and the wet density of the best water capacity($\gamma$)

Interrelated formula: $I_p=116.47-50.77\gamma$ (3)

Interrelated coefficient: 0.84

13 CONCLUDING REMARKS

It could be summarized as follows in line with the abovementioned:

13.1 The lithology of Nihewan bed is chiefly composed of clayey soil with sand-clay as its main component, and only 7.1% of clay is of poor engineering characteristics.

13.2 The sand is of compact type, and that of coarser particles is of poor gradation and of non-uniformity in respect of engineering category. The clayey soil is of over-consolidated type and with medium-lower compressibility. There is no seismic liquefaction for the strata.

13.3 In line with engineering requirements, the soil of Nihewan bed is of normal type in regard of its content of easy-dissolved salts and that of organic matter.

13.4 A small amount of clay with yellow-green and gray colour is of expansion type, and with poor properties of stability against water, and it is liable to collapse in such places as bank slopes. The rest is of favourable stability against water. Provided that a proper slope-cut ratio could be adopted, the stability of slope could be maintained.

13.5 The clayey soil could be employed as filling material below the roadbed surface in general. During construction, the optimum water content and thickness of filling soil should be maintained and controlled properly, and compacting machinery and rolling times should be duly determined so that optimum compaction could be realized.

13.6 The bearing capacity for clayey soil of Nihewan bed could be determined in line with the shear strength index.
The relationship between tensile and compressive strengths for selected sandstones as influenced by index properties and petrographic characteristics

La relation entre les résistances à la traction et à la compression pour des grès sélectionnés et l’influence des propriétés indices et caractéristiques pétrographiques

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Environmental Mitigation Group, Stow, Ohio, USA
Abdul Shakoor
Department of Geology and the Water Resources Research Institute, Kent State University, Ohio, USA

ABSTRACT: Based on a study of six Pennsylvanian sandstones from northeast Ohio, it was determined that the ratio of tensile strength to compressive strength for the sandstone rocks varies from 4.5 to 9.4 percent, with an average of 7 percent. The relationship appears to be influenced predominantly by the amount of matrix and cement. The greater the amount of matrix and/or cement, the higher the ratio of tensile to compressive strength. Other variables found to correlate significantly (r = ≥ 0.81) with both types of strength are the percentage of angular grains, percentage of straight grain-to-grain contacts, and the index properties of density and absorption. The results show that the frequently used assumption of tensile strength of rocks being 10 percent of their compressive strength is not always valid.

RÉSUMÉ: L’investigation dans six grès de l’âge Pennsylvanies de l’Ohio nord-est a démontrée que le ratio de la résistance de traction à la résistance compressive est entre 0.045 et 0.094 avec une moyenne de 0.07. La somme de la matrice et du ciment sont très important à la valeur de cette ratio. Le plus de la matrice et du ciment, le plus grande est le ratio de la résistance de traction à résistance compressive. Les autres variables qui correspondent à la niveau de 0.81 où plus avec les deux sortes de la résistance sont les pourcentages des grains angulaires, les attouchements droits entre les grains, et les indices de l’absorption et de la densité. Les résultats montrent que l’assumption simple que la résistance de traction des roches est 10 pour-cent de la résistance compressive n’est pas toujours valide.

INTRODUCTION

The tensile strength and unconfined compressive strength are frequently used properties in rock engineering (Bieniawski, 1974; Jumikis, 1979). The tensile strength of rocks, however, has not been studied as extensively as the compressive strength. This may be attributed to the complicated procedure, and the associated cost, of the direct tensile strength test as specified in American Society for Testing and Materials (ASTM) method D-2936 (ASTM, 1990). In practice, the tensile strength is often assumed to be 10 percent of the compressive strength value (Jumikis, 1979). This assumption, however, is not always correct. The tensile strength can be either more or less than 10 percent of the compressive strength. This is especially true of the sandstone rocks because of the extreme variations in their texture and mineral composition. The purpose of the research presented in here was to determine the relationship between tensile and compressive strengths for selected sandstones, and to investigate the influence of index properties (density, absorption) and petrographic characteristics (grain size and shape, grain-to-grain contact, mineral composition) on the two types of strength.

Previous work on tensile-compressive strength relationship, and the effect of index properties and petrographic characteristics on the two types of strength, has produced somewhat inconsistent results. Dube and Singh (1969) found that the orientation of the bedding planes with respect to stress direction had a marked influence on the tensile strength of disc-shaped specimens. Dube and Singh (1972) also discovered that the tensile strength of sandstones decreased with an increase in humidity and porosity, and increased with an increase in the amount of matrix. Mercier et al. (1985) found the tensile strength of granitic rocks to be inversely proportional to quartz content. In his study of the Fell Sandstones
of England, Bell (1978) found that texture was more important than mineral composition in influencing both the tensile and compressive strengths and that an increase in pore space tended to reduce the strength values. Hartley (1974) found that intergranular bonding and porosity were the main factors affecting strength of sedimentary rocks. Fahy and Guccione (1979) concluded that rocks with smaller mean grain size had higher strength values and that degree of grain-interlocking was more important than mineralogy. Howarth and Rowlands (1986) determined that mechanical properties of rocks were significantly influenced by their texture. Hoek (1965) discovered that higher stresses were required to cause failure in rocks with an interlocking texture. Price (1960) found that compressive strength increased with an increase in quartz content whereas Fahy and Guccione (1979) did not find a strong correlation between quartz content and strength. D'Andrea et al. (1965) found that there was a significant positive correlation between the index properties, notably the density, and the tensile and compressive strengths. Shakoor and Bonelli (1991) found that compressive strength, tensile strength, and Young's modulus values for sandstones were closely related to their density, percent absorption, total pore volume, and type of grain-to-grain contact.

RESEARCH METHODS

Sample Collection and Preparation

Six Pennsylvanian sandstones including the Buffalo sandstone, Grafton sandstone, Lower Mahoning sandstone, Saltzburg sandstone, Sharon sandstone, and Upper Freeport sandstone were collected for this study from northeast Ohio, U.S.A. Large blocks of these sandstones were obtained directly from the outcrops, marking the bedding orientations prior to block removal. The blocks were cored in the lab using an NX-size (2 1/8" or 5.4 cm) coring bit. All cores for compression testing were drilled perpendicular to the bedding, cut to have a length-to-diameter (L/D) ratio of 2.0-2.5, and lapped to meet the specifications outlined in ASTM method D-2938. The disc-shaped specimens for the tensile strength test were cut in two orientations, one parallel and the other perpendicular to the bedding (Figure 1). Ten cores and ten discs in each orientation were prepared from each sandstone, ten being the minimum number of test specimens required for best estimates of average strength values (Yamaguchi, 1969; ASTM, 1990). All cores and discs were oven dried at 105°C for 24 hours before testing.

Figure 1. Direction of applied stress and the resulting failure plane with respect to bedding.

Laboratory Testing

The Brazilian test (ASTM D-3967) was used to measure the tensile strength whereas ASTM method D-2938 was used to determine the unconfined compressive strength for the sandstones studied. The Brazilian test is an indirect test in which a disc-shaped specimen is compressed to failure, inducing tensile stress in the disc perpendicular to the loading direction. For this study, the test was performed with the applied load acting both perpendicular and parallel to the bedding orientation.

In addition to the two types of strength, index properties of specific gravity, absorption, and density were determined for three samples of each sandstone, and the average values were obtained. The specific gravity and absorption values were determined using ASTM method C-97 whereas bulk density was computed by multiplying the specific gravity with the density of water.

Petrographic Examination

Three thin sections of each sandstone, cut in three mutually perpendicular directions, were examined to study the petrographic characteristics. Grain size was determined using the technique described by Hutchinson (1974). Grain shape was evaluated visually using a chart developed by Pettijohn et al. (1984). The chart identifies five categories: angular, sub-angular, sub-rounded, rounded, and well-rounded. A modal analysis was performed on each thin section, according to the procedure proposed by Hutchinson (1974), to compute the percentages of matrix, cement, and hard and soft grains in each sandstone. The purpose of the modal analysis was to
determine the general composition, not the mineral composition. For the mineral composition, the grains counted were categorized as quartz, feldspar, mica, rock fragments, and heavy minerals. The grain-to-grain contacts were categorized using a classification scheme developed by Blatt (1982). The classification recognizes four types of grain-to-grain contacts including the point or tangential, straight or long, concavo-convex, and sutured contacts (Figure 2). The point contacts represent the lowest strength and sutured contacts are expected to yield the highest strength.

Figure 2. Nature of grain-to-grain contacts (after Blatt, 1982).

The statistical program SYSTAT\textsuperscript{®} was used to determine the mean, standard deviation, and variance values for each measured property. A regression analysis was performed, using SYSTAT\textsuperscript{®}, to determine the relationship between tensile and compressive strengths, with tensile strength considered as the dependent variable. In addition, the index properties (density, absorption) and petrographic characteristics (grain size, grain shape, percent cement, percent hard or soft grains, type of grain-to-grain contact) were also correlated with the strength values using a linear regression model. In these correlations, the strength values were the dependent variables. The strength of each relationship was measured in terms of the correlation coefficient (r) value. The r value must be > 0.81 for a correlation to be statistically significant at the 95 percent confidence level. However, an r value of 0.70 is often considered as the lower limit for estimation purpose in engineering practice (Fahy and Guccione, 1979). Therefore, all correlations with r values in excess of 0.70 were tabulated and graphed.

RESULTS

Engineering Properties

Tensile Strength: Table 1 summarizes the engineering properties of the sandstones tested. The Sharon sandstone has the lowest mean tensile strength of 374 psi (2579 kN/m²) whereas the Salzburg sandstone exhibits the highest mean tensile strength of 1,966 psi (13,558 kN/m²), when tested perpendicular to the bedding. When the applied stress is parallel to the bedding, the Sharon sandstone again has the lowest mean tensile strength of 268 psi (1,848 kN/m²) and the Salzburg sandstone has the highest mean tensile strength of 1,465 psi (10,103 kN/m²). In general, the tensile strength with the applied stress parallel to the bedding is lower than that with the applied stress perpendicular to the bedding. These results are similar to those reported by Dube and Singh (1969) who also found lower tensile strength values parallel to the bedding. The two exceptions are the Grafton sandstone and the Upper Freeport sandstone both of which have higher tensile strength values when tested parallel to the bedding. This may be attributed to the highly cross-bedded nature of these sandstones which made it almost impossible to cut the discs truly parallel or perpendicular to the bedding.

Table 1. Mean values of tensile strength, compressive strength, bulk density, and absorption.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Mean Tensile Strength (psi)</th>
<th>Mean Compressive Strength (psi)</th>
<th>Bulk Density (pcf)</th>
<th>Absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>374</td>
<td>5161</td>
<td>131.7</td>
<td>6.6</td>
</tr>
<tr>
<td>Grafton</td>
<td>357</td>
<td>6039</td>
<td>137.3</td>
<td>5.4</td>
</tr>
<tr>
<td>Lower</td>
<td>685</td>
<td>7273</td>
<td>146.3</td>
<td>4.1</td>
</tr>
<tr>
<td>Meamling</td>
<td>689</td>
<td>7377</td>
<td>139.2</td>
<td>5.1</td>
</tr>
<tr>
<td>Upper</td>
<td>437</td>
<td>550</td>
<td>132.2</td>
<td>5.1</td>
</tr>
<tr>
<td>Freeport</td>
<td>903</td>
<td>1460</td>
<td>151.6</td>
<td>2.8</td>
</tr>
<tr>
<td>Salzburg</td>
<td>1966</td>
<td>3207</td>
<td>166.6</td>
<td>0.7</td>
</tr>
</tbody>
</table>

* 1 psi = 6.895 KPa
** 1pcf = 0.062 Mgm³

495
Unconfined Compressive Strength: Table 1 also shows the results of compression testing. The mean compressive strength ranges from a low of 5,163 psi (35,604 kN/m²) for the Sharon sandstone to a high of 22,259 psi (153,500 kN/m²) for the Saltzburg sandstone.

Index Properties: The Sharon sandstone exhibits the lowest bulk density (131.7 pcf or 2.1 Mg/m³) and the highest absorption (6.6%) whereas the Saltzburg sandstone has the highest bulk density (165.6 pcf or 2.65 Mg/m³) and the lowest absorption (0.7%) (Table 1). These data suggest a possible correlation between index properties and the corresponding strength values.

Petrographic Characteristics

Grain Size and Grain Shape: The results of grain size and grain shape analyses are given in Table 2. It should be noted that only quartz grains were measured for grain size and grain shape analyses because of their abundance and ease of identification. The Sharon sandstone has the largest mean grain size of 0.344 mm and the Saltzburg sandstone has the smallest mean grain size of 0.101 mm (Table 2). Also, Sharon sandstone has the lowest proportion of angular grains (9.6 percent) and the highest proportion of rounded grains (90.4 percent) whereas the Saltzburg sandstone has the maximum number of angular grains (48.7 percent) and the minimum number of rounded grains (51.3 percent) (Table 2).

Table 2. Results of grain size and grain shape analyses.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Mean grain Size (mm)</th>
<th>Angular Grains (%)</th>
<th>Rounded Grains (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>0.344</td>
<td>9.6</td>
<td>90.4</td>
</tr>
<tr>
<td>Grafton</td>
<td>0.294</td>
<td>21.8</td>
<td>78.2</td>
</tr>
<tr>
<td>Lower Mahoning</td>
<td>0.146</td>
<td>20.5</td>
<td>79.5</td>
</tr>
<tr>
<td>Upper Freeport</td>
<td>0.208</td>
<td>23.1</td>
<td>76.9</td>
</tr>
<tr>
<td>Buffalo</td>
<td>0.266</td>
<td>21.8</td>
<td>78.2</td>
</tr>
<tr>
<td>Saltzburg</td>
<td>0.101</td>
<td>48.7</td>
<td>51.3</td>
</tr>
</tbody>
</table>

Mineral Composition: Table 3 shows the mineral composition of the sandstones studied. The quartz content ranges from 80.1 percent for the Upper Freeport sandstone to 94.2 percent for the Sharon sandstone. Based on mineral composition, and using the Folk's classification (Folk, 1974), the Sharon sandstone can be classified as a quartzarenite, Saltzburg sandstone as a sublitharenite, and the remaining four sandstones as subarkoses.

Table 3. Results of mineral composition analysis.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Quartz (%)</th>
<th>Feldspar (%)</th>
<th>Mica (%)</th>
<th>Rock Fragments (%)</th>
<th>Unknown (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>94.2</td>
<td>4.4</td>
<td>0.7</td>
<td>0.7</td>
<td>0.0</td>
</tr>
<tr>
<td>Grafton</td>
<td>82.9</td>
<td>6.3</td>
<td>6.1</td>
<td>4.6</td>
<td>0.0</td>
</tr>
<tr>
<td>Lower Mahoning</td>
<td>78.1</td>
<td>12.0</td>
<td>3.8</td>
<td>5.9</td>
<td>2.1</td>
</tr>
<tr>
<td>Upper Freeport</td>
<td>80.1</td>
<td>10.3</td>
<td>9.0</td>
<td>9.6</td>
<td>0.9</td>
</tr>
<tr>
<td>Buffalo</td>
<td>89.2</td>
<td>7.2</td>
<td>1.8</td>
<td>1.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Saltzburg</td>
<td>80.1</td>
<td>3.2</td>
<td>10.9</td>
<td>4.1</td>
<td>2.3</td>
</tr>
</tbody>
</table>

Cement and Matrix: Table 4 shows the percentages of hard and soft grains, matrix, and cement in each sandstone, as determined by the modal analysis. The Lower Mahoning sandstone has the highest percentage (19.2 percent) of matrix and the Saltzburg sandstone has the highest percentage (50.9 percent) of cement. By contrast, the Sharon sandstone has no cement and very little (1.4 percent) matrix. Both cement and matrix tend to bind the grains together thereby adding to the rock strength.

Grain-to-Grain Contact: Table 5 shows that the Saltzburg sandstone has the highest percentage (56.2 percent) of point contacts, Sharon sandstone has the highest percentage (64.5 percent) of straight contacts, the Buffalo sandstone has the most (11.5 percent) concavo-convex contacts, and the Upper Freeport sandstone has the highest percentage (47.0%) of sutured contacts.

Table 4. Results of modal analysis.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Hard Grains (%)</th>
<th>Soft Grains (%)</th>
<th>Matrix (%)</th>
<th>Cement (%)</th>
<th>Matrix + Cement (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>81.8</td>
<td>9.9</td>
<td>1.4</td>
<td>0.7</td>
<td>1.4</td>
</tr>
<tr>
<td>Grafton</td>
<td>64.0</td>
<td>9.9</td>
<td>8.2</td>
<td>1.8</td>
<td>10.0</td>
</tr>
<tr>
<td>Lower Mahoning</td>
<td>61.1</td>
<td>17.3</td>
<td>19.2</td>
<td>0.1</td>
<td>19.3</td>
</tr>
<tr>
<td>Upper Freeport</td>
<td>69.4</td>
<td>9.1</td>
<td>18.7</td>
<td>2.4</td>
<td>21.1</td>
</tr>
<tr>
<td>Buffalo</td>
<td>79.0</td>
<td>3.3</td>
<td>4.5</td>
<td>7.2</td>
<td>11.7</td>
</tr>
<tr>
<td>Saltzburg</td>
<td>38.3</td>
<td>7.2</td>
<td>3.5</td>
<td>30.8</td>
<td>54.4</td>
</tr>
</tbody>
</table>

Table 5. Frequency of different types of grain-to-grain contacts.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Point (%)</th>
<th>Straight (%)</th>
<th>Concavo-Convex (%)</th>
<th>Sutured (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>12.4</td>
<td>64.5</td>
<td>10.5</td>
<td>12.6</td>
</tr>
<tr>
<td>Grafton</td>
<td>9.2</td>
<td>61.3</td>
<td>7.6</td>
<td>22.0</td>
</tr>
<tr>
<td>Lower Mahoning</td>
<td>13.7</td>
<td>57.3</td>
<td>2.2</td>
<td>26.8</td>
</tr>
<tr>
<td>Upper Freeport</td>
<td>3.7</td>
<td>43.7</td>
<td>6.6</td>
<td>47.0</td>
</tr>
<tr>
<td>Buffalo</td>
<td>0.8</td>
<td>44.5</td>
<td>11.5</td>
<td>43.2</td>
</tr>
<tr>
<td>Saltzburg</td>
<td>56.2</td>
<td>19.1</td>
<td>0.9</td>
<td>23.7</td>
</tr>
</tbody>
</table>
RELATIONSHIP BETWEEN TENSILE AND COMPRESSION STRENGTHS, INDEX PROPERTIES, AND PETROGRAPHIC CHARACTERISTICS

Tensile Strength versus Compressive Strength

The tensile strength, measured in two mutually perpendicular directions, exhibits a strong linear correlation with the compressive strength of the sandstones studied (Figure 3). The correlation coefficient is slightly higher \( r = 0.97 \) when tensile strength is measured with the applied stress acting perpendicular to the bedding than with the stress acting parallel to the bedding \( r = 0.93 \). The ratios of tensile strength to compressive strength values, expressed as percentages, are shown in Table 6. The table shows that tensile strength values of the six sandstones studied are not 10 percent of their compressive strength values, as is generally assumed. Instead, the tensile strength ranges from 5.9 to 9.4 percent of the compressive strength for those sandstones stressed perpendicular to the bedding, and 4.5 to 8.6 percent for those sandstones stressed parallel to the bedding. The Lower Mahoning sandstone, which has the highest percentage of matrix (19.2 percent) also has the highest ratio (9.4 percent) of tensile strength to compressive strength. This supports the previous findings of Dube and Singh (1972) who found that tensile strength increases with an increase in the percentage of matrix. Overall, the tensile strength to compressive strength ratio is higher (7.2 percent) when the applied stress is perpendicular to the bedding than when it is parallel to the bedding (6.6 percent).

![Graph](image)

Figure 3. Relationship between tensile and compressive strength.

Table 6. Tensile strength expressed as a percentage of compressive strength.

<table>
<thead>
<tr>
<th>Sandstone</th>
<th>Tensile Strength Perpendicular to Bedding</th>
<th>Tensile Strength Parallel to Bedding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharon</td>
<td>7.1</td>
<td>5.2</td>
</tr>
<tr>
<td>Grafton</td>
<td>5.9</td>
<td>6.8</td>
</tr>
<tr>
<td>Lower Mahoning</td>
<td>9.4</td>
<td>8.6</td>
</tr>
<tr>
<td>Upper Freeport</td>
<td>5.9</td>
<td>7.9</td>
</tr>
<tr>
<td>Buffalo</td>
<td>6.2</td>
<td>4.5</td>
</tr>
<tr>
<td>Salzburg</td>
<td>8.8</td>
<td>6.6</td>
</tr>
</tbody>
</table>

Tensile Strength versus Index Properties and Petrographic Characteristics

Table 7 shows the results of regression analysis for tensile strength versus index properties and petrographic characteristics. Only those correlations are reported which are significant at the 95 percent confidence level. Both bulk density and percent absorption are closely related to tensile strength \( r \geq 0.94 \). Among the petrographic characteristics, percentage of cement plus matrix shows the best correlation with tensile strength (Figure 4). The percentage of angular grains in a given sandstone also significantly influences its tensile strength (Figure 5).

Compressive Strength vs Index Properties and Petrographic Characteristics

Table 7 also shows the correlation coefficient values for the significant relationships between compressive strength and index properties or petrographic characteristics. The strongest positive correlation is between compressive strength and bulk density \( r = +0.96 \) whereas the strongest negative correlations are exhibited by percent absorption \( r = -0.96 \) and

![Table](image)

Table 7. Results of regression analysis for tensile and compressive strength versus index properties and petrographic characteristics.
Figure 4. Relationship between tensile strength and percentage of matrix plus cement.

Figure 6. Relationship between compressive strength and bulk density.

Figure 5. Relationship between tensile strength and percentage of angular grains.

Figure 7. Relationship between compressive strength and absorption.

percentage of straight grain-to-grain contacts \( r = -0.93 \). The relationships that bulk density, percent absorption, and percent straight contacts have with the compressive strength are shown in Figures 6, 7, and 8, respectively. Similar results have been reported by Shakoor and Bonelli (1991). Other petrographic characteristics that significantly correlate with compressive strength include percentage of cement \( r = +0.92 \) and percentage of angular grains \( r = +0.88 \) (Table 7).

Figure 9 compares the photomicrographs of the Sharon sandstone (9-a), the Buffalo sandstone (9b), and the Saltzburg sandstone (9-c). The abundance of rounded grains, open voids, and point contacts in the Sharon sandstone (9-a), coupled with the lack of cement, clearly explains why it has the lowest strength of the six sandstones. On the other hand, the high strength of the Saltzburg sandstone can be attributed to its dense texture, abundance of angular grains, and high percentage of calcareous cement (9-c). The intermediate, but relatively high, strength of the Buffalo sandstone is primarily due to the presence of numerous sutured contacts (9-b).
Figure 8. Relationship between compressive strength and percentage of straight grain-to-grain contacts.

PREDICTION EQUATIONS

Stepwise linear regressions were performed to develop prediction equations for estimating tensile and compressive strength values from the known values of index properties and petrographic characteristics. All variables that exhibited $r$ values of $\pm 0.7$, or higher, when correlated with compressive or tensile strength were used in the stepwise regression. This resulted in the development of the following prediction equations:

Tensile Strength ($\perp$) (psi) = $-3471.4 - 15.8$ (percent Angular Grains) $+ 23.3$ (percent Cement) $+ 30.2$ (Bulk Density); $r = 0.99$  

(1)

Tensile Strength ($\parallel$) (psi) = $111.7 - 2.8$ (percent Angular Grains) $- 8.5$ (percent Straight Contacts) $+ 13.9$ (percent Matrix + cement) $+ 5.4$ (Bulk Density); $r = 1.0$  

(2)

Compressive Strength (psi) = $23026 - 3057$ (percent Absorption); $r = 0.96$  

(3)

The equations for predicting tensile strength from the known compressive strength, as indicated by the relationship shown in Figure 3, are:

Tensile Strength ($\perp$) (psi) = $-142 + 0.09$ (Compressive Strength); $r = 0.97$  

(4)

Tensile Strength ($\parallel$) (psi) = $59 - 0.06$ (Compressive Strength); $r = 0.93$  

(5)

Figure 9. Photomicrographs of the Sharon (9-a), the Buffalo (9-b), and the Saltzburg (9-c) sandstones. Notice the abundance of rounded grains, open voids, and point contacts in the Sharon Sandstone (9-a) which explains its low strength. The dense texture, abundance of angular grains, and presence of cement, as seen in photomicrograph 9-c, contribute to the high strength of the Saltzburg sandstone. The intermediate strength of the Buffalo sandstone is due to the presence of numerous sutured contacts (9-b).
CONCLUSIONS

The conclusions of this study can be summarized as follows:

1. Both tensile and compressive strengths increase with increasing bulk density and decrease with increasing absorption.

2. Both tensile and compressive strengths increase with decreasing grain size, decreasing percentage of straight grain-to-grain contacts, increasing percentage of angular grains, and increasing percentage of matrix and/or cement.

3. With a few exceptions, the index properties and petrographic characteristics affect the tensile and compressive strengths in similar manners.

4. The tensile strength for the sandstones studied is approximately 7 percent of their compressive strength.

REFERENCES


Collapse of a compacted clay under controlled suction
Collapsus d’une argile tassée sous une succion contrôlée

O.M. Vilar
University of São Paulo, São Carlos, Brazil

ABSTRACT: This paper describes the results of laboratory tests carried out to clarify the influence of matric suction reduction in the gradual collapse observed on a compacted clayey soil. Both conventional oedometer tests where the sample was flooded after loading and suction controlled oedometer tests were performed. Partial collapses under matric suction reduction are analyzed and it is shown that wetting-induced strains were more significant at the lower suctions and depend on overburden pressure. For the lower overburden pressures the soil tended to swell, while for the larger pressures the strains were due to collapse of the soil structure. It is also shown that deformations at zero suction are closer to the results obtained when the samples were inundated in the conventional oedometer test.

RESUMÉ: Cette communication décrit les résultats des essais réalisés en laboratoire pour montrer l’influence de la réduction de la succion sur l’effondrement graduel remarqué dans un sol argileux compacté. On a pratiqué les essais oedométriques conventionnels, dans lesquels l’échantillon était inondée après être chargée, et aussi les essais oedométriques avec succion contrôlée. On a analysé les effondrements partiels obtenus par la réduction de la succion et on a montré que les déformations induites étaient plus importantes sous suctions faibles et qu’elles dépendent de la charge appliquée à l’échantillon. Sous des charges faibles, le sol a eu la tendance de se gonfler, alors que sous des contraintes plus fortes les déformations étaient dues à l’effondrement de la structure du sol. On a aussi montré que sous succion nulle les déformations sont pareilles aux résultats obtenus lorsque les échantillons étaient inondées dans l’essai oedométrique conventionnel.

INTRODUCTION

Wetting-induced volumetric strains at constant overburden pressure can occur in a variety of natural and compacted non saturated soils. When a soil swells by the addition of water it is known as expansive or swelling soil, while when it densifies one says that it has suffered a collapse being a collapsible soil.

The physical reasons for each of the phenomena are different. Expansive soils are related to clay minerals that swell in the presence of water, while collapsible soils are characterized by an open unstable structure, where soil or aggregate particles are held in their place by matric suction or any kind of temporary bonding agent, such as soluble cements.

When water is added, the matric suction is reduced and the bonding agents may be softened or even eliminated, thus causing shear failures at the interaggregate or intergranular contacts as well as volumetric decreases (Dudley, 1970).

Compacted soils can show both swell or collapse, depending on their compaction characteristics and overburden pressure. Poorly compacted soils, in a loose state and/or drier than optimum are those immediately associated to collapse phenomenon, (Barden & Sides, 1969; Collins & Mc Gown, 1974), but even in well compacted fills the problem can arise, when overburden pressures are large (Lawton et al., 1989).

As it is well-known, the main laboratory test for identification of collapsible soils is the standard oedometer test where the sample is loaded to a stress
of interest and then soaked. To inundate the sample is probably the worst condition for a collapsible soil. However it must be recognized that there are many situations where soils will not reach total saturation, but only changes in moisture from its natural or as-compactcd moisture content (Alonso et al., 1987). This is specially true in arid and semi-arid regions and in compacted fills not built for water reservation.

Increasing moisture content, without reaching saturation, causes reduction in matric suction that can be responsible for relevant deformations in a soil, as it was shown by Escario & Saez (1973).

This paper describes the results of laboratory tests carried out to clarify the influence of matric suction reduction in the gradual collapse observed on a compacted clayey soil. The tests were conducted in a suction controlled oedometer where the suction is imposed to the sample according to the axis translating technique.

SOIL AND METHODS

The soil used in the investigation was a clay which show expansive characteristics in spite of its low content of montmorillonite clay minerals, being 42 meq/100 g its Cation Exchange Capacity as reported by Presa (1982). The liquid and plasticity limits were 71% and 36%, respectively, and shrinkage limit was 22%. Standard Proctor (ASTM D698) parameters were maximum dry density of 1.325 g/cm³ and optimum water content of 33.6%, and particle density was 2.70 g/cm³.

The samples were statically compacted in a ring having 70 mm in diameter and 20 mm in height, to a dry density of 1.23 g/cm³ and water content of 22%.

The test program consisted of conventional oedometer tests were the sample was soaked after loading and suction controlled tests, using the apparatus developed by Escario (Escario & Saez, 1973) which is shown in Figure 1.

This oedometer is based on the axis translating technique (Hilf, 1956) and is similar to the pressure plate apparatus used in soil physics for determining the moisture retention characteristics of soil.

To apply the desired suction, air pressure is raised inside the chamber. This air pressure will act too in the air present in the voids of the soil. The base of the soil specimen is in contact with water at atmospheric pressure through the cellulose membrane and inferior porous stone. As the cellulose membrane is impermeable to air, but not to water, there is a flow of water until equilibrium. At this time the suction is equal to air pressure, since suction corresponds to the difference between air and water pressures, and in this case water pressure is zero (water is at atmospheric pressure).

After compaction of specimen and mounting of apparatus, an initial overburden pressure of 40 kPa and the as-compactcd suction (3500 kPa) were applied to the sample. When equilibrium under the applied suction was reached, the sample was loaded to an overburden pressure of interest, then it was subjected to reduction in matric suction in stages, and the corresponding changes in height of the sample were measured.

Figure 1: Suction Controlled Oedometer (Escario & Saez, 1973)

To optimize the time available for testing it was decided that each stage of suction could take four or five days, unless some special condition could occur. In this case, as when suction was reduced to zero, the stage continued for nine to ten days.

CONVENTIONAL OEDOMETER TESTS

Figure 2 shows the compression curves obtained in conventional oedometer tests where the samples
Figure 2: Strain versus overburden pressure curves for soaked after loading collapse tests

were soaked after load. \( H_0 \) is the initial height of the sample and \( H_i \) is the height at the end of each stage of load. Total overburden pressures when the samples were inundated ranged from 80 kPa to 1280 kPa. Assuming that the sample soaked under 80 kPa of overburden pressure corresponds to the saturated condition it can be noted that this curve corresponds approximately to an inferior boundary of collapse strains.

It is worthy to note that depending on overburden pressures the soil can swell or densify by the addition of water. Collapse starts at overburden pressures larger than 160 kPa and collapse strains increase with overburden pressures at least until the larger overburden stress used in the tests (1280 kPa).

A similar behavior is observed in some lateritic soils from Brazil when air dried (Vilar et al., 1985), but it must be stressed that this soil tested at its natural moisture content shows collapse strains increasing with overburden pressures until a determined value. After that, the strains decrease (Vargas, 1973; Vilar, 1979) in a different way than that observed in the compacted soil under study.

**SUCTION CONTROLLED TESTS**

Figure 3 summarizes the results obtained in the suction controlled tests. In this figure the strains, referred to the height \( H_d \) the sample reached after equilibrium under load and initial suction, are plotted against suction for the overburden pressures used in the tests.

As it can be seen, for the lower surcharges (less than 160 kPa) the specimens tended to swell, irrespective of suction decrease. In the early stages of matric potential reduction (from 3.5 to 1.5 MPa) the deformations were negligible, but when the suctions were lesser than 1.5 MPa they increased appreciably.
reaching a change in height of about 2.5% (at zero suction) for the specimen at 80 kPa of overburden. Considering the larger surcharges (inferior to 600 kPa) it can be noted that until suction was not reduced below 1 MPa, the samples tended to swell in a way similar to that observed on the lower surcharges. When suction is reduced to 0.5 MPa the soil tend to collapse and collapse strains are crescent reaching its maximum at zero suction.

For the surcharge of 640 kPa, the strains were of collapse nature irrespective of matric potential. As can be noted, strains at zero suction are crescent with overburden stresses. These results show how collapse strains developed under a gradual decrease in matric potential. So, depending on overburden stress, strains due to collapse can be relevant when soil is wetted but without reaching total saturation.

In Figure 4 wetting induced deformations obtained in conventional oedometer tests and in suction controlled oedometer are compared. Deformations under controlled suction plotted are those related to zero suction and it can be seen the good agreement between values measured in one and other test. A similar behaviour is related by Justo et al. (1984) concerning swelling deformations Some factors can be responsible for the behavior observed. One is referred to the deformations commanded by expansive clay minerals and the others are related to soil structure (particle or aggregate arrangement; size and distribution of pores), to the volume of water that reaches the sample and the overburden stress. When few water reaches the sample swelling effects prevail although the volume of pores is large. If overburden pressures are not large, soil structure is able to withstanding the loads even at zero suction and swelling takes place.

Figure 3: Effect of suction decrease on strains at constant overburden pressures
When overburden pressures are higher and more water reaches the sample, collapse strains prevail. Large loads overcome the tendency for swelling and soil structure collapses since it cannot sustain these loads without the aid of high values of matric suction. It can be expected that as void ratio decreases, collapse will be more difficult to occur even at large loads. The proximity between particles or aggregates will be little, dry density will be large and expansive forces will dominate, since expansive stresses are influenced by dry density (Chen, 1975).

Figure 4: Comparison between wetting induced deformations measured in conventional oedometer test and in controlled suction oedometer (at zero suction).

Figure 5 shows the result of a test where the specimen was subject to cycles of wetting and drying. When the sample is wetted for the first time (suction decrease) collapse takes place. When drying to 1 MPa of matric potential a considerable volume reduction took place. This reduction cannot be assigned only to shrinkage of soil, since an undesirable compression have occurred immediately after air pressure was raised in this test. But, what is of interest is the void ratio at this stage, much lower than initial void ratio. Now, when the sample is wetted again, there is a tendency for swelling. The amount of swelling is not large, since the high overburden pressure prevent large volume increase. In the other cycles the behavior is similar, but shrinkage is lesser.

![Graph showing strain vs. total stress and overburden stress](image)

CONCLUSION

Poorly compacted soil of expansive nature can both expand or swell depending on overburden pressures and on the level of matric potential reduction.

For low surcharges (less than 160 kPa) the soil showed an expansive behaviour both in conventional oedometer test and in suction controlled test. For higher pressures, when matric potential is higher (above 1.5 MPa) there is a tendency of swelling, but when suction is reduced below that value, gradual collapse takes place with decreasing suction. Strains
due to gradual wetting of the soil are maximum at zero suction and the values obtained at this condition in controlled suction tests are closer to that obtained in conventional oedometer tests.

When overburden stresses were larger than 640 kPa strains due to water addition were of collapse nature, irrespective of matric potential.

ACKNOWLEDGEMENTS

The author is indebted to Drs. Jesus Saez Auñon; Ventura Escario and Carlos Oteo Mazo for their suggestions and for the facilities put at his disposal at the Laboratorio de Geotecnia - CEDEX in Madrid, Spain, where the tests were performed.

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Effet de la teneur en calcite sur la résistance résiduelle d’une argile monominérale
Effect of calcite content on the residual strength of a monomineral clay

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RESUME: Des études récentes ont montré que la différence dans l’angle résiduel $\varphi_r$ peut être lié au type de minéral argileux et à la chimie de l’eau interstitielle. La modification d’un de ces paramètres avait une influence directe sur la valeur de l’angle résiduel. Dans la présente étude, un autre paramètre (la teneur en calcite) est introduit. Cette étude est faite sur une argile monominérale en utilisant le cisaillement annulaire. Les essais ont montré qu’il existe une relation inverse entre les limites d’Atterberg et la teneur en calcite. Il a été aussi montré que l’angle résiduel augmentait avec l’augmentation du taux de calcite.

ABSTRACT: Recent studies have shown that this difference in the residual angle of friction is more linked to the clay mineral and the chemistry of the pore water. Modification of one of these factors has a direct influence on the value of the residual angle of friction. In the present paper, another factor is introduced and studied namely the calcite content. This study has been conducted using the ring shear method, the soil was a natural clay commercially available (kaolinite). The testing has shown that an inverse relationship links the Atterberg limits and the calcite content. Moreover, it was found that the residual angle of friction increased as the calcite content increased.

I INTRODUCTION

Le phénomène de la réactivation saisonnière des glissements de terrains et les problèmes de l’altération à long terme et de la réduction de la résistance au cisaillement des pentes argileuses, du fait des changements environnementaux, n’ont pas été très clairement compris et expliqués depuis longtemps. Ces dernières années, une attention particulière a été dévolue à l’explication de ces phénomènes, notamment par la tentative de mieux évaluer les facteurs constants ou dynamiques qui influencent la résistance au cisaillement résiduelle des sols argileux. Kenney (1967, 1977) peut être considéré comme le premier à avoir montré l’influence critique de la minéralogie des argiles sur la résistance au cisaillement résiduelle. Il a notamment montré que l’angle de cisaillement résiduel d’un sol, contenant un taux élevé de particules argileuses de faibles dimensions (montmorillonite), était plus faible que celui d’un sol contenant un minéral
argileux de dimension plus grande (kaolinite) ou des particules non argileuses. Chattopadhyay (1972) a trouvé que les particules de sol de forme plates donnaient des angles de frottement résiduels de faibles valeurs. Alors que les sols ayant des particules en forme d'aiguilles ont des angles de frottement résiduel élevés. Les minéraux d'argile peuvent, en général, différer par la taille, la surface spécifique et par leurs caractéristiques physico-chimiques. Mitchell (1976) a montré que l'augmentation de la surface spécifique du minéral argileux correspond à une réduction de la taille des particules et du coefficient de frottement du minéral. Townsend & Gilbert (1974) ont montré que l'angle de frottement résiduel d'un sol argileux croît de manière remarquable quand son indice de plasticité se situe en deçà d'une certaine valeur seuil. Vaughan & Walbancke (1975) et Vaughan et al. (1978) ont montré que le facteur qui influence l'indice de plasticité d'une argile naturelle d'activité moyenne est la proportion de minéraux argileux de forme plates dans le sol étudié. Lupini et al. (1981) ont relié le mécanisme de cisaillement des sols argileux à la forme et à l'orientation des particules d'argiles. Ainsi le mode de cisaillement turbulent est associé à une forte proportion de particules arrondies ou de particules plates, sans orientation préférentielle, et à fort coefficient de frottement interne. Le mode de cisaillement de glissement est relié, quant à lui, à des particules de forme plates, à faible coefficient de frottement interne et ayant une orientation très prononcée. Ce dernier mode se caractérise par une surface de cisaillement de faible résistance. Enfin entre ces deux modes extrêmes, il existe un mode intermédiaire appelé le mode transitoire et qui met en jeu, à la fois, le mode turbulent et le mode de glissement. À la lumière de son étude, Moore (1991) a confirmé que la résistance au cisaillement résiduel est influencée par la minéralogie des argiles et par la chimie de l'eau interstitielle constituante. Il a donc conclu qu'on ne peut pas considérer la résistance résiduelle comme une propriété immuable pour un sol donné mais, qu'au contraire, elle peut varier sous certaines conditions, avec le temps. Un des paramètres qui est susceptible de varier en fonction du temps et des conditions environnementales est notamment la minéralogie du sol. La minéralogie peut changer, par exemple, si une réaction chimique se produit entraînant la disparition ou la transformation totale ou partielle des minéraux initiaux. Ce phénomène peut survenir, par exemple, lors de la décalcification des argiles calcaireuses en milieu acide. Un des processus de réaction possible peut être l'attaque de ces sols par de l'acide carbonique qui est considéré comme le solvant le plus important de la calcite. En effet, le gaz carbonique CO₂ présent dans l'atmosphère peut se dissoudre dans de l'eau pour donner de l'acide carbonique CO₃H₂ suivant la réaction chimique:

\[ \text{CO}_2 + \text{H}_2\text{O} \leftrightarrow \text{CO}_3\text{H}_2 \]

Au contact du calcaire CaCO₃, il y'a formation de bicarbonate de calcium Ca(CO₃H₂) qui est un sel soluble dans l'eau:

\[ \text{CO}_3\text{H}_2 + \text{CO}_3\text{Ca} \leftrightarrow (\text{CO}_3\text{H})_2\text{Ca} \]
La réaction globale étant:

\[ \text{CO}_2 + \text{H}_2\text{O} + \text{CO}_3\text{Ca} \rightarrow (\text{CO}_3\text{H})_2\text{Ca} \]

C'est notamment Hawkins & McDonald (1992) qui ont soulevé, dans leurs travaux sur la résistance résiduelle du sol de Fuller Earth à Bath (Grande Bretagne), le problème crucial de la décalcification potentielle des argiles calcarières par attaque acide. Ils ont montré que c'est la fraction calcaire de taille limonéreuse qui a une influence sur l'angle de cisaillement résiduel. Un seuil de teneur en calcite de 28 % a été mis en évidence pour l'argile de Fuller de Bath (composée principalement d'ililité). Si la teneur en calcite est inférieure à ce seuil l'argile se comporte dans le mode de cisaillement par glissement alors que si la teneur en calcite est supérieure au seuil de 28 % c'est le mode de cisaillement transitoire qui l'emporte. De même, ils ont aussi montré que l'indice de plasticité est inversement proportionnel à la teneur en calcite. Un seuil Ip équivalent à la teneur en calcite de 28 % est défini à 40 %.

Le but de cette présente étude est de vérifier les conclusions de Hawkins et McDonald (1992) dans le cas d'une argile monominérale (la kaolinite) et en suivant un processus inverse de celui adopté par ces auteurs c.à.d. d'étudier le "calcifier" progressivement la kaolinite.

2 ETUDE EXPERIMENTALE

Le sol choisi pour la présente étude est de la kaolinite commerciale pure. L'analyse de diffraction aux rayons X a permis de mettre en évidence un pourcentage de kaolinite d'environ 94 % avec la présence de quelques traces d'ililité, de feldspaths et de calcite. De la calcite pure (97,4 %) est ensuite mélangée à la kaolinite pour obtenir des échantillons de kaolinite avec des taux de calcite aux environs de 0, 10, 20, 30 et 40 %. La détermination du taux de carbonate de calcium contenu dans les différents échantillons est réalisée par la méthode de titrage au CO2 à l'aide d'un calcimètre Dietrich Fruehling. L'eau utilisée pour la préparation des échantillons est une eau distillée. Les essais d'identifications sur les différents échantillons préparés sont résumés sur les figures 1, 2 et 3. De plus, le tableau 1 résume les propriétés de ces échantillons préparés pour différentes teneurs en calcite et donne aussi les résultats des essais de cisaillement.

La résistance au cisaillement résiduelle est déterminée par l'utilisation d'un appareil de cisaillement annulaire de type Bromhead (Bromhead, 1979). Pour tous les essais réalisés, une vitesse de cisaillement constante de 0,024 °/min et des contraintes normales de 50, 100 et 150 kPa sont utilisées. Les contraintes de cisaillement sont mesurées à l'aide de deux anneaux dynamométriques dont les comparateurs électroniques sont reliés à un système d'acquisition de données. Le calcul de la résistance au cisaillement résiduelle se fait suivant la méthode décrite par Bromhead (1979). Les échantillons de sol remaniés sont, au préalable, consolidés par étapes jusqu'à l'obtention de la contrainte normale finale désirée. À la fin de la consolidation initiale, on forme un plan de cisaillement en
Tableau 1 : Résultats des essais pour les différents échantillons utilisés.

<table>
<thead>
<tr>
<th>Échantillon</th>
<th>CaCO₃ (%)</th>
<th>φ₀ (%)</th>
<th>φ₀ (%)</th>
<th>Iₚ (%)</th>
<th>% &lt; 2 µ</th>
<th>φ'ᵣ(°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.6</td>
<td>64.0</td>
<td>34.7</td>
<td>29.3</td>
<td>66.0</td>
<td>7.5</td>
</tr>
<tr>
<td>2</td>
<td>10.3</td>
<td>50.5</td>
<td>26.7</td>
<td>23.8</td>
<td>62.5</td>
<td>8.7</td>
</tr>
<tr>
<td>3</td>
<td>19.7</td>
<td>47.0</td>
<td>24.8</td>
<td>22.2</td>
<td>59.0</td>
<td>9.5</td>
</tr>
<tr>
<td>4</td>
<td>31.1</td>
<td>43.8</td>
<td>23.8</td>
<td>20.0</td>
<td>48.0</td>
<td>15.9</td>
</tr>
<tr>
<td>5</td>
<td>41.2</td>
<td>36.6</td>
<td>20.8</td>
<td>15.8</td>
<td>46.0</td>
<td>18.5</td>
</tr>
</tbody>
</table>

Faisant tourner l'anneau inférieur mobile par rapport à l'anneau supérieur plus ou moins fixe de 5 rotations complètes. Les anneaux dynamométriques sont ensuite alignés et mis en contact avec le couvercle supérieur de façon à ce que les axes des anneaux forment avec lui un angle de 90°. L'échantillon est ensuite reconsolidé et cisailé.

3 DISCUSSIONS DES RESULTATS

Les courbes granulométriques des échantillons étudiés (fig. 1) montrent bien que l'augmentation du taux de calcite a pour résultat d'élérer la proportion des particules de taille limoneuse (comprises entre 0.002 mm et 0.074 mm, si on se réfère à la norme ASTM) ou limono-sableuse.
(en considérant la norme AFNOR, c.a.d. les particules comprises entre 0.002 et 0.080 mm). La fraction argileuse (% < 2 µ) chute de 66.0 % (pour 2.6 % de CaCO₃) à 46 % (pour 41.2 % de CaCO₃). La figure 2 montre que les limites d'Atterberg varient de façon inversement proportionnelle à la teneur en calcite (CaCO₃). L'écart entre les limites de liquidité et de plasticité, c.a.d. l'indice de plasticité, diminue, au fur et à mesure, que la teneur en calcite augmente. Cette diminution est plus ou moins linéaire.

En faisant passer la teneur en calcite de 2.6 % à 41.2 %, la limite de liquidité passe de 64.0 % à 36.6 %, la limite de plasticité chute de 34.7 % à 20.8 % et enfin l'indice de plasticité s'abaisse de 29.3 % à 15.8 %. La figure 2 montre le même type de variation que celle trouvée par Hawkins et McDonald (1992). La représentation des limites d'Atterberg sur l'abaque de plasticité de Casagrande (fig. 3), montre que les échantillons ayant une teneur en calcite de 2.6 % et de 10.3 % appartiennent au domaine de haute plasticité alors que les autres échantillons se situent dans le domaine de moyenne et basse plasticité.

La figure 4 représente les variations de l'angle de cisaillement résiduel en fonction de la teneur en calcite. On remarque que l'angle de cisaillement résiduel augmente d'une valeur initiale de 7.5°, pour 2.6 % de teneur en calcite, à une valeur de 18.5°, pour une teneur en calcite de 41.2 %. De plus, de cette figure, il apparaît clairement qu'il existe deux zones distinctes de variation de l'angle de cisaillement résiduel.

La première, correspondant à une teneur en calcite comprise entre approximativement 0 % et 20 %, pour laquelle la variation de l'angle de cisaillement résiduel est faible (2° environ); et une deuxième zone, avec une teneur en calcite supérieure à 20 %, où les variations de l'angle de cisaillement résiduel sont très frappantes, donnant, par exemple,

![Figure 2 Variations des limites d'Atterberg en fonction de la teneur en calcite.](image)

<table>
<thead>
<tr>
<th></th>
<th>Presente étude</th>
<th>Hawkins et McDonald (1992)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \omega_1 )</td>
<td>□</td>
<td>◯</td>
</tr>
<tr>
<td>( \omega_p )</td>
<td>■</td>
<td>●</td>
</tr>
</tbody>
</table>

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une augmentation de l’angle de cisaillage résiduel de 9° quand la teneur en calcite varie de 19,7 % à 41,2 %. Par conséquent, on peut dire qu’il existe une valeur seuil de teneur en calcite, comprise entre 20 % et 30 %, qui marque un changement net du comportement mécanique du sol étudié. Il est à rappeler que Hawkins et McDonald (1992) ont situé cette valeur seuil à 28 % de teneur en calcite pour le sol de Fuller Earth composé principalement d’illite. A titre de comparaison, les valeurs trouvées par Hawkins et McDonald (1992) sont reportées sur la figure 4 et illustrent une variation similaire de l’angle de cisaillage résiduel en fonction de la teneur en calcite. Le seuil en terme d’indice de plasticité est estimé, quant à lui, être situé entre 20 % et 22,2 %, alors que Hawkins et McDonald (1992) le situent pour leur sol à 40,0 %. La fraction d’argile des échantillons nous permet, quant à elle d’émeter des hypothèses sur le mécanisme de cisaillage de ces échantillons (fig.5). Il est intéressant de noter que la correlation présentée par Vaughan et al (1978) montre des discontinuités. Celles-ci peuvent être attribuées aux différents mécanismes régissant la résistance résiduelle. Toutefois, en se référant aux résultats obtenus par Kenny (1977), Vaughan et al (1978) et Lupini et al (1981), on peut dire que les échantillons de 1 à 3 (% CaCO₃ = 2,6 % à 19,7 %) se cisaillent dans le mode à glissement, par contre les échantillons 4 et 5 (% CaCO₃ = 31,1 % et 41,2 % respectivement) se cisaillent dans le mode transitoire, car une faible variation de la granulométrie conduit à une variation sensible de la valeur de l’angle de cisaillage résiduel.
4 REMERCIEMENTS

Cette étude a été réalisée dans le cadre du programme de recherche intitulé: "Investigation des propriétés mécaniques des pentes instables dans la région d'Alger". Ce programme est financé par le Ministère de l'Éducation Nationale, bourse n° J1602/7/93. Les auteurs remercient le LNHC (Alger) pour son excellente collaboration. Le support technique de laboratoire a été assuré par Mr Ali Amer et Miles Aissa Mamia, Ali Chaouche, étudiant(e)s à l'institut de Génie Civil.
BIBLIOGRAPHIE


Measurement of swelling strain of Gault clay from England
Mesure de la déformation de gonflement d’une argile de Gault (Angleterre)

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ABSTRACT: The three dimensional swelling strain of Gault clay samples were determined by using an improved model of three-dimensional swelling apparatus recently developed by British Geological Survey. All Gault clay samples showed significant amount of swelling. It was found that the rate of swelling strain normal to bedding was greater than the rate of swelling strain parallel to bedding. For all samples the greatest rates of swelling occurred during the first two days. After that swelling rate was reduced and small amount of swelling strain developed for 7 to 10 days in an approximately logarithmic fashion. It was found from the X.R.D. results that the Gault clay samples contain kaolinite, illite, small amounts of montmorillonite, calcite and some other accessory minerals. The presence of small amounts of montmorillonite have a marked effect on the swelling strain development of Gault clay.

Résumé: La déformation à trois dimensions des échantillons de l’argile Gault a été déterminée par l’usage du modèle perfectionné de l’appareil de déformation à trois dimensions, récemment réalisé par l’Etude Géologique britannique (British Geological Survey). Il s’est trouvé que le taux de déformation de renflement perpendiculaire à la stratification était plus grand que le taux de la déformation parallèle à la stratification. Pour tous les échantillons, les plus grands taux de renflement se sont produits pendant les premiers deux jours. Après ça, le taux de renflement a réduit et une petite quantité de la déformation de renflement a développé pour 7 jours dans une façon approximativement logarithmique. Il s’est trouvé des résultats des diffractions des rayons X que les échantillons contiennent kaolinite, illite, une petite quantité de montmorillonite, calcite et des autres minéraux accessoires. La présence d’une petite quantité de montmorillonite a un effet important sur le développement de la déformation de renflement.

1 INTRODUCTION

The swelling strain behaviour of a soil or rock is important to know the hydraulic responses of the ground. Great difficulties generally arise when foundation on clays start to swell under a finished engineering work. This paper describes the results of a laboratory investigation made on Gault clay samples by measuring the three-dimensional swelling strain of some undisturbed Gault clay samples collected from three boreholes at different depths. Swelling strain characteristics of a soil may be related to the type of clay mineral, arrangement of particles and the presence of any materials which can cement the particles together (Warkentin & Bozozuk, 1961). Natural moisture content and moisture content after 3D (three-dimensional) swell tests, X-ray diffraction (XRD) analyses have also been carried out on all samples. All the obtained results are compared and evaluated. Undisturbed samples of the Gault clay from different boreholes at different depths have been collected from the British Geological Survey to carry out this work. The borehole location of each borehole sample is shown in Table 1.

2 GENERAL GEOLOGY OF GAULT CLAY

In an undisturbed state Gault clay is a stiff, fissured, dark blue or greyish homogeneous heavy clay with some silty particles. It was deposited under marine conditions during the lower cretaceous period (Garratt and Wale, 1985). Later due to the erosion of Wealden anticline, Gault clay was reexposed at the ground surface. Upper Gault layer was disturbed by Devensian glaciation.
Table 1. Location of samples

<table>
<thead>
<tr>
<th>No.</th>
<th>Depth</th>
<th>Place in England</th>
<th>National grid ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.00-8.45</td>
<td>Eastbourne</td>
<td>TQ6058-0131</td>
</tr>
<tr>
<td>2</td>
<td>17.00-17.45</td>
<td>As above</td>
<td>As above</td>
</tr>
<tr>
<td>3</td>
<td>18.75-19.00</td>
<td>Klondyke</td>
<td>TL5940-7010</td>
</tr>
<tr>
<td>4</td>
<td>62.55-62.80</td>
<td>Arlesey</td>
<td>TL1887-3463</td>
</tr>
</tbody>
</table>

*S* = Sample

...and solifluxion.

3 MINERALOGY

The minerals identified in all the Gault clay samples are listed in Table 2. The detailed mineralogy of the Gault Clay samples has already been described by Hossain (1992).

Table 2. Results of X-ray diffraction

<table>
<thead>
<tr>
<th>Sample no.</th>
<th>Identified minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ill, Kao, Ill-l, Cal, Chl</td>
</tr>
<tr>
<td>2</td>
<td>Kao, Qtz, M, Chl, Cl, Cal, Ill</td>
</tr>
<tr>
<td>3</td>
<td>Ill, Kao, Llo, M, Chl, Qtz, Cal</td>
</tr>
<tr>
<td>4</td>
<td>Kao, Cl, M, Chl, K, Cl, Ill, Qtz</td>
</tr>
</tbody>
</table>

Note: Ill = Illite, Kao = Kaolinite, M = Montmorillonite, Cal = Calcite, Chl = Chlorite, Qtz = Quartz, Cl = Clinohlore, Llo = Liotite, K = Cl = Kaolinite-clinohlore.

In some samples, illite is the most dominant clay mineral but in some samples kaolinite is the most dominant mineral. No halloysite but a small amount of montmorillonite was also found in different samples at different depths. Non-clay minerals such as calcite are commonly present in large quantities.

4 EXPERIMENTAL METHODS

4.1 Apparatus

There are various limitations on the existing methods of determining the swelling strain of a soil sample. Various designs of apparatus have been proposed by various authors for measuring three-dimensional swelling strain under saturated conditions. British Geological Survey developed an improved model of three-dimensional apparatus for measuring the three directional swelling strains developed in an unconfined, undisturbed rock specimen. The main features of the three-dimensional apparatus are shown in figure 1.

![Figure 1](image_url)

Fig. 1 British Geological Survey three-dimensional swell test apparatus

The basic principle of this apparatus is similar to the ISRM (1979) apparatus for the measurement of swelling strain in an unconfined specimen. The shape of the cell is different from the ISRM apparatus. The cell is capable of containing the specimen filled with water. A vertical dial gauge reading 0.002 mm mounted on the top side of the specimen which is perpendicular to the bedding was used to measure the swelling along the vertical axis of the specimen. Two other dial gauges may also be employed at the same time to measure the swelling displacement in two other horizontal directions. The specimen may take the form of a right cube. Porous plates of ceramic ware are generally employed on the three sides of the cubic specimen to allow water access at different sides of the specimen. This apparatus was used in the laboratory for measuring the swelling strains on different samples of the Gault clay.

4.2 Test Specimens

A trimmed cubic shape sample with approximate size length of 60 mm was prepared very carefully from each undisturbed sample. The top and bottom of each trimmed cube was marked. The sample was then placed in the 3-D Swell test apparatus. The 3-D box was flooded to permit swelling. The amount of each swelling strain increment and the corresponding time was recorded to plot the graph of time versus strain percent as described by ISRM (1979) and Harper et al. (1979). Readings were taken until an equilibrium condition was reached. The saturated sample was then used to determine the moisture content.

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5 RESULTS AND DISCUSSION

According to ISRM (1979),

Swelling Strain Index = \( \frac{d}{L} \times 100\% \)

Where \( d \) = maximum swelling displacement recorded during the test, \( L \) = initial thickness of the specimen.

For each sample, axes described as horizontal were obtained approximately in the plane of bedding and axes described as vertical were obtained approximately perpendicular to bedding. Table 3 represents the results of swelling strains of different tests.

Table 3. Results of Swelling strains developed along different directions

<table>
<thead>
<tr>
<th>Sample</th>
<th>N.M.C.</th>
<th>H.C.</th>
<th>Swell Max., Min., Max.</th>
<th>Time after</th>
<th>Horizontal Principal Strain %</th>
<th>Vertical Principal Strain %</th>
<th>Vertical Axis Strain %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>%</td>
<td></td>
<td>days</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>21.42</td>
<td>27.82</td>
<td>9</td>
<td>0.07</td>
<td>0.10</td>
<td>0.32</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>22.78</td>
<td>32.57</td>
<td>7</td>
<td>0.25</td>
<td>0.15</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>26.15</td>
<td>38.64</td>
<td>10</td>
<td>0.18</td>
<td>0.14</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>24.93</td>
<td>30.21</td>
<td>7</td>
<td>0.18</td>
<td>0.13</td>
<td>0.45</td>
<td></td>
</tr>
</tbody>
</table>

Note: N.M.C.=Natural moisture content, H.C.=moisture content, HPS=Horizontal principal strain, VAS=Vertical axis strain, Max.=maximum, Min.=minimum.

The swelling strain versus time curves are shown in figures (2-5). Figures 2-5 are plotted in semilogarithmic form to show the time-strain relationships. All samples swelled for 7-10 days. The rates and magnitudes of swelling for different samples are different. In all samples, the rate of swell normal to bedding exceeded the rate of swell parallel to bedding (figs. 2-5).

The vertical swelling strain was maximum for the sample from a depth of 17.00-17.45m. The minimum vertical swelling strain was observed for the sample from a depth of 8.00-8.45m. (Fig. 6).

For all samples the greatest rates of swelling occurred during the first two days. After that swelling rate was reduced and small amount of swelling strain developed for 7-10 days in an approximately logarithmic fashion (Table 3). No noticeable disintegration was observed during swelling. The ratio of the strain rate in all samples normal to bedding to that parallel to bedding varied from approximately 0.1-0.62. All samples except that from 17.00-17.45m. depth showed consistent vertical swelling strain. The difference between the maximum horizontal principal strains developed in all other samples was substantially less (figure 7). Swelling of all samples completely ceased after approximately 7-10 days. The test results demonstrate that the swelling strain rate normal to bedding is about 3-4 times that observed parallel to bedding.

The laboratory temperature was 20-22°C during the tests. The natural moisture
contents of the Gault clay samples lie between 21% to 26%, whereas after 3D tests moisture content values lie in between 28% to 39% (Table 3).

From the XRD analysis it is evident that the analysed samples do not differ in their mineral types. But there are some differences in the amount of each clay mineral present.

6 CONCLUSIONS

The difference between the moisture contents before and after swelling strain tests was small.

In the three-dimensional swelling strain measurements, all samples swelled for 7-10 days.

The rates and magnitudes of swelling strain for different samples were different. In all samples, the rate of swelling normal to bedding exceeded the rate of swelling parallel to bedding.

For first few days, swelling strain rate was maximum, after that it was reduced.

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The clay minerals identified by XRD were kaolinite, illite, chlorite and small amount of montmorillonite which is associated with chlorite or illite. This small amount of montmorillonite may have a marked effect on the swelling strain development of Gault clay. Calcite and quartz had little influence on the studied swelling strain behaviour of the Gault clay samples.

Clay mineral hydration with the expansion of illite, kaolinite and small amount of montmorillonite occurred during these operations. Therefore, swelling strain development in Gault clay may have a marked effect on the geotechnical behaviour of Gault clay.

ACKNOWLEDGEMENT

The author is indebted to Mr. M.G. Culshaw, Chief Engineering Geologist of the British Geological Survey for his assistance, direction and also for providing samples in this study. The author also expresses his heartiest gratitude and sincere thanks to Mr. A.C. Lumaden and Dr. J. West of the University of Leeds, U.K. for their generous help and valuable suggestions in completing this work.

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Types génétiques des argiles et leur compactage dynamique
Genetic types of the clays and their dynamic compaction

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N. Goro
ISP Nr 3, Tirana, Albanie

RESUME: Il s’agit de l’étude des relations entre les propriétés de compactage et l’origine de quelques types génétiques des argiles, utilisées dans la construction des noyaux et des écrans de barrages des usines hydroélectriques. On trouve quelques relations entre les paramètres de compactage et propriétés physiques des argiles d’un type génétique donné utilisées dans la construction des ouvrages en terre.

ABSTRACT: The generalization of the dependence of the dynamic compaction properties on the origin of their formation of some mineralogical kind of clays, used in the construction of the dam core of hydropower stations is treated here. Some correlations are drawn between compaction parameters and physical properties of a given clay, which is used in the construction of embankments.

INTRODUCTION

Le choix du type d’un barrage dépend beaucoup de la nature du matériau en place ou à proximité du futur ouvrage. Dans le cas des barrages en terre, on utilise largement l’argile dont la quantité et la qualité sont prises en compte dans les projets de construction. Pour ce faire, il faut d’abord évaluer leurs propriétés de compactage et dans le cas échéant, il faut les améliorer. Ces propriétés dépend d’une part de la nature des minéraux argileux, de leur texture et composition et d’autre part de la matrice de la roche. Autrement dit, ces propriétés dépendent des conditions de formation des minéraux argileux. Le but de cet étude est l’établissement des relations probables entre ces propriétés des argiles et leurs conditions de formation.

1. TYPES GENETIQUES ET PROPRIETES GEOTECHNIQUES DES ARGILES.

Les gisements étudiés sont ceux de Qyrsaq (Vau i Dejës), Koman et Floq (Elbasan). Cernicë (B.Curri) et Hjeda (Shkodër).

I. A Qyrsaq on rencontre des argiles formées dans des marécages à partir de l’alteration des dépôts de flysch et flyschoidaux. Leur matrice est constituée de carbonates et/ou d’hydroxydes de fer ou d’aluminium. Leur texture est massive et leur granulométrie très hétérogène. Les grains à diamètre supérieur à 5 mm, constituent de 5 à 15% du poids sec des minéraux. Parfois ce chiffre atteint les 27%.

II. Les gisements de Koman et de Floq sont formés par l’alteration des dépôts flyschoidaux. Il contient des argiles formées par alteration qui sont autochtones ou bien très peu déplacées de leur endroit de formation. A Floq, ces argiles sont déposées en trois couches. La première, qui est en contact avec les roches saines contient jusqu’à 50% de minéraux argileux de diamètre supérieur à 5 mm, diminue progressivement de bas en haut pour atteindre la limite de la troisième couche, où ce chiffre est de 20% du poids sec des minéraux.
III. Les argiles de Cernice sont formées par l'alteration des dépôts conglo-meratiques du Néogène du fossé de Tropoja. Ce gisement est constitué d'argiles rouges à structure macroagréagats et qui contiennent des calcaires non altérés. Le passage des conglomerats aux argiles rouges est graduel.

IV. Le gisement de Njeda est formé dans un étang lacustre, recevant du matériel du fleuve Drini. Les minéraux argileux des gisements en question sont le kaolin et l'illite (fig. 1). Entre autres, ils contiennent un pourcentage élevé d'oxydes de fer et d'aluminium. Le coefficient d'alcalinité est faible (0.11). Le contenu en cations Ca²⁺, Mg²⁺, Na⁺ et K⁺ varie de 9.9 à 13.3 mg/l eq pour 100 gr d'argile sèche. De même la matière organique et les sols dissous se trouvent en faible quantité (respectivement 0.32-1.5% et 0.016-0.036%). Il en résulte que ces argiles sont formées dans un climat continental avec un pH acide (de 6.6 à 7). Les minéraux argileux sont transformés en des minéraux de composition plus simple et plus résistante comme c'est le cas des oxydes trivalentes du fer et de l'aluminium qui constituent l'essentiel de la matrice où sont dispersés les grains d'argile. À Cernice et Fioq ces derniers atteignent le diamètre de 2 mm. De même, les liens structuraux entre les grains d'argile sont très solides grâce au pourcentage élevé du limon (Fe²⁺, Al³⁺), de la faible capacité de l'échange des ions du complexe d'absorption et des pellicules d'eau coloidale qui les entourent.

De par leur composition minéralogique, les argiles des gisements en question ont un faible indice de plasticité:

<table>
<thead>
<tr>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>W₃</td>
<td>44.7-47.7</td>
<td>37.9-38.7</td>
<td>54.0</td>
</tr>
<tr>
<td>W₅</td>
<td>26.9-27.7</td>
<td>22.2-23.0</td>
<td>34.0</td>
</tr>
<tr>
<td>I₇</td>
<td>16.8-20.0</td>
<td>14.7-14.9</td>
<td>20</td>
</tr>
</tbody>
</table>

Malgré la variation de 16.5 à 27.4% de la fraction argileuse, les indices de plasticité des gisements I, II et IV ne varient pas beaucoup l'un de l'autre. Ceci est valable surtout pour la limite inférieure et le nombre de plasticité. La conclusion qu'on en tire c'est que dans les les argiles formées par alter- nation les agregats, absorbent une certaine quantité d'eau comme s'ils étaient désintégrés et donnent une plasticité faible malgré la présence des grains d'argile.

2. COMPACTAGE DYNAMIQUE ET RESULTATS OBTENUS.

Pour étudier l'effet du compactage des argiles en question, elles ont été soumises à des pressions de 50, 62.5, 80 et de 100 t/m³ dans des conditions de masse volumique maximale et de teneur en eau optimale.

La fig. 2 indique que les variations de compactage obtenus avec la même énergie s'expliquent par les caractéristiques géologiques et géotechniques des lits d'argiles.

Ainsi, sur la fig. 2 on observe que pour la même énergie de com- pactage (62.5 t/m³) les argiles formées par l'alteration des dépôts de flysch (Fioq, Njeda), pour une teneur en eau optimale de valeur non élevée (19 à 20.5%) ont une masse volumique sèche de 1660 à 1680 kg/m³. Par contre, pour les argiles de Koman et Qyrsaq, ces valeurs vont de 1680 à 1600 kg/m³ pour une teneur en eau optimale de 24%. Enfin, à Cernica ces valeurs sont respectivement de 1600 kg/m³ et 30%.

Il en résulte que ce minérais argileux hétérogènes dont le pour- centage de l'argile (20-25%) est presque le même que celui du sable et deux fois plus petit que celui de la fraction du limon, sont caracté- risées par une masse volumique sèche élevée et une faible teneur en eau optimale.
Fig. 2. Courbes de compactages des argiles de gisements à l’énergie de 62.5 t-m/m³

D’autre part le compactage les rend très résistants au fractionnement. On peut en déduire que sous la pression élevée exercée sur les minérais argileux, l’argile remplit les vides entre les grains limoneux.

Les minéraux argileux où la fraction argileuse est dominante (cas de Cernica) sont plus résistants face au compactage. De plus après le compactage, ces minéraux deviennent poreux et se fractionnent facilement. Dans le cas des argiles de Hjeda où la fraction des limons est élevée, après le compactage on obtient un sol très poreux qui se fractionne aisément.

de ce gisement sont déposés en trois couches a, b, c (du haut en bas). dont les indices de compactage pour une énergie de 62.5 t-m/m³, (fig. 3) sont les suivants:

<table>
<thead>
<tr>
<th>a</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Y_d$ (kg/m³)</td>
<td>1680</td>
<td>1650</td>
</tr>
<tr>
<td>$w$ (°)</td>
<td>21</td>
<td>22</td>
</tr>
</tbody>
</table>

L’étude montre que les indices de compactage diminuent de haut en bas. Il parait que cette variation de compactage serait dû à la fraction de flysch qui n’est pas affectée par l’alteration et empêche ainsi le compactage maximal des argiles.

Fig. 3. Courbes de compactages des argiles du gisements de Floq à l’énergie de 50, 62.5, 80 et de 100 t-m/m³. a- première couche b- seconde couche c- troisième couche.

Pour mieux mettre en évidence le rôle joué par le degré d’alteration sur les propriétés de compactage des argiles nous avons étudié les argiles formées par l’alteration des dépôts de flysch à Floq. Les argiles

Un autre fait qui met en évidence l’existence de cette fraction de flysch, est l’utilisation de l’endommètre. Pour une charge verticale de 20 bars, la déformation des minéraux argileux atteint la valeur

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de 10%. Même après cette déformation, la courbe de compactage a tendance à baisser. Ceci montre que sous l’influence de la charge, les fractions de flysch non altérées se détruisent. Pour renforcer cette conclusion, un échantillon de la couche a, après avoir resté 48h dans de l’eau (pour étudier l’effet de la désagrégation du minerai en présence de l’eau), a été soumis à une énergie de 62.5 t-m/m³. La courbe de compactage obtenue (fig. 3a) indique une augmentation de la masse volumique (de 1680 à 1700 kg/m³) par rapport à celle qui correspondait à l’état naturel du minerai (la courbe en tiré, fig. 3a) et une diminution de la teneur en eau optimale (de 21 à 20%) et de la déformation (de 10 à 8%).

Ensuite ces argiles ont été soumises à des compactages d’énergie variable. Les résultats obtenus montrent que l’augmentation de cette énergie est accompagnée d’une augmentation de la masse volumique sèche et d’une diminution de la teneur en eau maximale (fig. 3, 4 et 6). Ainsi, pour les charges de 50, 62.5, 80 et 100 t-m/m³, les masses volumiques maximales des argiles de Mjeda vont de 1630 à 1720, celles de Fiq de 1680 à 1760 et enfin celles de Qyrsq de 1560 à 1730 kg/m³.

Le cas des argiles de Cernica est différent. Pour une augmentation de la charge de 50 à 100 t-m/m³, la masse volumique sèche de ces argiles ne varie que très peu (de 1520 à 1680 kg/m³). De plus, après avoir été soumises à ces charges, les minéraux argileux de Cernica se détruisent aisément, ce qui est en désaccord avec le pourcentage élevé de la fraction argileuse.  

Fig. 5. Courbes de compactages des argiles du gisement de Cernica à l’énergie de 50, 62.5, 80 et de 100 t-m/m³.

A notre avis, ceci s’explique par le fait que dans ce cas, la teneur maximale en eau moléculaire serait proche de la teneur en eau optimale et d’autre part par l’existence d’agrégats de colloïdes de fer et d’aluminium entourés d’une pellicule d’eau colloïdale qui empêche les grains de s’approcher.

Quand les charges verticales dépassent la valeur de 8 bars, ces agrégats se détruisent et la déformation des argiles devient très appréciable.

La construction des ouvrages argileux exige un grand nombre d’analyses des propriétés de compactage des argiles. Ces analyses, à part le coût élevé, demandent un temps relativement long à être effectuées. C’est pourquoi on a utilisé des analyses simples, rapides et pas très coûteuses comme c’est le cas des limites d’Atteberg, pour découvrir les relations entre ces dernières et la masse volumique sèche ou la teneur
en eau optimale. Ainsi, sur la fig. 6 on représenté les relations existant entre la limite de plasticité et la teneur en eau optimale pour les argiles de Cernica, Gyrsaq et Floq, sur la fig. 7 les relations entre la masse volumique sèche et la limite de plasticité, et enfin sur la fig. 8, les relations entre le nombre de plasticité et la limite de liquidité.

Dans tous les cas cités ci-dessus, ces relations sont représentées par des courbes linéaires qui sont décalées l'une par rapport à l'autre et non parallèles.

En général, on peut dire que l'augmentation des valeurs des limites de liquidité et de plasticité se suit d'une augmentation de la teneur en eau optimale et d'une diminution de la masse volumique sèche maximale. Ces relations sont:

- Pour les argiles de Cernica
  \[ W_{op} = 6 + 0.44 W_1 \quad \text{et} \quad W_{op} = 0.62 + 9.8 W_p \]
  \[ \delta = 1.98 - 0.0084 W_1 \quad \text{où} \quad I_p = 5.3 + 0.297 W_1 \]

- Pour les argiles de Floq
  \[ W_{op} = 0.6 W_1 \quad \text{et} \quad W_{op} = 7.4 + 0.59 W_p \]
  \[ \delta = 1.96 - 0.0084 W_1 \quad \text{où} \quad I_p = 7.8 + 0.187 W_1 \]

Fig. 6. Relation entre la teneur en eau optimale et les limites de plasticité et de liquidité. *- argiles de Cernica; **- argiles de Floq; +++- argiles de Gyrsaq 5.

Fig. 7. Relation entre la masse volumique sèche maximale et la limite de l'liquidité. *- Argiles de Cernica; **- argiles de Floq; +++- argiles de Gyrsaq 5.

Fig. 8. Relation entre l'indice de plasticité et la limite de l'liquidité. *- Argiles de Cernica; **- argiles de Floq; +++- argiles de Gyrsaq 5.

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Le décalage des droites les unes par rapport aux autres s’expliquent par les conditions différentes de formation et le degré de l’altération des argiles. De leur côté, ces dernières, s’expriment par la minéralogie et les propriétés physiques des argiles.

L’étude et la connaissance des relations ci-dessus fait que pour les argiles ayant la même origine de formation, une seule mesure laboratoire des limites de plasticité ou de liquidité est en principe suffisante pour connaître les paramètres du compactage.

Ces paramètres sont nécessaires surtout dans les premières phases de préparation de projets de construction d’ouvrages où il faut savoir si leurs valeurs sont convenables ou bien s’il faut les améliorer pour satisfaire les exigences du projet.

CONCLUSIONS

1- Les propriétés de compactage dynamique des argiles dépendent de l’origine de leur formation. De son côté, cette origine détermine la minéralogie, la granulométrie, le degré de cimentation des grains et les éléments chimiques qui participent au complexe d’absorption des grains de minéraux.

2- Il existe des relations entre les paramètres de compactage dynamique des argiles et leurs paramètres physiques. Ainsi selon l’origine de formation des argiles, on peut déterminer les relations entre la teneur en eau optimale et les limites d’Atteberg, ainsi que celles entre à masse volumique sèche et la limite de liquidité.

La connaissance de ces relations et de quelques mesures simples des paramètres de plasticité est d’une grande utilité dans les premières phases de projets d’ouvrages en terre.

3- La limite de plasticité et celle de liquidité augmentent avec la teneur en eau optimale et diminuent avec la masse volumique sèche maximale du compactage.

4- Les paramètres de compactage dynamique des argiles étudiées dépendent du degré d’altération, du rapport sablo-ilmano-argileux et du rapport entre les agrégats et les grains de minéraux désagrégés par le processus d’altération.


REFERENCES


Grain size characterization of dune sand in Saudi Arabia
La granulométrie marquante des dunes sableuses en Arabie Saoudite

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ABSTRACT: Different dune types tend to associate together in different locations in Saudi Arabia. Parabolic dunes exist with dome and barchan dunes in Al Jafurah sand sea in the east while linear dunes exist with barchan dunes in An Nafud sand sea in the north and partly in Jeddah area in the west. The grain size parameters namely mean size, median, sorting, graphic skewness, graphic kurtosis, uniformity coefficient and grading coefficient vary within each dune moving from the windward side to the leeward side. They are similar for the associated dunes except for the linear dunes which grow by selective erosion of the older barchan dunes. Different statistical techniques including multivariate analysis were applied on the grain size parameters of samples representing different dune types. None of the applied techniques could differentiate between the sand from the parabolic, dome or barchan dunes. The variations in grain size parameter values within one dune could have had the same magnitude as those between the different dune types. Partial separation could only be attained for the linear dunes from the other dune types.

RÉSUMÉ: Des types différents de dunes ont tendance à s’associer dans des sites différents en Arabie Saoudite. A l’est, dans la mer sableuse d’Al Jafurah, des dunes paraboliques existent avec des dunes de forme de dôme et de croissant, tandis que, au nord, dans la mer sableuse d’An Nafud, et partiellement à l’ouest, dans la région de Jeddah, des dunes linéaires coexistent avec celles de forme de croissant. Les paramètres granulométriques notamment la moyenne, la médiane, le classement, l’asymétrie, l’angulosité, le coefficient d’uniformité et le coefficient de grade varient dans chaque dune quand on passe du côté de vents vers l’autre direction. Ces paramètres sont les mêmes pour tous les types de dunes à l’exception de dunes linéaires qui grandissent à l’aide d’érosion sélective de dunes croissantes anciennes.
Des techniques chiffrées avec des analyses multivariantes ont été appliquées aux échantillons représentatifs de types différents de dunes. Aucune des techniques appliquées eu le pouvoir de distinguer parmi les types de dunes. Les variations en valeurs granulométriques dans la même dune ont une magnitude comparable aux types différents des dunes. La séparation partielle est obtenue seulement au cas des dunes linéaires.

1 INTRODUCTION
The problem of sand drifting and dune migration is of special interest in Saudi Arabia as approximately one third of the country is covered by moving sand. Dunes exist either as coastal dunes as the case along the Red Sea coast or within sand seas as in the Nafud to the north, Jafurah to the east, Ad Dahna in central Saudi Arabia and Arub el-Khali in the south. The sand grain size is a major factor that affects the value of the threshold velocity (Chepil, 1945) and consequently the overall rate at which sand moves. Zingg (1952),
Bagnold (1953), and Lettau and Lettau (1978) included the grain size in their sand movement equations together with the wind speed and the threshold velocity. Shehata et al. (1993) believe that the grain size as well as other parameters such as relative density, mineralogical composition and consequent the sand specific gravity, salt content and moisture content affect the threshold velocity of sand. Therefore, sand grain size in Bagnold’s equation affects the sand movement once as being a parameter in the sand moving equation and once as an affecting factor on the threshold velocity.

Sand dune types exist in different associations at different localities in Saudi Arabia. For example, in the Jafurah sand sea, in the Eastern Province, parabolic dunes exist in association with barchan and dome dunes. Field observations and aerial photo studies indicated that a parabolic dune may get detached from its erosional area, develop a slip face and become a barchan dune. A barchan dune may, on the other hand, lose its slip face, and become a dome dune. In the Great Nafud sand sea, linear dunes exist parasitically on the top of the barchan dunes and move faster than them. They grow on the expense of the selective erosion of the finer sand off the older barchan dunes. In Jeddah area, only linear dunes exist with insignificant number of small barchan dunes.

The grain size distribution of sand have been used to characterize depositional environments (Folk, 1966; Visher, 1969). It was used by Abu El-Ella and Coleman (1985) to discriminate between six different environments within one river delta. The objectives of this research are to characterize different dune types in terms of the grain size parameters and to explain the field observations for the preferred association of certain dune types at different localities in Saudi Arabia.

2 SAND SAMPLING

Sand samples were collected on profiles along the center-lines of the barchan, dome and parabolic dunes and across the linear dunes. Four to six samples were collected along each profile depending on the dune type and size. In case of the dome and parabolic dunes, one or two samples were taken in their erosional areas while the rest were taken from the depositional areas. The samples were collected to represent the top 5 cm layer of each dune.

3 GRAIN SIZE PARAMETERS

The investigated grain size parameters are four sedimentological namely; mean size ($M_z$), standard deviation (sorting) ($\sigma_I$), graphic skewness ($Sk_I$) and graphic kurtosis ($K_g$) (Folk, 1980), and four engineering namely; effective size ($E_s$), median ($M_d$), uniformity coefficient (Cu) and gradation coefficient (Cc) (Torszeghi and Peck, 1968). The following are the equations that define these parameters:

- **$M_z = \phi_{16} + \phi_{50} + \phi_{84} \over 3$**
- **$\sigma_I = \phi_{84} - \phi_{16} + \phi_{95} - \phi_{50} \over 4$**
- **$Sk_I = \phi_{16} + \phi_{84} + 2\phi_{50} \over 2(\phi_{84} - \phi_{16})$**
- **$K_g = \phi_{95} - \phi_{50} \over 2.44(\phi_{75} - \phi_{25})$**
- **$E_s = D_{10}$**
- **$M_d = \phi_{50}$**
- **$Cu = D_{60} \over D_{10}$**
- **$Cc = (D_{30})^2 \over D_{10} * D_{60}$**

It should be noted that the sedimentological parameters use the
\(\phi\)-value of the percent finer curve while the engineering parameters use the grain sizes in mm on the percent retained curve.

These parameters were calculated for 86 sand samples representing four different dune types. The investigated dune types were barchan, parabolic and dome dunes in Al Jafurah sand sea in east Saudi Arabia, linear and barchan dunes in An Nafud sand sea in the north and linear dunes east of Jeddah in western Saudi Arabia.

4 GRAIN SIZE PARAMETER VALUES

Figure 1 shows that the variations in any grain size parameter is dependent not only on the dune type but also on the location of the sample within each dune.

Across the linear dunes, the mean grain size increases from the flanks toward the crest. The flanks are somewhat finer than the crests in a pattern to that reported by Lancaster (1986) for the linear dunes in southeastern Kalahari. The sand is generally moderately sorted and gets slightly poorly sorted at the base of the slip face. Compared with dunes elsewhere, these dunes seems to be less sorted than for example the Namib dunes studied by Lankester (1986). Most of the linear dune sands are nearly symmetrical but gets negatively fine skewed at the base of the slip face. The windward end starts as very leptokurtic to platykurtic at the crest and at the bottom of the slip face.

Across the barchan dunes the mean grain size is generally fine and increases windward. The sand is moderately well sorted to well sorted. The worst sorting is observed at the crest, a pattern noted in the Nebraska Sand Hills by Ahlbrandt and Fryberger (1980) and in central Namib by Watson (1986). This suggests that the crest is an accretion area as suggested by Bagnold (1953). The sand is nearly symmetric at the windward face but finely skewed at the crest. It is also platykurtic to very platykurtic on the windward face getting leptokurtic on the crest.

Across the dome dune is similar to the barchan dune except the pattern present in the windward face is repeated in the leeward face. At the crest the sand is fine grained, poorly sorted, finely skewed and very platykurtic. Moving away from the crest the sand gets medium grained, moderately well sorted, nearly symmetrically skewed and mesokurtic. Comparison with dome dunes elsewhere is difficult due to the lack of information.

Across the parabolic dunes the sand is generally medium grained, poorly sorted, finely skewed and platykurtic. However the erosional area of these dunes are generally fine to medium grained, moderately well sorted, nearly symmetrical and mesokurtic.

The variations in the median values within the different dune types follows more or less the variation trends of the mean. Due to the lack of information, no correlations could be made for Cu and Cu elsewhere.

Although variations in grain size parameters exist within single dunes, some variations may also exist between different dune types. Statistical analyses were performed to test for any variations. Table 1 gives the summary statistics of the various grain size parameters for the different dune types. It gives the minimum, maximum, mean value, standard deviation (SD) and coefficient of variability (Cv) of each parameter. Table 2 gives the same information for all the dune types collectively. The correlation matrix of the different parameters for all the sand samples is given in Table 3.

From Tables 1 and 2 it is obvious that the grain size parameters are similar for at least the parabolic, dome and barchan dunes. Some of the variables of the linear dunes are also similar to those of the other dune types. It can be concluded that we are dealing with only one population specially the Cv values are low for the different variables in each dune type and is still low for all the dunes collectively.

The physical meaning of the grain size parameter values (Table 4) interpreted using Folk (1980) and Terzaghi and Peck (1968) classifications again indicates the similarity between the different dune types specially the parabolic,
dune and barchan dunes. They are medium to fine grained, moderately sorted to moderately well sorted, fine skewed to strongly fine skewed or nearly symmetrical, mesokurtic and poorly graded. These characteristics are similar to the characteristics of the dune sand in south, east and north of Jeddah studied by Binda (1983) and in south-east Red Sea coastal area studied by Taj et al. (1990). They are also similar to the drifting sand characteristics at al-Hasa Oasis (Abolkhair, 1985) in the Eastern Province.
Table 1: Summary statistics of grain size parameters for each dune type.

<table>
<thead>
<tr>
<th></th>
<th>From</th>
<th>To</th>
<th>Mean</th>
<th>SD</th>
<th>Cv</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Linear dunes (N=21)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D10</td>
<td>.08</td>
<td>.18</td>
<td>.12</td>
<td>.03</td>
<td>23.45</td>
</tr>
<tr>
<td>Mz</td>
<td>1.64</td>
<td>3.03</td>
<td>2.36</td>
<td>.38</td>
<td>15.97</td>
</tr>
<tr>
<td>Md</td>
<td>.21</td>
<td>.51</td>
<td>.34</td>
<td>.08</td>
<td>22.65</td>
</tr>
<tr>
<td>SkT</td>
<td>.32</td>
<td>.86</td>
<td>.51</td>
<td>.13</td>
<td>26.27</td>
</tr>
<tr>
<td>Kg</td>
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<td>1.77</td>
<td>.50</td>
<td>.54</td>
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<tr>
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<td>2.05</td>
<td>.27</td>
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</tr>
<tr>
<td>Cc</td>
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<td>1.07</td>
<td>.22</td>
<td>20.65</td>
</tr>
<tr>
<td><strong>Parabolic dunes (N=24)</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>.11</td>
<td>.02</td>
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</tr>
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<td>10.97</td>
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<td>Md</td>
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<td>.14</td>
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<tr>
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<td>.93</td>
<td>.17</td>
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</tr>
<tr>
<td>Kg</td>
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<td>.14</td>
<td>.16</td>
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<td>1.18</td>
<td>34.09</td>
</tr>
<tr>
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<td>1.03</td>
<td>.24</td>
<td>23.16</td>
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<td><strong>Dome dunes (N=19)</strong></td>
<td></td>
<td></td>
<td></td>
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</tr>
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<td>D10</td>
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<td>.13</td>
<td>.04</td>
<td>30.55</td>
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<tr>
<td>Mz</td>
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<td>.71</td>
<td>.16</td>
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<td>Kg</td>
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</tr>
<tr>
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<td>1.49</td>
<td>1.03</td>
<td>.24</td>
<td>23.16</td>
</tr>
<tr>
<td><strong>Barchan dunes (N=22)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>D10</td>
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<td>.26</td>
<td>.12</td>
<td>.04</td>
<td>37.78</td>
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<td>2.01</td>
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<td>9.84</td>
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<td>Md</td>
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<td>.95</td>
<td>.53</td>
<td>.20</td>
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<tr>
<td>SkT</td>
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<td>.76</td>
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<td>26.48</td>
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<td>.19</td>
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<td>2.70</td>
<td>.74</td>
<td>27.55</td>
</tr>
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<td>Cc</td>
<td>.83</td>
<td>1.99</td>
<td>1.15</td>
<td>.29</td>
<td>25.71</td>
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</tbody>
</table>

Table 2: Summary statistics of grain size parameters for all the dune types (N=86).

<table>
<thead>
<tr>
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<th>FROM</th>
<th>TO</th>
<th>MEAN</th>
<th>SD</th>
<th>Cv</th>
</tr>
</thead>
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<td>.12</td>
<td>.03</td>
<td>29.82</td>
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<tr>
<td>Mz</td>
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<td>3.03</td>
<td>2.05</td>
<td>.33</td>
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</tr>
<tr>
<td>Md</td>
<td>.21</td>
<td>1.00</td>
<td>.58</td>
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<td>37.37</td>
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<td>.32</td>
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<td>.78</td>
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<td>Kg</td>
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<td>184.37</td>
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<td>Cu</td>
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<td>6.25</td>
<td>2.81</td>
<td>1.03</td>
<td>36.79</td>
</tr>
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<td>Cc</td>
<td>.37</td>
<td>1.99</td>
<td>1.03</td>
<td>.28</td>
<td>26.91</td>
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Table 3: Correlation matrix for the grain size parameters for all the dune types.

<table>
<thead>
<tr>
<th></th>
<th>D10</th>
<th>MZ</th>
<th>MD</th>
<th>SI</th>
<th>SkT</th>
<th>KG</th>
<th>Cu</th>
<th>CC</th>
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<tr>
<td>D10</td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td>MZ</td>
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<td>1.00</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MD</td>
<td>-04</td>
<td>-65</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>SI</td>
<td>-33</td>
<td>-47</td>
<td>87</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SkT</td>
<td>-23</td>
<td>30</td>
<td>-27</td>
<td>27</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>KG</td>
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<td>-10</td>
<td>-02</td>
<td>-20</td>
<td>-03</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cu</td>
<td>-53</td>
<td>-40</td>
<td>68</td>
<td>78</td>
<td>03</td>
<td>-02</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>CC</td>
<td>-24</td>
<td>04</td>
<td>-36</td>
<td>-32</td>
<td>13</td>
<td>-23</td>
<td>-22</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 4: General textural description of the dune sand

<table>
<thead>
<tr>
<th></th>
<th>Linear</th>
<th>Parabolic</th>
<th>Dome</th>
<th>Barchan</th>
</tr>
</thead>
<tbody>
<tr>
<td>MZ</td>
<td>Fine sand</td>
<td>Medium sand</td>
<td>Medium sand</td>
<td>Medium sand</td>
</tr>
<tr>
<td>SI</td>
<td>Moderately sorted</td>
<td>Moderately sorted</td>
<td>Moderately sorted</td>
<td>Moderately sorted</td>
</tr>
<tr>
<td>SkT</td>
<td>Strongly fine skewed</td>
<td>Fine skewed</td>
<td>Nearly symmetrical</td>
<td>Fine skewed</td>
</tr>
<tr>
<td>KG</td>
<td>Mesokurtic</td>
<td>Mesokurtic</td>
<td>Mesokurtic</td>
<td>Mesokurtic</td>
</tr>
<tr>
<td>Cu</td>
<td>Poorly graded sand</td>
<td>graded sand</td>
<td>graded sand</td>
<td>graded sand</td>
</tr>
</tbody>
</table>

Table 5: Generalized distance between the different dune types

<table>
<thead>
<tr>
<th></th>
<th>Linear</th>
<th>Parabolic</th>
<th>Dome</th>
<th>Barchan</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>0.00</td>
<td>9.00</td>
<td>8.84</td>
<td>4.11</td>
</tr>
<tr>
<td>Parabolic</td>
<td>0.00</td>
<td>0.00</td>
<td>1.27</td>
<td>1.72</td>
</tr>
<tr>
<td>Dome</td>
<td>0.00</td>
<td>2.44</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>Barchan</td>
<td>0.00</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6: Factors controlling sand dune characteristics

<table>
<thead>
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<th>Factors</th>
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<th>F2</th>
<th>F3</th>
</tr>
</thead>
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<tr>
<td>Eigenvalue</td>
<td>3.08</td>
<td>1.24</td>
<td>0.75</td>
</tr>
<tr>
<td>Eigenvalue - %</td>
<td>51.4</td>
<td>20.7</td>
<td>12.5</td>
</tr>
<tr>
<td>Cumm. eigenvalue-%</td>
<td>51.4</td>
<td>72.1</td>
<td>84.6</td>
</tr>
<tr>
<td>MZ</td>
<td>0.665</td>
<td>-0.437</td>
<td>-0.134</td>
</tr>
<tr>
<td>MD</td>
<td>-0.945</td>
<td>0.088</td>
<td>-0.52</td>
</tr>
<tr>
<td>SI</td>
<td>-0.933</td>
<td>-0.083</td>
<td>0.119</td>
</tr>
<tr>
<td>SkT</td>
<td>0.122</td>
<td>0.812</td>
<td>-0.544</td>
</tr>
<tr>
<td>KG</td>
<td>-0.830</td>
<td>0.070</td>
<td>0.062</td>
</tr>
<tr>
<td>Cu</td>
<td>0.418</td>
<td>0.612</td>
<td>0.643</td>
</tr>
</tbody>
</table>

The mean values of the variable D10 were very similar in the four examined dune types. Since the factor D10 is indirectly evaluated using the parameter Cu (equation 7), it is suggested to be disregarded during the multivariate analysis. Similarly, the graphic skewness (SkT) consistently showed values greater than 100 for the coefficient of variability. It was also recommended to be discarded leaving only 6 variables for the multivariate analysis.

The correlation matrix for each dune type as well as that for all the dunes (Table 3) consistently showed positive correlation between, the median grain size (MD), the standard deviation or sorting (SI), and the uniformity coefficient (Cu).

5 MULTIVARIATE ANALYSES

The selected six grain size parameters, namely; mean size, median, sorting, skewness, uniformity coefficient and gradation
Coefficient were subjected to three multivariate analyses in a trial to characterize the different dune types in terms of the grain size parameters. These analyses were discriminant analysis, factor analysis and cluster analysis.

5.1 Discriminant analysis

This analysis was performed to determine the generalized distance (Rao, 1965) between the different groups forming the different dune types. Table 5 gives the calculated generalized distance between linear, parabolic, dome and barchan dunes.

The generalized distance results indicate that the linear dunes are relatively distant from the parabolic and dome dunes and in fact it does not associate with them in any of the studied locations. The linear dunes are relatively close to the barchan dunes which form the source of sand to the linear dunes. The parabolic, dome and barchan dunes have closer distances and they associate together. This substantiates the fact that there is no difference in the characteristics of the sand that forms these three dune types. Their presence in different geomorphological forms could be more related to wind energy and presence of vegetation rather than the grain size parameters.

5.2 Factor analysis

The factor analysis was performed to extract the main factors that control the sand dune characteristics based on the selected grain size parameters. Table 6 shows that the first three factors contribute 85% of the total system information. It also gives the load of the different grain size parameters on each factor.

From Table 6 it is obvious that Factor F1 is positively loaded with \( M_g \) and \( C_c \) and negatively loaded with \( \sigma_f \), \( M_d \), and \( C_u \). Factor F2, on the other hand is positively loaded with \( K_g \) and \( C_c \) and negatively loaded with \( M_g \). Factor F3 is positively loaded with \( C_c \) and negatively loaded with \( K_g \).

A maximum separation of the linear dune samples from the rest of the dunes was attained by plotting F1 against F2 (Fig. 2). The other three dune types were inseparable neither in this plot nor in plotting any other two factors. Again this confirms the previous findings that the linear dunes mode of development is different from the other dunes.

Fig. 2 Plot of Factor 1 against Factor 2.

5.3 Cluster analysis

The cluster analysis was performed in a trial to group similar samples under separate clusters. Fig. 3 shows the result of the analysis in the form of a dendrogram. It is obvious that the linear dune samples have been partly separated from the other dune types while all the formed clusters included a mixture of samples from the parabolic, dome and barchan dunes. The linear dunes concentrate in clusters Ia and Ib and is mixed with parabolic, dome and barchan dunes in cluster Ib. All the other clusters contain a mixture of parabolic, dome and barchan dune sand. This also confirms that only the linear dunes can be partially separated from the other dune types, a result already obtained from both discriminant analysis and factor analysis.
selective erosion of the underlying barchan dunes, have more or less different grain size parameters and could be statistically partially separated from the other dune types.

7 REFERENCES


6 CONCLUSIONS

Field observations and statistical analyses indicated that the sand that is forming the parabolic, dome and barchan dunes is very close in its grain size characteristics. The variations in grain size parameters within a single dune could be as large as those between different dune types. In other words, the grain size parameters do not affect the formation of a certain dune type. Dunes may have different form or type without any changes in the characteristics of the forming sand. However, the studied linear dunes, being developed on the expense of

Fig. 3: Dendrogram for all the sand samples.


Both the internal friction angle and the cohesion of cohesive soils and soil types determined by means of the static cone penetration test (CPT)

Autant l’angle de frottement interne que la cohésion de sols et types de sol cohésifs déterminés au moyen de l’essai de pénétration d’un cône statique (EPC)

Meng Gaotou
Department of Hydrogeology & Engineering Geology, China University of Geosciences, Wuhan, People’s Republic of China

ABSTRACT: First, the results and data of laboratory and in-situ testing are presented and statistical analysed; then, the soil classification charts and shear strength parameters ($
\phi$, $c$) of the cohesive soils are expressed as function of the end resistance ($q_e$) and side friction ($f_s$) determined by the static cone penetration test (CPT).

$$q_e = \arctg (0.0069 \sqrt{q_s} - 0.1023)$$

$$c = n \sqrt{f_s} - b$$

Where $\phi$ is the internal friction angle, degree ($\phi$); $c$ is the cohesion, kPa; $q_s$ is the end resistance, kPa; $100 < q_s < 8200$ kPa; $f_s$ is the side friction, kPa.

When $16 < f_s < 80$ kPa, $a = 12.14, b = 32.77$; i.e. $c = 12.14 \sqrt{f_s} - 32.77$

When $1 < f_s < 9$ kPa, $a = 5.47, b = 3.80$; i.e. $c = 5.47 \sqrt{f_s} - 3.80$

They show that the expressions are quite good to evaluate the internal friction angle and cohesion of cohesive soils. The soil classification charts developed in this paper are also very good for identifying soil type from $q_s, f_s, F_r (F_r = f_s/q_s \times 100\%)$.

The advantages of this method are also discussed.

RESUME: D’abord, les résultats et les données au laboratoire et essais in situ sont présentés et analysés statistiquement; puis, la carte de la classification des sols et les paramètres de la force du cisaillement aux sols cohésifs ($\phi$, $c$) sont exprimés par la fonction de la résistance extrême ($q_s$) et de la friction latérale ($f_s$) déterminée par l’essai de pénétration du cône statique (PCT).

$$q_e = \arctg (0.0069 \sqrt{q_s} - 0.1023)$$

$$c = n \sqrt{f_s} - b$$

où $\phi$ est l’angle de friction interne, degré ($\phi$); $c$ est la cohésion, kPa; $q_s$ est la résistance extrême, kPa; $100 < q_s < 8200$ kPa; $f_s$ est la friction latérale, kPa.

Quand $16 < f_s < 80$ kPa, $a = 12.14, b = 32.77$; i.e. $c = 12.14 \sqrt{f_s} - 32.77$

Quand $1 < f_s < 9$ kPa, $a = 5.47, b = 3.80$; i.e. $c = 5.47 \sqrt{f_s} - 3.80$

Nous avons montré que les expressions sont très bien pour l’évaluation de l’angle de friction interne et de la cohésion aux sols cohésifs. La carte de classification des sols développée dans ce papier est aussi bien pour l’identification du type des sols à partir de $q_s, f_s, F_r (F_r = f_s/q_s \times 100\%)$.

Les avantages de cette méthode sont aussi discutés.

1 INTRODUCTION

The static cone penetration test (CPT) is a rapid, e-
but there have been few correlations between CPT data and the shear strength of cohesive soils until now. So the correlation between the end resistance, $q_e$, and the internal friction angle, $c$, and between the sleeve friction, $f_s$, and the cohesion, $c$, have been established in this paper. (The $c$, $q_e$ are determined by the direct shear test on 'undisturbed' samples taken from drilling.) This is a significant research work.

It is well known that the traditional method to determine the values of $c$ and $q_e$ is laboratory testing on sample. The sample are inevitably disturbed due to sampling, transporting and making specimens in laboratory. Therefore parameters, $c$, $q_e$, sensitive to the disturbance of sample produce considerable man-made errors. especially to the undisturbed soil samples difficulty to be taken, such as sludge, tailing sludge and so on. Even the 'undisturbed' sample test is doing with difficulty for laboratory direct shear testing, while the vertical pressure is applied, the sample will be pushed out from the shear box very easy, so the testing is failed. These happen frequently in engineering investigation for soft clay ground. If the values of $c$, $q_e$ are determined by CPT, the CPT is not only suited for soft soils, but also for hard soils, and the repeatability and accuracy of the CPT data are fairly good. During Engineering investigation, if a few sampling for the direct shear test are companied with CPT for the same type of soil, then the precision of $c$, $q_e$ determined by CPT will be improved. Therefore this research work can lead CPT to more widely used, to reduce the number of samples, save the investigation expenses, reduce the investigation time.

2. THE METHOD OF SELECTING PRIMARY TESTING DATA

The CPT data, $q_e$, $f_s$, and samples taken from the adjoining borehole are at the same depth for the same character of the same soil layer. The distance between the hole centers of CPT and adjoining sampling borehole is $2-3\text{m}$, the CPT graph pattern is referred to selecting the steady values of the graph as representative values of $q_e$, $f_s$. If the graph pattern is Sawtooth like, then the mean values of $q_e$, $f_s$ are taken as representative ones. During penetration process, the characters of soils around the cone are all affecting the values of $q_e$ and $f_s$. (It is formed a failure zone like a pear shape around the cone.) This influence should be concerned. So the mean values of $q_e$ and $f_s$ between 8 diameters of the sleeve above the cone tip and 4 diameters length beneath the cone tip are taken as representative ones, meanwhile, the values of $c$, $q_e$ are selected from the undrained and unconsolidated direct shear test values in order to corresponding with rapid penetration speed at 1.2m per minute.

3. THE PARAMETERS OF COHESIVE SOIL SHEAR STRENGTH DETERMINED BY CPT

3.1. The correlation analysis between $q_e$ and $q_e$.

Direct shear test and CPT were widely used in the tailing dam investigation in Tonglushan, Dayan County, Central China. The dam site is located in the west lakeside of the Dayan lake. There are mainly alluvial and lacustrine cohesive soils in the ground. Twenty four groups of $q_e$ and $q_e$ in more homogeneous soil layers are selected for drawing a scatter plot. The scatter mid line drawn is an approximate exponential function curve. If $q_e$ is changed to $\sqrt{q_e}$, then the curve is changed to a straight line as showing in figure 1.

Through analysis with the method of least square, the corresponding correlation is formed as following:

$$q_{pp} = 0.0069 \sqrt{q_e} - 0.1023$$

where, $q_{pp}$=internal friction angle, $\phi_{pp}$=end resistance, $kP_a+400kP_a < q_e < 8200kP_a$

The correlation coefficient, $r = 0.978$

The standard deviation, $s = 0.0276$

So $q_{pp} = \arctg(0.0069 \sqrt{q_e} - 0.1023)$

3.2 The correlation analysis between $c$ and $f_s$.

The cohesion value of normal and over consolidated soils depends on the interparticular bound water attraction force. It is found that the value of $f_s$ is approximately equal to the value of $c$, especially in high plastic soil. In space with increasing of the plastic index value of cohesive soils, the value of $c$ increases along
with the value of \( f_i \), or otherwise, the \( c \) value of the sandy soils of very low plasticity index decreases to zero along with the value of \( f_i \) increasing. Because there is shear strength in the bound water, during penetration, the probe sleeve mainly scrapes along the bound water film; the lower the moisture content, the less the loosely bound water content is, the relatively higher the strongly bound water content, so the higher the shear strength of the soil, the higher the value of \( f_i \). In sand, there is little bound water, so the probe sleeve measuring \( f_i \) mainly scrapes with the surfaces of sand particles. Although the value of \( f_i \) is much higher than that in clay, the value of \( c \) in sand is approximately zero.
The 19 groups of c and f, data are selected from testing data in the alluvial and lacustrine cohesive soils, then these data are drawn to a scatter plot; the 9 groups of c-f, data are also selected from the marine sludge in the Shenzhen ground, most China; the data are also drawn to a scatter plot in the same graph. c-f, values of the sludge concentrates in the low-left corner of the graph, because of smaller values. The correlation of f, c are not linear relationship, but if f, is transformed into $\sqrt{f}$, then $\sqrt{f} - c$ are a good linear relationship (see the Figure 2.)

After analysing with method of least square, the correlations are shown as following separately:

For alluvial, lacustrine cohesive soils,

$$c = 12.14 \sqrt{f} - 32.77$$  \hspace{1cm} (3)

1kP $a$ $f$ $< 10kP$, the $r = 0.940$.

For marine sludge,

$$c = 5.47 \sqrt{f} - 3.80$$  \hspace{1cm} (4)

1kP $a$ $f$ $< 80kP$, the $r = 0.957$.

The $r$ values of the correlation (3), (4) are very high. It is shown there are closely correlations between c and $\sqrt{f}$. They are also linear correlations. So they can be expressed with one correlation as following.

$$c = a \sqrt{f} - b$$  \hspace{1cm} (5)

Where $a$ and $b$ are the coefficients related to the type and origin of tested soils.

4. SOIL TYPES DETERMINED BY CPT

Soil types determined by CPT is one of very good methods to identifying soil type. It has been usually using one or two parameters, like $q$, $F_P$ (friction ra-
Fig 4. Muck (sludge) chart by CPT

5. CONCLUSION

The recommended method in this paper is primary attempt. It is necessary to be tested by more information in different areas.

The values of $q_e$ and $f_s$ should be the mean values in the certain depth range of testing hole, as it has been described in this paper. The method recommended in this paper is practicable. It may produce some error while it is used, but it is not surprising. The accuracy of the method will be improved gradually with information and data accumulated as well as conducting thoroughgoing research work.
The in situ swelling test in a hole
L’essai de gonflement in situ dans un trou

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ABSTRACT: An In-hole Swelling test Instrument (ISI) has been developed. Its probe consists of a force transmitted machinery, an electrical measuring system, and an antidiintegration and diversion device. Using this instrument, an in-situ swelling test for montmorillonized granite was conducted for a pumped storage station. The results showed that the swelling pressure of the montmorillonized granite is related to the fracture and microfracture intensities and groundwater pressure.

Un instrument de test de gonflement à l’intérieur de trou a été réalisé. Sa sonde consiste en un mécanisme de transmission de force, un système de mesure électrique et un appareil d’antidéintegration et de diversion. Grâce à cet instrument, les tests de gonflement in situ pour le granite montmorillonisé ont été faits à la station de générateur de réservoir de pompage. Les résultats montrent que la pression de gonflement du granite montmorillonisé est relative à l’intensité de fracture, à l’intensité de microfracture et à la pression des eaux souterraines, peut-être aussi à la pression de confinement.

1 INTRODUCTION

The swelling of rock and soil always does much harm to the foundations and underground constructions. It is an important factor must considered in design. Up to now, most of the swelling tests are conducted in laboratory. Because the swelling is related to the nature environmental factors closely, such as temperature, moisture, water acidity, confining pressure, etc., which are difficult to simulate in laboratories, and in the sampling, transportation, and the preparation of the samples tested in laboratories, it is difficult to maintain the nature state completely, so it is necessary to study the in-situ methods and instruments to test the characters of swelling rocks and soil.

With this view, since 1981 Hohai University and Geodata S.r.l. began to jointly develop the in-situ swelling indexes test instruments measuring in bore hole. In this paper the informations of one of the In-situ Swelling test Instruments (ISI) and the results of the in-situ tests by use of ISI are given.

Using ISI, the main obtaining index is swelling pressure. It also can obtain the maximum swelling capacity. Obviously, here the concepts of the swelling pressure and the maximum swelling capacity are different from those in laboratories. These are two new swelling indexes, which may be possesssses of more direct meaning for the design of the linings and of deep foundations.

2 STRUCTURE AND OPERATIVE PRINCIPLES OF ISI

2.1 Structure of ISI

ISI consists of a probe which can be put into a bore hole and a control instrument placed on the ground. The two parts are linked up with a special double shielded cable including 37 wires. (fig. 1)

The probe consists of a force transmitted machinery, an electrical measuring system, and a diversion and antiintegration device. (fig. 2)

The force transmitted machinery can be driven by a micromotor placed in the probe. When the machinery open, six force transmitted plates can be nestled closely to the wall of hole, and the electrical...
measuring system can be just placed at the normal operating situation.

![Diagram](image)

**Figure 1**

- **Cable**
- **Bore hole**
- **Probe**

**Figure 2**

- **Cable**
- **Micrometer**
- **Pressure measuring cells**
- **Displacement measuring gauges**
- **Force transmitted plate**
- **Comb sheets**
- **Nylon fabric cover**

The electrical measuring system includes six pressure cells and three displacement gauges. They can measure the swelling pressures and the swelling capacities in three directions which cross at the angles.

For ensure the in hole swelling test, the antidissipation and diversion device is set as the cost of probe. It consists of two layers. The inner is a group of comb sheets, and the outer is a nylon fabric cover. It avoids the tangential force acting on the surface of every force transmitted plate, which may occur when the rock is swelling.

The control instrument is possessed of some functions as to control the probe operation, to amplify the signals and A-D convert, to display and to record, etc.

The instrument is fit for AC supply with $220 \pm 40$ V, 50 Hz.

2.2 Testing process

The testing of J31 need to synchronize with drilling.

The drilling need to use diamond bit. Before every time running the tools into the hole, the diameter of the bit must be measured precisely.

The testing process is as follows:

1. When drill to determined testing point (pre-determined or determined by the variation of bottom-hole pressure), lift drilling tools; put the probe to the testing point as quickly as possible. Operate the motor to push the force transmitted plates to nestled closely to the wall of hole, and the probe is in working order. Make pre-pressure about 0.01 MPa. At that time the diameter of the hole (D) shown by displacement gauges is about equal to the diameter of bit ($D_b$), i.e.

   $$D = (D_b + 0.01) \pm 0.01 \text{ (mm)}$$

Record the initial readings of all measuring cells and gauges, the beginning time of the working order of probe, and the drilled time of the testing point accurately.

2. If the hole is a dry hole, inject water and maintain the hole water level. Regard the injecting time as the beginning time of working order.

3. Observe and record the readings of all cells and gauges at regular intervals.

4. If the hole diameter contract over than 0.1 mm, operate the motor to press the wall of hole for recovering the hole diameter.

5. When the swelling pressure readings become constant and the hole diameter maintain the initial size, it may be considered that the swelling has reached to stable.

6. Operating the motor in reverse direction to unload step by step; observe and record the swelling capacities.

7. When the test is finished, contract the probe, record the end readings.

8. Lift out the probe; go on drilling, to conduct next test. If the rock is dissintegratable, it is need to ream and to case before the continuative drilling.
3 IN-SITU TEST

3.1 The environment of the tests

The tests of ISI were conducted at the site of Guangzhou Pumped Storage Station. This power station is the largest pumped storage station in China. All the underground constructions of the station are constructed in a Mesozoic granite mass. Along most of the faults in the mass, the meso-epithermal alterations occurred in different degrees, which are characterised by sili
cation, carbonation, illitisation, kolinisation, and montmorillonisation. In most of the alteration zones, the content of alteration minerals is 10 to 30 % of the whole rock, the maximum content is 70 %; the content of montmorillonite is 0 to 4 % of the whole rock, the maximum content is 13 %. Because the granite suffered clayi
cation, the porosity of the rocks increased to 3 to 7 times in general ( Li Cheng el. 1991 ).

A few in-situ large-scale swelling tests have been conducted in a prospecting adit.

The tests showed that the swelling pressures ranged from 0.11 to 0.48 MPa, relating to the content of montmorillonite.

Because the large-scale tests need to expand much money and long time, it is difficult to conduct enough such tests for different alteration zones. Moreover, the testing environments of the largescale tests are also different from the environments of actual engineering. So that, the test of ISI was considered for the design of the second phase of the project.

The test hole was drilled in a construction tunnel. The hole top is about 70 m below the groundsurface, where it can be considered that there is no weathering effects. For some reasons, the test hole is an oblique hole with 70° of hole de
flexion. The designed hole depth is 31 m.

The depth of first test section is 6.10 to 6.35 m. In all the process of drilling and testing, a few groundwater outflowed from the hole. It shows that the groundwater table is somewhat higher than the hole top.

3.2 The results of ISI tests

3.2.1 The swelling of the altered granite is very unequal. It varies depending on the content of montmorillonite, and also depending on fracture intensity and microfracture intensity. As the content of montmorillonite being 3 to 4 % and the microfractures being well developed, the swelling pressure reached about 0.12 MPa.

3.2.2 Comparing with the laboratory tests using the drilling cores of in-hole test sections as the samples, the swelling pressures from ISI tests are higher than those from laboratory tests obviously.

It maybe results from:

1. In the process of drilling and preparation, it is difficult to protect the samples from moisture. If the altered granite samples are moistened and expanded, their volumes is almost cannot be recovered.

2. The thickness of samples tested in laboratories is thinner than the thickness of swelling rock layer of the hole wall.

3. In ISI tests, the swelling pressure may include a part of ground stress, especially when the rock is soft rock and the test point is placed very deep.

3.2.3 Comparing with the in-situ large-scale swelling tests, the swelling stabilized times of ISI tests are shorter obviously. It may be caused by:

1. The water pressure maybe makes the infiltration capacity bigger, so the swelling process become faster. The tests show that where the fracture intensity is higher, the swelling stabilized time is shorter.

2. The mechanical boundary condition of ISI test is different from that of large-scale swelling test. In ISI testing, the plastic ring around the hole is stabilized easily. When the plastic ring is stable,
the swelling will also be tending towards stable.

4 DISCUSSION

The In-hole Swelling Test Instrument is a new instrument. The tests by use of this instrument are only a few. But the initial tests have revealed some important problems about swelling mechanism and the effects of some environmental factors on swelling.

These problems need to be studied and deliberated based on a large number of the tests by use of ISI from now on.

4.1 About microfracture

The tests show that the microfractures play an important role in swelling process. In addition to tectonic stress, in altered rocks, the microfractures may also be caused by alteration. They are always in irregular shape and may find on cores by eye. Their width ranged from $n \times 10^{-6}$ to $n \times 10^{-7}$ mm which could be measured under microscope. Both sides of the microfractures mainly are coarse crystallized minerals. If clay swell, the microfractures will expand. These expansions cannot be recovered by any way.

It is believed that the effects of microfractures in altered rocks maybe different from those in soft rocks in which the content of clay is over than 30%.

4.2 About water pressure

When the microfractures dilate, the groundwater with higher pressure will infiltrate into the rock more easily. So the groundwater pressure may be an important factor affecting the swelling, which need to be considered in design. In this way, the temperature and the water quality also may be the affecting factors, which need to study by tests.

4.3 About the thickness of swelling ring around the holes and cavities

Before a large amount of in-situ tests, it is too early to discuss this problem. The previous tests show that in addition to the lithologic characters, excavation size, exposure time before lining, etc., the fracture intensity, the microfracture intensity and groundwater pressure are also the affecting factors of the thickness of swelling ring.

4.4 About the effects of ground stress

In underground constructions the effects of ground stress are involved to many engineering problems. The deep in-situ swelling tests also may meet the effects of ground stress.

For soft rock with obvious plastic deformation, the swelling pressures getting from ISI tests certainly including a part of ground stress. For hard rock the ground stress may expand the microfractures and affect the test results indirectly. So the swelling indexes getting from ISI tests could be looked as a kind of mixed indexes.

These indexes may be more meaningful for underground engineering.

All above problems will be solved only after a large numble of in-situ tests.

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The time effect on the undrained cohesion recovery in clayey soils, using the vane test

L’effet du temps sur la récupération de la cohésion non drainée dans des sols argileux, en utilisant l’essai au scissomètre

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ABSTRACT: Three different soft clay formations characterized from a mineralogical and geotechnical point of view were tested by laboratory vane tests. The repetition, at specific time intervals, of the shear test of the same samples after different consolidation histories underlined the effects of the undrained cohesion recovery. The phenomena which differed from clay to clay showed a clear relationship with the consolidation and with the "rest" period between two consecutive tests. Among the various possible reasons for such a process, the thixotropic phenomena, to compare with clays characteristic, seem to be important. Likewise, the displacement magnitude of the shear surface (cylinder), seem to be related to the destructuration and reorganization of the solid skeleton, in terms of cohesive undrained bonds.

RESUME : On a testé au scissomètre de laboratoire, trois différents formations d’argile molle qui avaient été déjà caractérisées du point de vue minéralogique et géotechnique. La répétition du test à la coupe, par intervalles de temps précis, sur la même échantillon après différents cycles de consolidation, a souligné les effets du recouvrement de la cohésion non drainée. Les phénomènes, qui sont différents d’une argile à l’autre, ont montré une claire relation avec la consolidation et avec la période de repos entre deux tests consécutifs. Parmi les plusieurs raisons possibles pour un tel procès, ce sont les phénomènes thixotropiques, comparés avec les caractéristiques des argiles, qui sont très importants. De même, la quantité de déplacement de la surface de la coupe (cylindre) semble être en relation avec la déstructuration et la réorganisation du squelette solide, en ce qui concerne les liaisons cohésive non drainées.

1 INTRODUCTION

The undrained cohesion values supply analytic data of fundamental importance when dealing with the properties of clays. Where short term tests are required, the numerical quantification of undrained cohesion in terms of peak and residual values, in known experimental conditions, represents the usual datum determined in-situ or in the laboratory. On the other hand, the process of post-shear reconstruction of interparticle bonds in time, represents an unknown even today, and has rarely been dealt with in the specific literature. Nevertheless, the problem itself cannot be ignored, even when only considering the importance of the recovery of the undrained cohesion over a period of time when taken from ultimate residual shear values to peak values.

Such a situation finds a practical application when shear displacement in a soil has been observed and the consequent undrained cohesive values which could be found range between the peak and residual shear values. It is the time function itself, over and above the numerous components of the system, which intervenes and plays a fundamental role on the reconstruction and reorganization of the cohesive bonds, until finally, in extreme situations, it recreates similar pre-shear constitutive conditions (Vaid & Campanella, 1977). Thixotropic and sensitivity phenomena have been related to various factors and properties, such as water content, plasticity and the activity, consolidation, nature and composition of the sediment, as well as the presence of different mineral clays (Mitchell & Houston, 1969). Where different micro-structural behaviour is concerned, other factors are present such as salt concentration, pH, C.E.C. specific surface, zeta potential,
isolectric point, organic matter, hydrolysis etc. In the present paper, three different clayey formations occurring in the Italian Apennine have been taken into consideration: Caotico Complex clay, Colombaccia clay and Pliocene clay formations. For each, a preliminary X-ray diffraction analysis on the 2 μm passers and some general and geotechnical characterization have been carried out. Shear tests were performed using vane tests in the laboratory on remoulded and reconsolidated samples at 50, 100, 150 kPa. By repeating the tests using pre-determined pauses, and by rotating the blade of the instrument which was firmly inserted in the cylindrical mould sampler, it was possible to obtain the peak and residual cohesion values in a known and constant water content condition.

The contents discussed in this paper depend on five main phenomena which partially interact with each other:
1) effects of pore pressure variations along the cylindrical shear surface of the vane;
2) effects of pore pressure modification due to the shear speed used;
3) variations of pore pressure related to the end of the consolidation process (primary and secondary);
4) cohesion recovery in thixotropic terms;
5) total or partial destructuration of the solid skeleton connected with shear displacement magnitude.

Although the data obtained in this present paper using the vane test supply quantitative information concerning the above-mentioned phenomena, they do not permit investigation into the possible pore pressure variations which could arise during the experiment.

By comparing the results obtained, it appears possible to put forward the hypothesis that the last two factors, listed above, play a determining role in the processes illustrated.

As for the cohesive recovery in function of the consolidation adopted, already pause times ranging between 20 to 70 hours in direct shear tests demonstrated considerable increases in tertiary clay blocks (Rybički, 1991).

2 STATE OF THE ART

In the literature there are relatively numerous experiments in-situ to determine the undrained shear strength using the field vane. Chandler's recent paper (1988) represents a very significant reference for its thorough analysis of the state of the art. In particular, it deals with the most relevant implications to be found using the field vane, i.e. the disturbance to the soil due to vane insertion, the consequences of pause periods and the shear rates adopted, and the anisotropy and consolidation of the soil.

As for the possible correlations with different experimental procedures such as the use of different triaxial apparatus, the contribution of numerous authors has been of significant value (Anderson & Woods, 1976; Kirkpatrick & Khan, 1981; Wroth, 1984; Jamiołkowski, et al., 1991). Earlier scientific papers have widely dealt with the influence of the different blade sizes of the instrument, shear rates and "rest" periods adopted before commencing experiments.

The insertion of a field vane in the soil provokes a disturbance which is all the smaller the smaller the ratio between the transversal area of the blades and the transversal area of the shear cylinder, while, according to Plaate (1966), the area ratio should not be more than 15%.

Today, a perimeter ratio $R = 4C/rD$ is usually used, with $D$ equal to the diameter of the shear cylinder (blade diameter) and $C$ equal to the vane blade thickness. By adopting blades $D = 10$ mm and $C = 0.7$ mm, in the present experiment, the result of $R$ is optimal and less than 0.09 (La Rochelle et al., 1973).

The insertion effects of the field vane can be synthesized from the following factors:
- destruction of interparticle bonds and destructuration in sensitive clays which produce a significant reduction in the undrained strength (Mitchell & Houston, 1969);
- local migration of particles and the consequent increase of pore pressure around the shear cylinder (Kimura & Saitoh, 1983).

When considering the "rest" period between the insertion of the blades and the beginning of the tests, Aas (1965), noted an increase of 20% to 50% of undrained strength in function of the sensitivity for "rest" periods longer than standard (5 mins). This could be due to the partial dissipation of pore pressure and therefore to the increase of effective pressure. The time required to reach 90% of maximum values varies from 10 to 20 hours depending on the type of clay.

However, it remains constant after a "rest" period of 24 hours (Torstenson, 1977).

The coefficient of consolidation $C_v$ influences the test results due to the fact that its high values are caused by the greater possibilities of local drainage.
paths (Roy & Leblanc, 1988).
The time factor also influences the shear rate of the soil. Increases of 20% in undrained shear strength have been observed when standard testing conditions (6 deg/min rotation) are increased to 60 deg/min (Calding & Odestand, 1950; Perlow & Richards, 1977; Wiesel, 1973).
Blight (1968) discussed the rates of rotation imposed on the instrument in undrained shear conditions using the field vane.

In his paper which dealt with the concept of "spheres of influence" The works of Masui & Abe (1981) and Roy & Leblanc (1988), also refer to the same subject. By taking the value of the consolidation coefficients Cv and the diameter D of the blades used, the rotation rate to be imposed on the instrument can be deduced in order to obtain undrained conditions (degree of drainage U < 10%). By using angular rates of 200 deg/min for the three clays being tested, maximum rupture times ranging between 12 to 21 seconds were reached, which, according to the various Cv values obtained certainly result as being undrained shear conditions (U < 10%).

Finally, through comparison of bi-tri-dimensional back-analyses obtained on slopes, a correction of undrained strength values through empirical coefficients related to the plastic index, soil anisotropy and strain rate effect, has been suggested for the field and laboratory vane (Bjerrum, 1972; Lunne et al., 1977; Menzies & Merrifield, 1980; Azzouz et al., 1983; Mesri, 1989-1993; Morris & Williams, 1993).

### Table 1 - Characteristics of the tested clays.

<table>
<thead>
<tr>
<th>FORMATIONS</th>
<th>A</th>
<th>B</th>
<th>C</th>
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<tbody>
<tr>
<td>σ'v (kPa)</td>
<td>W (%)</td>
<td>W (%)</td>
<td>W (%)</td>
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<td>K-Feldspar (%)</td>
<td>8</td>
<td>4.8</td>
<td>2</td>
</tr>
<tr>
<td>Plagioclase (%)</td>
<td>3</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Calcite (%)</td>
<td>-</td>
<td>36</td>
<td>35</td>
</tr>
<tr>
<td>Dolomite (%)</td>
<td>-</td>
<td>2.6</td>
<td>5</td>
</tr>
<tr>
<td>Phyllosilicates + Acc. (%)</td>
<td>72</td>
<td>45</td>
<td>45</td>
</tr>
<tr>
<td>Illite-Smectite (%)</td>
<td>15</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>Smectite (%)</td>
<td>50</td>
<td>60</td>
<td>25</td>
</tr>
<tr>
<td>Kaolinite (%)</td>
<td>30</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>Illite (%)</td>
<td>5</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>Chlorite (%)</td>
<td>-</td>
<td>15</td>
<td>20</td>
</tr>
</tbody>
</table>

3 METHOD

The soils analysed came from the clayey formation of the Caotic Complex clay, Colombacci clays and Pliocene clays, all samples outcropping in the Marche Apennines with analogous strata and morphometrical attitudes.
The tested formations were remoulded in the laboratory and passed through a sieve 0.42 mm, so as to eliminate any possible heterogeneity present. They were then mixed in distilled water with a percentage ranging between Wp and WI and left to one dimensionally consolidate in special stainless steel moulds with drainage holes, at respectively 50, 100 and 150 kPa (critical state conditions).
The following table 1 summarizes the water content as various degrees of consolidation were reached, as well as some of the mineral and geotechnical properties of the three clayey deposits.

Co-relationships and discussions on the data presented in the table have been thoroughly dealt with in previous papers (Elmi et al., 1985; Gori & Vanucci, 1987; Gori, 1989).

After a consolidation time of 60 hours and after a further "rest" period of 48 hours, the consolidation cylinders, the guide ring and the soil which exceeded the top of the mould were removed. The various moulds were then closed with metallic caps, complete with lock, in order to block any possible swelling up of the clay over a period of time and to maintain the same consolidation pressure which had been adopted. On each cap, a cut was made in the centre in the form of a cross to allow the insertion of the vane blades, co-axial to the mould itself, without having to re-open the container.

A plastic arbor collar with a hole in the centre and blocked by silicone kept the blades in exactly the same position, in such a way as to be able to constantly mobilize the same rupture surface (cylinder), for repeated shear tests. Once the vane blades had been inserted in the moulds, a pause of
Table 2 - Consolidation data of the tested clays

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>$\sigma'_v$</th>
<th>Wn</th>
<th>$C_v$</th>
<th>$t_{50}$</th>
<th>$A_H$</th>
<th>$\alpha_{\text{max}}$</th>
<th>$t_r$</th>
<th>T</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kPa</td>
<td>%</td>
<td>cm²/s</td>
<td>min</td>
<td>cm</td>
<td>°</td>
<td>sec</td>
<td></td>
</tr>
<tr>
<td>Caotic Clays</td>
<td>100</td>
<td>61</td>
<td>$3.3 \times 10^5$</td>
<td>100</td>
<td>0.150</td>
<td>39</td>
<td>12</td>
<td>$4 \times 10^{-4}$</td>
</tr>
<tr>
<td>Colombian Clay</td>
<td>100</td>
<td>43</td>
<td>$4.5 \times 10^4$</td>
<td>8</td>
<td>0.165</td>
<td>70</td>
<td>21</td>
<td>$9.5 \times 10^{-3}$</td>
</tr>
<tr>
<td>Pliocene Clay</td>
<td>100</td>
<td>34</td>
<td>$3.3 \times 10^4$</td>
<td>10</td>
<td>0.130</td>
<td>70</td>
<td>21</td>
<td>$6.9 \times 10^{-3}$</td>
</tr>
</tbody>
</table>

48 hours was adopted before beginning the experiment, so as to dispel any pore pressure connected to the disturbance produced on the sample. Between the various tests, all the moulds were immersed in distilled water at constant temperatures.

As far as the shear rate is concerned, undrained conditions were considered when the degree of drainage U resulted much less than 10% (Blight, 1968). Such a hypothesis is obtained when the time factor T is less than 0.05. By using the equation $T = \frac{t_r C_v}{D^2}$, where $t_r$ is the rupture time during the test and D is the diameter of the vane blades, the data showed in table 2 have been obtained when rotating the instrument at 200 deg/min.

They confirm that the adopted rate was more than suitable so that the shear rate of the clay being tested can occur in undrained conditions. Further confirmation of the procedure followed can be had by comparing the shear values (1st and 2nd shear during the first day of testing) obtained in the same clayey sediment when this sediment was subjected to the same consolidation pressure.

However, these values referred to an average of three moulds which resulted as being very homogeneous among all three.

Table 3 shows the shear schema adopted, where it is possible to note the time factor with pause between two consecutive tests, varying between 1 and 6 days. Therefore, in all, 27 moulds were tested for each clay formation, 9 of which consolidated at 50 kPa, 9 at 100 kPa and 9 at 150 kPa.

4 RESULTS

The data included in the following graphs, in each case, represent the arithmetical average of three values. The data are presented in the cartesian plane, where $\alpha = $ angular deformation values of the vane spring and where $d = $ rest period in days between one shear and another, following initial shearing. The values $\alpha$ multiplied by the blade dimensionless and by the constant of the spring used on the vane, equal to 0.32, supply the data of undrained cohesion expressed in kPa.

The graphs of the figures refer to the three different values of the adopted one dimensional consolidation pressures. The solid lines represent
the angular values of the first shear (peak value) and of the following re-shears repeated over a period of time (first residual values). On the other hand, the dashed line represents the ultimate residual angular values obtained through further and subsequent rotation of the blades, carried out 2 minutes after determining each previous residual value.

In order to analyse the results obtained more thoroughly, the relative variations $\Delta \alpha_1$ between the residual values (first residual values minus ultimate residual values) on the same shear day are discussed. In the same manner with $\Delta \alpha_2$ are show the values which refer to the differences between the first residual values obtained at the various pauses and the ultimate residual value relative to the previous test.

In short, in comparison to the anticipated pattern of the phenomenon shown in figure no.1, $\Delta \alpha_1 = (\alpha_n - \alpha_{n+1})$ represents the variation or relative loss of undrained resistance while $\Delta \alpha_2 = (\alpha_n - \alpha_{n-1})$ expresses the variation or relative increase of undrained resistance.

![Fig.1 - Anticipated pattern of the angular $\alpha$ value of the shear Vane test versus rest period following initial shearing.]

Therefore, as far as the anticipated model is concerned, the ultimate residual shear value should remain constant over a period of time, while the values of the first residual shear should increase when "rest" times between two consecutive tests are longer. Furthermore, these values (first residual shear) should remain constant, even when repeating the test with the same time interval between two pairs of consecutive shears. The different behaviour that emerges from the data discussed, over and above the problem of the complete soil destructuration and the eventual thixotropic recovery, can be attributed to other factors, such as:

1) phenomena of variations of pore pressure which can occur along the vane shear cylinder;
2) effects of pore pressure variations due to primary and secondary consolidation processes;
3) the influence of the horizontal shear stress during the consolidation of the sample (Ladd, 1991).

In spite of the above discussion on the choice of shear rate in function of the drainage, where the first of these factors is concerned, it is important to underline the significance of these phenomena - produced by rotation of the vane blades - on the shear surface, and even more so on the top and bottom surfaces of the clay cylinder.

In such a situation it is possible to obtain laboratory data which is not comparable to results from back-analysis, especially for over-consolidated soils. This has recently been experimented using triaxial apparatus on London clay sediments (Atkinson & Richardson, 1987).

In the second case, the information relative to the secondary consolidation effects on the undrained conditions and on the shear time, is explained by introducing the terms $C\alpha$ - compressibility related to time - and $Cc$ - compressibility related to the effective stress - (Mesri & Castro, 1987).

Adopting the ratio $C\alpha/Cc = 0.04$ an undrained cohesion value at a shear time of one hour in a vane test represents 85% of what could have been obtained in the same test carried out in one minute (Mesri, 1987). The end of the secondary consolidation produces a decrease of effective stress on the soil which is proportional to the length of time of the phenomenon with the consequent assumption of an excess of pore pressure (Mesri & Choi, 1979). The link of undrained cohesive value with the degree of overconsolidation of the soil has often been investigated (Jamiolkowski et al., 1985; Ladd et al., 1977). The results are expressed in linear form and subject to slight differences concerning exponent $m$, generally ranging between 0.80 and 1.35.

Unfortunately, on these subject matters and especially in the third above mentioned factor, due to the limits of the apparatus used, the laboratory vane experiments do not permit in depth knowledge which, could perhaps, clarify the terms of the problem. Therefore, the data discussed, although quantitative at the present state of the art, should only be considered from a qualitative point of view (Gori, 1991).

In this way, the procedure seems to have little influence on the phenomenon. In fact, for the same
clay at the same consolidation, the data relative to the average of three values of the peak shear and of the three residual shear values obtained on the first day of testing, are very close, with only a difference of a few degrees.

In the following comment, $\Delta \alpha_1$ and $\Delta \alpha_2$ variations, the absolute $\alpha$ values for each clay, in function of the "rest" times between the various tests and the different consolidation adopted will be considered contemporarily. Since it was impossible to thoroughly investigate the causes of the phenomena, the following will refer only to the results obtained from the various consolidation adopted and the displacement magnitude.

4.1 Caotic Clays

Referring to the $\alpha$ variable in Figs. 2, 3, and 4, it is possible to observe that the initial peak value is strictly connected to the pressure of the consolidation adopted, while, its consequent residual value appears to be independent of it.

In the successive shear carried out with pauses of one, two and three days, it is possible to observe an increase in the $\alpha$ values, proportional to the consolidation pressure adopted, and in the same way, a decrease of the corresponding ultimate residual value. Furthermore, the latter tends to stabilize itself, in the same way as the first residual value does, both in cases of longer pauses and in cases of repetition of the shear with the same lapse of time. Therefore, considering the $\Delta \alpha_1$ value, it is possible to deduce that when the consolidation pressure is increased, from the first day of shear, it produces an increase of the $\Delta \alpha_1$ variation. The data concerning $\Delta \alpha_1$, however, do not immediately reach the more frequent values. This occurs only after repeated shears. Although after every vane shear test operation resetting is carried out by a rotation of up to 360 deg, it is probable that the two shear operations carried out on the first day are not sufficient to produce the complete desctructuration of the clay, notwithstanding the original remoulding. The "relict" forms tend to reorganize themselves by remobilization of interparticle attractive bonds - the longer the "rest" period the greater the intensity of the latter.

In the following shear, the $\Delta \alpha_1$ value tends to stabilize itself at about 5 deg - 6 deg, showing a considerable increase with consolidation of 150 kPa.

As far as the $\Delta \alpha_2$ values are concerned, the most significant variation can be observed on the 4th day. With a three day pause period between two consecutive shears, the effect of an evident reconstruction of cohesive bonds is produced compared to that produced in "rest" periods of one or two days. By varying the "rest" time, the $\Delta \alpha_2$ values are slightly lower compared to $\Delta \alpha_1$ values and tend to increase the stronger the consolidation, as it can clearly be seen, for the 150 kPa line.

4.2 Colombacci Clays

When compared to the previous clay, the $\alpha$ values are clearly higher and very distinct at the various consolidation pressures, (Figs.5, 6 and 7) both for the peak value and, surprisingly, for the first and ultimate residual values. The $\alpha$ values always increase during pause periods of one, two or three
days in relation to the pressure adopted on the moulds. In time, they then tend to stabilize themselves. The ultimate residual values tend to decrease sharply during pause periods of one, two and three days and even during longer pause periods between two consecutive tests, although this is much less evident.

As far as $\Delta \alpha_1$ is concerned, during the first shear day, the relative loss of undrained strength results all the greater the stronger the consolidation. This phenomenon is very evident, for different "rest" times, although with some overlapping of the lines relative to 100 and 150 kPa. This pattern is still confirmed, especially if referred to data obtained with consolidation of 50 kPa. The $\Delta \alpha_2$ variations deriving from the relative strength recovery and in spite of the above-mentioned overlapping lines, generally show increasing values using the adopted consolidation pressure. This phenomenon is particularly evident with pause periods of 4 - 5 days (from 8 - 10 deg for 150 kPa).

4.3 Pliocene Clays

This sediment seems to show some what anomalous behaviour in some respects when compared to the clays illustrated previously. The peak and residual $\alpha$ values are the highest during the first day of experiments and they confirm their dependence on consolidation.

**Figs. 5, 6, 7** - Colombaci clays. Angular $\alpha$ variation versus different rest period following initial shearing.

**Figs. 8, 9, 10** - Pliocene clays. Angular $\alpha$ variation versus different rest period following initial shearing.
pressure. This aspect is less evident compared to the other two above mentioned clays. By repeating the shear with one day pauses the $\alpha$ values further decrease both in the first and ultimate residual value. This trend attenuates and disappears during longer pauses between two consecutive shears (Figs. 8, 9 and 10). In time, the ultimate residual value has the tendency to decrease, above all for short pauses (up to four days), between consecutive shears.

The data relative to the $\Delta \alpha_1$ variation underline a certain dispersion of values as the time between tests increases, with some overlapping of the line relative to the different adopted consolidation. The value of $\Delta \alpha_1$ goes as far as 16 deg with pauses of five days and with consolidation of 150 kPa and even 18 deg for pause periods of three days and 100 kPa. Some negative $\Delta \alpha_2$ values show the same pattern as for the $\alpha$ value. In particular, this anomalous behaviour is connected with the fact that the first residual value obtained on the second shear day, should at least be equal to the value obtained on the first shear day and certainly not inferior. Only during successive shears the $\Delta \alpha_2$ values become positive even with pauses of one day only, (except for the 50 kPa line, relative to the fifth day). This fact can be explained by considering the magnitude of the displacement of the shear cylinder. This magnitude is probably not yet sufficient to produce the total destructuration of the clay. However, it is evident that a slight increase of the $\Delta \alpha_2$ values is related to the increased length of the "rest" period between two successive tests, while the overlapping of the lines at different consolidations do not allow any conclusions to be drawn concerning their behaviour.

On the basis of what has been discussed up to the present on the three clays being tested, the following conclusions can be drawn:

- the rotation of the instrument during the shear phase with the same adopted rate produces different effects related to the consolidation to which the sampling moulds have been submitted;
- the recovery of the undrained cohesive residual strength tends to increase with the length of the "rest" period between two successive shear tests and with the consolidation adopted on the sample;
- the cohesive bond re-organization is strictly related to the magnitude of displacement of the shear cylinder, i.e. the total clay destructuration does not always occur within similar rotational values.

In the case of Pliocene clay, for example, greater displacement is necessary to stabilize the first and ultimate residual values. Furthermore, the pause effect, if made to last more than one day between two consecutive tests, shows slight but significant influences on the cohesive recovery process. Such an assumption can explain the behaviour observed, if it is related to the reduced activity of this clay; ($A = 42\%$), to the reduced values of consistence limits; to the grain-size loam and clay distribution and finally to the minute percentage of smectite (25%) and illite (20%).

On the other hand, in the Caotic Complex clay, the activity (75%), the higher clay fraction (80%) with 50% smectite and the high values of consistence limits produce an immediate reaction in the soil where the pause periods and cohesive recovery are concerned. This occurs in such a way that even only after one day and moreover in the following days an appreciable variation of the angular $\alpha$ value, strictly related to the adopted consolidation, can be observed.

Finally, in the Colombacci clays with a low activity, grain-size distribution and index properties similar to Pliocene clays, but with illite (15%) and elevated quantities of smectite (60%), a marked and immediate trend towards cohesive recovery, strictly related to the adopted consolidation pressure, can be observed. Furthermore, the last residual values clearly tend to decrease until they stabilize themselves after long pause periods.

Before concluding, some aspects of this research paper make it seem possible to put forward the hypothesis of the existence of a thixotropic phenomenon. On this subject, undrained cohesive recovery, similar to that dealt with in the present paper, has been observed in Detroit clays with similar sensitivity and analogous pause periods, with identical magnitude (Skempton & Northey, 1952).

At present, this same research is being carried out with an Vane borer and the first results are comparable.

5 CONCLUSION

The observations discussed up to this point seem to confirm the relevant implications to be inferred
from the micro-structural behaviour of clays in relation to their mineralogical and geotechnical properties. These phenomena are clearly evident where undrained residual cohesion, time and consolidation factors are concerned and have direct consequences on the reconstitution of interparticle bonds.

In spite of the difficulty of being objective when interpreting the phenomenon under investigation and considering the effects of the variations of pore pressure during the shear phases and the eventual relationships with secondary consolidation processes, some essential aspects emerge from this present research:

- dependence of the peak undrained cohesive value, first and ultimate residual value on the consolidation pressure adopted during the laboratory vane tests, caused by the magnitude of the displacement of the shear cylinder and the variations of pore pressure;
- influence of the activity, grain-size distribution and above all the composition of the clay fraction compared to the cohesive recovery, probably in thixotropic terms;
- time relationship between different "rest" periods in consecutively repeated shear tests and residual cohesive recovery;
- conditioning of the significance of the sensitivity of the clays in the above-mentioned terms.

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New analysis method of hydraulic testings for three dimensional fracture modeling in rock mass
Nouvelle méthode d’analyse d’essais hydrauliques pour le modelage tridimensionnel des fractures dans un massif rocheux

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OYO Corporation, Tokyo, Japan

ABSTRACT: In hydraulic investigation of fractured rock mass, it is necessary to identify three dimensional hydraulic characteristics of individual fracture. Pulsation Test system has been developed to identify three dimensional permeability distribution, and applied to actual site investigations successfully. In Pulsation test numerical analysis, it is significant problem to import geological information such as geometrical fracture network and single hole permeability testing results into numerical model for initial guess in inversion analysis.

In order to estimate appropriate hydraulic characteristics and unknown parameters in actual transient hydraulic testing results, two analyses that are based upon analytical solution of transient radial flow in infinite reservoir and finite reservoir has been developed aiming at total hydraulic analysis. Successful solutions have been acquired in numerical experiments and some actual site investigations.

RÉSUMÉ: En investigation hydraulique des roches fissurées, il est nécessaire d’identifier la perméabilité de la fracture individuelle en troisième dimension. 'Pulsation Test' a été développé pour identifier la distribution de la perméabilité en troisième dimension, et il a été appliqué dans quelques investigations actuelles. Dans l’analyse numérique du essai, il est problème important d’introduire: information géologique; géométrie de la fracture, résultat de l’essai de perméabilité dans des puits pour l’inversion numérique.

Pour estimer la qualité hydraulique et les paramètres inconnus en essai de perméabilité, deux analyses qui sont basées sur les solutions analytiques des courants radials en aquifère fini et infini sont développées. Des succès en expérimentations numériques et investigations sur place furent réussis.

1. INTRODUCTION

Concerning to the hydraulic investigation of rock mass, it is very important to identify distribution of hydraulic properties aiming at grouting programs in dam sites or underground spaces. In actual hydraulic investigation, conventional single-hole steady state well testing such as Lugeon test have been widely applied. However, for the investigation of inhomogeneous and fractured rock mass, it is indispensable to identify geometrical distribution of hydraulic properties or continuity of fractures.

For this purpose, the authors have been developing total hydraulic testing and analyzing system using transient data of pressure and flow rate performance. One is new crosshole hydraulic test named Pulsation test system, and the other is transient data analysis of single hole well test.

Pulsation test is new investigation method belonging to cross-hole interference test with numerical inversion. The authors have applied this system to some actual site investigations.

Through actual site investigations with pulsation test numerical inversion, the authors are
aware of the importance of appropriate initial hydraulic model and boundary conditions. In this reason, single hole transient well test analyses have been developed to identify initial piezometric pressure and continuity of single fracture.

2. PULSATION TEST SYSTEM

2.1 Testing procedure

A schematic view of Pulsation test is illustrated in Figure 1. Pulsation test is proceed by cyclic water injection and shut-in into ground at an interval isolated by double packers.

![Schematic diagram of Pulsation test system](Image)

Fig.1 Schematic diagram of Pulsation test system

Pressure induced by water injection transmits to receiving points according to hydraulic characteristics in testing region. The injection flow rate, pressure at injection interval, injection pressure on the surface, and pressure at each receiving interval are monitored and recorded simultaneously by personal computer.

2.2 Testing apparatus

Pulsation test apparatus consists of pressure pulse generator and receiver device. The authors have developed two kinds of pressure pulse generator for saturated and unsaturated zone. Pressure pulse generator for saturated zone is introduced in another paper (Kondoh et al 1991). In this paper, pressure pulse generator for unsaturated zone would be introduced.

In unsaturated zone, pressure pulse induced by water injection can not be transmitted through unsaturated air-filled ground or fractures. For this reason, the authors have developed air injection pulse generator called Air Pulsation Test device. Block diagram of Air Pulsation Test device is shown in Figure 2.

![Testing apparatus of Air Pulsation test](Image)

Fig.2 Testing apparatus of Air Pulsation test

Air Pulsation Test device consists of air compressor, air quality control unit, pressure control unit, and drill-hole unit. Air quality unit is installed to remove oil mist and dust in compressed air. Pressure control unit consists of regulator valve for pressure control, shut-in valve for cyclic shut-in, and air flow-meter. Three sets of flow-meter that have different measuring range are installed in pressure control unit for the sake of availability to rock masses wide ranging in permeability. The measuring range of each flow-meter is shown below.
In numerical analysis, the basic equation of one phase compressible fluid flow through compressible inhomogeneous media is utilized, which is known in equation (1).

\[
\nabla \left( \frac{K}{\mu B} \nabla (P - \rho Z) \right) = \frac{\partial}{\partial t} \left( \frac{\phi}{B} \right) + q \cdots \tag{1}
\]

where, \( K \) : permeability of rock mass [L^2]
\( \mu \) : static viscosity of fluid [ML^{-1}T^{-1}]
\( B \) : formation volume factor of fluid [-]
\( P \) : piezometric pressure [ML^{-1}T^{-2}]
\( \rho \) : column density of fluid [ML^{-2}T^{-2}]
\( Z \) : depth [L]
\( q \) : sink/source term [T^{-1}]
\( t \) : time [T]
\( \phi \) : porosity of rock mass [-]

Formation volume factor denotes the volumetric ratio of fluid in the same mass under standard pressure and underground.

In pulsation test analysis, finite difference method is utilized for numerical calculation. In forward calculation, spatial permeability distribution \( K \) is given in advance, and transient pressure changes \( P(t) \) at each spatial point caused by water injection, drainage, excavation of tunnel or underground space is calculated. However, in pulsation test numerical inversion, transient pressure changes at receiving points \( (P_i(t)) \) at \( i = 1, n \) are given in advance and spatial permeability distribution \( K \) is unknown and calculated with nonlinear least square method.

In numerical inversion, pressure performance at injection and receiving point is calculated with forward simulation, and permeability distribution would be modified in order to minimize objective function (squared sum of residual shown in equation (2)).

\[
J = \sum_{i=1}^{n} \sum_{j=1}^{m} \left( W_{ij} \left[ P_{\text{obs}}(i,j) - P_{\text{cal}}(i,j) \right] \right)^2 \cdots \tag{2}
\]

where, \( n_0 \) : number of injection and receiving points
\( n_t \) : number of time step
\( P_{\text{obs}} \) : observed pressure at point \( j \) and time \( i \)
\( P_{\text{cal}} \) : calculated pressure at point \( j \) and time \( i \) in forward simulation
There have been some studies about numerical inversion in groundwater problem [Carrera and Neuman 1986, Yang and Watson 1988]. However, in numerical inversion of inhomogeneous rock mass, due to large numbers of unknown parameters, efficient algorithm with minimum computing time (minimum numbers of forward simulation) must be introduced. For this reason, quasi-Newton method with optimum control is applied in pulsation test analysis [Tosaka et al. 1991, 1993]. Flow chart of numerical inversion is shown in Figure 4.

In this analysis, the authors aim at application in highly inhomogeneous rock mass, unknown parameter is not the permeabilities of finite difference grids, but inter-grid permeabilities between each grid and adjacent grid. Schematic view of inter-grid permeability is shown in Figure 5.

### 2.4 Treatment of fractures

Groundwater flow in fractured rock mass is conducted by flow paths consisted of fractures. However, in pulsation test numerical analysis, basic equation is based upon flow in porous media, and spatial discretization is based upon three dimensional finite difference grid system. Therefore, fracture with arbitrary angle can not be reconstructed directly in mathematical and geometrical model. In order to import geological information such as fracture network geometry into porous media analysis, the authors introduce fracture network...
into initial model of numerical analysis.

Fig. 6 Schematic diagram of fracture network in Pulsation test numerical analysis

Schematic view of fracture modeling is shown in Figure 6. In this modeling, initial hydraulic aperture of each fracture is assumed from Borehole TV and the result of hydraulic test, and inter-grid permeability distribution is calculated from equation (3)

\[ K_{ij} = \sum_{j \neq i} \frac{(2b_j)^3}{12D} \]  

where, \( K_{ij} \) : inter-grid permeability between grid i and j+1 [L²]
\( n \) : number of fractures intersecting the boundary of adjacent two grids
\( 2b_j \) : aperture of fracture j [L]
\( D \) : thickness of grid boundary [L]

2.5 Field application of Pulsation test

Pulsation test system has been applied to site investigation in fractured rock mass. This investigation aimed at identification of main fractures in welded tuff. In this example, Pulsation test with water injection and air injection has been utilized to saturated and unsaturated zone respectively. Location of drill-hole and scan line is illustrated in Figure 7.

There have been no obvious difference be-

Fig. 7 Scanline location in Pulsation test
tween saturated and unsaturated zone in geological condition or fracture distribution. So, the authors suppose that permeability of

Fig. 8 An example of Water Pulsation test

Fig. 9 An example of Air Pulsation test

both region is not so different from each other.

Two examples of pulsation test with water
injection (Water Pulsation test) and air injection (Air Pulsation test) is shown in Figure 8 and Figure 9. From these results, receiving pressure performance, especially delay of response from injection shows great difference from each other.

3. SINGLE HOLE TRANSIENT DATA ANALYSIS

In pulsation test numerical analysis, it is significantly problematic to import geological information such as geometrical fracture network and single hole permeability testing results into numerical model for initial guess in inversion analysis.

However, in conventional hydraulic testing analyses, steady state flow condition is assumed to estimate hydraulic conductivity. On the other hand, actual testing in site investigation, flow condition can not reach steady state, and some parameters such as initial hydraulic pressure or geometrical extent of permeable zone is unknown.

In order to estimate appropriate hydraulic characteristics and unknown parameters in actual transient hydraulic testing results, the authors have developed two analysis methods using transient data obtained in injection well test. One is Variable Rate Analysis which are based upon analytical solution of transient radial flow in infinite reservoir, and the other is Limited Reservoir Analysis that is based upon analytical solution in finite reservoir.

3.1 Variable Rate analysis

In actual well test, both flow rate and piezometric pressure can not reach steady state in practical time especially in unsaturated zone. Pressure and flow rate performance is extremely different between saturated and unsaturated zone with the same permeability.

An example of flow rate performance in multi-stage constant pressure test obtained in three dimensional numerical simulation is shown in Figure 10. Variable rate analysis is developed in order to identify accurate permeability of saturated and unsaturated zone and initial piezometric pressure around test section with actual transient data.

The basic solution in dimensionless pressure and flow rate is shown in equation (4).

\[
P_i - P_{w_i} = \frac{H_i}{2\pi k \phi h} \left[ \sum_{j=1}^{N} \left( q_j - q_{j,i} \right) (t_{N} - t_{j-1}) \right] + s \quad \ldots (4)
\]

And, dimensionless pressure is shown in equation (5).

\[
P_{w_{N}} = \frac{1}{2} \left( \ln(t_N - t_{j-1}) + \ln \frac{k}{\mu \phi c_w \mu} + 0.80907 \right) \quad \ldots (5)
\]

where, \( P_i \) : initial piezometric pressure [ML^{-1}T^{-2}]
\( P_{w_{N}} \) : piezometric pressure at injection point in time step N [ML^{-1}T^{-2}]
\( k \) : permeability [L^{2}]
\( q_j \) : flow rate in time step j [L^{3}T^{-1}]
\( B \) : formation volume factor of water [-]
\( \mu \) : static viscosity of water [ML^{-1}T^{-1}]
\( \phi \) : porosity [-]
\( c_w \) : total compressibility of rock mass and water [M^{-1}LT^{2}]
\( t_j \) : time at time step j [T]
\( s \) : skin factor [-]

![Flow Rate (l/min)](image)

Fig.10 A result of multi-stage constant pressure test
\( \alpha \): correction factor in unsaturated zone [-]

In conventional method, initial piezometric pressure is assumed to be known from water table in well. However, especially in fractured rock mass, water table in well does not represent initial piezometric pressure within each fracture. In this method, initial pressure and permeability are treated as unknown parameter, and nonlinear optimization is applied to maximize relativity coefficient between left hand side and right hand side of equation (4) from transient pressure and flow rate data.

An example of this analysis with transient data in saturated zone in Figure 10 is shown in Figure 11. Initial pressure and permeability is 1.67 atm and 1.0x10^-4 cm/s, and calculated result is 1.661 atm and 9.44x10^-5 cm/s, these results are in good accordance with each other, and unknown parameters (permeability and initial pressure) are calculated with sufficient accuracy.

![Fig.11 Result of variable rate analysis in saturated zone](image-url)

Another example from transient pressure data in Figure 10 is shown in Figure 12. In this result, it is shown that inclination of plot in pressure build-up and draw-down stage differs obviously with each other. This result is notable feature of this analysis in unsaturated zone, and it is a criterion of ground-water condition around test section.

In multi-stage pressure test in unsaturated zone, transient data only in build-up should be utilized for accurate data analysis.

3.2 Finite reservoir analysis

If the extent of permeable zone such as fracture is limited, pressure and flow rate performance is influenced by boundary of flow path. In this paper, pressure performance of constant rate injection test into single fracture or single aquifer is calculated with radial numerical analysis. In this calculation, permeability is assumed as 1.0x10^-4 cm/s, and radius of reservoir is set from 5m to 50m, and thickness of reservoir is 1 cm. Calculated transient pressure performance is shown in Figure 13.

![Fig.13 Result of constant rate test](image-url)

In this figure result of R=500m (nearly infinite) is illustrated. From this result, if the radius of reservoir is 5m, injection pressure rises rap-
idly about in 1 minute after injection started due to the boundary of aquifer. According to the effect of boundary, permeability would be under-estimated with conventional method of infinite reservoir.

In order to estimate extent of aquifer and appropriate permeability, the authors have developed Finite reservoir analysis. Basic equation of radial constant rate test is shown in equation (6) with same notation as equation (4).

\[
p_t - p_{sf} = \frac{B_t}{2 \pi kh} q \cdots (6)
\]

And, in this method, dimensionless pressure is expressed in equation (7).

\[
P_d = \frac{1}{2} \left[ \frac{1}{4t_D} \right] - \frac{1}{4t_{D_0}} \exp \left( -\frac{1}{4t_{D_0}} \right) \cdots (7)
\]

\(Ei(-x)\) is exponential integral function approximated as equation (8).

\[Ei(-x) = \int_{-x}^{\infty} e^{-u} du = \ln(x) + 0.5772 \cdots (8)\]

And, dimensionless time in equation (7) is expressed as equation (9) and (10).

\[t_D = \frac{kt}{\phi \mu c r_e^2} \cdots (9)\]

\[t_{D_0} = \frac{kt}{\phi \mu c r_e^2} = t_D \frac{r_e^2}{r_e^2} \cdots (10)\]

where, \(r_e\) : radius of reservoir

In conventional analysis, reservoir radius is assumed to be known, and permeability is estimated with type curves. On the other hand, permeability and reservoir radius would be calculated simultaneously with least square method. Flow chart of this analysis is shown in Figure 14.

Fig. 15 Result of Finite reservoir analysis (R=50m)

A result of this analysis is shown in Figure 15. This result is calculated from the data (R=50m) in Figure 10. In this result, the first half of transient pressure data do not indicate linearity, but the second half have linear relationship in this

Fig. 14 Flow chart of Finite reservoir analysis analysis.

This result indicates that in the reservoir with large radius, effect of boundary appears after early stage with infinite radial flow.

Numerical experiments of Finite reservoir analysis are carried out in many cases of permeability and reservoir radius with transient data from simulation of 1 hour injection. And comparison of reservoir radius of model with calculation are shown in Figure 16. In this figure, most of all result is in the area with 10% error. However, in some cases with low permeability and large radius, reservoir radius is underestimated. These results seem to be caused by the reason that effect of boundary is not sufficiently reflected to the transient data in 1 hour injection. And more accurate solution would be obtained.
Reservoir Radius
(calculated)

\[ Y = 0.5 \times Y \]

\[ Y = 1.1 \times Y \]

\[ X = 0.5 \times 10^{-6} \text{cm/s} \]

\[ X = 1.0 \times 10^{-6} \text{cm/s} \]

\[ X = 2.0 \times 10^{-6} \text{cm/s} \]

\[ X = 3.0 \times 10^{-6} \text{cm/s} \]

\[ X = 4.0 \times 10^{-6} \text{cm/s} \]

\[ X = 5.0 \times 10^{-6} \text{cm/s} \]

\[ X = 6.0 \times 10^{-6} \text{cm/s} \]

\[ X = 7.0 \times 10^{-6} \text{cm/s} \]

\[ X = 8.0 \times 10^{-6} \text{cm/s} \]

\[ X = 9.0 \times 10^{-6} \text{cm/s} \]

\[ X = 1.0 \times 10^{-5} \text{cm/s} \]

\[ X = 2.0 \times 10^{-5} \text{cm/s} \]

\[ X = 3.0 \times 10^{-5} \text{cm/s} \]

\[ X = 4.0 \times 10^{-5} \text{cm/s} \]

\[ X = 5.0 \times 10^{-5} \text{cm/s} \]

\[ X = 6.0 \times 10^{-5} \text{cm/s} \]

\[ X = 7.0 \times 10^{-5} \text{cm/s} \]

\[ X = 8.0 \times 10^{-5} \text{cm/s} \]

\[ X = 9.0 \times 10^{-5} \text{cm/s} \]

\[ X = 1.0 \times 10^{-4} \text{cm/s} \]

\[ Y = 0.9 \times Y \]

\[ Y = 1.1 \times Y \]

\[ Y = 10 \times Y \]

\[ Y = 0.1 \times Y \]

\[ 10^{-2} \]

\[ 10^{-1} \]

\[ 10^{0} \]

\[ 10^{1} \]

\[ 10^{2} \]

\[ Reservoir radius (m) \]

\[ Permeability (cm/s) \]

---

Fig. 16 Numerical experiment of Limited reservoir analysis (Comparison of reservoir radius)

Fig. 17 Numerical experiment of Limited reservoir analysis (Comparison of permeability)

with measurement of longer time.

Comparison of permeability of model with calculation is shown in Figure 17. In most of all cases, permeability of model is realized in the calculation. However, in some cases with high permeability, permeability can not be calculated with tolerable error. In least square calculation of these cases, squared summation of residual is not sensitive to permeability change, and this seems to be a reason to these errors.

In these calculations, Limited reservoir analysis is entirely applicable to single fracture or aqifer with 10^-6 to 10^-2 cm/s in permeability. So, this method would be useful in hydraulic test for not only in fractured rock mass but permeable thin sand in mudstone.

4. CONCLUSION AND FUTURE REMARKS

In this paper, the authors have presented new cross-hole and single hole well test analysis.

Pulsation test three dimensional permeability inversion would grant useful information in three dimensional permeability distribution in rock mass. However, for three dimensional hydraulic modeling of fracture network, geological and hydraulic characterization of individual fracture would be necessary.

The new analyses of single hole well test analysis proposed in this paper are useful method for hydraulic characterization of single fracture. And these methods present useful information about boundary condition for three dimensional inversion such as water table in rock mass. So, these methods would be useful in construction of initial guess of permeability distribution of three dimensional inversion in Pulsation test.

However, there are many problems to be solved in these methods including Pulsation test as follows.

1. Development of measuring system and numerical inversion for two-phase flow for more realistic phenomena in unsaturated zone.

2. Development of single hole well test analysis for fracture network in order to identify hydraulic and geometrical characterization around test section.

3. Introduction of geophysical data such as well logging or geotomography into three dimensional inversion.

4. Three dimensional modeling technique for interpretation of three dimensional permeability distribution to three dimensional fracture network.

Now, the authors are developing and refining these analytical methods to integrate the system.
for characterization and modeling of three-dimensional hydraulic characterization and modeling of fractured rock masses.

REFERENCES


Permeability tests on kaolinite and a natural clay liner

Essais de perméabilité sur kaolinite et sur une membrane de sol argileux naturel

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ABSTRACT: A programme of laboratory tests has been undertaken to investigate the change in permeability of clayey soils due to exposure of saline solutions. Remoulded samples of commercially available kaolinite and of a natural soil, occurring at the base of Coimbra sanitary landfill, were prepared using distilled deaired water and salt solutions at varying concentrations. The permeability tests were performed in consolidated samples using a reducing head permeameter. The results showed no significant changes in permeability due to the increase of the chemical solutions concentration.

RÉSUMÉ: On a executé un programme d'essais en laboratoire pour investiguer les altérations de perméabilité deux sols argileux soumis à des solutions salines. On a préparé des échantillons de kaolinite et de sol naturel qu'on a trouvé dans le enfouissement sanitaire de Coimbra, en employant de l'eau distillée et salées à des concentrations variées. Les essais de perméabilité ont été faits sur des échantillons consolidés à l'aide d'un perméamètre à charge réductrice. Les résultats de ce programme montrent que les changements de perméabilité aucun accroissement des concentrations salines soient devenus négligeables.

1 INTRODUCTION

The disposal of waste materials is a matter of increasing public concern. Sanitary landfills were conceived to minimise the risk of pollution due to waste disposal contaminants. However, the construction of sanitary landfills, in sites where the geologic and hydrogeologic features do not guarantee an environmental protection, requires the installation of a lining system. To be efficient the sealing soils that are a part of the lining system, should have low permeability and should not change under the action of the leachate. To investigate the influence of chemical solutions on the permeability of clayey soils laboratory tests were performed. The soil samples were prepared either with distilled and deaired water (DDW) or with salt solutions. Potassium nitrate and potassium sulphate were the salt solutions used in the tests. The nitrate and sulphate anions were chosen because both can be founded in leachates from sanitary landfill (Attewell, 1993; Yong et al., 1992), and the potassium cation was chosen in both cases to reduce the number of variables. The samples were firstly consolidated, under different vertical stress, and then the permeability tests were carried out in a reducing head permeameter.

This paper reports: the permeability changes with the salt solution concentration; the influence of increasing consolidation pressures on permeability; and the soils permeability using DDW compared with those using salt solutions.

2 EXPERIMENTAL PROGRAM

In this work two different soils were used, a commercially available kaolinite in a powder form and a natural occurring soil. These soils were initially classified through index tests and results are summarised in table 1.

The natural soil was firstly pulverised, by using a pestle, and the particles larger than 4.75mm were removed according to BS 1377:Part 6:1990.
Table 1 - Results of the index tests

<table>
<thead>
<tr>
<th></th>
<th>Kaolinite</th>
<th>Natural clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>% gravel</td>
<td>-</td>
<td>5.6</td>
</tr>
<tr>
<td>% sand</td>
<td>-</td>
<td>34.4</td>
</tr>
<tr>
<td>% silt</td>
<td>-</td>
<td>32.0</td>
</tr>
<tr>
<td>% clay</td>
<td>100</td>
<td>28.0</td>
</tr>
<tr>
<td>Wt (%)</td>
<td>60</td>
<td>49</td>
</tr>
<tr>
<td>Wp (%)</td>
<td>33</td>
<td>25</td>
</tr>
<tr>
<td>Ip = Wt - Wp</td>
<td>27</td>
<td>24</td>
</tr>
<tr>
<td>Gs</td>
<td>2.59</td>
<td>2.49</td>
</tr>
<tr>
<td>Classification</td>
<td>MH</td>
<td>CI</td>
</tr>
<tr>
<td>(Plasticity Chart)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Care was taken to remove the air from the samples. Each sample was 100 mm diameter and 30 mm thick and was consolidated under different stress of 50, 100, 200 or 400 kPa. The vertical load was applied and maintained until the primary consolidation finished (usually 24 hours). Before removing the vertical load the thumb screw on the piston was tightened up to avoid swelling of the soil samples.

Then the cell was set up with a cell top plate and connected, in the top, to the pressure chamber and in the bottom with the burette. The system was carefully deaired and filled either with DDW or with chemical solutions at selected concentration. Whole soil samples were immersed in fluid solution appropriate to the particular test. The solutions of potassium nitrate and potassium sulphate were prepared in agreement to molarity principles at 0.001M, 0.01M and 0.1M concentrations. Permeability determinations were then carried out on a reducing head permeameter (figure 2).

This permeameter was designed to test very low permeability soils. An air pressure applied in the bladder and controlled by regulator valve was used to exert a required pressure upon the solution. It was then transmitted at the top of soil samples provoking a flow through the sample and up into the burette. During the tests readings of reducing head were taken as a function of the time. A solution movement of more than 200 mm was always required before the test was considered complete. The permeability was calculated for each load applied during consolidation.

It was assumed that the specimen was fully saturated and the flow was governed by Darcy's law. Therefore the permeability coefficient (K) was calculated from the falling head equation considering the effective pressure head values:

\[
K = 2.3\frac{al}{At} \log_{10}\frac{h_1}{h_2}, \quad \text{(Eq. 1)}
\]

- \(a\) is the cross sectional area of the burette in \(\text{mm}^2\)
- \(A\) is the cross sectional area of the soil in \(\text{mm}^2\)
- \(l\) is the length of sample in \(\text{mm}\)
- \(h_1\) and \(h_2\) are the differential heads in \(\text{mm}\), at the start and finish of the time interval \(t\)
- \(t\) - time in seconds

Figure 1 - Cross section of the permeability cell. (1) Connecting rod; (2) o-ring (3) solution; (4) piston; (5) sample; (6) drainage holes; (7) solution; (8) base; (9) tap; (10) porous disc; (11) loading disc; (12) thumb screw; (13) top cell piece; (modified from McDermott and Selby, 1990).
Once the permeability test was complete the cell was disconnected from the system. The final thickness of the sample was recorded and the moisture content determined. The results were then used to calculate the void ratio of the samples.

Figure 2 - Reducing head permeameter. (1) Consolidation/permeability cell; (2) sample; (3) base; (4) compressed air supply; (5) air pressure regulator; (6) security valve; (7) connection valve; (8) bladder; (9) DDW or solution tank; (10) solution inflow; (11) burette; (12) meter rule; (13) pressure transducer; (14) air valve (15) solution pipe; (16) air pipe.

3 RESULTS

The permeability results obtained from the experimental work are summarised in figures 3, 4, 5 and 6. A logarithmic scale was used for the permeability values. In each figure a upper and lower bound are plotted which correspond to the lowest (50 kPa) and highest pressures (400 kPa).

First to be examined was the influence of increasing salt solutions concentrations on permeability. DDW, potassium nitrates and potassium sulphate at 0.001, 0.01 and 0.1 M concentrations were used. The results showed only very small permeability changes with the increase of the chemical concentrations. However, the kaolinite showed a decrease on permeability with both solutions especially after the concentration exceeded 0.01 M. The natural soil showed a slight reduction in permeability with increasing potassium sulphate; conversely, the soil showed increased permeability with higher concentrations of potassium nitrate. There was no obvious reason to explain the opposite trends presented by kaolinite and natural soil.

Permeability tests performed by McDermott (1989) on different soils (kaolinite, montmorillonite, illite and a naturally occurring soil) to sodium nitrates revealed some small permeabilities changes with increasing salt solutions. In that work only montmorillonite showed a significant change in permeability as function of increasing solutions concentrations. This was explained by the anionic exchange of hydroxyl ions within the montmorillonite minerals for nitrate ions. Similar results were obtained by Hawthorn (1991) using the same soils as McDermott with aluminium sulphates.

Then was analysed the permeability change as function of increasing vertical stress. The consolidation pressures used were 50, 100, 200 and 400 kPa. It was noteworthy that for the same consolidation pressure the void ratios found showed a repeatable result, as presented in tables 2 and 3. This was true for the kaolinite and the natural soil. The repeatable void ratios were an
essential requirement for the test program. Throughout the experiments the void ratio for each soil sample, at the same consolidation pressure, showed no significant variation and hence no important change in permeability can be attributed to the void ratio.

![Graph showing permeability results of the kaolinite soil using DDW and potassium nitrate.](image1)

**Fig. 3 - Permeability results of the kaolinite soil using DDW and potassium nitrate.**

![Graph showing permeability results of the kaolinite soil using DDW and potassium sulphate.](image2)

**Fig. 4 - Permeability results of the kaolinite soil using DDW and potassium sulphate.**
Fig. 5 - Permeability results of the natural soil using DDW and potassium nitrate.

Fig. 6 - Permeability results of the natural soil using DDW and potassium sulphate.

Some permeability tests were repeated under the same conditions with the same solution concentration. They were performed to ensure that the method was accurate. Repeated tests showed a reproducible result.

4 DISCUSSION

The effects of salt solutions on permeability of clay soil can be evaluated with Gouy-Chapman theory (Mitchell, 1976 referred Daniel and Shackelford, 1987) which states that the thickness of double diffuse layer (t) varies with the dielectric constant of the pore fluid (ε), the electrolyte concentration (n₀) and the cation valence (z) as follows:

\[
t \propto [\varepsilon/(n₀ z^2)]^{1/2}
\]  
(Eq.2)

Change in permeability due the chemical solutions is attributed to the change in soil structure due to the change in double diffuse
layer (Meegoda and Rajapake, 1991). As the diffuse double layer of water and cations expands, the permeability decreases because the flow channels become smaller. In our case the dielectric constant of the liquid is relatively constant, the cation valence is the same in both solutions (K⁺), and thus the permeability is dependent on electrolyte concentration. Increasing of electrolytic concentration reduced the thickness of double layer, increasing the permeability.

Kaolinite clay minerals are characterised by a small isomorphic substitution which explains their small reactivity. The X-ray technique was used as well in the natural soil with nitrates to measure the change in basal d-spacing due to increasing the solution concentrations. The results revealed that the basal d-spacing did not change indicating that there was no replacement of silicate ions for nitrates ions, confirming the non reactive character of clay minerals present in tested soils.

The results also showed that the permeability decreased as consolidation pressure was increased. The explanation for this trend can be attributed to the void ratio decrease as result of increasing vertical stress, confirming that the permeability is primarily dependent on the void ratio (Favaretto and Moraci, 1991).

5 CONCLUSIONS AND FINAL REMARKS

A few conclusions that might be draw from this experimental work are:
(1) the permeability of kaolinite and the natural soil seemed not to be influenced by salt solutions used. However the small range of concentration used does not allow a generalisation;
(2) the permeability values showed to be dependent on void ratio, decreasing when it decreased as direct consequence of the increment of consolidation pressures;
(3) the permeability results using DDW or chemical solutions did not show a significant difference.

Although a small number of tests were performed to evaluate the permeability of soils with salt solutions as permeants, it appears that permeability of the two soils tested is much more affected by the density of the soils than the nature of pore fluid. From a practical point of view this is very important since the protection of ground water could be guaranteed to small concentrations of salt solutions. The authors suggest that more tests of similar nature should be carried out using if possible real leachates, and montmorillonite rich soils.

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REFERENCES


Mesure de la pression des sols dans des modèles réduits de sable centrifugés
Measurement of the pressure of the soils in centrifuged reduced sand models

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ABSTRACT: Centrifuge modeling is the most efficient method for well simulating self-weight forces in reduced scale models. But, the field of geostatic vertical stresses in centrifuged sand samples is only supposed easy to know and control and, in addition, horizontal stresses are always totally unknown, as they depend on a lot of different parameters such as sand density, wall stiffness, overconsolidation ratio, G level.

RESUME: La modélisation en centrifugeuse est la méthode la plus efficace pour simuler les forces dues au poids propre des sols dans des modèles réduits. Mais, le champ de contraintes géostatiques verticales régnant dans des échantillons de sables centrifugés est seulement supposé connu et contrôlé, en plus le champ de contraintes horizontales est généralement mal défini, puisqu’ils dépendent de plusieurs paramètres comme la densité du sable, la rigidité des structures d’encaissement, le degré de surconsolidation et l’accélération centrifuge G.

1 INTRODUCTION

La centrifugation de modèles réduits de massifs de sols est la méthode la plus performante pour simuler les forces réelles dues au poids propre des sols (Garnier 1988). Elle permet notamment de reproduire la formation de terrains (sables et argiles) qui se sont déposés et consolidés sous poids propre.

De nombreux problèmes de géotechnique sont abordés par cette technique: glissement de terrains, creusements de tunnels, stabilité des fondations et souterrains. Les résultats expérimentaux ainsi obtenus peuvent contribuer à la validation de modèles numériques, à condition que déplacements, déformations et contraintes soient correctement mesurés.

La mesure des contraintes est la plus délicate car elle nécessite l’introduction de capteurs dont la rigidité est presque toujours différente de celle du sol dans lequel il est placé conduisant à des erreurs importantes (concentration de contraintes sur le capteur ou effet de voûte autour du capteur).

L’objet de cette communication est de présenter les résultats de la réponse de capteurs de pression totale soumis aux contraintes géostatiques verticales de référence que la centrifugation d’un modèle de sable permet de créer. Après avoir comparé la réponse de deux types de capteurs, on étudie l’influence de plusieurs paramètres sur le capteur le plus performant.

2 PRINCIPE DES ESSAIS EN MACROGRAVITE

E. Philips a été le premier en 1869 à proposer d’effectuer des expériences en macrogravité. Cette idée a été mise en application une soixantaine d’année plus tard en URSS et en USA. Ce n’est qu’à partir de 1970 que la modélisation en centrifugeuse a vraiment commencé à se développer dans le domaine du génie civil.

Une expérience sur modèle réduit physique est représentative du comportement de l’ouvrage réel, si les conditions de similitude sont respectées.

Pour les milieux continus, dans l’hypothèse de faibles déformations, les équations d’équilibre s’écrivent:
\[
\sum_{i} \frac{\partial \sigma_{ij}}{\partial x_j} + \rho \left[ g_i - \frac{d^2 \xi_i}{dt^2} \right] = 0 \quad (2.1)
\]

\(X_j\) : désigne les coordonnées ;
\(\sigma_{ij}\) : composante du tenseur de contraintes ;
\(\xi_{ij}\) : composante du tenseur de déplacement ;
\(g_i\) : composante du champ de gravité ;
\(\rho\) : masse volumique ;
\(t\) : le temps.

Si les échelles du modèle par rapport au prototype sont :

\[
\sigma^* = \sigma / \sigma', \quad l^* = X / X', \quad \rho^* = \rho / \rho',
\]

\[
g^* = g / g', \quad \xi^* = \xi / \xi', \quad t^* = t / t'.
\]

Les équations d’équilibre s'écrivent :

\[
\alpha^* \sum_{i} \frac{\partial \sigma^*_{ij}}{\partial x_j} + \rho \rho^* \left[ g^* g' - \frac{\xi^*}{t^*} \frac{d^2 \xi^*}{dt^2} \right] = 0 \quad (2.2)
\]

Ces équations demeurent formellement identiques aux précédentes si on a :

\[
\sigma^* = g^* \rho^* \quad \text{et} \quad \xi^* = g^* \xi^2
\]

Le comportement rhéologique de certains matériaux dépend étroitement du niveau des contraintes auxquelles ils sont soumis (géomatériaux, sols, matériaux granulaires, poudre). Il est alors indispensable de conduire les essais sur modèles à des niveaux de contraintes identiques à ceux de l’ouvrage réel, d’où la condition supplémentaire \((\sigma^* = l)\).

En utilisant en outre les mêmes matériaux aux mêmes densités \((\rho^* = l)\), les conditions de similitude (2.1) se réduisent à :

\[
g^* \rho^* = l
\]

\[
l^* = l^*
\]

Ceci revient tout simplement à augmenter les forces massiques dans le rapport inverse de l’échelle de longueurs.

3 DISPOSITIF EXPERIMENTAL

3.1 La centrifugeuse et le modèle

Les essais sont réalisés sur la centrifugeuse du Laboratoire Central des Ponts et Chaussées de Nantes (France). Ses principales caractéristiques sont : un bras de 5.5m de longueur, réduisant ainsi les gradients d’accélération sur la maquette ; une nacelle carénée, ce qui limite les effets aérodynamiques sur le modèle ; et un ensemble de liaisons électriques et fluides assurant, pendant la rotation, la commande et le transfert vers la salle de contrôle des mesures réalisées sur modèle.

Tous les essais ont été réalisés sur du sable de Fontainebleau dense d’un poids volumique moyen de 16 kN/m3. Ce sable est un matériaux fin siliceux, dont les caractéristiques sont rassemblées sur le tableau 1.

| Tableau 1 : Caractéristiques du sable de Fontainebleau |
|-------------|------------|-----|-------|-----|
|             | \(\gamma_s\) | \(\gamma_d\) | e max | \(\gamma_d\) | e min |
| (kN/m3)     | (kN/m3)    | (kN/m3)      |       | (kN/m3)      |       |
| 26.44       | 13.64      | 0.940        | 16.83 | 0.615        |

3.2 Les capteurs

Un bon nombre de cellules de pression a été développé pour mesurer les pressions dans les sols. La plupart d’entre elles sont conçues afin de minimiser l’influence d’inclusion sur le champ de contraintes réel.


La mesure des pressions dans les sols n’est représentative des pressions réelles que si la déflexion de la membrane sensible des capteurs est limitée, si le sol environnant le capteur est peu remanié, si les conditions de mise en place sont adaptées (Hadala in Hvorslev 1976), (Jarrett 1992); et si les caractéristiques géométriques et mécaniques (rapport surface sensible-surface totale, épaisseur-diamètre totale, rigidité du capteur-rigidité du sol) sont également respectées.

Les essais présentés dans cette communication, ont été réalisés avec deux catégories de capteurs :

1. Les premiers ont un mode d’action direct et un système de mesure électrique : le sol agit
directement sur la membrane sensible portant les jauges de contraintes. Quatre capteurs Kyowa PS-A, de 6mm de diamètre et 0.6mm d’épaisseur, et un capteur JPB en céramique de 38mm de diamètre et 8mm d’épaisseur ont été utilisés. Ces capteurs ont été enfouis dans le sable.

2. Les secondes possèdent un mode d’action indirect et un système de mesure électrique : un liquide incompressible (mercure ou huile de silicone) transmet la pression due au déplacement de la membrane sensible au corps d’épreuve du capteur. Six capteurs Kyowa BE-C de 30mm de diamètre et 9mm d’épaisseur, et deux capteurs Geokon de 40mm de diamètre et 2mm d’épaisseur ont été utilisés. Une partie d’entre eux est enfouie dans le sable, l’autre est encastrée à la surface de structures de différentes rigidités.

3.3 Programme de chargement

Après mise en place des capteurs au sein du modèle réduit, la centrifugation (de 1 à 100G) est réalisée suivant les cycles de la figure 1 : un premier cycle de charge-décharge réalisé par palier de 10G, un second cycle réalisé par palier de 20G, et un troisième cycle de charge-décharge réalisé en continu.

![Figure 1. Profil d’accélération.](image1)

Des essais préliminaires avec un plus grand nombre de cycles ont montré qu’au delà de trois, la réponse des capteurs est à peu près stabilisée.

4 RESULTATS

On présente tout d’abord l’influence du type de capteurs, c’est-à-dire de son mode d’action, sur la qualité de la mesure de la contrainte géostatique verticale. On examine ensuite, pour le type de capteur sélectionné, l’influence de divers paramètres : rigidité des encastrements, mode opératoire, et effet de bord sur la mesure des contraintes au sein d’un massif sollicité en macrogravité.

4.1 Choix du type de capteur

La figure 2 présente la réponse de deux capteurs à action directe au cours de deux cycles de charge-décharge. On constate la non linéarité de la relation contrainte géostatique-contrainte mesurée, une forte hystérésis lors de la décharge, une sous estimation de la contrainte géostatique au cours du chargement, et la forte contrainte résiduelle en fin de premier cycle pour l’un des capteurs (JPB).

Cette sous estimation et cette hystérésis, est due à une mauvaise communication des contraintes à la membrane sensible. En effet la déflexion de cette dernière peut être trop importante (capteur JPB) ; de même les caractéristiques géométriques et mécaniques du sol et du capteur peuvent ne pas respecter un certain nombre de condition (diamètre total, surface sensible, épaisseur du capteur, module équivalent du capteur) : ici le capteur Kyowa PS-A a un diamètre trop faible.

![Figure 2. Réponse type des capteurs à action directe.](image2)

L’effet de la raideur relative sol-capteur sur la mesure des pressions est bien connu depuis les travaux d’Askegaard (1981), dont les résultats obtenus par une approche analytique simplifiée ont été confirmés par calculs aux éléments finis (Clayton et Bica, 1993).

Les réponses des capteurs à action indirecte sont par contre linéaires (figure 3). Les courbes sont caractérisées par une légère hystérésis, et une sous ou surestimation qui n’est ici que de ±5% .

D’une manière générale, sur l’ensemble des essais réalisés avec ce type de capteurs, cet écart par
rapport à une réponse parfaite est en moyenne de ± 10%, suivant les conditions de manipulation et la position de la cellule de pression dans le massif.

Ces capteurs ont des caractéristiques géométriques et mécaniques qui respectent les recommandations des auteurs (faible déflexion de la surface sensible, rapport surface sensible-surface totale voisin de 1, surface de contact sol-diaphragme très importante...).

![Graphique](Figure 3. Réponse type des capteurs à action indirecte.)

Les réponses des capteurs à mode d'action indirect sont donc meilleures que celles des capteurs à mode d'action direct. En effet ce mode permet de transmettre au corps d'épreuve du capteur une pression hydrostatique correspondant à la moyenne des pressions réelles extérieures transmises au capteur ; alors qu'avec le mode direct le principe de la mesure suppose que cette pression réelle est uniformément répartie, à résolution égale, il conduit aussi à des capteurs de plus grande rigidité.

Dans la suite de l'étude seul le premier type de capteurs sera utilisé.

### 4.2 Influence du mode opératoire

On a étudié la réponse de capteurs d'une part enfouis dans le sol, d'autre part encastrés dans un plancher horizontal.

On présente l'influence du mode de pose des capteurs enfouis (a), l'influence de la rigidité des enca斯特ment sur les capteurs encastrés (b), et l'influence du mode d'utilisation des capteurs, c'est à dire enfouis ou encastrés, sur la qualité de la mesure des contraintes géostatiques verticales (c).

#### a) Mode de pose des capteurs enfouis


Les erreurs dues à la méthode de pose des capteurs dans le sol peuvent être calibrées si la méthode adoptée conduit dans tous les cas à des résultats répétitifs.

Hadala recommandait de poser le capteur sur une surface de sable plane ; Askegaard proposait de préparer la surface du sol à l'aide d'un disque métallique identique à la cellule du capteur utilisé ; Jarrett prévoit de perturber manuellement le sol et d'enfoncer le capteur au même endroit ; Hvorslev trouvait que l'application d'une pression résiduelle, durant ou après l'installation du capteur, améliore le contact avec le sol et conduit à la stabilisation de l'erreur de mesure.

On a utilisé deux méthodes de pose des capteurs enfouis au sein du sol "set on surface" et "tamping in". Avec la première le capteur est simplement posé puis recouvert d'une couche de sable non compactée ; avec la seconde le capteur est enfoncé puis recouvert d'une couche de sable compactée.

Les résultats (tableau 2) montrent que, la méthode "set on surface", conduit à des écarts beaucoup plus admissibles.

### Tableau 2. Influence du mode de mise en place.

<table>
<thead>
<tr>
<th>Capteur</th>
<th>Fourchette d'erreur (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Set on</td>
</tr>
<tr>
<td>Kyowa BE-C</td>
<td>-10 à +25</td>
</tr>
<tr>
<td>Geokon</td>
<td>-10 à +20</td>
</tr>
</tbody>
</table>

Cette méthode est préférable car, avec la seconde on exerce d'abord une pression initiale sur la membrane du capteur, puis on compacte le sol qui l'environne ; cela crée un point dur en modifiant la densité locale.

Avec la méthode "set on surface" on ne fait qu'introduire un élément rigide dans le sol, sans modifier sa compacité locale. L'erreur observée ne traduirait que cet effet d'inclusion.

#### b) La rigidité des structures d'enca斯特ements

La pose de capteurs encastrés dans un plancher au fond ou sur les bords du modèle est destinée d'une part à tester leur efficacité pour la mesure des pressions aux contact sol-structures, d'autre part à
étudier l’influence de la rigidité des structures sur la répartition des contraintes au contact sol-structure.

Dans le cas des capteurs encastrés dans un mur ou dans un plancher, on peut obtenir des mesures correctes si les enca斯特ments sont parfait, si la surface sensible du capteur affleure parfaitement la surface du massif d’enca斯特rement, si la rigidité du mur ou du plancher est inchangée, et si la déflexion de la surface sensible du capteur est limitée.

Des capteurs de type Kyowa BE-C ont été encastrés dans deux massifs en bois et métallique. La comparaison des résultats obtenus avec deux capteurs (figure 4), met en évidence le fort sous enregistrement des contraintes réelles et une forte hystérésis dans le cas du plancher en bois. Par contre la réponse du capteur encastré dans le massif métallique est proche de la contrainte géostatique.

Figure 4. Influence de la rigidité des structures.

Les pressions mesurées dans le cas du massif de bois sont nettement plus faibles à cause de la plus forte différence de rigidité sol-capteur-enca斯特rement (Boulebâne et al 92). De plus, le capteur étant placé dans la partie fléchie, une part des contraintes géostatiques est reportée vers les parties tendues du plancher, par développement de voûtes dans le matériau.

Pour le massif métallique, la différence de rigidité capteur-enca斯特rement est minime, et la déflexion est limitée, les mesures sont ainsi beaucoup moins perturbées.

Par conséquent, dans l’interprétations des mesures des pressions de contact sol-structures, on portera une grande attention à la position des capteurs vis à vis des parties tendues et fléchies.

c) Le mode d’utilisation des capteurs

Dans un même modèle on a placé des capteurs de même type (Kyowa BE-C), les un enfouis, les autres encastrés dans un massif métallique.

Théoriquement, les capteurs encastrés ne perturbent pas le sol, une bonne précision devrait être obtenue. Par contre les perturbations causées par les capteurs enfouis (inclusion rigide dans un milieu moins rigide) sont plus importantes, les mesures sont censées être moins précises.

Les résultats obtenus sur un des modèles testés sont présentés sur les figures 5 et 6 pour le premier chargement.

Figure 5. Capteurs enfouis.

Figure 6. Capteurs encastrés.

On observe que, dans les deux cas, les réponses sont semblables et situées dans une fourchette d’incertitude de ±10%. Les facteurs qui peuvent augmenter cette incertitude sont des défauts d’enca斯特rement, d’enfouissement et d’horizontalité des capteurs.

D’une manière générale, l’examen de tous les essais réalisés montre une tendance au sous enregistrement des capteurs encastrés dû à des reports de contraintes sur les massifs.
d’encastrement ; et au sur enregistrement des capteurs enfouis dû au phénomène inverse.

4.3 Influence des effets de bord

La mesure des contraintes géostatiques au sein de modèles réduits centrifugés peut être perturbée à proximité des parois verticales du modèle.

Afin de mettre en évidence cette influence, on a procédé à un talutage des massifs : après avoir effectué un essai classique comportant trois cycles de charge-décharge par rotation à 100G, on talute les bords du massif suivant le schéma de la figure 7, de manière à garder la même intensité des contraintes appliquées aux capteurs, tout en éliminant les frottements du massif concerné sur les parois du conteneur. Le massif de sable est alors soumis à de nouveaux cycles de charge-décharge.

Figure 7. Profil de talutage du modèle.

Figure 8. Influence de bord sur la mesure des contraintes.

On compare les résultats (figure 8) du dernier cycle du massif non taluté avec les résultats du premier cycle du massif taluté. Une différence légère n’apparaît que pour de fortes solicitations, elle n’excède pas 2%.

Ceci montre que dans ces modèles réduits centrifugés, et pour des parois rigides, les effets de bords restent négligeables quand le capteur est placé à une distance au moins égale à 0.25 fois la largeur du conteneur, lorsque l’épaisseur du massif n’excède pas lamoïté de sa largeur.

5 CONCLUSION

La modélisation par la centrifugation constitue un outil efficace pour l’étude expérimentale des problèmes d’interactions sols-structures. Cette méthode connaît actuellement un développement considérable et de nombreux travaux sont entrepris pour améliorer la connaissance des caractéristiques des massifs de sols centrifugés.

La détermination expérimentale du champ de contraintes géostatiques demeure cependant difficile car les résultats dépendent de plusieurs paramètres. L’étude présentée montre clairement que le choix du type de capteurs est primordial. En effet, les cellules de contraintes à mode d’action indirect sont recommandées. Leur réponse est caractérisée par une bonne linéarité, une faible hystérésis, et une sur ou (sous) évaluation de ±10% par rapport à une réponse idéale.

Pour les capteurs enfouis, l’influence de la méthode de mise en place est importante. La méthode "set on surface" est souhaitable.

Pour les capteurs encastrés, l’influence de la rigidité des structures et de la qualité des encastrements est primordiale.

La tendance générale est de sous enregistrer dans le cas des capteurs encastrés, et sur enregistrer dans le cas des capteurs enfouis.

Quand il est placé à une distance de la paroi supérieure à 0.25 fois la largeur du conteneur, le capteur est très peu influencé par un effet de bord.

La mesure expérimentale du champ de contraintes géostatiques est donc possible, bien qu’elle ne soit pas parfaite. En effet, il est difficile d’envisager la suppression de l’influence de paramètres tels que les dimensions des cellules, qui n’autorisent pas une mesure ponctuelle des pressions, et l’effet d’inclusion d’un corps rigide (capteur) dans un milieu moins rigide (sol).

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Influence of strong dynamic impacts on physical-mechanical properties of dispersive grounds
Influence des vibrations sur les propriétés physiques et mécaniques des sols dispersives

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ABSTRACT: As it was shown by experimental studies strong vibrational impacts render considerable influence on physical-mechanical properties of dispersive grounds: elastic waves velocities, attenuation constants, wetness in an aeration zone, stress conditions of ground massif and cone penetration resistance.

RÉSUMÉ: Comme des études expérimentaux montrent, la forte vibration influence considérablement sur les propriétés phisico-mécaniques des sols: absorption, humidité, état de contrainte du massif, ainsi que résistance en pénétration conique.

Traditionally it is supposed that physical-mechanical properties of dispersive grounds are described by parameters which though showing considerable space variations, in general external conditions do not change graphically. The aim of this report is to show that it is not true for all cases. In particular strong dynamic impacts change conditions of media where they propagated.

Before some remarks concerning the title of the report are necessary. In physics are traditionally taken external conditions describe as PVT-parameters. In this report only vibration impacts influence on physical-mechanical properties are discussed. Such temperature impact are excluded. When speaking about dynamic impacts, the author is stressing two moments. First, that this impact has a trigger, threshold character. If the intensity of impact is lower than some threshold level, then vibration impact on media parameters can be neglected. Such suppositions are coordinated with linear theory of elastic waves propagation. Recently many studies appeared in which insufficiency of linear suppositions for strong motion effects in dispersive grounds is shown. Second nonstationary of process is stressed. The majority of results cited below were received in long-time vibrational impacts,
which should be taken into consideration in evaluation of impulse impact (like earthquake or blust) influence upon dispersive grounds properties.

1 VIBRATIONAL IMPACT ON SEISMIC PARAMETERS

Seismic parameters include: velocity longitudinal and stress wave, density and attenuation constants. Vibrational impact on longitudinal waves velocity on dispersive grounds study is described in paper by Alyoshin and Kovalskaja (1989). Vibration produced by seismic prospecting vibrator SV-1C/100 situated at different distances from measuring place at a limit from 0 to 100 meters. The longitudinal waves velocity is measured by ultrasonic sounding technique on the distance from radiator to receiver about 10 cm. The moment of exciting of ultrasonic pulse was synchronized with the phase of vibrator waves, so as to make measuring the velocity of elastic waves separately in a pulse of tension and in a phase of compression. The results of experiments are shown in fig.1. It appeared that the difference of velocity in phase tension and compression is rather big: increase of velocity in compression phase is +10%, and its decrease in tension phase is -20%. The differences are decreased with the growing distance from vibrator plate to the place of velocity measuring. At a distance about 60 m these differences are disappearing. The stress in vibrational load is evaluated by the formula:

$$\delta = \varphi \nu \cdot \dot{x}$$

where $\varphi$, $\nu$ - accordingly density and elastic wave velocity. Minimal load influencing on elastic wave velocity is evaluated by 2000 Pa.

Another similar phenomenon concerns attenuation constant, which is considered constant and independent from vibration intensity. The experiment shows that it is not so: frequency-dependence attenuation parameters are different on different intensity of vibrational loads. It is important that on strong vibrational impact frequency dependence of attenuation is absent, so as the oscillations of low frequency equally attenuate with distance. The frequency dependence of attenuation begins from distance about 80 m.

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**Fig.1** Velocity of longitudinal wave versus distance from vibrator to place of measuring of velocity
The fluid and gas transfer effects on strong dynamic impacts in dispersive grounds are noted. Wetness migration in dispersive grounds under vibrational influence was studied experimentally by Alyoshin and Kudrjavtsev (1992). Experimental method is similar to the above-mentioned, but in a place of measuring, the samples of ground were taken at a distance of 0.5 m between them and their wetness was measured. Vibrational source was placed on a distance from place of measuring varying from 0 to 70 m. Wetness of sand in aeration zone before and after vibrational impact was measured. In Fig.2 is shown differential total (summarized value for all depths of sampling) wetness versus distance from vibrator. The graph shows threshold character of this effect. Minimal impact intensity influencing wetness which exceeds phone data, is nearly 1 W/sq.m.

Increasing of differential total wetness of aeration zone grounds agrees with the supposition that vibration causes capillary raising from ground water level.

In Fig.3 is shown differential total wetness versus frequency of vibration. It is obvious that it is the maximum graph on frequency about 30 Hz. This frequency value corresponds to the idea of structural resonance.

![Graph showing differential total wetness versus frequency of vibration](image)

**Fig.3** Differential total wetness versus frequency of vibration

Results of these experiments correspond to experiments on vibrational impact upon oil deposits for oil input increasing (Nicolaev et al., 1989). The same as in the above-mentioned case there is a remarkable frequency dependence of this effect and the dominant frequencies about 20 Hz are determined. Essential difference of both experiments is in difference of intensity: vibrational impact in grounds is stronger than the ones in oil massif.
3 VIBRATION INFLUENCE ON STRESS CONDITION OF MASSIF

The obscurity of concrete mechanism of vibrational influence on oil input increasing explained performance of experiment with using two vibrators. One - for vibrational impact on oil deposit, comparatively low frequency, powerful and another - for phase control of velocity of oil deposit, comparatively high frequency, low intensity.

Phase method of elastic waves velocity control are characterized by high relative accuracy, at more than 10 times higher than ones of usual measuring by impulse method.

Results of experiment are described in paper by Alyoshin et al. (1993). Because of harmonic influence the corresponding low frequency component must be appearing in Fourier spectrum of phase shifts. But it was not noticed which is quite natural, because the intensity of impact is lower the threshold level when elastic wave velocity changes.

Simultaneously it was clear that differential phase shifts in phone time when only high frequency vibrator was acting and in working time when both vibrators were working are essentially different.

It may be supposed that during vibrational impact change of stress condition of massif: media gradually relaxed to new stress condition. The evaluations showed that this change is about 5% of geostatic stress.

The change of stress condition may be the cause of oil input increasing, because varied stress condition changes massif strain, and fluid and gas are pressed out of oil deposit.

4 VIBRATION INFLUENCE ON CONE PENETRATION RESISTANCE

Cone penetrating tests are an effective method of engineering geological investigations, in particular mapping liquefaction susceptibility (Liquefaction of Soils During Earthquakes, 1985). It help to determine the frontal and lateral resistance of cone penetrating into ground massif with standard force. The frontal resistance correlates with deformation module. As it was shown before vibration essentially influenced elastic modules of grounds. It can be expected that vibration impact influences cone penetration resistance. The experiment confirm this supposition.

The cone penetration sounding is produced three times in each place: without vibration, in presence of strong and of weak vibration.

The vibrational impact was produced by seismic vibrator, placed on different distance from site of cone penetration sounding. In fig. 4 some results of experiment are shown. It is obvious that even a weak vibration remarkably decreased the cone penetration resistance. The other essential result is that degree of influence varied for
different grounds: in sand this difference between cone resistance with and without vibration is more than the ones in sandy-loam or clay.

- Vibrational impact on oil deposits with the aim of input increasing and decreasing of the danger of rock shocks and other phenomena.
- Method of seismic failure zone location forecast by the difference in vibrational influence on cone penetration characteristics.

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Engineering properties of a peat deposit in Japan
Propriétés techniques d’un gisement de tourbe au Japon

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ABSTRACT: The engineering and the other properties of a peat deposit from the ground surface to the depth of about 12 m were investigated and tested in field and laboratory. Based on the data taken from the investigations and tests, the many relationships among the properties of peat were proposed taking into account the effect of depth of ground. These relationships may be very useful to estimate the properties of peat deposit in Ishikari area, Hokkaido, Japan.

RESUME: Les caracteristiques techniques et autres d’un sol tourbeux entre la surface et une profondeur d’environ 12 m ont ete etudiées et vérifiées sur le terrain et en laboratoire. A partir des données prises lors de ces études et essais, il a été proposé de tenir compte de l’effet de l’épaisseur du sol dans les nombreuses relations établies entre les propriétés de la tourbe. Ces relations peuvent être très utiles pour estimer les propriétés de la tourbe dans la région d’Ishikari, à Hokkaido, au Japon.

1 INTRODUCTION

It is known that peat is fibrous and highly compressible compared with most mineral soils. The general condition of peat deposit is typically that it has a very high water content and an extremely low bearing capacity. Peat ground is generally ununiformity on both the vertical and the horizontal directions.

The cost of sampling and performing laboratory tests of peat and underlying clay is large in comparison with that of many other common soils, and in many cases it may be disproportionate to the total engineering cost for a given project. Therefore, it would be helpful if various peat properties such as the compression index could be correlated with some easily determined characteristics such as the index properties. The relationships between the engineering and the index properties of peat had been reported in past [Kogure, 1977; Sasaki, 1977]. We can not, however, find the report taken into the effect of depth of peat ground.

The engineering and the other properties of a peat deposit from the ground surface to the depth of about 12 m were investigated and tested in field and laboratory. Based on data taken from the investigations and tests, the many relationships between the properties of peat were proposed taking into the effect of depth of ground. The effect of depth was taken into as the character of the unit weight. These relationships may be very useful to estimate the properties of peat deposit in Ishikari area, Hokkaido, Japan.

2 OUTLINE OF PEAT DEPOSIT

The samples of peat tested were obtained from the peat ground in Ishikari area, at location near Sapporo, Hokkaido, Japan. The in-situ investigations carried out at the same area. A typical soil profile of the peat ground was shown in Fig.1. From Fig.1, the stratum from the ground surface to the depth of about 3 m is peat. The peat stratum is fibrous, and we call "surface peat stratum" in this study. The peat in Ishikari area is sedge peat and the major botanical constituent is one or more specific of sedge. According to the description of peat categories by Radforth, the peat considered in this study falls within category 13. That is, the peat consists of coarse
fibres crisscrossing fine-fibrous.

The stratum from the depth of about 3 m to about 7.5 m is alternation of strata of clay and silty clay contained peat fibers. The stratum from the depth of about 3 m to about 7.5 m is called "medium alternation strata" in this study. The stratum from the depth of about 7.5 m to about 11 m is peat. This peat layer is also fibrous, and is called "lower peat stratum" in this study. The stratum under the depth of about 11 m is fine sand or sandy silt, and these materials are considerable stable strata. The stratum under the depth of about 11 m is called "lower alternation strata" in this study.

From the considerations mentioned above, the peat ground considered in this study may be divided into the four layers on the direction of the depth as follows:

(1) Surface peat stratum
(2) Medium alternation strata
(3) Lower peat stratum
(4) Lower alternation strata.

In this study, the characteristics of the surface peat stratum, the medium alternation strata and the lower peat stratum are considered.

3 SOME PROPERTIES OF THE PEAT DEPOSIT

Dutch cone penetration and vane shear tests were conducted at the field of the peat deposit in Ishikari area. The penetration resistance by Dutch cone $q_c$ and the shear strength by the vane test $\tau$ were shown in Fig.2A.

The samples of laboratory tests were obtained by the thin-wall piston sampler. The inside diameter of the sampler used was 70mm and the outside diameter was 73 mm. The following measurements were conducted for the samples obtained:

(1) Specific gravity
(2) Unit weight
(3) Water content
(4) Ignition loss
(5) Degree of decomposition
(6) Other properties.

The tests of mechanical properties such as consolidation, direct shear and permeability tests were conducted for the undisturbed peat samples. The apparatus and the conditions of tests are the standard type, and the detailed explanations are omitted.
The variation of the void ratio \( e \) the water content \( w \) and the ignition loss \( \text{Lig} \) on the vertical direction were shown in Fig. 2B. The values of the wet density \( \rho_t \), the dry density \( \rho_d \), and the specific gravity \( G \) were shown in Fig. 2C. The compression index \( C_c \) obtained from consolidation test and the coefficients of permeability \( k_v \) and \( k_h \) obtained from permeability test were shown in Fig. 2D. Where, \( k_v \) and \( k_h \) indicate the coefficients of permeability on the vertical and the horizontal directions respectively.

Fig. 3 shows the relationships between the void ratio \( e \) and the ignition loss \( \text{Lig} \). The relationships obtained from the surface peat, the medium alternation strata and the lower peat were together shown in Fig. 3.

In the range of the ignition loss from about 30 to 89%, the void ratios of the surface peat strata are from 4 to 5.5. That is, there is different in the values of the void ratio of the peat according with the depth in spite of the same value of the ignition loss, and it may be seen that the relationship between the void ratio and the ignition loss cannot be simply determined.
The relationship between the water content \( w \) and the ignition loss \( L_{ig} \) is shown in Fig. 4. There is different in the values of the water content of the peats according with the depth in spite of the same value of the ignition loss.

The facts mentioned above suggest that the relationships between the physical properties must be taken into the effect of the depth of peat deposit. However, it may be seen that the relationships between the properties obtained from the peat samples taken from the same depth are favorable.

4 RELATIONSHIPS BETWEEN PHYSICAL PROPERTIES USING PHASE SYSTEM

A model of the phase system of saturated peat soil is shown in Fig. 5 using the values determined by tests. Based on the model and the notations shown in Fig. 5, the organic content \( C_o \) is

\[
C_o = \frac{G_o V_o \gamma_w}{(G_s V_s + G_o V_o) \gamma_w} = \frac{G_o V_o}{G_s V_s + G_o V_o} \tag{1}
\]

The dry unit weight \( \gamma_d \) is

\[
\gamma_d = \frac{(G_s V_s + G_o V_o) \gamma_w}{(G_s V_s + G_o V_o) \gamma_w} \tag{2}
\]

The void ratio \( e \) is

\[
e = \frac{V_w}{V_s + V_o} = \frac{1 - (V_s + V_o)}{V_s + V_o} \tag{3}
\]

The water content \( w \) is

\[
w = \frac{V_w \gamma_w}{(G_s V_s + G_o V_o) \gamma_w} = \frac{V_w}{G_s V_s + G_o V_o} \tag{4}
\]

From equations (1), (2) and (3), the following relation can be obtained.
\[ e = \frac{G_o \gamma_w^2 + [(1 - \frac{G_o}{G_s}) G_o^{-1}] \gamma_w \gamma_d}{[(\frac{G_o}{G_s} \gamma_w - 1) C_o + \gamma_w] \gamma_d} \quad (5) \]

The relationship between the void ratio \( e \) and the organic content \( C_o \) is represented by equation (5) using the parameter of the dry unit weight \( \gamma_d \). The measured values of \( G_o \), \( G_s \) and \( \gamma_w \) are 2.70, 1.50 and 1 tf/m³ (=9.8 kN/m³) respectively. \( G_o \) and \( G_s \) are the specific gravity of the soil particle and the organic matter respectively. \( \gamma_w \) is the unit weight of water.

Substituting these values in equation (5) we get

\[ e = \frac{2.70}{(0.80 C_o + 1) \gamma_d} - 1 \quad (5') \]

or

\[ \gamma_d = \frac{2.70}{(1 + e)(0.80 C_o + 1)} \]

where, the unit of \( \gamma_d \) is tf/m³.

The relationship between \( e \) and \( C_o \) is shown as the curves in Fig. 6 according with the dry unit density \( \gamma_d \). It is noted, from the curves shown in Fig. 6, that the value of \( e \) vary with \( \gamma_d \) in spite of the same value of \( \gamma_d \).

The measured results were also plotted in Fig. 6. Here, the ignition loss \( I_{ig} \) was used as the value of the organic content \( C_o \) for the measured results. It is seen that the measured results agree nearly with the theoretical curves.

On the other hand, from equations (1), (2), (3) and (4), the relationship among the water content \( w \), the organic content \( C_o \) and the void ratio \( e \) can be obtained as follows:

\[ w = \left[ 1 + \left( \frac{G_o}{G_s} - 1 \right) C_o \right] \frac{e}{G_s} \quad (6) \]

Substituting \( G_o = 2.70 \), \( G_s = 1.50 \) and \( \gamma_w = 1 \) tf/m³ in equation (6), we get

\[ w = (0.370 + 0.296 C_o) e \quad (6') \]

or

\[ e = \frac{w}{0.370 + 0.296 C_o} \]

Fig. 7 shows the relationship represented by equation (6'). It is noted, from the curves shown in Fig. 7, that the value of \( w \) increases according with the increasing the value of \( e \) in spite of the same value of \( C_o \). When \( e \) is constant, \( w \) is in proportion to \( C_o \). The measured results were also plotted in Fig. 7. It may be seen that the measured results agree nearly with the theoretical curves.

From equations (1), (2) and (4), the relation between the water content \( w \), the dry unit weight \( \gamma_d \) and the organic content \( C_o \) can be obtained as follows:

\[ w = \frac{\gamma_w}{\gamma_d} \left[ 1 - \left( \frac{G_o}{G_s} \right) C_o \right] \quad (7) \]
Substituting $G_o = 2.70$, $C_o = 1.50$ and $\gamma_w = 1$ tf/m³ in equation (7), the following relationship can be obtained.

$$ w = \frac{1}{\gamma_d} - 0.296 \frac{C_o}{C_o} - 0.370 $$

or

$$ \gamma_d = \frac{1}{w + 0.296 \frac{C_o}{C_o} + 0.370} \quad (7') $$

where, the unit of $\gamma_d$ is tf/m³.

The relationship represented by equation (7') is shown in Fig. 8. It may be seen, from Fig. 8, that $w$ becomes larger according to the decreasing $\gamma_d$ in spite of the same value of $C_o$. The measured results were also plotted in Fig. 8. It may be seen that the measured results agree approximately with the theoretical curves.

5 FORECASTING OF COMPRESSION INDEX

Many researchers have examined the possibility of forecasting the compression index of clays from a knowledge of other soil properties that can be more readily determined [Azzouz, 1976, Cozzolino, 1961, Murayama, 1958, Nishida, 1956]. On peat, several researchers have statistically examined the possibility of forecasting the compressibility [Kinosita, 1960, Maeguchi, 1965, NRC, 1969, Ohira, 1969].

Fig. 9 shows the relationship between the compression index $C_c$ and the void ratio $e$ of the peat and the other soils used in this study. The relationship between $C_c$ and $e$ is represented by using the method of least squares as follows:

$$ C_c = 0.244 e^{0.36} \quad (8) $$

From equations (5) and (8), we get

$$ C_c = 0.244 \left[ \frac{G_o \gamma_w^2 + \left[ (1 - \frac{G_o}{G_b}) C_o - 1 \right] \gamma_w \gamma_d}{\left( \frac{G_o}{G_b} \gamma_w - 1 \right) C_o + \gamma_w} \gamma_d \right]^{0.36} \quad (9) $$

Substituting the measured values of $G_o$, $G_b$ and $\gamma_w$ in equation (9), we get the following relation:

$$ C_c = 0.244 \left[ \frac{2.70}{0.80 G_o + 1} \gamma_d - 1 \right]^{0.36} \quad (9') $$

where, the unit of $\gamma_d$ must be used tf/m³.
Fig. 10 Relationship between compression index $C_c$ and organic content $C_d$

The relationship expressed by equation (9') is shown in Fig. 10. It may be seen, from Fig. 10, that the compression index $C_c$ decreases with the increasing the dry unit weight $\gamma_d$ in spite of the same value of the organic content $C_d$. The measured results were plotted in Fig. 10. It may note that the measured results agree nearly with the theoretical curves. It may be seen that the compression index is accurately predicted using both the organic content and the dry unit weight.

6 SUMMARY

The peat deposit discussed in this study is divided into the four layers on the vertical direction. The surface peat stratum exist from the ground surface to the depth of about 3 m and is fibrous peat. The medium alternation strata from the depth of about 3 m to about 7.5 m is composed of clay and silty clay contained peat fibers. There is the lower peat stratum from the depth of about 7.5 m to about 11 m, and the peat is fibrous. The strata under the depth of about 11 m are fine sand and sandy silt, and these soils are considerable stable.

The relationships between the void ratio and the ignition loss, or the water content and the ignition loss differ by the sampling depth of the peat in spite of the same value of the ignition loss. This fact suggests that the relationships between the physical properties must be taken into the effect of depth of peat ground.

A model of phase system of saturated peat was proposed. The model is represented by the physical values determined from tests. Based on the model, the relationship among the void ratio the organic content and the unit weight can be theoretically obtained, and the other relationships can be also obtained. The results of measurements agree with the theoretical relationships.

The compression index decreases with the increasing the dry unit weight in spite of the same value of the organic content. The compression index may be accurately be forecasted using both the organic content and the dry unit weight.

REFERENCES


Effect of calcium carbonate content on engineering properties of marly rocks
Effet du contenu en carbonate de calcium sur les propriétés des marnes

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Saga University Japan
Touraj Amirsoleymani
Mandro Consultant Engineers Company Tehran, Iran

ABSTRACT: Marl, a common sediment within carbonate sequences in Iran, is a mixture of clay minerals and calcium carbonate, and is changed to marlstone after lithogenesis process. Because of extreme sensitivity of marly rocks to weathering factors and consequently variation in its engineering properties, the existence of these rocks can create some problem in relation with engineering plans. To provide a better understanding of the effect of lithology on some engineering properties of marly rocks, the result of tests and log of 97 borehole (totally about 1770 meters) on these rocks of three geological marly formations of Iran collected and some sample prepared for complementary studying. Effect of calcium carbonate content on uniaxial compressive strength, consistency limits, dry density, slake durability index and slaking degree have been studied in this investigation.

RÉSUMÉ: La marnes en tant qu’un sédiment courant dans les séquences carbonatées de l'Iran est une mixture des minerais d’argile avec le calcium carbonate et il se transforme en marnes par l’intermédiaire des processus lithogénétiques. En raison de l’extrême sensibilité des roches de marnes en face des conditions atmosphériques et la modification consécutive de leurs qualités géotechniques, l’existence des roches en question dans la nature peut créer certains problèmes à l’égard des projets géotechniques. Pour mieux comprendre les relations entre lithologie et certaines qualités géotechniques des roches de marne, on a collecté les résultats et la documentation sur 97 sondes (en tout de 1770 mètres environ) dans trois formations de marnes de l'Iran. Plusieurs échantillons étaient préparés pour l'étude complémentaire. L'effet du contenu de la calcium carbonate sur la résistance uniaxiale à la compression, sur les limites des consistances d'Atterberg, sur le poids volumétrique sec, sur "slake durability index" et sur "slaking degree" étaient étudiés en cadre de ces recherches.

1. INTRODUCTION

The fundamental characteristics of a rock mass are generally laid down during lithogenesis. The diagenetic processes that take place are generally modified by factors such as deformability, permeability and the internal stresses developed within the volume of the rock mass undergoing lithogenesis. The physical properties, including the consistency and strength of the sediment before lithogenesis and as it progresses, are major factors in determining the final state of the rock mass.

Later factors such as surface to depth weathering in the rock mass tends to modify the original material (Arkin 1989).

Marly rocks are a mixture of clay minerals and calcium carbonate that classified in argillaceous rocks (or sediments). The main parts of argillaceous rocks are particles with diameter less than 0.004mm. The most of these rocks include a mixture of rock-flour and clay minerals. Marl usually include clay minerals that mixed with calcium carbonate; this carbonate has organic or biochemical origin (Greensmith 1979). Marl fundamentally is
defined as a mixture of clay minerals and calcium carbonate in which its carbonate content is between 35 to 65 percent (Pettijohn 1975), but usually this term used for any mixture of clay and calcium carbonate, such as Kupper marl in England that its calcium carbonate content varies among 0-30 percent (Bell 1983).

Engineering properties of marly rocks extremely vary with change in its composition (the type of clay minerals and clay/carbonate percent). This study is based on analysis of
investigation data of some engineering projects on three geological marly formation in Iran, in combination with some complementary studies. In a practical sense, the range in mechanical properties of clay mixtures often approaching condition of expansive clays is important in evaluating foundation and slope conditions formed by these materials. A better understanding of the developing changes of the mechanical properties with variation in composition and water content is important in establishing more accurate design parameters for engineering purposes (Arkin 1989). No attention to these characteristics of clay mixtures (such as marly rocks) sometime culminate in very extreme hazard, such as Vajont dam failure (Amirsoleymani 1987).

2. METHOD OF STUDY

This study is based on combination of two data group: 1) geotechnical data of three geological marly formations in three parts of Iran (Fig.1, Table 1) that gathered from geotechnical investigation report of some engineering project, 2) consistency limits, calcimetry, slake durability, and slaking test results on samples that prepared from these formations at different depths.

<table>
<thead>
<tr>
<th>Formation name</th>
<th>Number of boreholes</th>
<th>Total length of boreholes (meter)</th>
<th>Maximum depth (meter)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aghajari</td>
<td>2</td>
<td>60</td>
<td>30</td>
</tr>
<tr>
<td>Mishan</td>
<td>55</td>
<td>908</td>
<td>20</td>
</tr>
<tr>
<td>Tabriz</td>
<td>40</td>
<td>802</td>
<td>33</td>
</tr>
</tbody>
</table>

3. GEOLOGY

Geological description of studied formations are as follows:

3.1 Mishan formation

Mishan formation extends in south of Iran with a trend of NW-SE (in Zagros chain). Its type section is composed of 61 meters limestone interbedded with gray marl at the base and 649 meter gray weathered marl with competent limestone interlayers at the top. The age of Mishan formation is early to middle Miocene (Stocklin 1971). Samples of this study are from Mishan outcrops near Bandar Abbas city in south of Iran (Fig.1,No.1).

3.2 Aghajari formation

Extension of Aghajari formation is the same as Mishan formation and usually overlay Mishan formation. It is composed of brown to gray calcareous sandstone and red marls and siltstone with gypsum veins. The age of this formation is late Miocene-Pliocene (Stocklin 1971). Samples of this study are from Aghajari outcrops near Behbehian city in south west of Iran (Fig.1,No.2).

3.3 Lignite beds and lacustrine fish beds

This beds extent only in south of Tabriz city in north west of Iran (Fig.1,No.3). They composed of clay, sand, yellow and gray marl with two lignite beds. The age of these beds is Pocnian (Stocklin 1971). Hereafter, these beds is called Tabriz formations.

4. COMPLEMENTARY TESTS AND DATA ANALYSIS

X-ray analysis showed that the main clay mineral is montmorillonite in all samples. A typical chemical analysis of the three kinds of samples is shown in table 2. On the basis of calcimetry tests, the calcium carbonate content of studied marly rock samples are as follows:

- Aghajari formation: 49-55%
- Mishan formation: 29-35%
- Tabriz formation: 10-22%

The effect of change in calcium carbonate content on slake durability index, slaking degree, uniaxial compressive strength, qu, of studied marly rocks and consistency limits of powdered samples are presented in the following sections.
Table 2 Typical chemical composition of studied rocks

<table>
<thead>
<tr>
<th>Substance</th>
<th>Aghajari (%)</th>
<th>Mishan (%)</th>
<th>Tabriz (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>34.69</td>
<td>37.82</td>
<td>50.63</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>6.37</td>
<td>9.41</td>
<td>12.45</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.06</td>
<td>5.21</td>
<td>6.31</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.40</td>
<td>0.56</td>
<td>0.65</td>
</tr>
<tr>
<td>CaO</td>
<td>24.15</td>
<td>15.06</td>
<td>8.81</td>
</tr>
<tr>
<td>MgO</td>
<td>4.24</td>
<td>6.93</td>
<td>2.77</td>
</tr>
<tr>
<td>Na₂O</td>
<td>1.20</td>
<td>1.89</td>
<td>1.55</td>
</tr>
<tr>
<td>K₂O</td>
<td>1.28</td>
<td>1.71</td>
<td>2.15</td>
</tr>
<tr>
<td>SO₃</td>
<td>0.21</td>
<td>0.75</td>
<td>0.18</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>0.12</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>L.O.I.</td>
<td>24.07</td>
<td>20.65</td>
<td>14.08</td>
</tr>
</tbody>
</table>

4.1 Consistency limits

Consistency limits are characterized by plastic and liquid limits and plasticity and liquidity indices. The liquid limit represents the minimum water content at which the sediment flow under its own weight and the plastic limits the minimum water content at which the sediment is moulded without breaking. These limits control the consistency of the sediments as wetting condition change (Arkin 1989).

Samples powdered to pass through a 200 mesh sieve for consistency limits tests. Data analysis shows that plasticity of marly sediments be decreased with increasing in calcium carbonate content (Fig.2). Liquid limit and plasticity index is decreased in a linear manner with increasing in calcium carbonate content (Fig.3). The regression analysis results are:

LL = 84.09 - 0.84CaCO₃   n = 36   r = 0.81
PI = 61.13 - 0.79CaCO₃   n = 36   r = 0.84

Fig.3 Consistency limits in relation to calcium carbonate content
4.2 Slaking and durability

Durability of rocks is fundamentally important for all applications. Changes in the properties of rocks are produced by exfoliation, decrystallization (slaking), solution, oxidation, abrasion, and other processes. Fortunately, such changes usually act imperceptibly through the body of the rock and only the immediate surface is degraded in tens of years. At any rate, some index to the degree of alterability of rock is required (Goodman 1984).

Sansaki et al. (1981) classified the argillaceous rocks into six classes from A to F based on the degree of slaking in which rocks of class A remain without any change in contact with water and to class F the amount of cracking and disintegration be increased. Gamble (1971) proposed a slake durability classification (table 3) based on slake durability index.

<table>
<thead>
<tr>
<th>Table 3. Gamble's slake durability classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group name</td>
</tr>
<tr>
<td>Very high durability</td>
</tr>
<tr>
<td>High durability</td>
</tr>
<tr>
<td>Medium high durability</td>
</tr>
<tr>
<td>Medium durability</td>
</tr>
<tr>
<td>Low durability</td>
</tr>
<tr>
<td>Very low durability</td>
</tr>
</tbody>
</table>

* After one 10-minute cycle (dry weight basis)
** After two 10-minute cycle (dry weight basis)

The results of slaking and slake durability tests on studied marly rocks based on above classification is given in table 4.

<table>
<thead>
<tr>
<th>Table 4. Slake durability and slaking tests results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Formation name</td>
</tr>
<tr>
<td>Aghjari</td>
</tr>
<tr>
<td>Mishan</td>
</tr>
<tr>
<td>Tabriz</td>
</tr>
</tbody>
</table>

4.3 Uniaxial compressive strength

Increment of the calcium carbonate content lower the water absorption property and increase the dry unit weight, \( \gamma_d \), of marly rocks because of reducing the clay content. As a consequence, the uniaxial compressive strength, \( q_u \), of marly rocks is increased with increase in dry unit weight, \( \gamma_d \), and decrease in natural water content, \( W_n \) (Fig. 4&5).

![Fig. 4 Relationship between \( q_u \), \( W_n \), and calcium carbonate content in studied rock samples](image1)

![Fig. 5 Relationship between \( q_u \), \( \gamma_d \), and calcium carbonate content in studied rock samples](image2)

5. CONCLUSIONS

Clay/carbonate content and the type of clay minerals are the main factors that control the engineering properties of marly rocks. If the kind of clay mineral be constant, the engineering properties of these rocks improve with increasing in the calcium carbonate content.
REFERENCES:


Effect of sampling on mechanical properties of loess
Effet d’échantillonnage sur les paramètres mécaniques du loess

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ABSTRACT: The natural undisturbed landloess is characterized as a loose open structured macroporous soil. Experience gained during recent decades shows that the loess soil in some cases undergoes structural collapse and subsidence due to inundation. In order to find the explanation of such behaviour, numerous laboratory and field tests on loess soil were performed. On the basis of the obtained results it has been concluded that the sensitivity of loess to the subsidence due to wetting or saturation depends to a large extent on the initial dry density, initial water content and stress level acting during saturation. By the determination of density the loess can be classified as being either loose and susceptible to subsidence or sufficiently dense and unlikely to subside. It is to note that the loose loess can produce large differential settlements of buildings, which in most cases very dangerous. For that reason it is of great importance to determine the potential of collapsibility of loess with the satisfactory degree of precision. The results obtained by laboratory investigations indicate that loess deposits are very sensitive to the mechanical disturbance, and inadequate method of sampling can lead to wrong conclusions and to solutions which are on the unsafe side.

RESUME: Le loess terrestre en état naturel est caractérisé comme un sol de structure macroporeuse. Expérience montre que le loess dans certains cas, sous l’effet des contraintes provoquées par des batiments, peut atteindre des tassements excessivement grands. Dans le but de trouver l’explication de tels comportements, de nombreux essais de laboratoire et sur terrain ont été effectués. À la base des résultats obtenus, on a conclu que la sensibilité du loess au tassement dépend de la saturation depend conséquemment de la teneur en eau à l’état naturel, et à la fois du niveau des contraintes agissantes pendant la saturation. Aussi, les résultats obtenus montrent que le loess est très sensible au réaménagement mécanique et que les méthodes inadéquates d’échantillonnage peuvent amener aux conclusions fausses, qui, sont à la côté de l’insécurité.

1. INTRODUCTION

Loess is anolian sediment transported by wind from the flood plains of glacial rivers. The natural undisturbed loess, known as landloess, is characterized as a loose open structured macroporous soil composed of silt particles separated by clay coatings (Gibbs and Holland, 1960; Larionov, 1965).

This type of soil is often considered to be unstable as a foundation material because of its potential for large settlement and for loss of strength on wetting or saturation. The amount of settlement depending on the water content and the initial dry density has been discussed by Milović (1967, 1978). The state-of-the-art papers on collapsing soils are by Northey (1969) and comprehensive review of geotechnical investigations of loess is given by Lutnegger et al. (1979).

For the explanation of the behaviour of loess after wetting or saturation the initial density of loess and its initial water content are of primary importance. However knowing that loess is very sensitive to the mechanical disturbance, it is of great interest to use the adequate method of sampling in order to obtain the samples of high quality.

2. LABORATORY AND FIELD TEST RESULTS

In order to define the effect of the dry density and the water content of loess
samples on the shear and deformation parameters of loess deposits, numerous laboratory tests have been carried out on the undisturbed loess samples with various values of the initial dry density and water content. Hand carved blocks were removed from test pits. Figs. 1 and 2 show the relationship between the unconfined compression strength \( q_u \), water content \( w \) and dry density \( \gamma_d \).

![Fig.1. Relationship between the strength \( q_u \) and water content \( w \) for loess samples.](image1)

![Fig.2. Relationship between the strength \( q_u \) and dry density \( \gamma_d \) for loess samples.](image2)

The laboratory test results obtained on the undisturbed loess samples, cut from blocks in the vertical and horizontal direction, have shown that this kind of soil exhibits anisotropic properties. Fig. 3 shows the typical results of the unconfined compression strength \( q_u \).

Full lines indicate the results obtained on vertical samples (V) and dash lines the results on horizontal samples (H).

In Fig. 4, the relationship between the cohesion \( c \) and water content \( w \) is shown.

![Fig.3. Unconfined compression test results for samples cut in vertical (V) and horizontal (H) direction.](image3)

![Fig.4. Relationship between the cohesion \( c \) and water content \( w \).](image4)

By numerous consolidation tests it has been determined the amount of deformation that may occur. The collapse potential suggested by Knight (1963), is defined as:

\[
CP = \frac{\Delta H_C}{H_0}
\]  

(1)

where \( H_C \) = change in height upon wetting and \( H_0 \) = initial height. As a measure of the subsidence the following expression is often used:

\[
i_m = \frac{e_n - e_i}{n} = \frac{e_n}{n}
\]

(2)
where: $e_n$ = void ratio before saturation at the vertical stress $\sigma_h$; $e_n'$ = void ratio at the end of subsidence under the same vertical stress $\sigma_h$; and $i_m$ = dimensionless coefficient of subsidence. In Fig. 5 is shown the typical consolidation test result.

![Diagram of void ratio and vertical stress](image)

**Fig. 5.** Typical subsidence consolidation test result.

Consolidation test have been performed on several groups of loess samples, covering the large range of dry densities and initial water content. So, the coefficients of subsidence $i_m$ have been determined for five various values of dry density, for various degrees of saturation and for several stress levels upon wetting. For illustration, in Fig. 6. are shown the coefficients of subsidence $i_m$ for very low dry density (curve 1), and for high dry density (curve 2).

![Diagram of $i_m$ vs. dry density](image)

**Fig. 6.** Coefficients $i_m$ for low dry density (curve 1) and high dry density (curve 2).

In Fig. 7 is shown the variation of the compressibility modulus $E_d$ with the dry density $\rho_d$ of the saturated loess samples.

![Graph of modulus vs. dry density](image)

**Fig. 7.** Variation of the compressibility modulus with the dry density.

On several sites field load tests have also been performed. Typical results, indicating the effect of the dry density at the practically the same water content (curves 1 and 2), and the effect of water content at the same dry density (curves 2 and 3) on the stress-deformation relationship, are shown in Fig. 8.

![Graph of stress vs. dry density](image)

**Fig. 8.** Field load test results on loess.

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The effect of saturation of loess deposits on its penetration resistance is clearly shown in Fig. 9, where curve 1 is registered in loess with natural water content and curve 2 in saturated loess.

![Graph showing penetration test results in loess.]

Fig. 9. Static penetration test results in loess.

The laboratory and field test results clearly show the importance of two basic parameters of loess i.e. the initial dry density and initial water content. In other words, these two parameters govern the behaviour of loess under the applied load and in conditions of increased water content.

3. MECHANICAL DISTURBANCE OF LOESS SAMPLES

Results of several studies indicate that the method of sampling has a very significant influence on the quality of loess samples and on sensitive clays samples. It has been noticed that loess samples taken by thin walled tubes are unusually mechanically disturbed. For that reason a comparative laboratory testing has been made on piston samples and on samples from hand carved blocks in test pits. Typical unconfined compression test results are shown in Figs. 10 and 11.

![Graph showing unconfined compression test results.]

Fig. 10. Unconfined compression test results: (A) block samples with \( \tau_d = 12,9 \) kN/m\(^3\) and \( w = 12,2\% \); (B) piston samples with \( \tau_d = 15,1 \) kN/m\(^3\) and \( w = 12,6\% \).

![Graph showing unconfined compression test results.]

Fig. 11. Unconfined compression test results: (A) block samples with \( \tau_d = 16,5 \) kN/m\(^3\) and \( w = 17,4\% \); (B) piston samples with \( \tau_d = 16,3 \) kN/m\(^3\) and \( w = 16,5\% \).

Fig. 10 shows the results obtained on block samples with \( \tau_d = 12,9 \) kN/m\(^3\) and \( w = 12,2\% \) (curve A). From the same depth piston samples had \( \tau_d = 15,1 \) kN/m\(^3\) and \( w = 12,6\% \) (curve B). In this case of low density the mechanical disturbance leads to the overestimated values of \( q_u \) and \( E \). However, in the case of high dry density of loess deposits, block samples had \( \tau_d = 16,5 \) kN/m\(^3\) and \( w = 17,4\% \) and piston samples \( \tau_d = 16,3 \) kN/m\(^3\) and \( w = 16,5\% \). In Fig. 11 curve A was obtained for block samples and curve B for
piston samples. Here, the values of $q_u$ and $E$ for piston samples are considerably underestimated.

In Table 1 are given the values of $T_d$, $w$, $q_u$, and $E$, deduced from the unconfined compression tests, performed of block samples and on piston samples.

Table 1. Values of $T_d$, $w$, $q_u$ and $E$ for block (B) and piston (P) loess samples.

<table>
<thead>
<tr>
<th></th>
<th>$T_d$ (kN/m³)</th>
<th>$w$ (%)</th>
<th>$q_u$ (kN/m²)</th>
<th>$E$ (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>12,8</td>
<td>14,7</td>
<td>21</td>
<td>3500</td>
</tr>
<tr>
<td>P</td>
<td>15,7</td>
<td>12,4</td>
<td>147</td>
<td>9800</td>
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<tr>
<td>B</td>
<td>12,9</td>
<td>12,2</td>
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<td>2500</td>
</tr>
<tr>
<td>P</td>
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</tr>
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<td>B</td>
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<td>P</td>
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<td>P</td>
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<td>6000</td>
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</tr>
<tr>
<td>B</td>
<td>16,5</td>
<td>17,4</td>
<td>187</td>
<td>19200</td>
</tr>
<tr>
<td>P</td>
<td>16,3</td>
<td>16,5</td>
<td>80</td>
<td>5000</td>
</tr>
<tr>
<td>B</td>
<td>18,8</td>
<td>23,0</td>
<td>150</td>
<td>10700</td>
</tr>
<tr>
<td>P</td>
<td>15,6</td>
<td>22,6</td>
<td>107</td>
<td>2100</td>
</tr>
<tr>
<td>B</td>
<td>17,3</td>
<td>20,6</td>
<td>160</td>
<td>13300</td>
</tr>
<tr>
<td>P</td>
<td>16,7</td>
<td>23,1</td>
<td>86</td>
<td>2300</td>
</tr>
<tr>
<td>B</td>
<td>17,3</td>
<td>20,0</td>
<td>120</td>
<td>2300</td>
</tr>
<tr>
<td>P</td>
<td>17,7</td>
<td>21,2</td>
<td>180</td>
<td>7300</td>
</tr>
</tbody>
</table>

The obtained results show again that for loess samples of low dry density the values of $q_u$ and $E$, determined on piston samples, are overestimated, whereas for loess of high density these values are underestimated.

Concerning the deformation parameters, the effect of mechanical disturbance is also very pronounced. In Table 2 are shown some typical results of the modulus of compressibility $E_{so}$ and the coefficient of subsidence $I_m$ for block and piston samples.

Table 2. Values of $T_d$, $E_{so}$ and $I_m$ for block and piston loess samples.

<table>
<thead>
<tr>
<th></th>
<th>$T_d$ (kN/m³)</th>
<th>$q_u$ (kN/m²)</th>
<th>$E_{so}$ (kN/m²)</th>
<th>$I_m$ (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>12,0</td>
<td>30–60</td>
<td>1000–2000</td>
<td>0,072</td>
</tr>
<tr>
<td>P</td>
<td>14,8</td>
<td>120–280</td>
<td>3000–4000</td>
<td>0,13</td>
</tr>
</tbody>
</table>

As shown in the above Table, the deformation parameters, obtained on piston samples are on the unsafe side.

4. MECHANICAL DISTURBANCE OF CLAY SAMPLES

In order to evaluate by laboratory tests the effect of sample disturbance on the properties of Canadian sensitive marine clays, block, piston and Shelby samples were taken at several sites. Liquid limit of these clays varied between the limits $W_1 = 60–69\%$, plastic limit $W_p = 23–26\%$, water content $W = 60–75\%$ and sensitivity $S_e = 10–15$.

A large number of unconfined compression tests were performed and typical results for sensitive St-Simon and Nicolet clay are given in Table 3.

Table 3. Unconfined compression test results.

<table>
<thead>
<tr>
<th>Site</th>
<th>St-Simon (Quebec, Canada)</th>
<th>Nicolet (Canada)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compression strength, $q_u$ (kN/m²)</td>
<td>60±12</td>
<td>32±3</td>
</tr>
<tr>
<td>Block</td>
<td>41±6</td>
<td>28±5</td>
</tr>
<tr>
<td>Piston</td>
<td>25±6</td>
<td>22±4</td>
</tr>
<tr>
<td>Shelby</td>
<td>1,4</td>
<td>1,4</td>
</tr>
<tr>
<td>Strain at failure, $E_{fr}$ (%)</td>
<td>1,6</td>
<td>1,6</td>
</tr>
<tr>
<td>Block</td>
<td>1,9</td>
<td>3,0</td>
</tr>
<tr>
<td>Piston</td>
<td>5,0</td>
<td>4,2</td>
</tr>
<tr>
<td>Shelby</td>
<td>1,0</td>
<td>1,2</td>
</tr>
<tr>
<td>Young's modulus, $E$, (kN/m²)</td>
<td>750±400</td>
<td>4200±800</td>
</tr>
<tr>
<td>Block</td>
<td>5400±200</td>
<td>3200±500</td>
</tr>
<tr>
<td>Piston</td>
<td>3000±500</td>
<td>1100±300</td>
</tr>
<tr>
<td>Shelby</td>
<td>3000±200</td>
<td>3000±500</td>
</tr>
<tr>
<td>Ratio Shelby/Block, $q_u$</td>
<td>0,42</td>
<td>0,68</td>
</tr>
<tr>
<td>Ratio Piston/Block, $q_u$</td>
<td>0,68</td>
<td>0,87</td>
</tr>
<tr>
<td>Ratio Shelby/Block, $E$</td>
<td>0,40</td>
<td>0,26</td>
</tr>
<tr>
<td>Ratio Piston/Block, $E$</td>
<td>0,72</td>
<td>0,76</td>
</tr>
</tbody>
</table>

The load deformation curves obtained through the unconfined compression tests for the three types of specimens are shown in Fig. 12.

Two plate-loading tests were carried out under undrained conditions. Typical load settlement curve is shown in Fig. 13.

In these tests the depth of foundation was $D_F = 2,3\ m$ and the plate diameter $D = 0,61\ m$. The value of the elastic modulus $E=10000$ kN/m² was obtained using the influence coefficients for the elastic settlement of an anisotropic layer, produced by a perfectly rigid circular foundation.
which relates the forces acting at nodal points with their displacements.

It is known that the finite element method consists in decomposing a continuous medium in a finite number of elements. In this case each quadrangular element is subdivided into four triangles with a common nodal point in the center of the quadrangular element. The basic element is one triangle for which the displacement field is assumed to be a linear function of space coordinates. For the determination of componental stresses and displacements produced by a vertical uniform load over a flexible circular foundation, the stiffness matrix is given by:

\[
\begin{pmatrix}
D_e = \frac{E_v}{(1+\mu)(1-\mu-2\mu K)^2} & \mu_h K_h \mu v^2 & 0 \\
\mu_h (1+\mu K) & (1+\mu K) & 0 \\
\mu_h (1+\mu K) & 0 & (1+\mu K) \\
0 & 0 & 0
\end{pmatrix}
\]

where: \(E_v\) = modulus of elasticity in the vertical direction, \(\mu_h\) = modulus of elasticity in the horizontal direction, \(\mu_v =\) Poisson's ratio in the vertical direction; \(\mu_h =\) Poisson's ratio in the horizontal direction; \(K = \rho_h / E_v, \) and \(t = (1+\mu)(1-\mu-2\mu K)^2).\)

For all componental stresses and displacements the dimensionless coefficients have been calculated for several values of the ratio \(H/D,\) where \(H\) and \(D\) are the thickness of the compressible layer and the diameter of the circular foundation, respectively. The dimensionless coefficients have also been calculated for two values of \(n,\) where \(n = 1/K\) represents the degree of anisotropy. In Table 4 are shown the dimensionless coefficients \(I_w\) for the settlement of the center of the circular foundation.

### Table 4. Coefficients \(I_w\) for anisotropic soils

<table>
<thead>
<tr>
<th>(H)</th>
<th>(n = 0.50)</th>
<th>(n = 2.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(D)</td>
<td>(\mu_v = 0.30)</td>
<td>(\mu_v = 0.30)</td>
</tr>
<tr>
<td>0.50</td>
<td>0.334</td>
<td>0.441</td>
</tr>
<tr>
<td>1.00</td>
<td>0.523</td>
<td>0.686</td>
</tr>
<tr>
<td>2.00</td>
<td>0.651</td>
<td>0.853</td>
</tr>
<tr>
<td>3.00</td>
<td>0.691</td>
<td>0.908</td>
</tr>
</tbody>
</table>

Comparing these values \(I_w\) with those obtained for isotropic soil one may conclude that the settlement of the anisotropic soil with \(n < 1\) is smaller and with \(n > 1\) is greater than the settlement of the isotropic soil.
CONCLUSIONS

On the basis of the laboratory and field investigations the following conclusions can be made:

- loess deposits, as well as marine clays, are very sensitive to the mechanical disturbance, and the inadequate method of sampling can lead to wrong laboratory results;
- the sensitivity of loess to the subsidence due to wetting or saturation depends to a large extent on the initial dry density, initial water content and stress level acting during saturation;
- the prediction of the behaviour of structures founded on loess deposits must be based on the settlement calculation in natural moisture conditions and also in wetted or saturated state;
- in settlement calculations anisotropic properties of loess deposits must be taken into account.

REFERENCES


A new approach for determining soil activity using the water adsorption test
Une nouvelle approche pour la détermination des caractéristiques des sols en utilisant l’essai d’absorption

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K. Schetelig
RWTH, Aachen, Germany

ABSTRACT: In this work, the determination and mechanisms of the Water Adsorption Capacity of soils is defined by the ENSLIN/NEFF apparatus are described. Activity of fine grained soils is defined by the introduction of new soil physical parameters, called "Water Adsorption Index, and modified Water Adsorption Index". Both of these parameters are derived from the Water Adsorption Test.

A comparison with the definition of Activity by SKEMPTON 1953, including advantages and disadvantages is given. The validity of the new approach is documented with a large number of mineralogical, chemical and physical soil tests on tropical soils and data cited in the literature.

It is demonstrated that the definition of soil activity using the Water Adsorption Test is much more exact and more practical in describing the nature of the clay minerals in soils and their base complex.


Une comparaison entre cette nouvelle définition et le chiffre d’activité de SKEMPTON (1953) a été faite, des avantages et désavantages ont été discutés. La validité de la nouvelle définition de l’activité a été documentée à l’aide des études minéralogiques, chimiques et physiques faites sur des sols tropicaux, et l’aide de certaines données de la littérature.

Il a été montré que la nouvelle définition de l’activité du sol est plus sûre, plus claire et plus pratique pour la description des caractères significatifs des minéraux d’argile et leur garniture en cations.

1 INTRODUCTION:

Soil activity has a wide application in geotechnical engineering, diversifying from soil classification, settlement, consolidation, permeability, swelling and erosion behaviour, to waste deposits. Therefore, a reliable and simple procedure for estimating the activity of soils is of primary importance.

The results of many investigations about the geotechnical behaviour of fine grained soils have emphasized that the
controlling factors are the form of the crystal structure of the clay mineral, type and concentrations of the attached cations (base exchange complex) and the pore water—electrolyte content. These factors are not directly determinable with present geotechnical laboratory tests. Mostly they are described through soil activity.

The present definition of activity using the activity index from SKEMPTON (1953), considering the relationship between the plasticity index and the percentage of clay, has many drawbacks which will be discussed below. Exact determination of the clay minerals and the chemical composition of soils involves complex techniques, expensive devices and require skilled specialists. Thus, the principle objective of this study is to introduce a new approach, using a very simple soil test to define soil activity, which reflects reliably the type and amount of the clay minerals and their base exchange capacity. The test is called the Water Adsorption Test after ENSLIN/NEFF (1959, 1988).

2 MECHANISMS OF MOISTURE ADSORPTION

Retention of water by soils is a result of attractive forces between the minerals and liquid phases. These forces are encountered through three mechanisms:

- direct adhesion of water molecules to mineral surfaces by van der Waals forces;
- capillary binding of water;
- osmotic binding of water

Adhesion is the attraction of water molecules to mineral surfaces by various types of London—van der Waals forces. These are very short-range forces, which diminish with about the sixth power of distance. Thus only an very thin water layer is adsorbed in this manner around soil particles. However, the adhesion forces, together with the cohesive forces between water molecules, form the basis for capillary binding of soil water.

Capillary binding is governed mainly by the sizes and shapes and spatial distribution of voids in the soil skeleton (fabric texture). Binding of water in an capillary can be characterized with the potential energy of water molecules near the particle surfaces.

Osmotic adsorption of water in the surface of clay mineral platelets (so-called diffuse electrical double layer) is the chief mechanism causing water adsorption in fine grained soils. Clay mineral surfaces generally have a vallency of negative charge due to isomorphous substitution, broken bonds around edges of the silica-alumina units, or the replacement of the hydrogen of the exposed hydroxyl groups by a cation. The overall electrical equilibrium is then maintained by the existence of an excess of cations in close proximity to the surface of the solid platelets. Because of the strong surface charge density of the clay minerals, they have the capability of adsorbing molecules and ions from the surrounding soil solution. Due to their dipolar nature, water molecules orient themselves in the electrostatic field and attach to the clay surface. Because of the field strength decrease with distance from the clay platelet surface, the water molecules experience a net static electrical force in the direction of the clay surface. This force decreases gradually with distance from the surface and is zero at the end of the double layer. This mechanism is totally dependent on the type and amount of clay minerals and the base exchange complex.

3 NEW SOIL PARAMETERS FOR DETERMINING SOIL ACTIVITY:

The water adsorption capacity of fine grained soils reflects the nature and character of the clay minerals present. The Water Adsorption Test after ENSLIN/NEFF is a very simple, reliable, cheap, and significantly rapid test. The test estimates the water adsorption capacity (Wasserbindevermögen) \( W_p \), which is defined as the amount of water drawn by the sample \( (m_w) \), divided by the dry mass of the soil sample \( (m_d) \). The property \( "W_p\)" gives an indication of the specific surface area and the cations attached to the clay minerals.
A decisive influencing factor is also the amount of the clay fraction. In searching for a quantitative measure of this factor, a new physical soil parameter is introduced. This parameter is called "Water Adsorption Index" and is defined as:

\[
\text{WAI} = \frac{W_b \, (\%)}{< 2 \mu m}
\]

The WAI depends on the clay fraction, amount and type of the dominant clay mineral and the attached cations. That is to say:

\[
\text{WAI} = f \left( \text{clay fraction, type of the clay mineral, attached cations} \right)
\]

\[
= f \left( x, y, z \right)
\]

with \( x = \text{clay fraction}; \ y = \text{type of the dominant clay mineral}; \ z = \text{binded cations}. \)

If \( y \) and \( z \) are constant, the "\( f \)" is a hyperbolic function in the form:

\[
\text{WAI} = \frac{a}{x^a}
\]

where, \( a \) and \( n \) = constants and are called here Water Adsorption constants.

Figures 1, 2 & 3 depict the hyperbolic relationship between the Water Adsorption Index and the clay fraction for different clay minerals and natural soils. The values of the constants "\( a \)" and "\( n \)" have been estimated after performing non-linear regression (see Table 1).

Most of the fine grained soils (clay fraction > 30 %) have been found to possess a Water Adsorption Capacity "\( W_b \)" greater than 40 % (Pichler (1993); Neff (1959, 1988); Demberg (1991)). Therefore, it is suggested to modify the Water Adsorption Index in the following way:

\[
\text{WAI}_m = \frac{W_b \, (\%)-40\%}{< 2 \mu m}
\]

Table 1: Values of the constants "\( a \)" and "\( n \)" for different minerals and soils.

<table>
<thead>
<tr>
<th>clay mineral or soil</th>
<th>a</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>15,50</td>
<td>0,687</td>
</tr>
<tr>
<td>Halloysite</td>
<td>15,06</td>
<td>0,674</td>
</tr>
<tr>
<td>Illicite</td>
<td>10,68</td>
<td>0,503</td>
</tr>
<tr>
<td>Kaoliglimmer</td>
<td>09,57</td>
<td>0,465</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>08,04</td>
<td>0,302</td>
</tr>
<tr>
<td>lateritic soils</td>
<td>35,40</td>
<td>0,830</td>
</tr>
<tr>
<td>tropical black soils</td>
<td>15,22</td>
<td>0,557</td>
</tr>
</tbody>
</table>
With the soil parameters WAI and WAIm, the activity of fine grained soils will be determined. Based on the results of the mineralogical, chemical, and mechanical soil tests on pure clay minerals mixtures carried by Pichler (1953) and by Nawari (1992) and others cited in the literature, the following definition of soil’s activity is made:

<table>
<thead>
<tr>
<th>WAI</th>
<th>WAI&lt;0.60</th>
<th>non-active</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>WAI&lt;2.0, WAI_m&gt;0.60</td>
<td>normal active</td>
</tr>
<tr>
<td>1.40</td>
<td>WAI&gt;2.0, WAI_m&gt;0.75</td>
<td>normal active</td>
</tr>
<tr>
<td></td>
<td>WAI&lt;2.0, WAI_m&gt;1.40</td>
<td>active</td>
</tr>
</tbody>
</table>

**dominant clay minerals:**
- Kaolinite, Halloysite, Allophane
- Illite, Chloride
- Illite, Chloride
- Smectite, Mixed-Layer Minerals, Vermiculite
- Smectite, Mixed-Layer Minerals, Vermiculite

**Fig. 2** Relationship between WAI and clay fraction for black tropical soils

**Fig. 3** Relationship between WAI and clay fraction for lateritic soils

4 COMPARISON WITH SKEMPTON'S ACTIVITY INDEX:

In 1953 Skempton introduced the concept of soil activity as a means to reflect the nature of predominant clay minerals in the clay fraction. He defined the activity index as:

\[ I_a = \frac{I_p}{<2 \mu m} \%

where, \( I_p \) = plasticity index
SKEMPTON's scheme divided inorganic soils into inactive, normal active, and active according to the following limit values of \( I_a \):

- \( I_a < 0.75 \) inactive
- \( 0.75 < I_a < 1.25 \) normal active
- \( I_a > 1.25 \) active
The geotechnical difficulties and disadvantages with the application of SKEMPTON'S activity index will be summarized in the following:

(a)- The determination of the plasticity indices (Liquid Limit, Plastic Limit) is not only time consuming, costly, and has a poor reproducibility, but also inherent with subjective mistakes. This was already noted by many investigators (e.g. NORMAN (1958); SOWERS (1959); NUGENS & KOCKAERST (1967); SHERWOOD (1970); WHYTE (1982)). SOWERS et al. (1959) contained the most comprehensive list of faults associated with the liquid limit test:

- Difficulty of cutting a groove in some soils, particularly those containing silt or sand.
- Tendency of some soils to slide in the cup rather than to flow plastically.
- Tendency of certain soils of low plasticity to liquify with shock rather than to flow plastically.
- Sensitivity to small differences in apparatus such as the grooving tool form, hardness of the base, shape of the cup, and wear on the cup.
- Sensitivity to operator technique as the result of groove shape and alignment, cleanliness of the cup bottom and base, speed of operation, observation of point of groove closing, lack of proper adjustment, and thoroughness of mixing.

(b)- SEED, et al. (1964), YUDHBIR & SAHU (1988) and others had concluded that with SKEMPTON'S activity index it is not possible every time to detect the dominant clay minerals. For example, pure kaolinite changes its activity from 0.4 to about 1.0 with the addition of only < 5 % montmorillonite. One more example represents the illite-montmorillonite mixture. By increasing the montmorillonite fraction, there is no corresponding variation in (\(I_a\)) until the montmorillonite percentage approaches 70 %.

(c)- For tropical soils, plasticity indices are not material constant and show great variability with sample preparation techniques (ARNOLD, 1984 and VAEGAS, 1988).

5 DESCRIPTION OF SOIL ACTIVITY USING THE ACTIVITY DIAGRAM:

In order to establish engineering-oriented classification for activity of soils, based on the Water Adsorption Test, a new diagram has been developed. This diagram is named an Activity Diagram (see Fig. 4). The diagram is specified with the following lines:

![Activity Diagram]

Fig. 4 Activity diagram

L - Line: \( WAI_m = 0.60 \)
M - Line: \( WAI_m = 0.75 \)
N - Line: \( WAI_m = 0.87 \)
O - Line: \(< 2 \mu m = 15 \% \)
P - Line: \(< 2 \mu m = 30 \% \)
Q - Line: \(< 2 \mu m = 40 \% \)

The four main domains depicted in the Activity Diagram are explained as:

Low active:
Dominant clay minerals are, for example, kaolinite, halloysite,
allophane. Geotechnical behaviour:
weak swelling and shrinkage potential,
low compressibility, high shear strength and bearing capacity.
Medium active:
Predominant clay minerals are usually illite and chlorite. Soils in this domain have normal swelling and shrinkage behaviour, medium shear strength and compressibility. High active:
Soils are characterized with high swell and shrinkage potential, low permeability, low shear strength, and high compressibility. The principal clay minerals are mixed-layer minerals, vermiculite and smectite. Very high active:
Peculiarity of these soils are very high swelling potential, very impermeable, very low bearing capacity and shear strength, and strong susceptibility to dispersion.

6 PRACTICAL EXAMPLES AND VALIDITY OF THE NEW APPROACH:

6.1 Tertiary clay soils from Frankfurt/M.:
Based on the study by PETERS (1989) and KLIESCH (1990) on Frankfurt clay, the results of the chemical, mineralogical and physical investigations are summarized in Table (2). Using the suggested activity classification diagram (Fig. 5), it is indicated that Rupel clay (Rupelton) varies from "medium active" to "low active" clays. The Landschneckenmergel changes from "high active" to "medium active", and the Prosothenien strata (Prosothenienschichten) also show "medium active" to "high active" characteristics. These results compare very well with their mineralogical analysis.

6.2 Black tropical soils:
According to the investigations conducted by NAWARI (1992), shows a summary of the mineralogical and physical soil analysis on different black tropical soils from the Sudan. The activity diagram Fig. 6, indicates that tropical black soils diversify from "very high active" - "high active" to "medium active". The relationship between the mineralogical composition and the activity diagram of these soils is
Table 2: clay soils from Frankfurt:

<table>
<thead>
<tr>
<th>Location</th>
<th>Quartz (%)</th>
<th>Feldspar (%)</th>
<th>Calcite (%)</th>
<th>Kaolinite (%)</th>
<th>Illite (%)</th>
<th>Chlorite (%)</th>
<th>Montmorillonite (%)</th>
<th>Clay (%)</th>
<th>Wb (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Samples from Frankfurt/Oftenbach</td>
<td>24-39</td>
<td>4-10</td>
<td>4-10</td>
<td>5-8</td>
<td>16-23</td>
<td>3-5</td>
<td>5-11</td>
<td>50-70</td>
<td>69-86</td>
</tr>
<tr>
<td>(Rupelton)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Samples from Frankfurt/Preuningsheim</td>
<td>32-39</td>
<td>4-10</td>
<td>1-2</td>
<td>6-8</td>
<td>16-19</td>
<td>2-4</td>
<td>22-29</td>
<td>70-75</td>
<td>91-100</td>
</tr>
<tr>
<td>(Landeschneckenmergel)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Samples from Frankfurt/Ginnheim</td>
<td>25-36</td>
<td>4-8</td>
<td>8-21</td>
<td>6-9</td>
<td>11-18</td>
<td>3-4</td>
<td>15-18</td>
<td>61-71</td>
<td>75-90</td>
</tr>
<tr>
<td>(Prosothenienschichten)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

clearly seen.

6.3 Soils from Bad Goisern (Austria):
ROHN (1991) carried out a large number of chemical, physical and mineralogical investigations in studying the stability of natural slope in Bad Goisern (Austria). Representing these samples in the activity diagram Fig. 7, the following can be concluded: the Hasel Mountains samples lie in "high active" to "medium active" domains. The clay fraction of samples comprised mainly illite and chlorite. Four Fleckenmergel samples are specified as "very high active". Their mineralogical composition shows more than 40% montmorillonite-illite mixed layer. The other two samples are classified as "medium active" soils. The high carbonate content of these samples is the reason for the decreasing activity of the clay.

6.4 Lateritic soils:
STUBENDORFF (1986) had investigated more than 100 lateritic samples from more than 10 tropical countries. Fig. 8 shows that these soils are "low active" clay soils. The mineralogical tests prove that the clay minerals are dominantly kaolinite.
Fig. 8 Activity diagram Lateritic soils

CONCLUSION:

In this study the application of the Water Adsorption Test to define the activity of soils was introduced. The mechanisms of water adsorption by fine grained soils and their relationship to amount and type of clay minerals were described.

New soil parameters, Water Adsorption Index (WAI) and the modified Water Adsorption Index were defined. These parameters are a function of the clay fraction, and the kind and amount of clay minerals. The mathematical expressions governing these relationship were given. Using these indices, the Activity Diagram was developed to classify soil activity. The results of application of the Activity Diagram for different temperate soils and tropical soils correlate very well with their mineralogical compositions.

The suggested classification scheme has been shown to be reliable, significantly simpler, quicker, and less expensive than the present system of activity classification. Furthermore, the proposed approach of defining soil activity will be of an influential role in identifying and classifying tropical soils.

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The influence of clay and marl weathering on their physical and mechanical properties
L'influence du processus d'altération des argiles et des marnes sur leurs caractéristiques physiques et mécaniques

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ABSTRACT: The influence of Neogene clay and marl weathering on their physical and mechanical properties is considered in this paper for Belgrade area. Quantitative estimates of physical and mechanical changes are based on laboratory data for clay and marl sampled in drill holes and geophysical data for the same holes. Changes are considered only of the integral physical and mechanical properties, i.e. the properties which involve all or nearly all geologic characters.

RÉSUMÉ: La communication analyse l'influence du processus de la désintégration superficielle des argiles néogènes et de la marnes dans la région Belgrade en leurs caractéristiques physiques et mécaniques. L'évaluation des modifications des caractéristiques physiques et mécaniques s'effectue sur la base des données issues des essais de laboratoire des échantillons d'argile et de marnes provenant des puits ainsi que des essais géophysiques effectués dans ces mêmes puits. Seul sont analysées les modifications des caractéristiques physiques et mécaniques de caractère général.

1 INTRODUCTION

Clays and marls in Belgrade area, like in most of Neogene basins, have been greatly changed in processes of weathering, forming a surfaces crust of a variable thickness. Where on plateaus, it is covered with loess (fossil weathering crust) to a maximum depth of 14-16m forming the full weathering section (Fig.1.) ; it is shallower on slopes, where it has been carried away in sheet-washes and slides.

Weathering processes caused partial or complete changes in mineral composition, texture and structure, and consequently in physical and mechanical properties. The geometry of the crust (position and thickness) and mechanical behavior are particularly important for the building activity as manifested in the following:

a) Most of structures both on and under ground, are built up in the weathering crust of clays and marls. The interaction zone of ground and structure is often in the crust.

b) Consequence of mechanical disintegration and chemical decomposition are significant changes in the resistance to rock deformation, their hydrologic function and stability.

c) All deformations: settling, flow and creep begin

Fig.1. Full section through Neogene clay and marl weathering crust in Belgrade area.
and end in the surface weathering crust. Each sliding formed in it (there are hundreds of them) is a direct or indirect consequence primarily of its poorer physical and mechanical properties than those of the adjacent unaltered rocks. The deepest sliding in Belgrade area reaches its bottom, i.e. the interface of weathered and fresh rocks. That is why its entire depth should often be considered in any land stabilization project.

All the above mentioned has a strong bearing on the general building conditions, sometimes even on the operation conditions as well.

2 INVESTIGATION RESULTS

The processes of surface weathering in Neogene clays and marls of Belgrade area have led to greater or smaller changes in almost all of their physical and mechanical properties. The limited length of the paper does not allow a description of all physical and mechanical changes; therefore only those properties will be described which are integral in nature, which include changes of all or nearly all other properties.

2.1 Velocity of elastic wave propagation

Velocity of elastic wave propagation was measured on samples 20 cm long. Velocities of longitudinal (\( V_p \)) and transversal (\( V_s \)) waves were measured. The measurement results statistically processed showed that longitudinal wave velocities in clay and marl weathering crust ranged from 500 to 1788 m/s, or 1460 m/s on average. Two zones can be distinguished: one, with the high variation of longitudinal wave velocities (from 500 to 1778 m/s) and mean value of 1180 m/s - the zone of intensive fracturing or complete clay and marl weathering; and the other, zone of less fractured clay and marl gradually passing to fresh rock with longitudinal wave velocities from 1415 to 1704 m/s and mean velocity of 1580 m/s. In unaltered, fresh clays and marls, the range longitudinal wave velocities was from 1480 to 1800 m/s, the mean velocity 1613 m/s.

Measured longitudinal wave velocity in the same drill hole, at approximately same depths, showed a generally equal order of velocity magnitude as that on samples (Fig. 2).

Both laboratory and field results showed that these methods were efficient for identification of intensive fracturing or complete weathering zones. Propagation velocities of transversal waves (\( V_s \)) are essentially similar in differences for the weathering crust and the zone of fresh clays and marls. But their use in identification of weathering crust zones is more difficult.

Fig. 2. Diagram of longitudinal wave velocity variations with the depth in clays and marls of Belgrade area. \( V_p \) - velocity in ground, \( V_{p_1} \), \( V_{p_2} \), \( V_{p_3} \), \( V_{p_4} \) - velocities on samples.

2.2 Heat conductivity

Heat conductivity was measured on drill-hole samples, primarily for the definition of temperature effect, as a factor of surface weathering. The measurement results are given in Fig. 3.

The increased heat conductivities in the weathering crust are a result of fracturing and of water and air presence in the fractures. The heat conductivity range in the highly fractured zone (8.50 - 12.50 m) with air-filled pores was 1.80 - 2.05 W/mK. Heat conductivity was the highest (2.10 W/mK) in a less fractured rock zone (12.50 - 18.00 m) where cracks were filled with water. Heat conductivity decreased to 1.30 W/mK from this zone to the zone of prevailing gypsum to the transitional zone. It varied from 1.50 to 1.60 W/mK in unaltered gray marls. It follows from the above stated that the temperature effect on surface weathering rapidly decreases below the depth of about 16 metres.
2.3 Uniaxial strength

Uniaxial strength of clays and marls is basically an integral property almost all other properties of these rocks. Unlike the grain size, specific gravity, bulk density, porosity, moisture content and other properties, the uniaxial strength of weathering rocks is appreciably different from that of unaltered rock. Uniaxial compressive strength values for weathering rock samples and fresh rocks were

\[(2.0 - 7.5) \times 10^4 \text{N/m}^2 \text{ and } (4.0 - 24.0) \times 10^4 \text{N/m}^2\]

respectively. A rapid rise in the uniaxial strength was notable from the yellow-brown and gray marls boundary, i.e. from the weathering/fresh rock interface. It was considered normal in respect to the hydrogeologic properties and the ground water dynamics. Deep in the weathering crust, ground water flows mainly along cracks, turning them into the privileged break planes in rock specimens. The scatter of values was much greater in the fresh rock zone. From statistical analysis of the mentioned data I deduced that the variation in uniaxial compressive strength deep in the weathering crust could be approximated by a linear relation, which reads as follows (Fig. 4).

\[\sigma = -0.08h + 5.28\]

The large scatter of uniaxial strength values for the zone unchanged gray marls hardly allows its relation the depth. This broad scatter is a likely result of the high heterogeneity of the zone, and the consequent inability to have representative samples. In point of fact, any number of straight or curved lines can be drawn. My statistical analysis has shown that a linear relation may correspond to this zone, expressed by equations:

\[\sigma = 0.86h - 14.28 \quad \text{and} \quad \sigma = 0.16h + 10.77\]

Based on the distribution of uniaxial strengths of fresh marls, the relationship between the strength and the depth of samples may be expressed by a more complex function graphically represented in Fig. 4.

2.4 Shear strength parameters

Shear strength parameters, i.e. angle of internal friction and cohesion, essentially differ for the weathering crust and the fresh clay and marl zone. These two parameters have highly variable values for the weathering crust: angle of internal friction mainly
within the range $10^0 - 20^0$ and cohesion from 10 to 80 kN/m$^2$. The variations are results of dissection and degree of disintegration, i.e. heterogeneity and anisotropy of rocks by these parameters, and the size of the study specimen/area. The angle of internal friction and cohesion in the fresh clay and marl zone respectively vary from 15$^0$ to 25$^0$ and from 80 to 120 kN/m$^2$, locally even to 150 kN/m$^2$. The lower the angle of internal friction, the higher the cohesion, and vice versa. Values of these parameters are sometimes much lower in the field. This is explained by the size of the study specimen/area and the impossibility under laboratory conditions to take into consideration many factors affecting these parameters.

2.5 Dynamic modulus of elasticity

Dynamic modulus of elasticity was determined using the propagation velocity of elastic longitudinal waves on samples 20 cm long. The dynamic modulus of elasticity varied in the weathering crust from 470 to 1000 MN/m$^2$, and in the unaltered, fresh clay and marl zone from 1000 to 1423 MN/m$^2$ (Fig. 5.)

Generally scrutinized, the dynamic modulus of elasticity grows with the depth. The diagram also shows a clear boundary between the surface weathering crust and the fresh rock zone, by this parameter.

The static modul of elasticity of the weathering zone of clays and marls ranges between 16 and 24 MN/m$^2$, and in a unaltered, fresh rock mass between 150 and 200 MN/m$^2$. Approximately the

![Fig. 5. Diagram of dynamic modulus of clay and marl elasticity in relation to depth, calculated using wave velocity $V_p$ on samples from hole B - 1.](image)

![Fig. 6. Variations in geoelectrical, radioactive and temperature of clays and marls in hole B - 1.](image)
properties are almost entirely based on laboratory sample testings. The results, consequently, bear all the know inadequacies and possible errors in respect of sample representativeness in terms of scale, drilling technology and sampling, laboratory methods, etc.

Any study in situ would certainly give better results, but most of physical and mechanical properties are not determined in situ. For an as reliable estimate as possible of changes in the clay and marl physical and mechanical properties caused by processes of weathering, logging was made in the same drill hole from which samples for lab tests were used. Geoelectrical, thermal and radioactive measurements were taken and the results given in Fig. 6.

The measurement results have shown that geo-electrical and radioactive properties of clays and marls very well identify the weathering crust or the boundary of altered and unaltered rock properties. The measurements clearly separate the interface between weathering and transitional zones (depth about 22m). Natural radioactivity distinctly defines the boundary between weathered and fresh clays and marls (depth about 28m). The neutron-neutron log distinctly shows four weathering zones which have depths in good agreement with those of megascopic core records (about 12m, 18m, 22m and 28m).

3 CONCLUSION

It follows from the afore given analysis and deductions that weathering processes appreciably changed the physical and mechanical properties of Neogene clays and marls in Belgrade area. The changed character is manifested in color and grain size and all other properties (specific gravity, density, porosity, angle of internal friction, cohesion, uniaxial compressive strength), others (specific gravity, density, Atterberg limits of consistency) are slightly different from those of fresh rocks.

Clearly, all properties in the weathering crust have been more or less changed, as shown by the compressive strength as an integral property of rock and the velocity of elastic wave propagation, and by complete geophysical log. Physical properties of rocks (heat conductivity, electrical conductivity, self-potential, natural radioactivity, etc.) integrate all geologic properties of rocks and as such are a reliable indicator of the physical and mechanical character of rocks in the weathering crust.

Any complex field investigation should certainly include, in addition to laboratory tests, field surveys of elastic wave propagation velocity and logging of electrical conductivity, self-potential and radioactivity of rocks.

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Božinović, D. 1969. Geotehničke osobine terena u početku Beograda koje izgradju koherentni gornjemiocenski sedimenti, doktorska disertacija odbranjena na Rudarsko-geološkom fakultetu u Beogradu (Geotechnical Properties of Terrain in Belgrade Area Built-up of Coherent Upper Miocene Sediments, Doctoral thesis defended at the Belgrade Faculty of Mining and Geology).

ABSTRACT: Many clay soils in Poland, especially in the central and western part, possess a large potential for volume change. The aim of this study is to show how even small quantity of calcareous nodules plays an important part in volume changes of expansive clay. Such phenomena arises because of mineralogical effects, particularly clay mineralogical swelling due to hydration and dehydration process. Samples were collected from two different areas in Wroclaw where volume changes of soil affected low cost housing.

1 INTRODUCTION

It is a characteristic of clay minerals in clay soils that volume changes occur with changes in water content. Mainly such type of soils is found in the western part of Poland (see fig.1). Figure 1 shows borders of existing plicocene clays and territories of shallow deposition of drifts.

The resulting shrinkage and swelling cause damage to structures and, as pointed out by Burland (1984), low-rise housing is particularly vulnerable since the sub-strata are not heavily loaded and the structures themselves do not possess a great deal of stiffness. However, other structures, particularly those associated with the infrastructure, are also susceptible to volume changes effects. A considerable amount of small towns in south-west part of Poland are comprised of this type of buildings constructed 60 - 70 years ago and founded on strip footings between 0.7 to 1.0 m below ground level. It is hardly to say anything about total cost of repairing such houses in Poland, because buildings...
are own by different institutions (some are private, some belong to the town, and some belong to housing co-operatives).

This problem is similar to problems in other countries. Although it is particularly prevalent in arid and semi-arid countries, shrink-swell problems also affect countries in temperate zones like Poland, Romania or UK. The annual economic cost in the middle 1980s of the problem in the USA and Sudan as catalogued by Burland (1984) are respectively $6 billion to $8 billion and $6 million. A ten year drought (dry winters and autumns as well as warmer summers) caused in Poland rapid decrease in soil’s natural moisture till 2.5 m below ground level. We can compare thickness of this active layer to the similar one in the India, Australia, the USA - 3.0 m thick or Romania 2.5 m thick. But for "hot" countries, as Israel, this active layer is equal 6.0 m. fig.2) than as a result of dried up soil, building gets out of the tree. If sequence of operation mentioned above is reversal (see fig.3) than the part of the building situated near to tree settles more.

Driscoll (1983) notes that large trees may extend the depths to which desiccation takes place and suggests that rapid change in moisture content corresponds with a soil suction of approximately 10 kPa. Such processes are liable to become significant such that low-rise buildings are uplifted where a suction of 100 kPa is reduced by soil wetting.

As water is removed from the pore space of soils, the relict water exerts a surface tension force between the soil particles. Since the action is similar to an all round pressure being exerted on the soil, shrinkage, the magnitude of which depends on the value of the suction pressure, results.

![Diagram](image)

**Fig.2** The influence of the tree planted before the building was put up, 

a) when the whole building is under the influence of the tree

b) when the building is partly under the influence of the tree

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2 THE SHRINKAGE PROCESS

In Poland the decrease in soil’s natural moisture is caused by small amount of rain and snow falls, and mainly by taking up water through the roots of large trees growing near buildings. It was proved that a single tree dries soil in the radius equal 1.5 H, where H is the height of this tree. The type of damages of construction depends on the moment the house was built: before or after planting the tree nearby (see fig.2 and 3) Przystański (1991). If the tree was planted before the building was built (see

3 DAMAGE TO STRUCTURES AT WESTERN POLAND

In Wroclaw (700 000 of inhabitants), especially in the south eastern part of the town, many clay soils exist, they possess a large potential for volume change. That's why this is an important problem for the town. I would like to present two case histories of damaged buildings (details are presented in Pula 1993).
Fig. 3 The influence of the tree planted during usage of the building
a) when the whole building is under the influence of the tree
b) when the building is partly under the influence of the tree

3.1 Case history 1: Building in ribbon development

Extensive damage has been caused to a large number of properties constructed as a part of a low cost housing scheme west of Wroclaw. The buildings were of conventional construction with both external and internal load bearing walls in brick resting on strip footings between 1.2 and 1.5 m below ground level. The ground floors were unreinforced brick beam-framed floor and resting directly on the bearing walls. The second floors were constructed as wooden beam-framed floor. The rafter framings are also wooden. The length of this building is 298.50 m and its width is 16.20 m (see fig. 4).

The area is underlain by clays and silty clays from the Tertiary and pleistocene drift. In a few places, in length of the building, fine sand is also present.

The investigations included the detailed documenting of the damage to the structures together with topographic information and the presence of covered areas, drains and gardens. Along the front bearing wall grow many large trees whose roots decreased moisture of soil below its shallow foundation. In addition, soil profiles were investigated in trial pits and boreholes. In order to ensure that samples were not affected by building loads the borehole were positioned at least 0.5 m from the external walls. Undistributed block

Fig. 4 View of the front bearing wall.
samples were obtained from the inspection pits during the summer when the water content was near its minimum value. The set of geotechnical parameters is presented in Table 1.

Cracking of the structures were noticed ten years ago, thirty five years after occupation. In the external walls most cracks extended diagonally from above and below windows, above door openings and at the corners and near the apices of gable walls. Once formed the cracks progressively widened about 2 mm per year.

The internal walls were also affected with cracks extending above doorways and down the sides of the door frames, leading to the distortion of many door openings. Because of those damages, some inhabitants, in fear of their lives, were evicted from the part of this building. In Fig. 4 the front bearing wall can be seen, and in Fig. 5 and 6 - cracks of front as well as rear bearing wall.

This type of movement was consistent with the effects of swelling clays in which doming of the structure occurs due to a higher water content being maintained at the center and clays beneath the outer

walls of buildings being subject to fluctuating shrink and swell respectively in dry summer seasons and wetter autumn ones.

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>23 - 40</th>
<th>50 - 88</th>
<th>20 - 30</th>
<th>32 - 65</th>
<th>7 - 10.20</th>
<th>17 - 18.75</th>
<th>105</th>
<th>0.03 - 0.25</th>
<th>3.0 - 72</th>
<th>4 - 10</th>
<th>28 - 60</th>
<th>6.3 - 8.2</th>
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<td>Cohesion [kPa]</td>
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3.2 Case history 2: Detached houses

The shrink-swell problem is also danger to detached houses, especially if the trees were planted after building houses. The example of such a house is presented in fig. 7.

The two-storey building, one of a large number of properties built in the south part of town, was of conventional construction with both external and internal load bearing walls in brick. The investigation showed that the footings were only 1.1 m below ground level. The ground floors were unreinforced brick beam-framed floor and resting directly on the bearing walls. The second floors were constructed as wooden beam-framed floor. The rafters framings are also wooden. The length of the building is 15.50 m and its width - 9.90 m.

In 1993 a conventional shell-and-auger borings were drilled, two at the front of the building and two at the rear of the house. Results of drilling shows that the area is underlain by very thick (more than 25 m) layer of clays from the Tertiary and pleistocene drift with many calcite nodules. The full index of geotechnical parameters is presented in table 2. Because of the presence of great amount of calcite nodules sharply increases content of CaCO₃ in the soil samples.

The house was surrounded by trees, some of them were as much as 25 m in height.

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nat water content (%)</td>
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</tr>
<tr>
<td>Liquid limit (%)</td>
<td>42 - 63</td>
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<tr>
<td>Plastic limit (%)</td>
<td>14 - 27</td>
</tr>
<tr>
<td>Plasticity index (%)</td>
<td>20 - 47</td>
</tr>
<tr>
<td>Linear shrinkage (%)</td>
<td>18 - 34</td>
</tr>
<tr>
<td>Free swelling (%)</td>
<td>110</td>
</tr>
<tr>
<td>Liquidity index (%)</td>
<td>0.1 - 0.3</td>
</tr>
<tr>
<td>Cohesion [kPa]</td>
<td>30 - 50</td>
</tr>
<tr>
<td>Int. angle friction (°)</td>
<td>9 - 15</td>
</tr>
<tr>
<td>Clay &lt; 2 μm (%)</td>
<td>25 - 65</td>
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<tr>
<td>CaCO₃ (%)</td>
<td>10 - 13</td>
</tr>
</tbody>
</table>

Cracking of the structures were noticed six years ago, forty eight years after occupation. At the beginning minor cracking (up to 2 mm wide) had been reported, most of which was confined to the ground floor. In the external walls most cracks extended diagonally from above and below windows, above door openings and at the corners and near the apices of gable walls. Some of cracks are vertical.

Fig.7 Diagonal cracks extending from the door frame to the nearest window.

Particularly between building and it annex. Once formed the cracks progressively widened, more or less 1 mm per year. The internal walls were also affected with cracks extending above doorways and down the sides of the door frames, leading to the distortion of many door openings. Now this building needs heavy repair (fig 7 and 8).

In this case, like above, movement of structure was consistent with the effects of swelling clays during wet winter and shrinking during hot, dry summer.

4 DISCUSSION OF RESULTS

Soils which are presented above can be classified (according USAEWES classification of swell potential O’Niel and Poormoayed 1980) as a
medium class. For those soils quantitative analysis were also carried out to determine the calcite percent by chemical analysis. Average contents of CaCO₃ for soil in case history one is equal 8.2 % and for case history two 12.2 %.

After this, free swelling test for each sample of soil in oedometer were carried out. The sample was flooded with distilled water and allowed to swell until equilibrium was reached. This test gave as a value and curve of volume changes of free swell for the sample. Examples of such curves for both cases are presented in fig. 9 (for comparison third curve for 2.6 % of CaCO₃ is presented). The results of the oedometer tests are considered to have been affected by the presence of nodules of calcrite.

Various empirical and deterministic methods (Van der Merwe 1964, Weston 1980) exist for identifying clay deposits that are prone to swell behaviour but any classification does not take into account the percent of CaCO₃ in a sample.

It seems possible that the presented in fig.9 relationships could be typical of all clay soils with high contents of calcrite nodules.

Fig. 8 Cracks in the internal walls.

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Fig. 9 Changes of sample's height due to time in swelling process.
5 REMEDIAL MAINTENANCE

Different remedial measures were designed to restore the structural integrity of both buildings:
- due to the low cost nature of the building in the ribbon development, it was considered uneconomic to undertake comprehensive works. In such situation the owner has decided on partly demolition of the building. In the left part of the building underpinning to certain of the external and internal walls was extended to a depth of at least 2.0 m below the ground surface. The foundations were taken down to weathered bedrock by placing mass concrete where the bedrock was less than about 1.3 m below the base of the foundation. To reduce changes in the water content beneath the edges of perimeter paving extending 1.0 m from the outer walls was instaled.
- in case of detached house it was decided, first to remove the nearest high tree, which roots dried up the soil below the foundation. Next walls were underpinned by means of augered mini-piles. Apar from those actions the building was also anchored in two levels, using 22 mm steel bars and steel plates on each external walls. After that, severely damaged walls with cracks wider than 10 mm were repaired by stitching them together with 10 mm diameter mild steel bars that were grouted into 35 mm deep grooves formed in the mortar between the courses of the brickwork. The bars extended at least 500 mm on either side of the cracks and they were placed 200 mm apart.

6 CONCLUSIONS

Soils with a capacity for swelling cover nearly half of territory of Poland and are serious cause of foundation problems. The full economic impact of the damage to structures is difficult to quantify precisely, but is comparable with the costs of natural disasters. However since, the effects are most seriously concentrated on individual small structures, there has been relatively limited co-ordinated action. The case records that have been presented in this paper show the range of values of soil suction to be the building, expected in samples taken from clay sites where desiccation has occurred, and where there has been consequential building damage. Some of these records show that the desiccation caused by trees can be comparable with other reasons.

The data presented in this paper indicate that not only clay content index or liquid limit are important parameters for swell or shrinkage potential of expansive soils but also quantity of CaCO₃.

The phenomenon of shrinkage can be speeded up by roots of high trees growing near the shallow founded buildings. This is specially danger if, at the beginning small tree, was planted quite near (even a few meters) from the building. After twenty or thirty years roots of big tree completly will change geotechnical conditions above foundation of the building.

7 REFERENCES


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Change of hydraulic properties of muddy deposits during compaction: Assessment of mechanical and chemical effect
Changement des propriétés hydrauliques des boues pendant la compactage: Évaluation de l'effet mécanique et chimique

Tomochika Tokunaga, Shin-ichi Hosoya, Keiji Kojima & Hiroyuki Tosaka
University of Tokyo, Japan

ABSTRACT: To quantify permeability change during mechanical compaction, one dimensional consolidation experiments of artificially deposited or remolded muddy samples were carried out together with variable head permeability measurements at each consolidation equilibrium. Results show that porosity(ϕ)--permeability(K) relationships for respective samples are linear on double logarithmic scales over the range 0.25<ϕ<0.7, that is to say, porosity and permeability satisfy the relationship: K=K₀(ϕ/ϕ₀)ᵃ where K₀ is initial permeability, ϕ₀ is initial porosity, and a is constant. The relationship is confirmed by thorough review of published data. The equation will be useful to express permeability reduction by mechanical compaction from the range of soil to that of soft rock.

RESUMÉ: Pour quantifier les changements de perméabilité pendant la compaction mécanique, des expériences de consolidation unidimensionnelle ont été faites sur des échantillons de boues artificiellement déposées ou remodelées, ainsi que diverses mesures de perméabilité d'avancée variable à chaque équilibre de consolidation. Les résultats ont montré que, pour les échantillons respectifs, la relation porosité(ϕ)–perméabilité(K) était linéaire sur les échelles bilogarithmiques, sur la plage de 0.25<ϕ<0.7, autrement dit que le porosité et la perméabilité satisfaisaient la relation: K=K₀(ϕ/ϕ₀)ᵃ, où K₀ est la perméabilité initiale, ϕ₀ la porosité initiale et a une constante. La relation a été confirmée par la revue minutieuse des données publiées. Cette équation sera utile pour exprimer la réduction de la perméabilité par compaction mécanique, allant du sol à la roche tendre.

1 INTRODUCTION

Prediction of long–term fluid migration is becoming important for environmental evaluation associated with geological disposal of radioactive wastes and with other processes. Correct estimation of the change of hydraulic properties of sediments during geological processes is one of the keys for reliable prediction. For example, because muddy deposits contain large amount of pore water in their depositional stage, large amount of water is expelled from muddy deposits as sedimentary sequences are compacted, indicating that the process of expulsion of pore water from muddy deposits controls both fluid migration in the sedimentary basin and the mode of compaction of muddy horizons and of whole sedimentary sequences. Thus, the change of permeability of muddy deposits is prime importance to assess the evolution of the sedimentary sequences.

However, there are few studies which cover wide–ranged change of permeability, e.g., from muddy soil to hard rock. To this end, one dimensional consolidation experiments and thorough review of published data were carried out to determine the change of permeability by mechanical compaction. Muddy deposits are generally considered that their physical properties have anisotropies. Permeability also shows anisotropy especially
when compaction of the deposits progresses (e.g., Wilkinson & Shipley, 1972; Al-Tabbaa & Wood, 1987). However, considering fluid migration in alternating sequences of sandstones and mudstones which are typical in sedimentary basins, dominant migration direction of fluids in muddy sequences is perpendicular to bedding surfaces (Magara, 1978). Thus, the study focuses on the permeability which is perpendicular to maximum compression axis or to bedding surface.

Chemical processes during compaction are also important to assess the change of permeability and to assess other physical properties. Although we have been trying to unravel the effects of chemical compaction on permeability reduction of muddy deposits, it has been under investigation. In the following sections, we mainly discuss the effects of mechanical compaction on permeability reduction of muddy deposits.

2 EXPERIMENTS

2.1 Apparatus and experimental procedure

One dimensional consolidation apparatus shown in Figure 1 was used to carry out both consolidation and permeability measurements. Maximum compressive stress which can be applied by the apparatus is 500 kgf/cm².

Artificially deposited or remolded samples were inserted into the cylinder and load was sequentially applied. Pore water of the sample was drained through both the upper and lower drainage discs. Hydraulic heads of both the upper and lower outlets were set at the same value during consolidation to avoid generation of vertical gradient of porosity in the sample.

Permeability measurements were carried out when pore pressure of the sample became hydrostatic, i.e., the sample was consolidated to its equilibrium. Falling head permeability measurements were conducted by injecting water from lower side (Figure 1). For a falling head permeability test in which flow through the sample is entirely vertical, the vertical permeability can be calculated from the formula

\[ K = \frac{a}{A} \frac{L}{t} \ln \left( \frac{h_0}{h} \right) \]  

(1)

where \( K \) is the vertical permeability of the sample, assumed uniform and constant, \( a \) is the cross-sectional area of the permeability burette, \( A \) is the cross-sectional area of the sample, \( L \) is the height of the sample, \( h_0 \) is the initial difference of hydraulic heads between the upper and lower outlets, and \( h \) is the difference of hydraulic heads at time \( t \). Thus, permeability was obtained by plotting logarithm of \( h \) versus \( t \) and by getting the regression line using method of least squares (Figure 2).

Porosities of respective measurement stages were determined by back calculation from the final sample weight and dimensions. Thus, axial displacements during each stress increment were recorded by an linear variable displacement transformer (LVDT) attached in parallel with the loading piston (Figure 1).

Overall procedures of the experiments were shown in Figure 3.
2.2 Samples used in the experiments

Samples used in the experiments were crushed siltstone from the Plio-Pleistocene Kivada Formation of the Kazusa Group in Boso Peninsula, Japan; crushed Rochester shale in New York; and commercially available kaolinite powder (Table 1).

Samples were set by either the following procedures:

1. Large amount of pure water and powder samples were mixed and poured into the cylinder. They were kept until the particles of minimum size settled down. Settling time of the smallest particles was estimated by the Stokes' law. This initial condition was used to obtain similar fabric to the naturally deposited sediments. This condition is referred to as 'depositional setting' here.

2. Samples were prepared as slurries, with relatively smaller amount of pure water than the above. These slurries had porosities of approximately 0.8 and were viscous enough to prevent segregation during filling of the cell. This condition is here named as 'slurry setting'.

2.3 Results of the experiments

Typical experimental results of falling head permeability measurements by each consolida-

![Figure 2. Log h versus time relationship for a falling head permeability experiment (exp. 7).](image)

![Figure 3. Procedures of the experiments in this study.](image)

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Sample name</th>
<th>Initial condition</th>
<th>Specific gravity</th>
<th>clay content(%)</th>
<th>silt content(%)</th>
<th>D_{10}(μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>exp. 1</td>
<td>crushed Kiwada Fm.</td>
<td>ds</td>
<td>2.56±0.01</td>
<td>10.9</td>
<td>82.4</td>
<td>3.71</td>
</tr>
<tr>
<td>exp. 2</td>
<td>crushed Kiwada Fm.</td>
<td>ds</td>
<td>2.56±0.01</td>
<td>16.4</td>
<td>79.0</td>
<td>2.81</td>
</tr>
<tr>
<td>exp. 3</td>
<td>crushed Rochester shale</td>
<td>ds</td>
<td>2.77±0.01</td>
<td>30.9</td>
<td>69.1</td>
<td>2.40</td>
</tr>
<tr>
<td>exp. 4</td>
<td>crushed Rochester shale</td>
<td>ds</td>
<td>2.77±0.01</td>
<td>29.0</td>
<td>71.0</td>
<td>2.38</td>
</tr>
<tr>
<td>exp. 5</td>
<td>crushed Kiwada Fm.</td>
<td>ss</td>
<td>2.56±0.01</td>
<td>22.5</td>
<td>76.0</td>
<td>2.52</td>
</tr>
<tr>
<td>exp. 6</td>
<td>crushed Rochester shale</td>
<td>ss</td>
<td>2.77±0.01</td>
<td>29.8</td>
<td>70.2</td>
<td>2.39</td>
</tr>
<tr>
<td>exp. 7</td>
<td>kaolinite powder</td>
<td>ss</td>
<td>2.61±0.01</td>
<td>37.8</td>
<td>62.2</td>
<td>----</td>
</tr>
<tr>
<td>exp. 8</td>
<td>kaolinite powder</td>
<td>ss</td>
<td>2.61±0.01</td>
<td>37.8</td>
<td>62.2</td>
<td>----</td>
</tr>
</tbody>
</table>

Table 1. Samples used in the experiments. In the columns 'Initial condition', 'ds' indicates depositional setting and 'ss' indicates slurry setting respectively.

637
tion equilibrium are shown in Figure 2. Observed relationship between time and logarithm of $h$ are approximately linear, as expected from the form of expression (1). Measured vertical permeability data of several runs obtained from falling head experiments are plotted against porosity on double logarithmic axes in Figure 4. These results were all obtained from experiments on normally consolidated samples for which the vertical effective stress was being steadily increased and porosity was steadily falling. Error bars shown in Figure 4 were estimated from both errors derived from the measurements during experiments and errors derived from the least square fitting to obtain permeability. Results of each run show that the porosity($\phi$)-permeability($K$) relationships are linear on double logarithmic scales. Thus, an empirical equation:

$$K = K_0 \left( \frac{\phi}{\phi_0} \right)^a$$

(2)

where $K_0$ is initial permeability, $\phi_0$ is initial porosity, and $a$ is constant, is obtained from our experimental results.

On the other hand, soil engineers often use the different relationships between permeability and pore fractions. Tavenas, Jean and others (1983) evaluated several equations which relate permeability with pore fractions. They concluded that from a practical point of view, a relation

$$\log K = \log K_0 - \frac{e_0 - e}{C_k}$$

(3)

where $e_0$ is initial void ratio, and $C_k$ is the permeability change index (Tavenas, Jean and others, 1983), is excellent for initial void ratios less than 2.5 and for volumetric strains of practical interest in engineering problems.

In this context, our experimental results were compared with equation (3). Figure 5 shows the plots of the experimental results by void ratio as abscissa and logarithm of permeability as ordinate. If experimental results follow equation (3), plots should be linear in Figure 5. In case when we only consider void ratios greater than 1.0, equation (3) is surely excellent. However, considering rather large

![Figure 4. Relationships between permeability and porosity on double logarithmic scales.](image-url)
volumetric strains, equation (3) no longer matches measured permeability reduction trend. On the other hand, overall permeability reduction trend is well reproduced by the equation (2).

3 REVIEW OF PUBLISHED DATA

A lot of low-stress consolidation-permeability measurements have been conducted in the fields of soil engineering and agriculture (see references). There exist a few publications which report permeability measurements using high-stress consolidation apparatus (e.g., Shinjo & Komiya, 1984).

We extracted published experimental data from about 30 publications, which were included in references, and reviewed their results to compare the data with our measurements.

Part of the rearranged published data are shown in Figure 6. Absolute values of data were not adequate for direct comparison.

![Figure 5. Relationship between logarithms of permeability and void ratio.](image)

![Figure 6. Relationships between permeability and porosity on double logarithmic scales of laboratory consolidated experiments rearranged from published data (data sources are in references). (a) Remolded samples. (b) Initially undisturbed samples.](image)
because of wide variation of samples and of methods used in the measurements. However, change of permeability of most of the samples show that relationships between permeability and porosity of consolidation experiments of both remolded samples and initially undisturbed samples are almost linear on double logarithmic scales (Figure 6). These relationships are consistent with our experimental results. Thus, considering mechanical compaction of muddy deposits, equation (2) possibly explains the permeability reduction trends.

4 DISCUSSION

A new empirical equation (2) was compared with published permeability versus porosity data of naturally compacted samples to assess the possibility of extrapolation of the equation to natural condition. Data of naturally compacted samples were obtained from the following samples:

1. Well core mudstones in the Gulf of Mexico measured by Terzaghi's method from Bryant and others (1975).
2. Well core mudstones in Japan and Canada measured by direct method (details were not shown) from Magara (1978).

Each measured data was rearranged in the same way as we did in Chapter 3. Results of the rearrangement were shown in Figure 7 with the data of experimentally consolidated samples. Most of the naturally compacted mudstones are plotted at similar places with those obtained by laboratory consolidation experiments. Their overall permeability reduction pattern also follows that we determined in the previous discussions, when their porosities are grater than about 0.3. This observation indicates that equation (2) is possibly extrapolated to natural condition when porosity of the sample is grater than about 0.3. However, this consideration is preliminary because we have not yet understood time effects which are the most pronounced difference between laboratory consolidated samples and naturally compacted ones.

Data of naturally compacted samples show a little different pattern when porosities become less than about 0.3 (Figure 7). Change of permeability of naturally compacted samples seems to deviate from that of mechanically compacted ones. That is to say, permeability of naturally compacted samples tends to be larger than that of mechanically compacted ones at the same porosity values.

There are several possible explanations for the deviation.

One possible explanation is that the effect of chemical compaction, e.g., cementation, pressure-solution precipitation, and so on, becomes significant when porosity becomes lower value.

On the other hand, Inami & Hoshino (1974) measured the drained compressibility of argillaceous rocks, which were taken from both surface exposures and exploratory wells in Japanese oil bearing areas, and found that the compressibility changed regularly according to

![Figure 7. Relationships between permeability and porosity on double logarithmic scales of naturally compacted samples rearranged from published data (after Bryant and others, 1975; Magara, 1978; Dutta, 1988; Katsube and others, 1991).]
porosity when porosities were less than about 0.3 and that the compressibility became as large as that of liquid when porosities were greater than about 0.3 (Figure 8). They explained that the abrupt change of the compressibility indicated that the bulk physical properties of the samples changed from viscous fluid to plastic solid when their porosities decreased to be less than 0.3. Thus, the mechanisms of compaction changed when the porosity went through the value at around 0.3. This also is possible explanation of the difference of the change of permeability between experimentally consolidated samples and naturally compacted ones.

There are possibly other explanation for their deviation, and the problems have been under investigation.

5 CONCLUSIONS

The experiments and thorough review of published data lead to the following conclusions.

1. A relation

\[ \log K = \log K_0 - \frac{e_0 - e}{C_k} \]

which has been widely used in the field of soil engineering, is excellent for estimating permeability for the range of soil, in which porosities are greater than about 0.5. However, considering the range of soil to that of soft rock, the following empirical equation seems to better express porosity versus permeability relationships.

2. Linear relationships between \( \log \phi \) and \( \log K \) were obtained both from high-stress consolidation experiments and from thorough review of published data. Thus, a new empirical equation

\[ K = K_0 \left( \frac{\phi}{\phi_0} \right)^a \]

is proposed which expresses the change of permeability of muddy deposits by mechanical compaction from the range of soil to that of soft rock.

3. Naturally compacted muddy deposits follow the new empirical equation when their porosities are greater than about 0.3. Thus, the equation could be extrapolated to natural condition. However, permeability versus porosity relationships are diverted from the equation when porosities of the samples are less than about 0.3. Several explanations for the deviation are possible, and the problems have been under investigation.

ACKNOWLEDGEMENTS

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SI METRIC CONVERSION FACTORS
kgf/cm² × 98.0665* = kPa
md × 9.869233E-04 = μm²
*:Conversion factor is exact.
The application of the hoop tension test to a study of the residual stored strain energy within a jointed rock mass

L’application de l’essai de traction à l’étude de l’énergie de déformation résiduelle dans un massif rocheux fissuré

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ABSTRACT: This paper describes the variation of hoop tensile strength of rock with respect to the direction of paleo and active fields of in-situ stress, and residual stored strain energy.

RÉSUMÉ: Cette publication décrit la variation de la force ultime en tension des échantillons de roches chargées unidirectionnellement en dépendance de la direction du paleo et du champ de contraintes in situ, et en dépendance de l’énergie de déformation résiduelle.

1 INTRODUCTION

It is desirable to know the magnitude and distribution of residual stress because such stress can be considered either beneficial or detrimental to the mechanical behaviour of rock. A tensile residual stress would generally be considered more detrimental to rock strength than a compressive one, because rock has lower resistance to tensile stress. The significance of such residual stress has been studied since early 1970’s, by (i) mathematical modelling (Voight et al 1974, Holzhausen and Johnson 1979), (ii) experimental modelling and simulation (Varnes and Lee 1972, Holzhausen and Johnson 1979), (iii) acquisition of quantitative data from XRD (X-Ray Diffraction) (Friedman and Logan 1970), and stress relaxation measurements (Swolfs et al 1974). Unfortunately, the complexity of measurement and interpretation of residual stress makes it very difficult to be certain of the effect residual stress may have on rock behaviour, because previous measurements of residual stress show that the values obtained varied with the volume and the shape of the relieved boundary of rock (Engelder 1993).

The new approach described here, for the study of the distribution and magnitude of residual stored strain energy within samples of rock, taken from the jointed rock mass, is the application of the hoop tension test; this provides a measure of tensile strength, and work-done to failure in extension.

2 DESCRIPTION OF THE TECHNIQUE

The apparatus of the hoop tension test consists of two semi-cylindrical loading platen and an hydraulic piston to separate them. Failure generally occurs at the point of maximum tensile stress, which is on the plane of platen separation (Xu et al 1988, John et al 1991, Al-Samahiji 1992, Butenuth et al 1993). Pressure is directly controlled by a GDS Digital Controller with computer. Pressure and the position of piston are monitored and recorded. Load line displacement (or opening) of the platen is also recorded.

In this work, sets of carefully oriented samples were tested under a constant loading rate of 5.8 MPa/min and the work done to failure was calculated from the force-displacement curve (Fig. 1).

3 TEST ON THE CLEVEDON LIMESTONE

Hoop samples were cored from an antiform fold because such a fold is developed in response to previous stresses where basic orientation to the axis of folding is fairly well known (Price 1986, Price and Cosgrove 1990). Oriented samples from a fold would thus have different orientation with these directions and might be expected to have different values of work-done to failure. A fold in Carboniferous limestone in Clevedon, S.W. England was used for gaining such samples (Fig. 2). This limestone has an
age of 365-350 Ma (Green 1992). Dry density and effective porosity are 2.575 to 2.657 (g/cm$^3$) and 1.74 to 5.29 % respectively. Dip and dip direction of the fold limb sampled are 10° to 40° and N160° to N190° respectively. Joints on the fold limb were measured within an 1m radius from the sampled blocks; the dominant joints are cross joints (55% of total numbers of measured joints) and strike joints (28% of total number) (Fig. 3a). Oriented blocks were carefully taken from the limb of this fold and from them hoop samples were cored, as closely as possible to each other (Fig. 3a). The results are reported here with reference to the plane of the limb of the fold; see Fig. 3b.

A total of 13 samples (outer diameter 94 mm, inner diameter 43 mm, and thickness 47 mm) was taken. Seven samples were dried over P$_2$O$_5$ until no further change in their weight occurred, this being an alternative method for drying to avoid any release of stored strain energy during oven drying. The other

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**Fig. 1** Diagram showing the procedures of a hoop tension test on a set of oriented samples.

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**Fig. 2** Location and geology map of sampling site for Carboniferous limestone, at Clevedon.

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**Fig. 3** Relationship between orientation of sample and natural joint sets in the fold.
six samples were saturated with distilled water under vacuum. The result, plotted in Fig. 4, shows that much more energy was required to create tensile failure in the 0° and 90° directions, which are parallel to the cross and strike joints, respectively. If the tensile failure on the plane of 0° and 90° were controlled by jointing, the work done should be the lowest at 0° and 90°. Contrary to this, less energy is required to create tensile fracture in the directions 45° and 135° to the reference direction, where parallel joints seldom occurred on the fold. Therefore it is thought that the result was not due to the presence of the joints which bounded the sampling blocks: but what does it say about the possible stresses residual in the fold?

The other way to analyse this result is to consider the influence of the release of residual stored strain energy on tensile fracture during the hoop test. It has been well known that residual stored strain energy is much more easily released in the direction perpendicular to the joint orientation but is still locked in the direction parallel to joint orientation (Atkinson 1987). This locked energy will be released by further coring, cutting or during failure (Engelder 1993).

Thus the result shown in Fig. 4 can be possibly interpreted as follows: (i) since jointing commenced within the fold, residual stored strain energy has been released preferentially in the direction perpendicular to the major joints (0° and 90°); (ii) locked-in energy which had been preserved in other directions (e.g. 45° & 135°) was released during the hoop test making it much easier to create tensile fracture in these directions, viz 45° and 135°. Thus the direction in which the stored strain energy may be presumed to be least, required the greatest work to cause failure. The influence of water on the tensile fracture is also shown on the result but does not change these conclusions.

4 TEST ON THE CARNMENELLIS GRANITE

Carnmenellis granite, near the U.K. Hot Dry Rock Geothermal Energy Project Site was chosen as a location for samples because much information relevant to a study of stored strain energy is available, most especially measurements of in-situ stress magnitude and direction (Parker 1989). Samples were taken from the Carnsee quarry, which is 2.5 km from the Hot Dry Rock Test Site (Fig. 5). This granite has an age of 290-275 Ma and was emplaced during the latter part of the Hercynian Orogeny. A complex tectonic history followed; (i) lode and dyke emplacement associated with NE-NW compression, (ii) dextral wrench faulting along NW-SE trends, (iii) NNW-SSE compression during Alpine Orogeny (<70 Ma) (Edmonds et al 1975), and emergence to form part of the land mass of the U.K. from about the mid-Miocene (14 Ma).

The modal composition of the granite is 30% quartz, 30% alkali feldspar, 20% plagioclase, 10% muscovite, 6% biotite, tourmaline and andalusite as accessory minerals (Exley et al 1964). Grain size is 2 to 5 mm or less, with the feldspar megacrysts up to 10 to 15 mm in length. Feldspar megacrysts do not show oriented alignment in this sampling area although they do have a good alignment parallel to the cleaving way in other areas of the Carnmenellis granite. Dry density and effective porosity are 2.635 to 2.652 (g/cm³), 0.163 to 0.586 % respectively.

Major joint sets in the quarry are two subvertical, striking N65° to N245° and N165° to N345° and one horizontal. It has been well known that the quarrymen used the term quartering way, cleaving way, and tough way in the order of easy splitting (Hill and McAlister 1906). These three planes have been correlated with the three major joint sets;
Fig. 5 Location and geology map of sampling site for Carmmenellis granite, based on Parker (1989).

 quartzing way to the horizontal joint set, cleaving way to the N165° to N345° joint set, and tough way to the N65° to N45° joint set.

Oriented cores were taken from four adjacent holes at the bottom of the quarry by vertical in-situ coring. Sliced samples were marked with 5 different orientations measured clockwise from the true north; (i) 40° (parallel to $\sigma_3$), (ii) 65° (parallel to N65° to N245° joint), (iii) 100° (parallel to the direction of plane of maximum shear stress [$\sigma_1-(90°-\phi)/2$]) which in this case = 130°-20°, (iv) 130° (parallel to $\sigma_1$), and (v) 165° (parallel to the N165° to N345° joint).

4.1 P-wave velocity measurement

Six sliced solid cores were dried over $P_2O_5$ and their P-wave velocity measured with 15° intervals. Four of six samples (3-4, 3-6, 3-9, 4-6) showed a similar trend (Fig. 6) with maximum velocity parallel to 130° ($\sigma_1$ direction) and minimum velocity parallel to around the N45° to N60° ($\sigma_3$ direction or tough way direction). This result agrees well with other results of the relationship between in-situ stress and P-wave velocity (Plumb et al. 1984, Engelder and Geiser 1980). Sample no. 3-15 seems to have a similar but flatter trend. The trend of sample no. 4-1 differs and this might be because it was the sample nearest to the floor of the quarry (2cm beneath the existing floor) and could have been disturbed by previous quarry blasting.

Fig. 6 P-wave velocity of Carmmenellis granite as a function of direction of wave propagation relative to true north ($\theta = 0°$).
4.2 Brazilian tests

A total of 31 samples for Brazilian testing were prepared from the inner core of hoop samples. Two different diameters were used, 50.8 mm and 54.5 mm. The thickness of samples was varied from sample to sample but the diameter to thickness ratio was kept according to the ISRM Suggested Methods (Brown 1981). 18 samples were dried over $\text{P}_2\text{O}_5$ and 13 samples were saturated with distilled water under vacuum. The tests were conducted according to the ISRM Suggested Methods except for inserting soft wood between the loading platens and the sample instead of using a loading jig with appropriate radius of curvature.

The result, plotted in terms of tensile strength (MPa) does not show any particular differences in the value of stress at failure between 50.8 mm diameter samples and the 54.5 mm diameter samples. The average values from each group of oriented samples are plotted in (Fig. 7). The orientation of maximum tensile strength, i.e. $65^\circ$ is parallel to the tough way plane and orientation of lowest P-wave velocity, all of which agrees well. But the orientation of minimum Brazilian tensile strength is not as closely related to either $\sigma_1$ ($130^\circ$) or $\sigma_3$ ($40^\circ$), or the N165° to N345° joint set. The reason for this is not so clear but is suspected to reflect the combined consequences of relaxation from all the previous geological histories. Generally, dry and saturated sample show similar trends.

4.3 Hoop tests

A total of 36 samples (21 dried and 15 saturated) were prepared with two different sizes of inner diameter (57.7 mm, 61.1 mm) and the same outer diameter (120 mm). Thickness varied from sample to sample within the range of 27 mm to 59 mm. All these variations of dimension were tested to find whether there is any effect of sample size for a given grain size. The results show no particular effect of sample size. The mechanical work done to failure, divided by the total volume of the sample, is calculated and its average value for each oriented group is plotted in Fig. 8.

The average values show trends similar to those seen in the P-wave velocity and Brazilian tensile strength: i.e. the average value of work is required to
fail in tension is at a maximum parallel to the direction of tough way plane (N65°). The results from both saturated and dry specimens clearly show the average value of work to be at a maximum with failure in the N65° Tough way direction, agreeing with both P-wave velocity results and the results from Brazilian tests. The various minimum are less clearly related to any particular direction: perhaps they reflect the effects of previous geological history in the same way as do the Brazilian tests. Of particular interest is the higher work required to fail samples that had been saturated: if the conclusions from the samples from Clevelon can be taken as a guide, then the directions within a sample which can most easily lose their stored strain energy are likely to be the directions requiring the most work to fail. From this it can be suggested that saturating samples causes them to release stored strain energy in preferred directions.

5 CONCLUSION

The tensile strength of rock appears to vary with the direction of paleo and active in-situ stress. One interpretation of the results indicates that a sample which has lost its stored strain energy requires more work to fail it than one which has not.

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The Hoek-Brown $m_i$ constant in triaxial tests on some Italian rocks
La constante $m_i$ de Hoek-Brown en essais triaxiaux pour quelques roches italiennes

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ABSTRACT: The paper highlights the importance of executing triaxial tests for engineering-geological problems concerning rock material. 110 tests were carried out on rock cores drilled in nine different rocks in Italy. The Hoek-Brown $m_i$ constant was found to be the most significant parameter for describing the failure behavior of rock material. The scattering of $m_i$ values suggests that its evaluation on the basis of lithologic description alone is not advisable. Even when the uniaxial compressive strength of a rock is uniform, the $m_i$ constant and shear parameters can show great differences.

RÉSUMÉ : Cette étude met en évidence l'importance de l'exécution des essais triaxiaux dans les questions d'ingénierie-géologique concernant le matériaux rocheux. 110 essais furent conduits sur des noyaux rocheux excavés dans neuf roches différentes en Italie. La constante $m_i$ de Hoek-Brown résulte comme le paramètre le plus significatif afin de décrire le comportement à la rupture du matériaux rocheux. L'écartement des valeurs de $m_i$ suggère qu'une évaluation de la constante $m_i$ sur la base de la seule description lithologique n'est pas recommandable. Même quand la résistance à la compression monoaxiale d'une roche est uniforme, la constante $m_i$ et les paramètres de coupe peuvent montrer des différences importantes.

1 INTRODUCTION

When trying to solve engineering-geological problems concerning rock masses, the geomechanical characterization of rock material becomes an essential feature because, along with discontinuity characteristics, it determines the behavior of rock mass.

Of course it would be meaningless to try to classify failure criteria proposed by different Authors according to their credits, but it is also true that the Hoek & Brown failure criterion is the most frequently used in all of most practical engineering-geological applications.

As is known, both when it is applied to rock material and when it is applied to a rock mass, the Hoek-Brown criterion implies, among others, the measurement of the $m_i$ constant referred to intact rock.

For intact rock, the original Hoek-Brown failure criterion (1980) may be written in the following form:

$$\sigma_1 = \sigma_3 + Co(m_i \sigma_3 / Co)^{0.5} \quad (1)$$

whereas the modified criterion for jointed rock masses (Hoek et al., 1992) can be expressed in the form:

$$\sigma_1 = \sigma_3 + Co(m_i \sigma_3 / Co)^{0.5} \quad (2)$$

By using different equations, both versions enable the envelope curve to be calculated on the $\sigma$-$\tau$ plane and the values of instantaneous friction angle $\phi_i$ and of instantaneous cohesive strength $c_i$ to be calculated for a given value of effective normal stress $\sigma_n$.

The $m_i$ constant is measured by processing triaxial test data.

The paper deals with the results of 110 tests executed on rock cores drilled in nine different rock types in Italy. These tests were part of a wider survey that also included ultrasonic speed measurements, point-load strength tests, petrographic analyses and the determination of the main physical characteristics of rock material.

2 EXAMINED LITHOTYPES

The tests were carried out on rock cores drilled in Italy (Fig. 1) in different lithostratigraphic units: a) anhydrite and gypsum (San Giovanni Bianco Formation, Upper Carnico), b) calcareous marlstone (Pizzo di Brenno Fm., Maastrichtian), c) gypsum (Gessoso-Solfifera Fm., Messinian), d) sandstone (Macigno Fm., Oligocene), e) clayey marlstone
(Rocchetta Fm., Upper Oligocene-Lower Miocene), f) calcilutite (Maiolica Fm., Upper Titonian-Lower Aptian), g) cherry marly limestone (Rosso ad Aptic Fm., Kimmeridgian-Upper Titonian) and h) stratified chert (Radiolariti Fm., Upper Albian-Lower Kimmeridgian).

Fig. 1 Samples location (see text for explanation)

3 TEST EQUIPMENT AND SPECIMEN PREPARATION

All tests were made by using an oleodynamic rigid press with 1,500 kN load capacity for axial load application. Load was detected by a pressure transducer positioned on the hydraulic line. In triaxial tests, every specimen was wrapped into a rubber latex membrane and was put into a Hoek type triaxial cell.

Lateral confining pressure was applied by a 70 MPa hydraulic pump fitted with a pneumatic pressure stabilizer capable of balancing the pressure variations induced by specimen deformation. Axial and diametrical deformations were detected by using 20 mm electric strain gages applied directly onto the side surface of every specimen. All test data were stored by using a data logger with 6 seconds scan delay and recorded on data files for their subsequent processing.

For every sample, four specimens were normally prepared, three of which would be submitted to triaxial compression and one to monoaxial compression. The specimens were obtained by coring with diamond drill bit 82 mm wide diameter boring cores and then cutting them by rectified saw according to parallel bases. Specimen diameter is 54 mm. Specimen height to diameter ratio is always between 2 and 2.5.

Specimen side surface was polished by using rubbing paper n. 400 and treated with chlorotene degreaser in order to improve strain gage sticking.

4 COMpressive Tests

Monoaxial compressive tests were carried out according to ASTM D3148-86 standard by progressively increasing load with a constant gradient until failure occurred.

Triaxial tests were conducted according to ASTM D2664-86 standard. Lateral confining pressure was increased up to test value and, simultaneously, axial load was isotropically increased in order to prevent any specimen deformation. The tests were carried out by increasing axial load with a constant gradient until failure occurred.

5 CALCULATION OF $m_1$ CONSTANT

For every sample (made up by at least four specimens), a diagram was made concerning principal stresses at failure and failure envelope was calculated by adopting the original procedure proposed by Hoek & Brown. This procedure enables calculated failure envelope reliability to be assessed and immediately compared with experimental data. When experimental data are excessively different from the calculated failure envelope, or else when the data comparison coefficient is below a threshold value - which, in the cases examined, was fixed to 0.65 on the basis of former experiences - the failure envelope is reappraised after eliminating the anomalous datum.

At the end of this procedure, the geomechanical parameters typical of the examined sample are obtained - i.e. $C_0$, $m_1$, $\sigma$ - and the failure envelope is determined on the $\sigma/\sigma$ plane. From the failure envelope instantaneous friction angle and cohesion values can be obtained for determined normal stress values.

6 TEST RESULTS

Table 1 synthetically shows compressive strength, $m$, constant, instantaneous friction angle and instantaneous cohesion values calculated for a normal stress on the failure plane equal to 1 MPa, besides fundamental static parameters of elasticity.

The $\sigma/\sigma$ diagrams shown in figures 2 to 10 show the failure envelopes for the nine cases examined.

As is shown in the table and in the figures above mentioned, sensitive differences were recorded among the samples of two lithotypes - i.e. calcareous
Table 1: Synthesis of test results

The marlstone of Piano di Brenno Formation and sandstone of Macigno Formation. In both cases, tests enabled three different sample groups to be defined, which had very different mechanical characteristics at failure. Such differences are not highlighted by monaxial compressive strength values. They are only partially shown by considering the parameters of elasticity, whereas they are evident only on the basis of the $m_1$ constant of the failure criterion adopted.

In particular, as regards the Macigno Formation, in the three lithological-technical groups detected, Co is always very close to 105 MPa, $E_{s50}$ is equal to about 33 GPa and $m_1$ values vary from 3.1 to 8.0 and to 25.4, that is, about ten times between the most scanty class and the class having the best characteristics.

The differences between instantaneous friction angle and cohesion values for a normal stress - equal to 1 MPa - on the failure plane were also evident. Actually, $\phi_1$ varies from 31° to 49° and to 65°, whereas $c_1$ varies respectively from 32.0 to 20.9 and to 11.3 MPa. These average values indicate that the shear resistance variation in the sandstone examined is not only a variation in its absolute value but, above all, a variation in mobilization modes. Actually, from the first to the third sample group, an increase in friction and a decrease in cohesion was recorded. Therefore, the samples of the first group show a greater shear resistance at low stress levels and a lesser shear resistance at high stress levels than other groups. As a result, the first group sample failure envelope curve is more accentuated and is expressed

![Graph showing Principal stresses at failure]

Fig. 2: Sandstone (Macigno Fm.) - Principal stresses at failure
by a low $m_1$ value.

The cause of these sensitive variations in the mechanical behaviour of the three sample groups - which on a macroscopic level appear to be substantially uniform - was found in the different average granulometric composition of the rock. The microscope revealed that the samples having a prevailing finer grain size - with a modal diameter lower than 0.5 mm - have lower $m_1$ values, whereas the samples having a prevailing coarser grain size - modal diameter > 1.0 mm - show higher $m_1$ values. That led the researchers to assume that, for the examined sandstone, the greater degree of mutual embedding between grains along the failure surface that characterizes the samples having a greater grain size and the progressive elision of their edges for growing normal stress onto the failure plane produce a failure envelope with a not so accentuated curve (and, therefore, high $m_1$ values).

Conversely, in finer grain samples, the cohesion value is greater because the cementation links between the grains that make up sandstone are more developed.

while $m_1$ values vary from 4.4 to 15.1 and to 47.7, that is, more than ten times from the most scanty class to the class having the best characteristics. The differences between instantaneous friction angle and cohesion values for a normal stress - equal to 1 MPa - on the failure plane were also evident. Actually, $\phi_1$ varies from 36° to 55° and to 66°, whereas $c_1$ varies respectively from 5.0 to 3.7 and to 2.4 MPa.

In this case, too, the shear resistance variation in the marl examined is not only a variation in its absolute value but, above all, a variation in mobilization modes. Actually, from the first to the third sample group, an increase in friction and a decrease in cohesion was recorded.

These variations in the mechanical behaviour of the three sample groups were found to be dependent on the different percentages of carbonates and detrital material present in the three groups. In this case, too, only microscope analysis could clarify a situation no evidence of which had been found on a macroscopic level.

For all other lithotypes, no sample groups having different mechanical behaviors were recognised.

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**Fig. 3**: Calcereous marlstone (Piano di Brenno Fm.) - Principal stresses at failure

As to the calcereous marlstone of Piano di Brenno Fm., in the three lithological-technical groups found, Co is always close to 22 MPa, the higher $E_{95}$ value is 2.3 times greater than the lowest value measured.

**Fig. 4**: Anhydrite (S.Giovanni Bianco Fm.) - Principal stresses at failure
Fig. 5: Gypsum (San giovanni Bianco Fm.) - Principal stresses at failure

Fig. 6: Gypsum (Gessoso-Solfifera Fm.) - Principal stresses at failure

Fig. 7: Calciulite (Maiolica Fm.) - Principal stresses at failure

Fig. 8: Cherty Marly limestone (Rosso ad Aptici Fm.) - Principal stresses at failure
7 CONSIDERATIONS

Test results enable the following considerations to be made:

1. The $m_i$ constant was found to be the most significant parameter in order to describe the failure behavior of rock material. In particular, within lithotypes having the same compressive strength, the $m_i$ constant enables significantly different mechanical behavior to be distinguished.

2. In a few cases, the $m_i$ values measured during tests can differ substantially from published average values. That can be true both of samples that were not homogeneous in their mechanical behaviors and of samples that were substantially homogeneous.

Therefore, in any application problem, an exhaustive mechanical characterization of rock material should include the determination of the $m_i$ constant and of the failure envelope besides deformability characteristics. In other terms, in order to know the failure behavior of a rock, it may not be enough either to ascribe a $m_i$ value on the basis of literature or, as it often happens, to limit laboratory research to a definition of monoaxial compressive strength alone (which is by far the most known and used parameter), but it is necessary to execute triaxial compressive strength tests.

SYMBOLS

$c_i$: instantaneous cohesive strength
$c_s$: cohesive strength for a normal stress of 1 MPa
$C_0$: uniaxial compressive strength
$E_{so}$: tangent elastic modulus at 50% of failure stress
$E_{so}^*$: secant elastic modulus at 50% of failure stress
$m_i$: failure envelope constant
$\mu$: Poisson ratio (secant)
$\sigma$: normal stress
$\sigma_1$: tensile strength
$\sigma_3$: major principal stress
$\sigma_2$: minor principal stress
$\tau$: shear strength
$\phi_i$: instantaneous friction angle
$\phi_1$: friction angle for a normal stress of 1 MPa

ACKNOWLEDGEMENTS

Many thanks to Prof. R. Pozzi for his critical reading of this paper.

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Microfabric of Guadalquivir ‘Blue Marls’ and its engineering geological significance

Microstructure des ‘marnes bleues’ du Guadalquivir et sa signification en géologie de l’ingénieur

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ABSTRACT: A detailed microfabric study was carried out on samples from the so-called Guadalquivir "Blue Marls" located in Southern Spain using the scanning electron microscope technique. The study has shown that the overall microfabric features of these overconsolidated "Marly Clays" is predominantly a "clay matrix" type. The platy clay minerals within this matrix are arranged in a different manner the most common one being a randomly oriented "aggregate" of clay mineral particles. Some coated silt and sand sized grains mainly organism tests were also present sinking in the dense matrix, where the clay particles were oriented around them. Various areas of local parallel orientation of clay minerals in a "Face to Face" and "Edge to Edge" interaction are also present in all the samples studied. However, a pronounced parallel orientation of clays along the bedding plane was not apparent here. In addition to this the influence of the "Wetting and Drying" process in the microfabric of this soil was assessed.

Résumé: Un détaillé d’une Microstructure a été réalisée sur des échantillons de ce qui apparaît comme les "Blue Marls" du Guadalquivir, au sud de l’Espagne, en utilisant la technique de microscope électronique à balayage. L’étude a démontré que l’ensemble des éléments de microstructure de ces argiles marls surconsolidées est fondamentalement une "argile matrice" parmi ces matières, les argiles minérales se présentent dans un ordre différent dont le plus courant est un "agrégat" orienté au hasard. Certains minéraux de la forme de silt and and couverts minéraux argileux principalement de fossiles sont présents dans la dense matrice où des plaquettes d’argil tournent autour de eux. Plusieurs zones orientées parallèlement vers les minéraux argileux, se retrouvent aussi, dans tous les échantillons examinés. Néanmoins, une orientation parallèle significative de l’argile, au plan de la sédimentation n’a pas été très apparente. En plus, l’influence du processus de "wetting and Drying", dans la microstructure a été étudiée.

1 INTRODUCTION

The Guadalquivir “Blue Marls” formation has been deposited in the Guadalquivir basin, (Southern Spain) in stable marine waters during middle to upper Miocene with out any significant eustatic changes (Percogon 1962). These sediments have been overconsolidated (Liquidity Index less than 0) as a result of great depths of overlying sediments in the past without being lithified. In hand samples the main feature of the unworned soil is that it is a fine grained clay with massive and compact texture being hard to stiiff and a blue and medium grey coloured, but whereas weathering has occurred it is mottled Orange and brownish gray as a result of mineral oxidation. Numerous inclusions (oxidized Pyrite) are present throughout the clay, and also some fissures and vertical joints are common. Occasional almost horizontal sand and silt filled partings probably representing bedding planes are also present.

These highly plastic and stiff clays are extensive deposits in the major part of Guadalquivir basin and generally exhibit frequent engineering problems associated to superficial landslides mostly reacted with road cuts and rail ways in the area. Expansion is common in this type of clays containing more than 40 % expansive clay minerals.

Oteo and Sola (1993) reviewed stability of slopes on this soil emphasizing the
weathering with the increase of fissuring and influencing the mechanical behavior of the sediment. The main purpose of this paper is to study the microfabric under the scanning electron microscope and, to explain how the microfabric changes as a result of weathering process common in the area. For this purpose samples from the Guadalquivir blue marl were obtained from Cordoba in a fresh and intact state in large parafined blocks. In addition to microfabric studies tests were carried out on the index properties, and mineralogical and chemical composition of the soils.

2. INDEX PROPERTIES

The properties studied include natural moisture content, atterberg limits, dry density, Grainsize distribution and carbonate content.

The tests were performed on undisturbed samples and average values of these are shown in Table 1.

Table 1. Average Properties of the blue marls

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural moisture content</td>
<td>25%</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>65%</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>35%</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>37%</td>
</tr>
<tr>
<td>Activity</td>
<td>0.74</td>
</tr>
<tr>
<td>Classification</td>
<td>CH</td>
</tr>
<tr>
<td>Dry Density (g/cm³)</td>
<td>1.67</td>
</tr>
<tr>
<td>CaCO₃</td>
<td>25%</td>
</tr>
<tr>
<td>Percentage of particles &lt;74μ</td>
<td>98%</td>
</tr>
<tr>
<td>Percentage of Particles &lt;2μ</td>
<td>53%</td>
</tr>
</tbody>
</table>

A detailed data on geotechnical properties of this soil is found in Otero and Sola (1993).

3. MINERALOGY:

The mineralogical composition was determined using X-ray diffraction method of powder of the total sample, and oriented aggregates of the less than 0.002 mm fraction for clay mineral analysis. The semiquantitative procedure of Biscegy (1965) was followed to give the proportion of minerals present. It was shown a similar mineralogical composition with abundance of phyllosilicates ranging from 67 to 72% of the total sample. Calcite being the second mineral present in this soil with an average value of 17%. Quartz ranging from 6 to 10%, Feldespar, Dolomite and Gypsum in small amount (less than 5%) were also present. The most common clay minerals present in the 0.002mm fraction were smectite (Ca+ Montmorillonite) and illite in quantities ranging from 44 to 50% for Smectite and 34 to 40 % for Illite. Another clay mineral present was Kaolinite in amounts ranging from 6 to 9%.

Under the scanning electron microscope the clay mineral particles has an irregular form and flaky morphology. The major part of Carbonate minerals is biogenic mostly represented by Nanofossiles (Cocolith). Small amount of fibrous minerals (sepiolite) are also observed in some of the samples.

4. MICROFABRIC

The microfabric study was performed using the Jeol scanning electron microscope. Observation has made on both vertical and horizontal surfaces. Samples were prepared following the technique describer by Barden and Sides (1971). After air drying samples were fractured to expose fresh surface and applied (50 to 80) alication of peeling using the adhesive paper, next they were coated with gold in the sputter for five minutes. All samples were prepared in a same manner to avoid possible microfabric disturbance.

Observation of microfabric was made firstly on intact samples to asses the overall feature of the sediment and secondly on samples treated by humectation and desiccation cycles.

Microfabric of undisturbed samples

As it might be expected from natural sedimentary soils the microfabric feature of the undisturbed samples of the Guadalquivir blue marl is heterogeneous and complex where it has been identified several distinctive types of microfabric associations. However in this paper is discussed the most representative and typical microfabric of the soils and only typical micrographs are presented.

The most dominant and remarkable overall microfabric feature observed in all undisturbed samples is basically the "clay matrix" type which consists on aggregates of clays and group of clay minerals, silt and sand grains and microorganism sinking in the clay particles. A typical micrograph of this microfabric viewed under low magnification
is illustrated in fig (1).

Fig. 1: Clay Matrix.

As it is evident from the micrograph the matrix is fine grained and fairly dense and non-oriented where the voids are small and evenly distributed, inter. and intra-assemblage pores were not identified. This dense structure with an abundance of small voids of less than 1μm diameter is consistent with the compacted texture, and low liquidity index (less than 0) of the sediment where the large primary pores common in marine clays were collapsed under the overburden pressure. The large and circular pore seen in the middle of the micrograph are probably opened through organism locomotion indicating the bioturbation process occurring in the sediment.

In a detailed observation of larger magnifications of the matrix fig (2) it has been shown numerous fine silt particles and microorganisms sinking with in the flaky clay minerals that are arranged mainly in a random manner.

Other microfabric elements relatively abundant of this soils are the typical "Honeycomb" structure of Terzaghi and the "Cardhouse" type, the later being less abundant. Representative micrographs of this microfabric are shown in fig (3 and 4).

Fig. 2: Detail of the dense matrix. Notice randomness of clay particles.

Fig. 3: Typical "Honeycomb" microfabric.
The granular areas mainly consist of silt and sand sized quartz and calcite crystals arranged in a grain-grain particle contact fig (6). And in some of this areas there appeared to be the connector (consisting of randomly oriented clay particles) type microfabric described by (Collins and Mc.Gown 1974) that serves as a bridge between the silt and sand grained particles fig (7).

There exist also several areas of parallel clay minerals orientation in a turbostratic manner along a possible natural shear plane fig (5) indicating the existence of microscopic discontinuities which can influence the macrostructure of the sediment. The main possible natural shear plans appear normally forming 45 to 60 angle to the bedding and are suggested to be tectonic origin, a process that affects the area where this formation is found (Betic region, Southern Spain).

Fig. 4: "Cardhouse" Microfabric. Edge to Face association of individual clay platelets

Fig. 5: Parallel orientation of particles along a plan.

Fig. 6: Grain - grain contact of silt particle

Fig. 7: Flocculated clay connector
The microrganism basically composed by Coccolithis are arranged in an open structure (fig 8) giving abundant intratess pores. Some of the intratess porositis that can significantly influence the basic properties of the sediments are infilled with cristals of calcite, pyrite and even gypsum, giving a high resistance to the sediment Fig (9).

Fig. 8: Arrangement of Coccolithis in the soil.

The abundance of clay microfabric type in a random arrangement shown in this overconsolidated soils similar to the primary fabric is suggested to be due to the high amount of silt particles and cementing agents (high carbonate content) preventing the normal realignment of particles in this type of soils and to the biomechanical process of organisms (biturbation).

Fig. 10: Parallel orientation of clay minerals around the microrganism

In fig (10) is shown how the associated clay minerals around the infilled organism are arranged forming a dense parallel orientation.

Microfabric of soils after wetting and drying cycles

One of the important aspects of this study was to assess the influence of wetting and drying processes on the microfabric of this samples.

In visu observations and low scale magnification of samples after 25 wetting and drying cycle in the laboratory indicates that there is a dramatical increase of fissuring in the treated samples. The samples were very brittle without cohesion after drying. The increment of fissuring in the upper part of this sediments (where weathering has occurred) has been reported by Carlos oteo P.Sola (1993) in field observations.
frequent indicating the breakdown of these arrangements and disruption of the original matrix with the drying and wetting processes and abundance of individual clay particles.

This is consistent with the observation of Barden and Sides (1971b) in Keuper Marl. Here it was abundant the coating of silt particles by individual clay minerals.

In terms of mineralogical composition there is no indication of any significant changes in clay mineralogy as a consequence of this treatment.

CONCLUSIONS

Guadalquivir Blue Marls consist of mainly Phyllosilicates minerals of about 67%, and Calcite 17%, Smectite and Illite being the most dominant clay minerals.

The main microfabric feature of this sediment is a randomly oriented dense clay matrix type. There observe also abundant flocculated microfabric elements typical of soils from marine origin.

The randomness of clays and preservation of original flocculated microfabric in this overconsolidated sediment, is attributed to the abundance of silt particles, cementation agents preventing the realignment of the clay minerals and to the bioturbation process.

The local orientation of particles along a parallel plan indicating a natural shear zone which heavily influences the mechanical behavior (shear strength) is suggested to be tectonic origin common in the area.

It is observed that wetting and drying processes (weathering) modify the microfabric of the Guadalquivir blue marls by breaking down the randomly oriented stable microfabric and forming unstable aggregates consisting of individual particles. And also it has shown that fissuring intensity is increased by this processes.

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The influence of high pressures on structural changes of components and geomechanical properties of natural silicate soil

L’influence des hautes pressions sur des changements de structure de quelques composants des sols silicatés

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ABSTRACT: In this paper the influence of high pressing pressures up to 12 GPa on changes in X-ray diffractograms of some of the most important natural soil components such as calcite, kaolinite, K-feldspat, Na-feldspat and hematite has been analyzed. It has been shown that on the surface of polycrystalline particles of some components under the influence of high pressures a destruction of the structure and a formation of an amorphous layer occurs. This further means that the newly formed amorphous particles, under certain conditions can behave as colloid systems that with varied properties influence changes in geomechanical soil properties.

RÉSUMÉ: Dans cet article l'influence des hautes pressions, jusqu'à 12 GPa, sur des changements de structure de quelques composants de sol très importants: calcite, kaolinite, de potassium et sodium, feldspat et hématite, est analysée. Il est évident, d'après l'analyse par la diffraction des rayons-X, que sur la surface des particules polycristallines des ces composants, sous l'effet des hautes pressions, perdent la structure cristalline et la couche amorphe se forme. Les particules ainsi formées, c'est-a-dire des couches amorphes, peuvent se comporter, sous des conditions naturelles comme systèmes colloïdaux et par ces propriétés infleur sur des caractéristiques géomécaniques de sol.

1 INTRODUCTION

Observations of the behavior of a series of materials under the influence of high pressures cannot overlook mechanochemical processes that are a result of the influence of these pressures. In connection with this, Butjagain (1971) examined common process laws initiated by mechanical influences on different materials under the influence of static and dynamic strain, where the final effect of this influence can be besides material destruction an energy accumulation, that is, material structure deformation including the electrical structure. The described effects during mechanochemical treatment of the material make mechanochemical reactions approach reactions in plasma. This is why Tisen, Majer and Heinike (1967), suggested the tribo plasma model given in Fig. 1, as the model of mechanochemical processes. In agreement with this model, energy freed during minimal friction or shock, due to a low material thermal conductivity brings about not only a local temperature rise and material sublimation, but a formation of such a material state that is in the form of a mixture of electrons sand ions, i.e. plasma. Analysis performed by Battaglia (1990), show that due to mechanochemical processes, on the surface of quartz a surface layer of amorphous is formed, that is detected by a decrease of the intensities of characteristic reflections X-ray diffraction maximums.

As the load through soil is passed from one structural soil component to another thorough a small surface contact, on these places high pressures exist, as is shown in Fig. 2. High pressures equally exist in places where small-grained fractions (clay) are jammed between two large grains. Based on this and also on results obtained by Susic (1990) it can be expected that
in natural soil in places where high pressures exist an amorphous layer is formed.

For this purpose an X-ray analysis of some of the most frequent natural soil components before and after the influence of high pressures has been performed.

2 EXPERIMENTS

Calcite, caolinite, K-feldspar, Na-feldspar and hematite have been used for the analysis, as some of the most frequent natural soil components.

Calcite and hematite were in the form of polycrystalline powders of laboratory purity. Caolinite, K-feldspar and Na-feldspar originated from natural deposits in Strumice (Macedonia), Prokuplje (Serbia) and Cellice (Check republic), respectively.

The material samples, that were in the form of a powder were first pressed under the pressure of 0.5 GPa, forming pellets of a cylindrical shape, 1 cm in diameter and 1 cm in height. Then, two such pellets were put in a device, as shown in Fig. 3, in which pressing up to 12 GPa was performed.

An X-ray analysis of the samples was performed before and after pressing, using an automatic X-ray powder diffractometer (Philips model PW-1710). An Cu anode with a fine focus was used, the length of radiation was 1.54060A and a proportional counter (Xe) was used. For measuring the angles of the diffraction maximums and their intensities the basic PW-1877 program was used.

3 ANALYSIS OF RESULTS

In dependence of the pressing pressure X-ray analysis diffractograms given in Figs. 4-8 for every investigated component were obtained.

Obtained calcite diffractograms, given in Fig. 4, show that the intensity of the characteristic (104) reflection starts to gently decrease at pressures higher than 5 GPa. During this, the lattice constant d=3.03Å practically does not change. As, during this, the other reflections become less pronounced this shows that in calcite with a pressure increase a partial destruction of the crystal lattice occurs and the transformation calcite amorphous.

An analysis of the intensities of caolinite characteristic (001) reflections, d=7.10Å, Fig. 5, show that a decrease of these values is highest at pressures up to 5 GPa, while at higher pressures it is less pronounced. On caolinite diffractograms, one can also see a loss of some pronounced reflections. This analysis shows that caolinite becomes amorphous under the influence of high pressures.

On diffractograms of K-feldspar pressed under pressures up to 5 GPa, Fig. 6, one can clearly see a decrease of the intensity of the characteristic (002) reflection, d=3.25Å, and also the loss of some clearly pronounced reflections and the appearance of a series of weakly pronounced reflections, that shows that a transformation of crystal K-feldspar amorphous K-feldspar takes place. At higher pressures these processes can also be seen, but they are less pronounced.

In Na-feldspar, Fig. 7, with an increase of
Fig. 4 Calcite X-ray in dependence of the pressure

Fig. 5 Caolinite X-ray in dependence of the pressure

Fig. 6 K-feldspat X-ray in dependence of the pressure

Fig. 7 Na-feldspat X-ray in dependence of the pressure

Fig. 8 Hematite X-ray in dependence of the pressure

Fig. 9 A diagram of the changes of characteristic reflections intensities depending on the pressing pressure for: calcite, caolinite, K-feldspat, Na-feldspat, hematite
pressure a constant decrease of the intensity of the characteristic (002) reflection, \( d=3.19 \text{A} \) occurs, that is until a more pronounced transformation crystal Na-feldspar amorphous Na-feldspar.

Only in hematite, Fig. 8, with an increase of pressure, no relevant decrease of the intensity of the characteristic (104) reflection, \( d=2.69 \text{A} \) occurs, so one can say that hematite is quite stable under the influence of pressures up to 12 GPa, and that a significant transformation hematite-amorphous hematite does not occur.

If the dependencies of intensities of some components characteristic reflections from the pressing pressure are compared, that is shown as a diagram in Fig. 9, one can see that feldspats, first K-feldspat and then Na-feldspat are very sensitive to the influence of high pressures and that under pressures of only a few GPa start to become amorphous.

In caolinite amorphization is of a lower intensity and is mostly done wholly under pressures lower than 5 GPa. Different from previous cases, for the amorphization of calcite pressures higher than 5 GPa are needed. Only hematite proved to be quite inert to the influence of high pressures.

<table>
<thead>
<tr>
<th>Component</th>
<th>Characteristic parameter</th>
<th>Pressure (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcite</td>
<td>I (cts)</td>
<td>906 900 471</td>
</tr>
<tr>
<td></td>
<td>d (A)</td>
<td>3.0366 3.0320 3.0315</td>
</tr>
<tr>
<td>Caolinite</td>
<td>I (cts)</td>
<td>380 246 216</td>
</tr>
<tr>
<td></td>
<td>d (A)</td>
<td>7.1439 7.1325 7.1296</td>
</tr>
<tr>
<td>K-feldspar</td>
<td>I (cts)</td>
<td>1806 497 420</td>
</tr>
<tr>
<td></td>
<td>d (A)</td>
<td>3.3429 3.3410 3.3367</td>
</tr>
<tr>
<td>Na-feldspar</td>
<td>I (cts)</td>
<td>1945 1384 740</td>
</tr>
<tr>
<td></td>
<td>d (A)</td>
<td>3.1936 3.1929 3.1897</td>
</tr>
<tr>
<td>Hematite</td>
<td>I (cts)</td>
<td>174 199 159</td>
</tr>
<tr>
<td></td>
<td>d (A)</td>
<td>2.6951 2.6916 2.6896</td>
</tr>
</tbody>
</table>

A further analysis of obtained results, given in Table 1, showed that in all investigated components, the intensity of characteristic reflections (I) decreases with a decrease of pressing pressure, while the lattice constant (d) is practically unchanged. This fact shows that in the components an inner destruction of the crystal lattice under the influence of high pressures had not occurred, but only an amorphization of the polycrystal components particle surfaces.

4 DISCUSSION

From the given analysis one can see that the place in natural soil where high pressures will occur depends on the structure and mineral composition of the soil. The geological time factor has an important role in the occurrences and processes that appear under the influence of high pressures. Experimental results obtained by Zulkov (1968) indicate the importance of the time factor in mechanical processes. Results show that the longer the temperature and time are, a smaller force is needed for the destruction of the materials crystal lattice. This means that contact pressures between particles that have a static character, for a long enough time span, cause a surface destruction of the crystal lattice (amorphization), as under the short-term influence of high pressures.

Fig. 10 Double electrical caolinite layer: a) in the case of negative ion adsorption, b) in the case of positive ion adsorption

Our results show that under the influence of high pressures a surface amorphous layer formed on particle contacts, which as a consequence has changes in surface boundary phase properties. This further means that, having in mind, the complex structure and basic characteristics of the investigated silicates, it can be considered that the destruction of natural soil mineral crystals
under the influence of high pressures, leads to the appearance of particles of colloid dimensions as parts of the destroyed structure. These particles, under the influence of underground water, that contains an electrolyte, adsorb certain ions and become homogeneously charged, that leads to a formation of an encircling double electrical layer that stabilizes the formed colloid systems, as is shown in Fig. 10 in the case of caolinite. This way, a decreased viscosity on the contact boundary of the two solid phases, under the influence of high pressures, creates a metastable terrain state, i.e. a stable structure of clay minerals created by flocculation, in literature better known as "the house of cards", Fig. 11a, passes due to the above stated processes in an unstable dispersative structure, Fig. 11b, with a tendency to settle and slide. That the zone in which particles change into a dispersive structure is of colloid dimensions (for example 20 \mu m) is proved by Skempton's (1985) investigations. For earth movement at a macrolevel, i.e. of a greater scale, it is absolutely necessary that the above mentioned processes are joined as a whole in space. The given analysis shows that earth movement is influenced by a series of factors, such as the nature, length of the pressure action, meteorological conditions, seismic processes etc.

Fig. 11 A model of the clay minerals microstructure: a) flocculate state, b) dispersal directed state

In the end, one can say that our results and discussion show that occurrences and processes that take place in particles of natural soil components for a long enough (geological) time span, that further influences changes in geomechanical properties of natural soil. Also, this consideration of fundamental occurrences presents a contribution to understanding why in some almost unexpected locations after a shorter or longer time period soil settling or sliding occurs.

5 CONCLUSION

An X-ray analysis of natural soil components powders before and after the action of high pressures shows a decrease of characteristic reflections intensities on diffractograms. However, the characteristic interspace distances have not changed, that shows that phase transformations have not occurred in the materials, but an amorphous layer has formed on the polycrystalline particle surface. This new-formed state, with its diverse physico-mechanical, chemical and other properties and characteristics influences a change in engineering properties of natural soil, due to which natural soil can start to move, i.e. settle or slide.

REFERENCES


Fissured effect of clay soil mass
Effet des fissures sur le comportement des sols argileux

X.W. Hu, Q.F. Li, Z.S. Zao & D.F. Kong
Chengdu Institute of Technology, People's Republic of China

ABSTRACT: Fissured effect of clay in Chengdu, Hefei and Nanjing are stated in this paper. It is studied that fissure controls mechanical property of soil mass according to fissured effect of angle and quantity. The results show this fissured effect of soil mass is evident.

RÉSUMÉ: L'action contrôlée sur les propriétés mécaniques de corps sols, d'angle incliné et de nombre de crevasses dans les argiles de Chengdu, Hefei et Nanjing en Chine a été étudiée. Cette étude montre macroscopiquement que l'effet de crevasse dans les argiles est important.

1 INTRODUCTION

Fissured clay is a soil that has a crack. It is widely distributed in many countries of Asia, Europe, America and Africa. There is fissured clay in Southwestern, middle-southern, eastern and northern of China, too. Analyzing situation of fissured clay study, we find out that fissure controlling strength of soil mass is not profoundly studied. Obviously, studying engineering geological property of fissured clay systematically, we can effectively use it to avoid building accident, develop theory of geology and satisfy needs of engineering construction. In this paper, studying fissured effect of soil mass through angle and quantity, we obtain conclusion that this effect is outstanding.

2 FISSURED EFFECT OF ANGLE

2.1 Fissured effect of angle under the same state and enclosing pressure.

In order to illustrate fissured effect of angle clearly, we must remove interferences of other factors which possibly affect mechanical property of sample. So undisturbed sample must be kept natural moisture content and fissured angle a (that is an angle of fissured side and plane of maximum -main stress C1). Samples contained a=30°, 40° and 50° are tested under enclosing pressure 0.3=0.2MPa. Curve a-c shown figure 1.

From fig.1 we obtain the following conclusions:

1. The curve a-c appear mini-summit shape (curved 2) and mini-gradual-steady shape (curve3), or shape between summit and gradual steady. That is showed no evident effect of angle to curve a-c under

![Graph](image-url)

Fig.1 Curve a-c of undisturbed sample with different angle (a=0.2MPa)
1. a=30°; 2. a=40°; 3. a=50°
0.3=0.2MPa.

2. Sme of strain increase obviously with angle adding under the same enclosing pressure 0.3. For example, when c=0.3=0.2MPa, the strain of simple with a=30° is 2.3 times over that of sample with a=30°, 1.5 times over that of a=40°.

3. Sme of strength decrease with angle adding under the same pressure between a=30° 50°. For example, the strength of sample with a=30° is 1.25 times over that of sample with a=40°, 1.56 times over that of a=50°.

4. Total size of strain before sample broken increase with angle adding under the same pressure, between a=30° 50°. For example, the strain of sample with a=30° is 2.73 times over that of a=40°, 3.75 times over that of a=50°.

According to above analysis, it is evident that fissured angle influences deformation and strength of clay. Not only fissured angle effects deformation and strength of sample, but also it influences distinctly parameter of shear strength C1(cohesive force) and Fm (angle of internal friction). According to testing results of undisturbed sample with 1 fissure, C1
and $\phi_u$ vary with fissured angle increasing. (see table 1)

<table>
<thead>
<tr>
<th>angle ($^\circ$)</th>
<th>$C_u$ (MPa)</th>
<th>$\phi_u$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0^\circ$</td>
<td>0.105</td>
<td>22.5</td>
</tr>
<tr>
<td>$30^\circ$</td>
<td>0.075</td>
<td>8.0</td>
</tr>
<tr>
<td>$45^\circ$</td>
<td>0.064</td>
<td>6.0</td>
</tr>
<tr>
<td>$50^\circ$</td>
<td>0.057</td>
<td>5.5</td>
</tr>
<tr>
<td>$55^\circ$</td>
<td>0.054</td>
<td>4.0</td>
</tr>
<tr>
<td>$60^\circ$</td>
<td>0.058</td>
<td>10.2</td>
</tr>
<tr>
<td>$65^\circ$</td>
<td>0.045</td>
<td>13.1</td>
</tr>
<tr>
<td>$70^\circ$</td>
<td>0.038</td>
<td>12.9</td>
</tr>
</tbody>
</table>

From the above testing results between $\gamma=0-70^\circ$, it is showed that $\phi_u$ is changed from large to tiny, then gradually increased. The minimum $\phi_u$ is related to about $\gamma=60^\circ$. This change is conform to ordinary law of More-Coulomb’s strength theory. However, $C_u$ basically decreases with angle adding. This result is possibly caused by testing condition. Because while the sample with steep angle fissure is made and installed, part of sample above fissure easily slip a little along the fissured plane, so cohesive force $C_u$ is decreased and lead to lose reality.

2.2 Fissured effect of angle under the same state and different enclosing pressure

It is showed fig. 2 that testing results of sample with fissured angle $\gamma=30^\circ$, $40^\circ$ and $50^\circ$ (keeping natural moisture content) under different enclosing pressure ($\sigma_3=0.3$ or $0.4$ MPa). Comparing fig. 1 with fig. 2(a), (b), we may see the following effects of enclosing pressure to fissured angle:

1. Shapes of curve $\sigma_1-\sigma_3$ are entirely of mini-steady type with pressure adding. It is no once more gradually changed from mini-summit type of small angle to mini-steady type of steep angle.
2. Strength decreases with the angle adding under smaller enclosing pressure. But with pressure adding, the strength of sample with $\gamma=30^\circ$ obviously increases. However, strength of sample with $\gamma=40^\circ$ and $50^\circ$ basically keep up original value and are no distinctly changing. For example, the strength of sample with $\gamma=30^\circ$ is 1.90 times over that of $\gamma=40^\circ$ , 2.12 times over that of $\gamma=60^\circ$ under $\sigma_3=0.3$ MPa. Similarly, strength of sample with $\gamma=30^\circ$ is 1.70 and 1.50 times over that of $\gamma=40^\circ$, $50^\circ$ under $\sigma_3=0.4$ MPa.
3. Curve $\sigma_1-\sigma_3$ of sample with $\gamma=40^\circ$ and $50^\circ$, not only shape is similar, but also distance between curve is to shorten with enclosing pressure adding.

Fig. 2 Curve $\sigma_1-\sigma_3$ of undisturbed sample with different angle
(a) $\sigma_3=0.3$ MPa; (b) $\sigma_3=0.4$ MPa.
1. $\gamma=50^\circ$; 2. $\gamma=40^\circ$; 3. $\gamma=30^\circ$

Fig. 3 Curve $\sigma_1-\sigma_3$ of saturated and undisturbed sample with different angle
(a) $\sigma_3=0.2$ MPa; (b) $\sigma_3=0.3$ MPa

2.3 Fissured effect of angle under different state and enclosing pressure

It is very evident that different moisture content
influence fissured effect of angle. Testing results of saturated and undisturbed sample under $c=0.2\text{MPa}$ and $0.3\text{MPa}$ are showed fig. 3.

Comparing fig. 3 with fig. 1 and fig. 2(a), we can see that fissured effect of angle still exists. Effect of saturated sample is more distinctly weakened than that of natural sample. Shapes of curve are affected basically of mini-steady type, and no difference(fig. 3(a)). Curves are closed to each other. No differences of strength are not evident because of fissured effect of angle (see fig. 2(a)). Fissured effect of angle with saturated sample is nearly gradual gap between $\phi=50^\circ - 60^\circ$.

According to the above results, fissured effect of angle to clay mechanical property is obvious. But different enclosing pressure and moisture content will give some different influences to this effect.

3 FISSURED EFFECT OF QUANTITY

Quantity of fissure will surely effect mechanical property of soil mass. Testing results of fissured effect of quantity are showed fig. 4, fig. 5 and fig. 6.

![Fig. 4 Curve $\sigma_1-\sigma_2$ of undisturbed sample with different fissured quantity ($c=0.2\text{MPa}$)](image)

It is showed that shapes of curve are similar and gradually steady type from fig. 4. When $c=0.3\text{MPa}$, the shapes are almost alike. Total size of strain with curve 1 is slightly greater than that of curve 2 before sample is broken. But strength of curve 1 is evidently larger than that of curve 2, that is 1.5 times over. This result shows that sample with one or no fissure is different, but deformation and broken process are similar. It is likely that gray-white clay of fissured plane is very thin, so yellow parent clay is very important effect in deformation and broken course. However, because of fissured existence, strength of curve 2 sample decreases greatly.

![Fig. 6 Curve $\sigma_1-\sigma_2$ of undisturbed sample with different fissured quantity and angle ($c=0.4\text{MPa}$)](image)

Fig. 5 and fig. 6 show results of sample with different fissured quantity under the same enclosing pressure $c=0.4\text{MPa}$. When $\phi=60^\circ$, shape of curve is basically alike and approaching. Deformation is different, but no evident. It is likely that fissured angle of sample is more steep. But because fissured angle of soil mass is different on the spot, it will make mechanical property of soil mass with different fissured quantity complicated. Fig. 6 shows increase of deformation and decrease of strength are obvious, when fissured quantity adds. It standards one species of complicated natural conditions. Evidently, this contains fissured effect of angle, and not only fissured effect of quantity.

Fissured effect of quantity to parameter of shear strength of clay is distinct, too. When fissured quantity adds, $C_u$ and $\phi_u$ will decrease (see table 2).

| Table 2 Parameter of shear strength of undisturbed sample with different fissured quantity |
|----------------------------------|-------|-------|-------|-------|
| quantity                         | no fissure | 1 fissure | 2 fissure | 3 fissure |
| $C_u$ (MPa)                     | 0.451   | 0.075   | 0.009   | 0.004   |
| $\psi_u$ ($^\circ$)             | 27.2    | 5.0     | 11.0    | 7.5     |

4 CONCLUSION

1. The existence of fissure makes mechanical property of soil mass evidently decreased. Results of unconfined (uniaxial) compressive experiment show that strength of disturbed sample is 1.9 ~ 2.4 times over that of undisturbed sample under the same moisture content and density. It is illustrated that fissure decreasing effect is greater than that of particle interlocking force each other on clay strength.
2. It's evident that fissured effect of angle with the same quantity fissure on mechanical property when \( \alpha = 30^\circ \sim 50^\circ \). Strength decreases with angle adding. But this effect is influenced by testing enclosing pressure and sample moisture content. When \( \alpha = 50^\circ \sim 70^\circ \), shear strength is consistent with the More–Coulomb's theory. Strength decreases the minimum when \( \alpha = 45^\circ \sim 55^\circ \). Parameter \( \phi' \) of shear strength is changed similarly. But this testing results show that parameter \( C' \) (cohesive force) is gradually decreased with angle adding.

3. The mechanical property of clay is decreased with fissured quantity adding. It is showed that differences of deformation and strength are not evident. It is likely related to steep fissured angle of sample.

REFERENCES
New techniques for determination of mineral composition of clay fraction in soils
Des nouvelles méthodes pour déterminer le contenu minéralogique de la fraction fine des sols

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Institute of the Earth's Crust, Russian Academy of Sciences, Irkutsk, Russia

ABSTRACT: New techniques, a "program complex" and an "express-method", have been developed in the laboratory, which allow to estimate percentages of minerals in the clay fraction (< 1 μm). The program complex method consists in grouping the studied samples according to their mineralogical composition determined qualitatively by X-ray diffraction, and subsequent analysing major element chemistry and computing percentages of clay minerals through the simplex procedure within each group. With the express method the mineral composition of the clay fraction is found by matching the samples to master groups synthetized in a reference table where the classification is made according to base exchange capacities of clay minerals, humus content and swelling.

RÉSUMÉ: Des nouvelles méthodes ont été développées au laboratoire pour déterminer la quantité des minéraux argileux dans la fraction fine (< 1 μm). La méthode appelée "un complex du program" consiste en groupement des échantillons d'après leur minéralogie identifiée d'une façon qualitative par l'analyse diffractométrique aux rayons X suivi par des analyses chimiques et computation des teneurs des minéraux argileux à l'intérieur du chaque groupe à l'aide de la procedure simplex. Avec la méthode "express" on évalue la composition minéralogique de la fraction fine en mariant des échantillons aux groupes étalons consignés dans un tableau de référence où la classification est faite d'après capacités d'échange de base des minéraux argileux, la teneur de humus et le gonflement.

1 INTRODUCTION

Examination of clay minerals, which are the main fabric-forming components of soils, is of major importance for comprehensive engineering geological assessment of soils, as is confirmed by substantial theoretical, methodological and practical work (Riashchenko, 1990). By now mineral composition of the clay fraction of soils has been determined qualitatively by X-ray diffraction phase analysis accompanied by electron microscopy of crystals. The main shortcoming of these methods is that they do not allow quantifying the mineral contents. The paper describes two new methods of calculating percentages of minerals in the clay fraction (<0.001 μm) which were elaborated and tested due to join efforts of Laboratory of Permafrost and Soils Studies of the Institute of the Earth's Crust (Irkutsk) and Department of Hydrogeology and Engineering Geology of the Technological University (Irkutsk).

2 THE PROGRAM COMPLEX METHOD

The first step of studies by the program complex method is to group the study specimens according to sets of clay minerals they contain established by qualitative X-ray diffraction. As the second step, SiO₂, Al₂O₃, Fe₂O₃, CaO, MgO, Na₂O, K₂O contents and LOI are determined.
within each group. Then a special simplex processing procedure is run by a program of computing maximum error on the basis of the Chebyshev's criterion. The program was written by O.A.Khalilova based on the algorithm by I.K.Karpov; L.A.Kozmin from the Institute of Volcanology (Petrovlovsk-Kamchatskiy) did the final tuning. The software provides solution by the modified simplex method with binary constraints and enables repeated computing with the same mineral composition matrix and constraints on the variables.

Two problems have been solved for the data obtained in the Institute of the Earth's Crust (Irkutsk).

The first study was of the clay fraction from 35 specimens of Quaternary sediments and clay gauge sampled in Severo-Muyskiy and Baikalskiy tunnels of the BAM railway. Mineral contents in the clay fraction were quantified and examined in their relation with plasticity and swelling of soils (Suturin et al., 1986). The X-ray diffraction yielded the following grouping: hydromica+kaolinite (14 samples), hydromica+clorite+kaolinite (8 samples), hydromica (4 samples), kaolinite (3 samples), montmorillonite (1 sample), chlorite+kaolinite (1 sample). All the groups contained quartz and feldspar. A matrix of fundamental compositions was created for each group as a special case of the general matrix and percentages of minerals were computed by the simplex method. The results are shown in Table 1. The table contains amounts of minerals computed on the basis of the general matrix. In (a) there are theoretical compositions quantified without preliminary sorting by X-ray data; (b) represents the percentages of actual minerals calculated by the within-group matrices with the grouping made due to X-ray diffraction.

Table 1. Computed percentages of minerals in the clay fraction of soils from the Severo-Muyskiy tunnel of the BAM railway

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Mineral composition (from X-ray data)</th>
<th>Contents of minerals (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>a</td>
</tr>
<tr>
<td></td>
<td></td>
<td>proluvial-deluvial sediments</td>
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<td>23</td>
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<td>250</td>
<td>K, Hm</td>
<td>M10, C15, K74, Qu3, Fs9</td>
</tr>
</tbody>
</table>

Abbreviations stand for names of minerals: Hm = hydromica, Cl = chlorite, K = kaolinite, M = montmorillonite, Qu = quartz, Fs = feldspar. Numbers next to symbols of minerals are their percentages.
Therefore, the clay minerals from the samples analysed are predominantly chlorite, hydromica, and kaolinite. The Quaternary pro- luminal-deluvial loam contains from 39 to 58% of kaolinite and from 20 to 49% of hydromicas in kaolinite+hydromica assemblages; 42% of kaolinite, 33% of chlorite, 20% of hydromica in hydromica+chlorite+ka- olinite assemblages; the quartz content is within 7 to 9%. The content of hydromica may increase up to 75%, the content of kaolinite therewith drops down to 4% and that of feldspar increases up to 20%. The clay gauge is also characteristically dominated by kaolinite (from 36 to 60%) and hydromica (from 36 to 63%).

The percentages of clay minerals were analysed in their relation to plasticity and swelling of soils. 3 to 24% clay of total sample (18 specimens) yields very low or ab- sent plasticity (plasticity index of 2-10%) and swelling index of 0.010 to 0.086 cm³/g. The low plasticity must be due to the fact that the clay fraction is dominated by kaolinite. For instance, an un- likely low plasticity of 3% was observed in a sample containing 20% of clay where the clay fraction contained 47.8% of kaolinite, 34.3% of chlorite, 19.4% of hydromica and 1.4% of feldspar. Another sample with 24% of clay, which contained 62% of kaolinite, also showed plastic- ity as low as 6% inspite of the presence of montmorillonite in a quantity of 33%.

Plasticity and swelling are known to increase linearly with growing amounts of the clay component in soils. The samples analysed did not show this regularity. Soils con- taining as little as 3 to 7% of clay do not have plastic properties but demonstrate swelling values of 0.023 to 0.086 cm³/g. In this case the very percentages of minerals within the clay fraction must play the major role. For instance, high- est swelling was observed in a non- plastic specimen with a clay fraction of 6.3% of total sample, which consisted of hydromica (44.7%) and kaolinite (26.8%) as the dominant clay minerals, and of some feldspar (22%) and quartz (6%). The small portion and quartz+feldspar+kaoo- linenite mineralogy of the clay fraction are responsible for the absence of plastic properties of the sample; substantial amount of hydromica, which might contain swellable layers caused high swel- ling. A cluster analysis validated the swelling versus hydromica content relationship with a correla- tion coefficient of 0.36; plasticity index showed a correlation with portion of clay fraction at a ratio of 0.37.

The second case was of 54 speci- mens of clay from Quaternary sedi- ments and from Carboniferous-Permian sandstones, belonging to a coal-bearing formation, sampled in the area of influence of the Middle-Yenisei reservoir. The specimens were sorted into 13 groups with different sets of min- erals based on qualitative X-ray diffraction data. Each group was examined separately to determine mineral composition, fabric and properties of the loess soils. The quantified mineralogy, particle size distribution, plasticity, swelling, water content, porosity are given in Table 2 for seven samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth</th>
<th>% clay minerals</th>
<th>% clay</th>
<th>Ip</th>
<th>E_sw</th>
<th>Sr</th>
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</table>

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A synthetic review of the results obtained allows the following inference. The domination of chlorite among clay components yields low plasticity, high cohesion, and the absence of swelling and subsidence deformations (sample N107 from a depth of 1 m); increase in hydromica content to 46% leads to swelling and increase in soil strength (sample N107 from a depth of 3 m). The presence of the montmorillonite+kaolinite assemblage endows the soils with compressibility properties, low soil strength (down to 0.05 MPa), high plasticity and very low swelling (lower than 1%); (Table 2). In this case the kaolinite, the content of which attains 47 to 62%, impedes swelling and facilitates intergrain contacts, while the influence of montmorillonite, the portion of which amounts to 34-45%, becomes suppressed. The montmorillonite+hydromica assemblage (sample N 5162 from a depth of 1 m), imparts swelling (5%) and some subsidence capacity to the soils when the minerals are in a ratio of 20% montmorillonite to 80% hydromica; at the same time, the assemblage of montmorillonite and kaolinite in the ratio of 34-45% to 47-62%, respectively, has very little influence on swelling and subsidence capacity of the soils studied. In both cases the portion of the clay components was 25-27%.

3 EXPRESS METHOD

The method enables to determine mineral composition of clay components of soil by matching their base exchange capacities to that in master groups of samples synthesized statistically into reference tables. The reference tables render classification of samples according to cation exchange capacities of clay components in relation to the respective mineral compositions. Examination of 520 specimens of clay components of different types of soils in south East Siberia and Mongolia provided a basis for compiling the reference tables and setting up two master groups of samples. The specimens examined were taken through the following treatment: each was divided into two pieces whereby the clay fraction of one set of samples was examined by qualitative X-ray diffractometry and their duplicates were used to measure base exchange capacity through nephelometry and calorimetry. Particles smaller than 0.001 μm were sorted out by elutriation; the nephelometry and calorimetry was performed with the use of methylene blue dye (Kulichsky, 1977).

The presence and contribution of montmorillonite (vermiculite) (three divisions with contents of less than 10%, from 10 to 50% and more than 50-80%) were inferred from data on swelling and content of humus which is remarkable for its high physical and chemical activity.

The first master group consists of Cretaceous clays sampled in the vicinity of Sainshand town in Mongolia which were found to contain up to 90% montmorillonite and to have a base exchange capacity of 38 to 82 mg-equ per 100 g dry weight. The other master group includes clay fractions of void-filling loam from Quaternary coarse-blocky sediments sampled around Silibino (extreme North East Russia). These have main mineralogy bound to hydromica and chlorite and base exchange capacities ranging within 6 to 19 mg-equ.

The percentages of minerals in the clay fraction were inferred by matching the samples to the master groups based on the following assumptions: (a) an exchange capacity of 30-35 mg-equ at swelling of 4% and humus content of lower than 1% evidences of hydromica+chlorite mineralogy.
with some admixture of kaolinite; montmorillonite must be absent or very minor; (b) an exchange capacity in a range of 35 to 80 mg-eq/m at swelling higher than 4% and humus content lower than 1% is indicative of the presence of montmorillonite (or vermiculite) to the extent of 40-50% and higher; minor hydromica, kaolinite, chlorite may also be present; (c) a high exchange capacity of 35 to 80 mg-eq/m cannot be accounted for by mere predominance of montmorillonite (or vermiculite) at no swelling and humus content higher than 4% as such a fraction may be likewise composed of hydromica-chlorite with minor kaolinite and the high physico-chemical activity of the clay components may be due to the admixture of humus.

Grouping the clay components, less montmorillonite, according to their cation exchange capacities yielded the following classes: from 4 to 21 mg-eq/m (82% of all samples which belong to this class), from 22 to 32 mg-eq/m (50% of the samples), from 33 to 43 mg-eq/m (32% of the samples), from 44 to 57 mg-eq/m (44% of the samples), from 58 to 123 mg-eq/m (26% of the samples).

| Sample | Depth m | Base exchange capacity, mg-eq/m | Humus content, % | Effective swelling, % | Main mineralogy*
<table>
<thead>
<tr>
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<tbody>
<tr>
<td>1C</td>
<td>22.0</td>
<td>44</td>
<td>0.44</td>
<td>18.0</td>
<td>M, K</td>
</tr>
<tr>
<td>1B</td>
<td>02.0</td>
<td>55</td>
<td>1.70</td>
<td>02.1</td>
<td>Hm, K</td>
</tr>
<tr>
<td>4AP</td>
<td>15.0</td>
<td>52</td>
<td>0.41</td>
<td>00.1</td>
<td>Hm, Cl</td>
</tr>
<tr>
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<td>02.5</td>
<td>68</td>
<td>0.46</td>
<td>14.0</td>
<td>M, M+Hm, K</td>
</tr>
<tr>
<td>9548</td>
<td>08.0</td>
<td>68</td>
<td>2.64</td>
<td>08.0</td>
<td>M, Cl, Hm, K</td>
</tr>
<tr>
<td>228S</td>
<td>03.9</td>
<td>37</td>
<td>1.25</td>
<td>-</td>
<td>Cl, Hm</td>
</tr>
<tr>
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<td>10.0</td>
<td>12</td>
<td>0.27</td>
<td>-</td>
<td>Cl, Hm</td>
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<td>0.52</td>
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<tr>
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<td>-</td>
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<tr>
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<td>02.0</td>
<td>82</td>
<td>--</td>
<td>05.0</td>
<td>N</td>
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* the main mineralogy from a control test by qualitative X-ray diffraction

abbreviations of mineral names are the same as in Table 1

So, a sample which would fall into this or that class, can be judged to contain montmorillonite with a probability of 18% for the first class and of 74% for the 5th class. The method provides semi-quantitative results. Note that the results should be tested by X-ray examination of individual samples belonging to different classes of cation exchange capacity (Table 3).

CONCLUSIONS

The newly elaborated ways to quantify the amounts of clay minerals (< 0.001 μm) have advantages over the currently used qualitative methods of mineral composition determination as these are certain percentages of minerals rather than their mere presence or absence which are responsible for physico-chemical properties, strength and deformational capacity of soils. The program complex method allows comprehensive assessment of soils due to statistical processing of the available regional engineering geological data. The express method solves the problem of semi-quantitative studies of mineralogy of clay components of soils which provide a basis for classification of soils according to their properties in relation with mineralogical composition of their clay fraction. The express method can be of use for appraisal of building, ceramic and other materials. The methods are remarkable for their low cost and are of good economic efficiency when suitably combined.
REFERENCES


Dynamic instability of soils and rocks
Instabilité dynamique des sols et roches

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ABSTRACT: Dynamic instability means soil stiffness and strength degradation, and in wide sense - the increase of soil failure probability under dynamic loading in comparison with the static conditions. The mechanism of dynamic instability is a universal one for all soils and rocks - it is based on their ability to accumulate from cycle to cycle a certain part of undissipated energy and so is linked with the hysteretic properties of soils and rocks as microheterogeneous materials. Using thermodynamic equations the specific peculiarities of (a) dynamic dilatancy of cohesionless soils, (b) quasi-thixotropy of clay soils, (c) dilatant and thixotropic effects in low-cohesive soils and, (d) fatigue of rocks as the dynamic instability phenomena are analyzed.

RESUMÉ: Sous l’instabilité dynamique des sols on comprend la diminution de leur résistance en conditions des charges dynamiques par rapport aux charges statiques. En sens plus large c’est l’augmentation de la probabilité de destruction de sol sous la charge dynamique. Le mécanisme de l’instabilité dynamique est universelle pour tous les sols et il consiste en leur capacité d’accumuler de cycle en cycle une certaine partie de l’énergie non diffusée ce que étroitement lié avec leurs propriétés d’hystérésis comme des matériaux microhétérogènes. En utilisant des équations thermodynamiques des singularités spécifiques de manifestations de l’instabilité dynamique étaient montrés sous forme: (a) de dilatantie dynamique des sols pulvéralents, (b) de quasi thixotropie des sols argileux, (c) des effets dilatanto-thixotropiques en sols faiblement liés, et enfin, (d) de la fatigue des roches cristallines.

I GENERAL CONSIDERATION

Dynamic instability means soil stiffness and strength degradation, but in wide sense - the increase of soil failure probability under dynamic load in comparison with a static one. The only principal difference between dynamic load and a static one is the effect that in every next cycle of loading it is superimposed on already existing in material stress field and causes stress concentration in its most "weak" microvolumes. Proceeding on this concept dynamic load must be changing quicker than the stresses, caused in soil by this load, are dissipating. Not any cyclic or repeated loading can be considered as a dynamic one. Dynamic instability of any soil and rock is based on one unique mechanism - their ability to accumulate from cycle to cycle a certain part of undissipated energy. So, the accumulation of internal energy of system and, consequently, alteration of the ratio between external force and strength of structural bonds take place during dynamic loading. This ability of soils and rocks is linked with their hysteretic properties as microheterogeneous materials. This additional amount of internal energy appearing during dynamic loading, is concentrated within the most "weak" microvolumes of material, so called stress concentrators. And this is also
common for all soils and rocks. But the form in which internal energy is being accumulated is different for the soils and rocks with different type of structural bonds.

So, the general mechanism of dynamic instability of soils and rocks has an energy nature. And its general energy criterium is a universal one: the failure of structural bonds in the vicinity of stress concentrators begins on condition that the amount of the additional accumulated internal energy $\Delta W$ (in any form) reaches the value of their activation energy $\Delta W \geq E_{act}$.

All typical responses of soils to dynamic loading could be classified as presented in the upper part of fig.1. But these are only phenomena that could be usually observed directly as a result of dynamic loading. And in the lower part of the same figure five basic mechanisms of dynamic instability of soils and rocks are presented. The only characteristic one in rocks is fatigue but it has some peculiarities in different rocks. And there are more - 4 variants for soils. Generally speaking "dilatancy" means alteration of soil porosity during its shearing and it may be positive some volume increase and decrease of density are observed, but it also may be a negative one - which means compaction of soil and can result both in the increase of strength in dry or non-saturated sands and in liquefaction of loose saturated sands. So if we observe any changes in soil volume during shearing, we should speak about dilatancy.

2 DYNAMIC DILATANCY OF SANDS

Coarse-grained soils such as gravel could be considered to be dynamically stable in the absence of sufficient quantity of disperse filling. Dynamic response of sand can result in: 1) compaction (negative dilatancy) of loose sand with any degree of saturation; 2) liquefaction of saturated sands (loose and with medium density); 3) positive dilatancy in comparatively dense low-saturated sands resulting in their density decrease and consequent strength degradation; 4) some softening of dense saturated sands resulting from strain accumulation, not followed by liquefaction.

Using thermodynamic approach, we can analyze transformations of energy in the soil under dynamic loading. A quantity of energy transmitted to the soil in every cycle ($E_d$) is spent for elastic strains ($E_e$), to change distances among the particles (or porosity) and also to overcome friction among them ($\Delta W$). This friction results in heating of soil and part of energy is emitted in the form of heat ($\Delta U$). So, the energy balance equation for the arbitrary $k$-cycle of dynamic loading for elemental volume of sand could be written as following:

$$E_{d_{k}} = E_{e_{k}} + \Delta W_{k} + \Delta U_{k}$$

(2.1)

Particles become mobile on condition that friction along the elemental interparticle surface is overcome in $m$-cycle. Their movement is followed by the alteration of potential energy of the system $\Delta P_j$. And the energy balance equation for $m+1$-cycle:

$$E_{d_{(m+1)}} = E_{e_{(m+1)}} + E_{act} + \Delta W_{m+1} + \Delta U_{m+1} + \Delta P_{m+1}$$

(2.2)

and in every next $j$-cycle:

$$E_{d_{j}} = E_{e_{j}} + E_{act} + \Delta W_{j} + \Delta U_{j} + \Delta P_{j}$$

(2.3)

Taking into consideration very small value of specific surface in sands its increment resulting from fracture of interparticle bonds can be neglected. We can also neglect heat increments because its accumulation in porous sand from cycle to cycle is practically impossible.

It is obvious that additional internal energy $\Delta W_j$ can be accumulated only in the form of kinetic energy of oscillating particles:

$$\Delta W_j = \sum_{j} \Delta K_j = \Delta K_{ij}$$

(2.4)

Elastic strains can be neglected as interparticle contacts disappear, and then:

$$E_{d_{j}} = E_{act} + \Delta K_{ij} + \Delta P_{j}$$

(2.5)

For compaction potential energy decrease means the equivalent increase of kinetic energy:

$$-\Delta P_{j} = \Delta K_{pj}; \ E_{d_{j}} = E_{act} + \Delta K_{j} + \Delta K_{pj}$$

(2.5a)

So, as a result of volume decrease part of potential energy was transformed into kinetic one. And after the dynamic loading (i.e. $E_d = 0$):

$$-E_{act} = \Delta K_{pj}$$

(2.5b)

activation energy and, hence, the strength of new structure will be higher than initial one for the value of $\Delta K_{pj}$. For decomposition part of kinetic energy is, on the contrary, spent to increase a potential one:

$$E_{d_{j}} = E_{act} + \Delta K_{j} - \Delta K_{pj}$$

(2.5c)
TYPICAL SOILS AND ROCKS RESPONSE TO DYNAMIC LOADING

Fatigue failure  Dynamic compaction  Strength degradation

Partial strength degradation  Liquefaction (full strength degradation)

MECHANISMS OF DYNAMIC INSTABILITY OF SOILS AND ROCKS

Fatigue  Thixotropy  Quasi-thixotropy  Combined dilatant and thixotropic effects  Dynamic dilatancy

Fig. 1 Dynamic instability of soils and rocks: classification diagram
and after dynamic loading
\[-E_{\text{act}} = \Delta K_j - \Delta K_{\gamma j}\] 
(2.5d)
activation energy and strength of created soil structure will decrease in comparison with initial value for \(-\Delta K_{\gamma j}\). And so during the dynamic loading of cohesionless soil a part of energy is accumulated in the form of kinetic energy of its particles and is spent for their relative displacement that means the alteration of soil porosity \(n\).

The behaviour of soil is determined by grain-size composition, relative density, intensity of loading and moisture content. These factors control the value of activation barrier because the latter influences the coefficient of interparticle friction, and grain-size composition and density determine the interparticle contact area. And so, the probability of negative dynamic dilatancy increases with increasing moisture content and initial porosity over critical value. The influence of grain-size composition is not so distinct, because effects of mass, shape and roughness of grains must be taken into consideration. The major role must belong to the size of particles which controls weight and interparticle area. All these patterns of behaviour can be observed for the same soil, depending on its moisture content: at low moisture content soil performs positive dilatancy and decrease of strength, then gradual increase of moisture leads to some compaction and strength increase, and finally - over critical water content compaction is followed by liquefaction of soil.

So, the specific mechanism of dynamic dilatancy in cohesionless soils is transformation of a part of transmitted energy into kinetic energy of oscillating particles and its general criterium can be written in this form: \(\Delta K = E_{\text{act}}\), where \(\Delta K = \sum \Delta K_i\). In other words activation energy of structural bonds in cohesionless soils is a kinetic one. The generation of excess pore pressure is not a cause but a result of negative dynamic dilatancy, and liquefaction - is just a form of its appearance.

3 COHESIVE SOILS

The next specific mechanism of dynamic instability is thixotropy and, especially, quasi-thixotropy of cohesive soils. Thixotropy means full or partial failure of structural bonds in soil under dynamic loading, followed by their spontaneous recovery, water content, density and temperature of the system being constant. This phenomenon is observed in soils where coagulative interparticle contacts prevail.

A very important feature of thixotropic system is its full recovery - strength regains up to exactly its initial level in any number of cycles. This pattern of behaviour is specific in colloidal systems and clayey pastes. But natural undisturbed cohesive soils, as a rule, do not perform this important property and their regained strength is either less than initial one or exceeds it. And so it is more correctly to speak about quasi-thixotropy of such soils, taking into consideration that their dynamic response is based mostly on thixotropic processes, complicated, however, by some specific peculiarities. Three different cases of dynamic behaviour of cohesive soils are summarized in fig. 2.

3.1 Thixotropic system

For arbitrary \(k\)-cycle of dynamic loading energy balance equation (assuming \(\Delta U = 0\)):
\[E_{s_k} = E_{s_k} + \Delta W_k\] 
(3.1)
It is well-known that in cohesive saturated soils activation barrier (fig. 3) is controlled by the critical thickness of water film at the coagulative contact between the interacting particles - I mean distance range conforming the secondary potential minimum. Thus, if \(F_p\) - a force that must be applied to one of the particles to increase the distance between them for \(dr\) and to change forces balance in favour of pushing them aside, then the condition for failure of coagulative contact can be presented as:
\[\sum \Delta W_k \geq F_p dr = E_{\text{act}}\] 
(3.2)
In any cycle of loading after this condition was reached the disturbance of soil structure is continuing, and energy, being transmitted to any its elemental volume is spent: a) to hold up the achieved level of disturbance, b) for elastic strains and c) to increase the internal energy.

This excessive energy is accumulated in form of kinetic energy of oscillating particles (\(\Delta W\)). A well-known phenomenon of avalanche-like
Fig. 2  Strength regain in: 1 - thixotropic material, 2-3 - quasi-thixotropic natural clay soils.

Fig. 3  Integral energy of interaction between two small particles in the water versus distance (after Deryagin-Landau-Verway-Overbeek theory 1957).
failure of structural bonds in thixotropic system after a certain duration of loading, is connected with the decrease of energy expenditures to hold up the achieved level of disturbance because of the increase of surface energy $\Delta S$. This effect may be expressed in term of ($E_{\text{act}} - \Delta S$). One can imagine it as an increase in mobility of oscillating particles in any elemental volume of soil after some of bonds among them have failed.

Then, the energy balance equation for any j-cycle after condition (3.2) is reached:

$$E_{\text{ac}j} = E_{\text{e}j} + E_{\text{act}} - \Sigma \Delta S_j + \Sigma \Delta W_j$$  
(3.3)

So, the internal energy, being accumulated in kinetic form, is transformed to the extent of structure disturbance into a surface one, which has a considerable value for clay systems and provides the possibility of subsequent recovery of structural bonds with the same strength because if dynamic loading is stopped ($E_{\text{d}} = 0$), total surface energy equals activation energy of new structure:

$$\Sigma \Delta S_i = E_{\text{act}}$$  
(3.4)

This is the necessary condition of thixotropy (to simplify the discussion, activation energy is considered as a some average parameter for any elemental volume of soil). Thus the activation energy of structural bonds in cohesive soils is also accumulated in kinetic form and then is transformed into a surface one.

3.2 Quasi-thixotropic system

Case 2. Now, we have hardening which has thixotropic nature (fig. 2, curve 3). This is the most complicated and interesting case. It is known that in high-concentrated dispersed systems (natural clayey soil for example) particles are fixed at the distances which do not exactly coincide with the secondary potential minimum on the total interaction energy curve (fig. 3, point C) and the points, meeting their interaction energy values, are displaced to one or another side from it (points $A_{1,2,...}, B_{1,2,...}$). So, the increment of internal energy $\Delta W_j$ in every cycle of loading means the increase of particle mobility and probability of their interaction with maximum energy during the recovery period (it means that points, meeting their new interaction-energy would be located closer to the energy optimum after the regain). Thus we have the increase of the molecular interaction energy for the $\Delta E_{\text{eq}}$ value.

This process is going with the absorption of the equivalent quantity of kinetic energy:

$$E_{\text{eq}j} = E_{\text{e}j} + E_{\text{act}} - \Sigma \Delta S_j + \Sigma (\Delta W_j - \Delta E_{\text{eq}j})$$  
(3.5)

and after the loading ($E_{\text{d}} = 0$)

$$E_{\text{act}} = \Sigma \Delta S_i + \Sigma \Delta E_{\text{eq}i}$$  
(3.6)

And + $\Sigma \Delta E_{\text{eq}i}$ term expresses the effect of strength increase over initial level during thixotropic regain.

Thus, the excessive internal energy in natural cohesive soils, accumulated in kinetic form, is spent not only to create new surfaces but also to alterate particle interactive forces towards their optimum. Moreover, we have experimentally stated that if dynamic loading is intensive enough the destruction of microaggregates takes place. In the energy balance equation this process must be represented by the appearance of some additional quantity of surface energy $\Delta S_{\text{eq}}$. Hence, there appeared to be more particles, forming new soil structure during regain period in comparison with initial state. So, the final strength of the soil would be higher than the initial one because of the increase in the quantity of structural bonds in unit volume of soil - it is expressed by eq. (3.6a):

$$E_{\text{act}} = \Sigma (\Delta S_i + \Delta S_{\text{eq}i}) + \Sigma \Delta E_{\text{eq}i}$$  
(3.6a)

Finally, more complicated case: quasi-thixotropic regain of natural clayey soil with noticeable heterogeneity of structure: the diameter of pores in the same soil can vary by 3-10 times. It means that undisturbed soil contains specific microvolumes that are characterized by some excessive (in comparison to the average for a given soil level) amount of surface energy, stored in potential form ($\Delta S_p$). And this amount decreases the activation energy of these microvolumes, and of the soil itself: $E_{\text{act}} - \Delta S_p = E'_{\text{act}}$.

After the failure of structure during dynamic loading and the subsequent regain, a new structure would be more homogeneous because of the lack of unstable bonds with the low level of activation energy $E'_{\text{act}}$.

Thus quasi-thixotropic soil hardening over the initial strength level is based on 3 effects:
1) effect of increase of particle energy interaction in the secondary potential minimum (approaching the optimum);
2) disappearance of structural bonds with relatively low activation energy (thermodynamically it means the decrease of entropy); 3) increase in the quantity of interparticle contacts in unit volume.

Case 3 (fig. 2 curve 2) is typical for the soils in which structural bonds other than far coagulative ones prevail. In these soils part of the released molecular energy is either spent to change the distance between interacting particles, that means their transmission from the primary potential minimum to secondary one, or - for dense low-saturated clay soils - is even emitted in acoustic form (ΔE_ac):

\[ E_{e_j} = E_{e1} + E_{ac} + \sum \Delta W_j - \sum \Delta S_j + \sum \Delta E_{m,j} + \sum \Delta E_c \]  

Then if \( E_{ac} = 0 \)

\[ E_{ac} = \sum \Delta S_j - \sum (\Delta E_{m,j} + \Delta E_{c,j}) \]  

(3.7a)

The last term in eq. (3.7a) expresses the decrease of regained strength in comparison with initial level, since part of it was already irreversibly spent for the destruction of bonds with high activation energy. All this means ordering of the system towards its entropy decrease. Considering \( \Delta E_{ac} \) and \( \Delta E_{c} \) to be equivalent as denoting some part of particle interaction energy, the condition of quasi-thixotropic process in soil can be represented by:

\[ E_{ac} = \sum \Delta S_j + \sum \Delta E_{m,j} \]  

(3.8)

from which it is evident that quasi-thixotropy is caused exclusively by energy heterogeneity of undisturbed structure of natural cohesive soils and can be observed only when subjected to dynamic loading for the first time. And this effect has been really observed experimentally.

4 COMBINED DILATANT AND THIXOTROPIC EFFECTS IN LOW-COHESIVE SOILS

It has been found in late 1960-s that thixotropic properties could be observed to some extent in the soils with only 1.5-2% admixture of clay particles. These soils could conform both sands, silts and sometimes loams. So, there is obviously a group of soils intermediate according to their properties between cohesive and cohesionless ones which should be discussed separately as low-cohesive soils. Phenomenology of their dynamic instability has some very specific peculiarities.

1. The typical soils from this group - silty sands and sandy silts demonstrate very rapid strength degradation under dynamic load, following by their liquefaction if saturated and transformation into a liquid with very low viscosity. Because of this peculiarities they were called "quick sands" in 1930-s.

Strength regain after dynamic loading takes place against a background of poor compaction and is followed by moisture content alteration. This process progresses very slowly because of low clay particle content, from one hand, and very low permeability of soil, which prevents its compaction, from another. Total duration of regen period varies from several hours to several days, and the regained strength usually exceeds the initial one (mostly, because of some compaction).

3. Dynamic response of low-cohesive soils is strongly dependent on their moisture content - and they can perform both positive and negative dilatancy, followed by liquefaction a critical degree of saturation.

4. Low-cohesive soils can demonstrate extreme sensitivity to the vibrations with certain, very narrow frequency ranges in the interval 15-45 Hz, varying with moisture content and grain-size composition. This effect most probably has a resonant nature - we consider it to be an internal superharmonic resonance in the interconnected system of oscillating soil particles (Voznesensky et al. 1994). Thus the dynamic response of low-cohesive soil is a combination of dynamic dilatancy and thixotropic phenomena. Energy balance (if \( \Delta U = 0 \)) also can be at first described as:

\[ E_{e_k} = E_{e_k} + \Delta W_k \]  

(4.1)

It is obvious that in this system, where coagulative and friction bonds prevail, energy also could accumulat only in kinetic form of oscillating particles. And the condition at which the failure of structural bonds can begin must be represented as a combination of such conditions for cohesive and cohesionless soils. Then in any j-cycle of dynamic loading after this condition has been reached, the amount of energy
transmitted to elemental volume of soil is spent:
a) to support the achieved level of structure
disturbance ($E_{v0}$); b) for elastic strains of
undisturbed bonds ($E_e$) and
c) to increase internal energy of the system ($\Delta W$) - in kinetic form. The latter is, in turn, spent
to increase surface energy of soil ($\Delta S$) - to
destroy structural bonds - and to the subsequent
displacement of particles that means the
alteration of potential energy of the system ($\Delta P$):

$$E_{d_j} = E_{s_j} + E_{e_j} + \Delta W_j - \Delta S_j - \Delta P_j \quad (4.2)$$

Or, using the equations for cohesive and
cohesionless soils we come to the eq.(4.3) in
which terms typical both for thixotropic and
dilatant systems can be separated

$$E_{d_j} = E_{s_j} + (E_{e_j} - \Delta S_j) + (\Delta K_j - \Delta K_p) \quad (4.3)$$

Very high dynamic sensitivity of low-cohesive soil (i.e. low activation energy) is connected
with two aspects: with considerably less number
of coagulative contacts in unit volume in
comparison with cohesive soils from one hand,
and with lower inertia of particles in comparison
with clean sands. So such soils are initially more
"mobile" that would be obvious if we consider
activation energy not as an average value, but as
a sum of activation energies for $n$ structural
bonds and compare the obtained values for three
discussed groups of soils.

Positive dilatancy has poor probability to occur
in low-cohesive soils because of their cohesion, but liquefaction against a background
of some compaction is typical appearance of
their dynamic instability. Energy balance
equation for the last case may be written as:

$$E_{d_j} = E_{e_j} - \Delta S_j + \Delta K_j + \Delta K_p \quad (4.4)$$

At liquefaction activation energy is equivalent to
the total surface energy of the system and
so, after the regain new strength would exceed
the initial value because of increase in total
interparticle area resulting from compaction:

$$\Sigma \Delta S_j = E_{e_j} + \Delta K_p$$. It means that potential
energy emitted during the liquefaction of the soil
is, finally, transformed into a surface one and then
spent to establish new structural bonds.

Thus the energy criterion of dynamic
instability is the same for all soils and the
excessive internal energy is accumulated in
kinetic form of their particles in all cases. And
the differences among dilatant, quasi-thixotropic
and dilatantly-thixotropic processes are
concentrated in subsequent transformations of
different forms of energy. In other words, the
phenomenology of dynamic instability of soils
quite evidently refers to the energy of the
process.

5 FATIGUE OF ROCKS

Nowadays the regularities of fatigue phenomena
in rocks remain poorly understood. Because of
the lack of unique rock fatigue theory several
basic assumptions, developed for solid
homogeneous materials are usually used to
explain fatigue phenomena in rocks, but they
have only a limited application for such discrete
and heterogeneous media.

Our understanding of the rock fatigue
mechanism is based on the following concept.
The presence of primary heterogeneities of
various types (soft minerals' grains, large pores
of certain shape, etc.) causes the redistribution
of applied load and the appearance of the secondary
stress field which are concentrated on these
flaws. So, they appear to be "embryos" of fatigue
microcracks. This secondary stress field
is altered from cycle to cycle because of hysteretic
properties of rock. And in every next cycle those
arised microcracks are the most probable stress
concentrators. Once the tips of the propagating
microcracks are drawn nearer (observed as
microplastic strains accumulation) than some
critical distance, their local stress anomalies are
added and the total stress exceeds the rock
strength in this microvolume. This results in
microcracks coalescence and the crack, causing
rock failure, arises. So, dynamic loading creates
favourable conditions for stress concentration
within the most unstable microvolumes of rock,
resulting in its failure under significantly lower
loads in comparison with static loading. Rock
fatigue limit is obviously controlled by the
strength level of its major flaws which can vary
significantly.

Now let's analyze the dynamic load energy
transformations in any elemental volume of rock
considered as nonlinear-elastic material
containing structural bonds with different
strength. These bonds are primary
heterogeneities with different activation energy. It is well-known that in modern engineering practice fatigue limit of any material is determined from Veuler's curve. That obviously means that if maximum stress during the cycle doesn't exceed fatigue limit - no material strength alteration could occur. This assumption might not be always valid in application to such discreet heterogeneous media as rocks are. And maybe the most interesting question here is the possibility of fatigue phenomena if maximum dynamic stress is below fatigue limit - "low energy fatigue".

Within the modern framework of fatigue understanding it means elastic behaviour of material and its failure is impossible even if number of loading cycles becomes infinite. Non-linear elastic behaviour of rock means that we have closed hysteretic loop and so part of energy transmitted to any elemental volume of rock is spent for elastic strain and part - equal to the area of hysteretic loop - is absorbed in the rock. The size of the loop is proportional to the maximum stress. This energy can be accumulated only in the form of heat. Such heating was observed experimentally for rocks during dynamic loading. In accordance with kinetic theory of fatigue this heating is localized within microvolume of grain size in the vicinity of flaws - maximum stress concentrators. So their existence reduces dynamic stability of rock. When the value of accumulated energy reaches the activation energy of the closest flaw (let's call it first level activation energy) a submicrocrack arises:

$$\Sigma \Delta U_j = E_{\text{act}}^j$$  \hspace{1cm} (5.1)

As this submicrocrack is propagating part of energy is emitted in the form of elastic waves' energy resulting from elastic strains decrease with the arising of new surfaces ($\Delta E_{\text{rel}}$):

$$E_{\text{rel}} = E_{\text{rel}} + E_{\text{rel}} - \Delta E_{\text{rel}}$$  \hspace{1cm} (5.2)

According to the Griffith theory this energy released due to unloading of material in the vicinity of microcrack "flows" towards its tips and is spent there for further cracking. And in every next cycle:

$$E_{\text{rel}}(n+1) = E_{\text{rel}}(n) + \Delta U_{n+1} + \Delta E_{\text{rel}}$$  \hspace{1cm} (5.3)

Thus crack propagation process is energy profitable and the propagation would continue if:

$$\Delta E_{\text{rel}}(n+1) + \Sigma \Delta U_{n+1} \geq E_{\text{act}}^j$$  \hspace{1cm} (5.4)

but it would be arrested if $\Delta E_{\text{rel}} = 0$. It could happen when microcrack reaches the closest flaw with the higher level of activation energy. And eq.(5.3) means that the speed of microcrack propagation will decrease if $\Delta E_{\text{rel}} \sim 0$. It means that this flaw is "worn out" and relative hardening of rock takes place.

Further microcrack propagation or appearance of new ones can occur only after this higher activation energy level has been achieved. It is possible only with the increase of dynamic stress level. But if maximum dynamic stress is less than fatigue limit then the activation energy of this major part of structural bonds cannot be reached, and so rock failure is impossible.

Since intergrain bonds in rock can have various strength and so we could assume the existence of infinite number of activation energy levels. But the probability analysis shows that there exist no more than 3 distinguishable levels of activation energy for the rock without specific large inclusions: first (low), second (high or major) and critical (I mean those levels of activation energy that result in strength alterations which could be registered experimentally since the existence of no less than some critical quantity of discontinuities with low activation energy is necessary for macroscopically distinguishable strength reduction). The low energy rock fatigue can be really observed in the experiment even for monomineral rocks without cracks because the primary stress concentrators in them are pores. And the major role belongs not to the diameter but to the shape of pore, since in accordance with Kolosov-Ingliss solution the value of stresses concentrated on any cavity contour is controlled by its curvature radius. So, the highest stress anomalies must occur in the vicinity of the tips of slit-like pores. These ideas agree with our experimental data for different limestones. We obtained the decrease of static unconfined strength after only 10-20 cycles of dynamic loading below fatigue limit. Later relative hardening and, finally, stabilization (according to the condition (5.5) were observed. Microstructure analysis of these limestones revealed a considerable number of slit-like pores, concentrating stresses from 7 to 12 times exceeding the nominal applied stress value.
Thus, the internal energy is accumulated in rocks in the form of heat. If dynamic load doesn’t exceed fatigue limit, low energy fatigue can occur in some rocks that results in their partial strength degradation in first cycles of loading, but never leads to the failure. It is caused by the activation and following disappearance of a part of primary stress concentrators. High energy fatigue is characteristic for all rocks during dynamic loading over the fatigue limit.

6 SUMMARY

All soils and rocks can be classified according to the mechanisms of their dynamic instability since the differences in dynamic behaviour are distinctly pronounced in the regularities of accumulated energy transformations. Soils on one side and rocks - on the other differ principally in the forms of internal energy accumulation. In soils - it is kinetic energy of particles, and in rocks - heating. This accumulated energy can be further spent: 1) only to increase surface energy of the system (this is thixotropic material, but not a natural soil); 2) mainly to increase surface energy, followed by alterations in interaction energy (quasi-thixotropic soils - highly saturated clayey soils); 3) mainly to alter potential energy of soil particles (dilatant soils - pure sands); 4) to alter both surface and potential energy (dilatantly-thixotropic low-cohesive soils: silts, etc.); 5) mainly to acoustic energy emission (rocks and also dry lithified clays and loesses).

REFERENCES

Microstructure of some sediments of Mid-Pacific
Microstructure de quelques sédiments du fond du Pacifique

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Niu Zuomin  
Guangzhou Marine Geological Survey People's Republic of China

ABSTRACT: The samples in this study were collected from ocean floor about 5000m under the water by modern sampling instrument, with a sampling depth of 0--25cm. According to laboratory study these samples are Quaternary sediments made up of clays and organic remains. The sediments are characterized by high water content, high compressibility and the lowest strength. The samples were prepared for microstructural studies by freeze-drying method and were studied using a scanning electron microscope (SEM), yielding the following results: 1. Four types of microstructure were identified, comprising: 1) the skeletal type, 2) the skeletal-matrix type, 3) the honeycomb type and 4) the multiple honeycomb type which is a special type unknown so far in the literature; 2. All these four types are microstructures with high porosity and showing coagulation contact which were formed in the initial stage.

RESUMÉ: Les échantillons de cette étude ont été prélevés sur le fond océanique de 5000m environ sous l'eau, par un instrument d'échantillonnage moderne avec un profondeur de 0-25cm. Selon les études en laboratoire, ces échantillons appartiennent aux sédiments quaternaires comprenant les argiles et les débris organiques. Ils sont caractérisés par un teneur en eau élévé, une compressibilité élevée et une faible intensité. Les échantillons de l'étude microstructurale sont préparés sous la méthode de lyophilisation et étudiés sous un microscope électronique à balayage pour obtenir les résultats suivants: 1. Quatre types de microstructure ont été identifiés: 1) squelette, 2) squelette-matrice, 3) "nid d'abeilles", 4) "nid d'abeilles" en dimensions multiples qui est un type spécial inconnu dans la littérature présente; 2. Tous ces types de microstructure possèdent de forte porosité et se montrent un contact coagulatif formé en étape initial.

1 INTRODUCTION

With the exploitation of marine resources, a series of engineering geological problems have been posed, and an independent new branch of engineering geology began to appear during the past two decades. Most of the engineering geological researches have so far been done in the continental shelf regions. However, due to exploration and trial exploitation of solid mineral resources on the ocean floor by developed countries in recent years, a special problem has further been posed in marine engineering geology, namely the engineering geological study of oceanic basin, and one of its important scopes is the study of ocean floor sediments. In recent literature, engineering geological researches of ocean floor sediments are related to radioactive waste buried in the ocean floor as well as to marine survey of the polymetallic nodule regions on ocean floor. In late 1980s, China began to carry out marine geological survey in Mid-Pacific polymetallic nodule regions, and soon afterward performed engineering geological study on ocean floor sediments.

The samples used in this study were collected from 7°-15°N, 138°45'-153°30'W in Mid-Pacific where the water is mostly over 5000m in depth with the deepest of 5212m, only individual places being 4759m deep. Sampling was made by Guangzhou Bureau of
Marine Geological Investigation of Ministry of Geology and Mineral Resources using the ship "Haiyang IV" in a marine survey flight HY4-881. The instruments employed were the China made XD-1 box sampler and the US made 2450 and 2155 piston gravity samplers and 1890 boomerang corer. The samples were recovered from the superficial layer and the sampling depths range between 0 and 25 cm with a maximum of 50 cm.

The microstructure of the samples would definitely be influenced by the pressure and temperature conditions, although the sample quality was good in view of the advanced sampling equipment. Nevertheless, we still made a trial study on the microstructure of such kind of sediments.

2 GENERAL CHARACTERISTICS OF SEDIMENTS

According to N. M. Strakhov's hydrodynamic classification, the sampled region is an immense para-stationary region, i.e., hydrodynamically inactive region surrounded by subtropical gyre in Mid-Pacific Ocean. Literature shows that the normal rate for the deposition of deep sea sediments in Mid-Pacific Ocean is 0.2cm/10^3 a, and the depositional rate in this region obtained by No.2 NGIP on the basis of fossil organism zoning is 0.297cm/10^3 a.

According to the Ocean-going Team of No.2 NGIP, the sediments in this region include siliceous ooze (with a siliceous organism content greater than 30%), siliceous clay (with a siliceous organism content of 10-30%), calcium-bearing siliceous ooze (with a calcareous organism content of 5-10% in siliceous ooze) and deep sea clay formerly known as oceanic clay (with neither the organic remains nor the zeolite content exceeding 5%). Based on fossil record, these are all Quaternary sediments.

The samples have very high water content and good mouldability, with non-uniform colour distribution after air-dried. Siliceous ooze and clay are light and dark brown networks of spotted with white, probably resulting from the segregation of salt; calcium-bearing siliceous ooze is light brown, with dark brown network developed perpendicular cleavages and with worm holes 1 mm or less in diameter developed parallel to the bedding planes; deep sea clay is dark greyish brown indistinctly spotted with light colour. All samples show lumpy texture after air-dried.

In order to understand the common characteristics of the superficial sediments on ocean floor, comprehensive test has been made on soil samples using the currently widely employed routine methods. Some of the results are listed in tables 1, 2, 3 and 4.

Table 1. Grain (micro-aggregated composition) size composition of sediments

<table>
<thead>
<tr>
<th>sample name</th>
<th>sampling depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous ooze</td>
<td>0-25 cm</td>
</tr>
<tr>
<td>Siliceous clay</td>
<td>0-25 cm</td>
</tr>
<tr>
<td>Calcium-bearing</td>
<td>0-35 cm</td>
</tr>
<tr>
<td>Siliceous ooze</td>
<td>0-15 cm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Grain sizes</th>
<th>0.063-0.004 mm</th>
<th>0.004 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>21.72-78.218</td>
<td>77.89-19.57%</td>
<td></td>
</tr>
<tr>
<td>74.68-21.748</td>
<td>68.88-14.73%</td>
<td></td>
</tr>
<tr>
<td>90.66-82.19%</td>
<td>90.03-16.34%</td>
<td></td>
</tr>
<tr>
<td>62.91%</td>
<td>35.51%</td>
<td></td>
</tr>
<tr>
<td>87.69%</td>
<td>87.35%</td>
<td></td>
</tr>
<tr>
<td>33.50%</td>
<td>65.14%</td>
<td></td>
</tr>
<tr>
<td>90.57%</td>
<td>90.03%</td>
<td></td>
</tr>
</tbody>
</table>

Grain sizes are dominated by clay grain (<0.004 mm), which accounts for 35.5-78.68% and locally less than 20% (probably due to analytic error); whereas microaggregate sizes are largely 0.063-0.004 mm which accounts for more than 80%. This suggests that before applying the dispersing agent, the clay grains occur as anti-water micro-aggregates falling into the range of silt to fine sand grains.

Table 2. Clay mineral composition of sediments

<table>
<thead>
<tr>
<th>sample name</th>
<th>M</th>
<th>M*</th>
<th>I</th>
<th>K</th>
<th>Ch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous ooze</td>
<td>49.7%</td>
<td>32.8%</td>
<td>8.8%</td>
<td>8.7%</td>
<td></td>
</tr>
<tr>
<td>Siliceous clay</td>
<td>42.2%</td>
<td>41.7%</td>
<td>8.8%</td>
<td>7.3%</td>
<td></td>
</tr>
<tr>
<td>Calcium-bearing</td>
<td>54.4%</td>
<td>32.4%</td>
<td>4.5%</td>
<td>8.7%</td>
<td></td>
</tr>
<tr>
<td>Siliceous ooze</td>
<td>46.5%</td>
<td>25.4%</td>
<td>11.8%</td>
<td>16.3%</td>
<td></td>
</tr>
</tbody>
</table>

X-ray diffraction spectrum shows that the mixed layer mineral contents are rather high. However, the varieties of the mixed layer mineral cannot be identified since the samples were analyzed only after making the clay suspension into oriented section which was glycercine-saturated for 24 hours under 60°C, and no other special treatment had been made. Therefore, the mixed layer mineral was included into the same category with montmorillonite during
calculation. The above four types of sediments are all dominated by montmorillonite and mixed layer minerals with subordinate illite and minor amount of chlorite and kaolinite. This result is in conformity with the approximately 2% of K₂O content in the sediments as analyzed by No.2 NGIP and also with the Ocean Clay Mineral Distribution Map compiled by N. M. Strakhov.

It is evident from Table 3 that the total amount of exchangeable bases approximates to the exchange capacity, which conforms to the result of pH determination. Remarkably different from the soils from continent and continental shelf regions in exchangeable cation composition of the sediments, the samples have exchangeable Mg²⁺ and K⁺Na⁺ predominating. However, it is worth pointing out that Ca²⁺ still accounts for more than 30% and even 50% in the cation absorption aggregates. This is probably due to the high exchange energy of Ca²⁺, so that it is still difficult to be replaced by Na⁺ even though the sediments had long been in contact with the Na⁺-rich sea water. The reason for the clay grains to occur as anti-water micro-aggregates is on the one hand related to the high concentration of electrolyte in sea water and on the other hand related to the condensation of exchangeable Ca²⁺. The exchange capacity of the sediments is as high as 45.98-102.41 meq./100g soil, which is in conformity with its clay mineral composition. The specific surface area, 93.81-157.48 m²/g soil is slightly low, which may be related to the measuring method. The methyl alcohol molecules are as large as 25Å in diameter, so they can hardly be able to penetrate the supermicropores.

It is evident from the test result of the physico-mechanical properties of the sediments (Table 4) that the water contents are high, with some reaching 355.97%; the unit weights are low and not exceeding 1.3g/cm³; the void ratios are high, all being greater than 5 with individual values approximating to 10; both the liquid and plastic limits are high, with the former commonly greater than 100% and reaching as high as 190%; the compressibility is high, with the coefficient of compressibility attaining as high as 11.46 MPA⁻¹; the strength is low; and the undrained shear tests yield friction angles equaling to 0 and cohesion not exceeding 4 KPa.

### Table 3. Physicochemical properties and pH values of sediments

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>Exchangeable bases me/100g soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ca²⁺ Mg²⁺ K⁺Na⁺ Total</td>
</tr>
<tr>
<td>Siliceous ooze</td>
<td>19.24 7.87 27.58 54.69</td>
</tr>
<tr>
<td>Siliceous clay</td>
<td>21.76 2.72 15.45 39.93</td>
</tr>
<tr>
<td>Calcium-bearing</td>
<td>33.48 65.33 3.60 102.41</td>
</tr>
<tr>
<td>Siliceous ooze</td>
<td>16.41 18.24 16.10 50.75</td>
</tr>
</tbody>
</table>

|               | Exchange | Specific  |
|               | capacity | pH       |
|               | surface  | me/100g | area m²/gs. |
|               |          | soil     |             |
|               |          | 58.58    | 143.50      |
|               |          | 45.98    | 136.26      |
|               |          | 102.41   | 93.81      |
|               |          | 54.02    | 157.48      |
|               |          | 7.19     |             |
|               |          | 6.96     |             |
|               |          | 7.73     |             |
|               |          | 7.07     |             |

### Table 4. Physical and mechanical properties of sediments

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>W₀</th>
<th>Wₚ</th>
<th>P</th>
<th>a</th>
<th>Q</th>
<th>φ</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous ooze</td>
<td>162.89</td>
<td>1.11</td>
<td>2.55</td>
<td>5.40</td>
<td>355.97</td>
<td>1.27</td>
<td>2.75</td>
</tr>
<tr>
<td>Siliceous clay</td>
<td>204.15</td>
<td>1.20</td>
<td>2.56</td>
<td>5.80</td>
<td>314.50</td>
<td>1.26</td>
<td>2.75</td>
</tr>
<tr>
<td>Calcium-bearing</td>
<td>239.18</td>
<td>1.21</td>
<td>2.54</td>
<td>6.30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Siliceous ooze</td>
<td>245.56</td>
<td>1.21</td>
<td>2.79</td>
<td>7.20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3. Microstructural characteristics of sediments

3.1 Method of study

Since the water contents of the samples are very high, the normal dehydration procedure will cause volume shrinkage which results in the destruction of the primary structure. Therefore, the samples were prepared using low temperature and vacuum sublimation method (i.e. freeze drying). The samples were quickly frozen in a containing isopentane cooled by liquid nitrogen. So
that the pore water would form amorphous ice and no volume expansion would take place to influence the primary structure of the samples. The frozen samples were then put into a low temperature (-40°C) vacuum system (degree of vacuum 10⁻⁵ torr) to undergo sublimation for about 18 hours. After treatment, the surfaces of the samples were coated with gold film in a vacuum-coating equipment, and examined and photographed under a scanning electron microscope. Seventy-eight photographs of varied magnifications have been taken from 8 samples, and ten representative photographs are sorted out for description.

3.2 Characteristics of structural elements

The microstructural elements of the superficial sediments of the ocean floor bear an intimate relationship with the material components. According to No.2 MDIP, of the major material components of the sediments, clay material accounts for 50-90.7%, siliceous organic remains 5-42%, calcareous organic remains 0.1-15%, while other minerals all account for less than 18. It is obvious from the SEM photographs that two types of microstructural elements are recognizable, namely the microaggregates formed by microparticles of clay minerals and the largely siliceous (diatom, radiolaria etc.) and subordinately calcareous (coccolith and foraminifera) fragmental organic remains (photographs 1, 2, 3 and 4). As shown in the photographs, these organic remains and their fragments are of varied shapes and are highly variable in size. They can vary from several microns to several tens of microns, with individual exceeding a hundred micron in size.

Three types of clay particle micro-aggregates are discernible in the photographs:

1. Short columnar micro-aggregates consisting of smooth scaly particles. The outline is distinct and smooth, with a length of 1-2 micron and a width of 0.2 micron (photograph 9).

2. Irregular micro-aggregates 3-7 microns across, consisting of non-smooth scaly particles. The shape is irregular and the outline is not distinct, showing a crumpled surface and warped edge character (photographs 7, 8 and 9).

3. Globular micro-aggregates formed by crumpled scaly particles about 1 micron across and in face-edge contact with one another (photographs 5 and 6). Formed by clay particles infilling the foramina of foraminifera, the globes are regularly distributed at intervals of about 3-5 microns.

3.3 Microstructure types

Approximately four types of microstructure are formed by various organic remains and micro-aggregates. These are represented largely by skeletal type and honeycomb type.

1. Skeletal microstructure (photograph 3) is composed mainly of organic remains and fragments of different shapes and highly varied sizes. The structural elements are loosely and disorderly arranged, and the pores are of irregular geometry and circular holes are developed in the organic remains.

2. Skeletal-matrix type microstructure (photograph 4) is composed of remains of coccolith (ultramicro calcareous substance), diatomic fragments and spongey clay substances (matrix). The structural elements are loosely and disorderly arranged, and the pores are irregular in shape.

3. Honeycomb microstructure (photographs 7, 8 and 9) is developed in all of the four types of sediments mentioned above. The cell walls of the honeycomb are made up of micro-aggregates of clay particles, and the aggregates are in face-edge and edge-edge contact with one another. As a whole, no anisotropism is recognizable. The cells are equiaxial and are mostly 6-9 microns across with individual reaching 20 microns.

4. Multiple honeycomb microstructure (photograph 10) is an unique type composed of cells of varied sizes. The small ones are 0.1-1 microns, the medium ones are 2 microns while the large ones are as much as 6 microns in diameter. The cell walls are composed of micro-aggregates with crumpled surface, warped edge and blurred border. This type of microstructure is developed in two types of ooze sediments. It was probably formed through multiple coagulation and settlement during the slow sedimentation of colloid-like clay particles in sea water.

The overall characteristics of the honeycomb and multiple honeycomb microstructures are that their cell walls are thinner than those of the superficial sediments from continental shelf regions. This is probably due to the fact that the source material is not as richly supplied as in continental shelf regions, and in addition the concentration of clay suspension is lower.

All of the four types of microstructures mentioned above are highly porous initial structures. Sediments with such types of microstructures have especially high water content, and the structures were formed by coagulation. Consequently the sediments will have very low strength.
Photo. 1 Diatonic remains and fragments

Photo. 2 Radiolarian (siliceous) remains

Photo. 3 Skeletal microstructure formed by diatom and other organic fragments

Photo. 4 Skeletal-matrix microstructure formed by coccolith remains, diatonic fragments and clay microaggregates

Photo. 5 Globular microaggregates in foraminifera

Photo. 6 Enlargement of a globe in photograph 5
3.4 Origin of the microstructures

The microstructures of the above-mentioned ocean floor superficial sediments are of primary origin. In other words, they were formed during the deposition of the sediments. When coagulation took place in the clay suspension, the clay materials were deposited. The coagulation of microparticles in the clay suspension and the microstructures thus formed are controlled by many factors, including mainly the mineral composition of the microparticles, the dispersion degree, the geometry of microparticle and the medium conditions etc. The sediments described in this paper are dominated by clay materials and organic remains, and the formation of microstructures are closely related with the material component of the sediments. Sediment dominated by organic remains will form skeletal microstructure, while those dominated by clay materials will result in honeycomb type microstructure. The outline of clay grain microaggregates is closely related with the mineral composition of the clay: montmorillonite micro-aggregates have irregular outline and are characterized by crumpled surface and warped edge; while illite forms microaggregates with smooth surface and sharp and regular outline. As to medium conditions, it is well known that sea water is a medium with high electrolyte content (with the average salinity of 3.5%), which should be advantageous to the contraction of the diffuse layer of the clay particle and consequently to the coagulation of the particles. However, the high pH value (7.6) of the sea water on the contrary makes all the particles to be charged negatively and hence unable to coagulate because of the strong repulsion. As a result, the terrigenous particles had for a long period been in a suspension state in the vast ocean region until some favourable conditions occurred when they coagulated to form micro-aggregates and slowly settled with some organic remains which had been living in the ocean. In the course of settlement, these microaggregates coagulated further, and formed various types of microstructures when they were deposited on the ocean floor.
Owing to multiple coagulation, loose and soft sediments with the unique multiple honeycomb microstructure were finally formed.

4 CONCLUDING REMARKS

Microstructural study on the superficial sediments of the Pacific ocean floor has revealed the following four points:
1. The sediments display four types of microstructures, namely (1) skeletal; (2) skeletal-matrix; (3) honeycomb; and (4) multiple honeycomb, of which the multiple honeycomb microstructure is an unique type unknown so far in the literature.
2. The formation of the various types of microstructures is closely related with the material component, depositional conditions and medium characteristics of the sediments.
3. All these four types are initial microstructures with high porosity, showing coagulation contact.
4. The microstructures under study are the radical factor controlling the high water content, high compressibility and low strength of the ocean floor superficial sediments.

REFERENCES.


NOTE

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Strength of samples after their excavation
La résistance des échantillons après l’extraction

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Warsaw University, Poland

ABSTRACT: In result of strength testing of rock samples one can distinguish several stages of strength in each sample after its excavation from a rock massif: the stage of the minimum strength values (immediately after excavation, as a result of short-lasting elastic decompression and expansion with the same primary pore water content); the stage of hardening (after termination of decompression and expansion with decreasing of the primary pore water content and condensation and crystallization of the pore water solutions); the stage of the maximum strength values (indicating a balance of hardening and beginning of visible results of the recurrent atmospheric temperature, humidity and air pressure changes); the stage of the visible mechanical fatigue (the mechanical fatigue rate depends first of all on mineral and pore water solution contents, structures and textures of rocks and also on the rock occurrence conditions. The mechanical fatigue brings about total rock disintegration after some time).

RÉSUMÉ: En résultat des investigations de la résistance des échantillons des roches on peut distinguer les plusieurs étapes de la résistance des échantillons après les excavations du massif: l’état de la résistance minimum (après excavation comme le résultat de la décompression court-durable, élastique et extension avec la même humidité primaire, naturelle); l’état du durcissement (après la terminaison de la décompression et extension avec le décroissement contenu d’eau des pores primaire et la cristallisation des solutions d’eau des pores); l’état de la résistance maximum (indique l’équilibre du durcissement et du commencement de visible résultats des changements d’air atmosphérique, répétent les changements de la température et de l’humidité); l’état de la fatigue mécanique visible (la vitesse de la fatigue mécanique dépend avant tout des contenus: minéral et les solutions d’eau des pores, de la structure et de la texture des roches et aussi des conditions de leur existence. La fatigue mécanique donne en résultat la destruction total après quelque temps).

1. INTRODUCTION

Miners, stone-cutters and sculptors have known since a very long time that it is easier to form shaped stones and sculptures from the freshly excavated blocks from pits and quarries, and that the primary soft rocks turn to hard rocks after their excavation all the time, i.e. they harden. The hardening phenomenon of rocks after their excavation from a rock massif is a very complicated process, which is connected first of all with decompression, desiccation, substantial changes. The hardening phenomenon causes the following questions to be answered:

1. how long does hardening last,
2. what is the maximum value of the sample strength at the termination of hardening,
3. is the maximum value of the sample strength a constant rock parameter,
4. are atmospheric pressure changes able to cause the mechanical fatigue of rocks samples after termination of the hardening.

The model studies described below were executed to obtain answers to the above mentioned questions.

2. CHOICE OF THE ROCK FOR TESTING

The Upper - Cretaceous opoka marl from a quarry in Sulejów on the Vistula (Central Poland) has been chosen for the model studies of hardening (in the defined laboratory conditions), because:

1. its mineral content, structure and texture and its physical (especially mechanical) properties are well know (Kowalski 1961, 1966, 1975, 1992);
2. it is macroscopically (and the most part microscopically) homogeneous and isotropic, what is conducive to get strength values without great dispersion of results;
3. it is soft in its natural state and comparatively quickly hardening after its excavation, what does not require a greater pressure or a longer time of testing.
Fig. 1 Two result rows of strength and mechanical fatigue testing of rock samples: A - without latent weakness surfaces; B - with them. Samples: a - unloaded ones, b - loaded ones without shearing surfaces; with shearing surfaces: c - according to theoretical conical ones; d - partly concordant to latent weakness planes; e - distinctly concordant only to the tectonic joints measured in quarries; f - space situation of latent weakness surfaces in quarries; n - numbers of mechanical fatigue cycles (10, 25, 50, 100) and the second number after the dot - a phase in each mechanical fatigue cycle: 1 - phase of loading, 2 - phase of unloading. Strength of samples without and latent weakness surfaces: before mechanical fatigue - \( R_{\text{c,100}} \) and after \( n \) cycles of fatigue \( R_{\text{c,0ij}} \) - strength of samples with some latent weakness surfaces before mechanical fatigue - \( R_{\text{c,100n}} \) and after \( n \) cycles of fatigue - \( R_{\text{c,0in}} \). C - situation of each tested sample in quarry; N - North direction measured in quarry and designated on each sample top; SS - section of tectonic joint plane with a vertical quarry front; PP - strike of each tectonic joint.

3. METHOD OF STRENGTH TESTING

The uniaxial compression strength testing has been chosen to characterize strength of the tested marl. It is known that the strength values depend on the direction of the uniaxial compression (Kowalski 1961, 1966). For the tested marl in the air-dry state the mean strength value of the compressed samples in the direction perpendicular to bedding \( R_{\text{c,100}} = 14.6 \) MPa and in the direction parallel to bedding \( R_{\text{c,0in}} = 13.4 \) MPa and the coefficient of anisotropy of compressive strength \( A_c = \frac{R_{\text{c,100}}}{R_{\text{c,0in}}} = 0.92 \), although the tested marl is macroscopically homogeneous and isotropic. Similarly the coefficient of anisotropy of the compressive strength of the marl samples in the water-saturated state \( A_{\text{sw}} = \frac{R_{\text{c,100w}}}{R_{\text{c,0inw}}} = 0.88 \), when the mean values of the compressed samples strength: in the direction perpendicular to bedding \( R_{\text{c,100w}} = 8.7 \) MPa and in the direction parallel to bedding \( R_{\text{c,0inw}} = 7.7 \) MPa. So, the strict compliance with the condition to compress the tested samples always in the same (perpendicular to bedding) direction makes reduction of the strength testing results dispersion possible. Therefore, it was necessary to get (for strength testing) samples, which were orientated always perpendicular to their bedding as it is in the natural rock massif, where bedding is practically horizontal. So each rock monolith (about \( 0.4 \times 0.4 \times 0.3 \) m), which was cut out from the rock massif in the quarry, had the orientation marks on its surfaces, which surface is the top, which one the bottom with N-pointer. The samples (0.04 x 0.04 x 0.04 m) for strength testing were cut out from these monoliths with preservation of its spatial orientation.
in the natural rock massif.

There is another kind of the mechanical strength anisotropy in rocks (Kowalski, 1961). It is called the tectonic or crack strength anisotropy. This anisotropy is also a cause of a great dispersion of the strength testing results and their lowering. It was observed, that the lowest values of the uniaxial compression strength testing results of samples from the same rock monolith are always connected with clearly visible crack planes in the compressed and destroyed samples. These planes are vertical, perpendicular to bedding and parallel to the measured in quarry tectonic fissures and cracks (Fig. 1.C).

These planes disclose some latent surfaces or zones of some strength weakness, which have exist in rock massif; they are associated with tectonic planes of the massif. There are two principal kinds of crack surfaces to be observed, which arise in samples after their uniaxial compression strength testing (perpendicular to bedding): cone-shaped surfaces and crack planes (Fig. 2).

The strict exclusion from further considerations the lower compression strength values of the samples with the occurring after testing crack planes was necessary to escape a discussion about a influence of the tectonic rock anisotropy on general conclusions of this paper.

It is known, that the rock strength values depend on the changing moisture content in rocks. The coefficients of tested marl softening by uniaxial compression are: perpendicular to bedding: $M_{c,\perp} = \frac{R_{c,\perp}}{R_{c,\parallel}} = 8.7 \text{ MPa} : 14.6 \text{ MPa} = 0.6$ and parallel to bedding: $M_{c,\parallel} = \frac{R_{c,\parallel}}{R_{c,\perp}} = 7.7 \text{ MPa} : 13.4 \text{ MPa} = 0.57$. This great influence of moisture content in the marl on its strength had no effect on results of the executed studies. The strength values of the particular samples - $R_{c,\text{cut}}$ were determined in 10 succeeding series from the first series after 30 days and later in turn after 60, 90, 120, 150 180, 210, 240, 270 and 300 days (counting from the excavation day in the quarry). So, the moisture content in the tested samples was the same, in each series and did not influence the dispersion of the compression strength testing results in each separate series. Each tested sample after about 27 weeks (180 - 190 days) from its excavation day (from the quarry) was in the same air-dry state, i.e. its weight was constant. Hence, the observed diminution of uniaxial compression strength values after the samples hardening is associated with samples always in the same air-dry state. It made possible to avoid further considerations about influences of different moisture contents on the
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| $\Sigma R_{\text{est}}$ | - | 87.2 | 98.1 | 109.1 | 113.1 | 125.1 | 130.4 | 131.9 | 130.7 | 128.0 | 124.7 |
| $\bar{R}_{\text{est}}$ | (8.7) | 9.7 | 10.9 | 12.1 | 12.6 | 13.9 | 14.5 | 14.6 | 14.5 | 14.2 | 13.8 |
| $\Delta R_{\text{est}}$ | ± 1.9 ± 2.2 ± 2.4 ± 2.5 ± 2.8 ± 2.9 ± 2.9 ± 2.9 ± 2.8 ± 2.8 |
| $R_{\text{est max}}$ | (10.6) | 11.6 | 13.1 | 14.5 | 15.1 | 16.7 | 17.4 | 17.5 | 17.4 | 17.0 | 16.6 |
| $R_{\text{est min}}$ | (7.0) | 7.8 | 8.7 | 9.7 | 10.1 | 11.1 | 11.6 | 11.7 | 11.6 | 11.4 | 11.0 |
| $\bar{R}_{\text{est}}$ | (8.7) | 9.7 | 10.9 | 12.3 | 12.7 | 14.3 | 15.2 | 15.5 | 15.5 | 15.3 | 14.8 |
| $R_{\text{est}}$ | - | - | - | 10.7* | 11.5* | 12.5 | 13.1 | 13.0 | 13.3 | 12.9 | 12.7 |

Table 1. Compression strength values (perpendicular to bedding) of "opoka" marl samples $R_{\text{est}}$ MPa in time: t (from 30 to 300 days after excavation from the Sulejów quarry: their arithmetic means: $\bar{R}_{\text{est}}$ MPa; standard permissible deviations: $\Delta R_{\text{est}}$ MPa; calculated, standard, limit, strength values: maximum ones: $R_{\text{est max}}$ MPa and minimum ones: $R_{\text{est min}}$ MPa. Remark: Values in brackets are not measured but extrapolated.

obtained strength testing values after the samples hardening (Kowalski 1961, 1975), when some mechanical fatigue was already observed (i.e. after about 210 days after samples excavation from the quarry).

Each tested sample was always a regular cube (dimensions: 0.04 x 0.04 x 0.04 m exact to 0.001 m). The same cubic form and exact dimensions of samples made possible to avoid further considerations about influences at geometrically different sample shape on the obtained compression strength testing values.

It was important too that the compression strength testing of each sample was executed always on the same hydraulic press (C.S.B.29) with the same proper constant velocity (0.1 MPa s⁻¹) according to the obligatory standards by the same two operators.

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4. RESULTS

The uniaxial compression strength testing results ($R_{\text{est}}$) are given in Table 1. There are also the calculated for each series arithmetic mean values ($\bar{R}_{\text{est}}$) the permissible standard deviations from the mean values ($\Delta R_{\text{est}}$) for each series of the tested samples (i.e. series after: 30, 60, 90, 120, 150, 180, 210, 240, 270 and 300 days after the samples excavation day from the quarry and the permissible standard acceptable limit strength values: maximum ones ($R_{\text{est max}}$) and minimum ones ($R_{\text{est min}}$). The obtained results for each series (Tab. 1) are contained within these limits with a great margin. Therefore, the obtained results of the uniaxial compression strength testing are adequately precise according to standards and the calculated mean values formally credible. It is easy to notice in table 1, that the calculated arithmetic mean strength values and the
Fig. 3 Changes of compressive strength values within the opoka marl samples in time (from a moment of their excavation in a quarry at Sulejów). Axes: \( t_d \) - time one (in days); \( R_{\text{c,lt}} \) - strength one (in MPa). Strength values of samples: 1 - ones with conical shear surfaces; 2 - ones cracked along planes, that are parallel to the measured tectonic fissures in quarries. Strength values in defined moments - \( t \) (after sample excavation); 3 - mean values and the limit, permissible (according to standard) strength values; 4 - maximum and minimum ones. Diagrams of strength changes in time: 5 - arithmetic mean values; and laboratory determined strength values: 6 - maximum ones, 7 - minimum ones, and also calculated, limit (permissible according to standard) strength values: 8 - maximum ones and 9 - minimum ones. Remark: Strength values at the excavation moment are extrapolated, not measured.

measured minimum \( (R_{\text{c,lt, min}}) \) and maximum \( (R_{\text{c,lt, max}}) \) ones in the first series increase comparatively quickly (Fig. 3) until about 180 days (after the sample excavation day). So, the observed initial strength values increase confirms the known process of sample hardening during the first 180 days after their excavation from the rock massif in open and underground pits. It was described similarly among others (W. Fortunat, 1962, 1972). The collected in table 1 strength values in the first two series after 30 and 60 days proved to be homogeneous data set. But after 60 days after samples excavation one can divide the whole data set into subsets: the first one with greater strength values of samples, which crack along conical shear surfaces (GCS - Fig. 2 and Fig 1a) and the second one with the lower strength values of samples, which crack along vertical or almost vertical planes or some combination of parts of similar planes, that are parallel to the measured tectonic fissures and cracks in the rock massif (GPS - Fig. 2 and Fig. 1B and 1C). The origin of similar planes can testify to the existence of some latent planes of the strength weakness in rock massives (Kowalski 1961). These planes result from changes of the stress state during tectonic history of the rock massives. The two different subsets are separated in table 1 by dotted line.

The arithmetic mean strength values for each series of measurements are calculated as for the first subset of samples with conical shear surfaces \( (R_{\text{c,lt}}) \) as for the second subset of samples with vertical shear planes \( (R^p_{\text{c,lt}}) \).

The arithmetic mean strength values for series in the period from 180 till 240 days after the samples excavation are highest and almost constant; later on, after 240 days they decrease slowly. This remark refers as well to the whole data set \( (R_{\text{c,lt}}) \) as to the first subset \( (R^c_{\text{c,lt}}) \) and the second one \( (R^p_{\text{c,lt}}) \). However, it is better to take into further consideration the first subset data \( (R^c_{\text{c,lt}}) \) only. The second subset data \( (R^p_{\text{c,lt}}) \) could result from the latent strength weakness planes occurring in the marl samples as well as by its mechanical fatigue under some influence of the repeated atmospheric pressure changes, because the samples were constantly in the same laboratory conditions with practically constant room temperature and air humidity and only the atmospheric pressure was changing in time. Only the
atmospheric pressure changes could cause many times repeated loading and unloading of the marl samples (Kowalski, 1992).

Therefore some precise observations of the even small strength values decrease in the tested marl samples of they first subset (i.e. with the greater strength and without the latent strength weakness planes) could point more explicitly the atmospheric pressure changes as the cause of the observed mechanical fatigue in the tested marl samples (Fig. 4).

At last, it should be noticed that given in table 1 in brackets strength values in the excavation day from the quarry are not measured but only extrapolated; and that the two strength values, which are marked with crosses, are not the arithmetic mean values, but result of singular measurements.

5. HARDENING STAGES AND MECHANICAL FATIGUE

Each sample, which has been excavated from open or underground pit, is in quite different physical conditions than it was before in the rock massif. Its internal physical field, systems of internal forces, texture and structure have to be accommodated to its new external physical field. The accommodation of the internal, physical field of each sample to its external field is a very compound process, which is extended in time. Taking into considerations the above premises and findings of moisture content, weight, volume and uniaxial compression strength changes in time in the tested, spatially oriented samples of a Upper Cretaceous marl and also running atmospheric pressure changes at practically the same air humidity and temperature in the laboratory, it is possible to distinguish the following stages of strength changes in the samples in time, beginning from their excavation day from the open and underground pit:

1. The initial stage of sample hardening (i.e. the stage of the rapid strength increase), when the weight of samples is the greatest, when they are water-saturated and their volumes are increasing as a result of decompression in new conditions after their excavation from a rock massif. The shear surfaces under the uniaxial compression influence in samples are always conical, as in isotropic bodies. The stage persists two months (beginning with the day excavation from the quarry). This period is too short to manifest a mechanical fatigue under the influence of atmospheric pressure changes.

2. The main stage of sample hardening (i.e. the
stage of very rapid strength increase, like before). The moisture content and weight of samples decrease (slightly, more slowly), but the volume of samples increase unnoticeable in practice. Almost every sample shows conical shear surfaces after the uniaxial compression, but the particular samples in the particular measurement series reveal cracks along vertical planes, which are parallel to the measured tectonic cracks and fissures in the excavation quarry. These samples show some strength anisotropy of the tested marl at that time. This stage lasts about three months and begins about the 90th day and ends about the 150-th day since the sample excavation day. Occasional occurrence of latent strength weakness planes might manifest incipient mechanical fatigue of tested samples under the atmospheric pressure changes (i.e. the repeated loading and unloading).

3. The late stage of sample hardening (i.e. the stage of the slower strength increase). The weight and moisture content in samples decrease very slowly. The volume changes are unnoticeable. A little more samples crack along vertical planes, which are parallel to the tectonic cracks fissures. The strength anisotropy increase in the tested samples is observed, though most of samples preserve their primary strength isotropy. This stage lasts about one month and begins on about the 150-th day and ends about the 180-th day (after sample excavation day from quarry). Then, the distinct increase of the samples number, which manifest latent strength weakness planes might testify the mechanical fatigue increase.

4. The final stage of samples hardening (i.e. the stage of the highest strength values stabilization). The weight and moisture content decrease are minimal. The volume increase of samples unnoticeable. Almost one third of the tested samples crack after uniaxial compression along vertical planes, which are parallel to the measured tectonic cracks and fissures in the quarry, but the remaining (70%) samples crack along conical shear strength surfaces. This stage lasts about 1.5 month and begins on the 180-th day and ends about the 225-th day after the sample excavation day. Presence of many latent strength weakness planes in the tested samples and the highest strength values stabilization might manifest some equilibrium between two main processes: rock hardening after excavation and mechanical fatigue under atmospheric pressure changes.

5. The stage of the predominant rock mechanical fatigue manifestation. The tested samples after their hardening are in the air-dry state having constant weight and volume and are characterized by very slow strength decrease in time. At the very beginning of this stage about 45% of the tested samples manifest latent strength weakness planes, which demonstrate the tectonic strength anisotropy increase in time. This stage begins at about 225-th day after the excavation day. Then, in comparison with the air humidity and temperature constancy in the laboratory only the atmospheric pressure changes are practically the only variable factor in the external physical field around the samples, which might cause mechanical fatigue of them and as its result their strength decrease.

6. GENERAL CONCLUSIONS

The above results and executed analyses of many published and archival rock compression strength values as well as more general publications (Duncan 1969; Jaeger 1979; Kidybinski 1982; Kieslinger 1960; Müller 1963; Obert and Duvall 1967; Talobre 1957; Thiel 1989; Charlez 1991 and others) allow to formulate the following general conclusions for each rock type, which had been excavated from its rock bed (from its rock massif) and later existed in the atmospheric air:

1. The rock strength is not a constant parameter, characterizing a rock in each conditions of its existence; therefore, strength values of a rock after its excavation from its natural bed as samples differ from the primary strength of the rock in its natural rock massif.

2. The physical field (of pressure temperature, humidity) in a rock sample, to be excavated from the natural rock massif, is simply a part of the whole physical field of that rock massif and therefore the internal and external field of the same sample are in reality only one continuous physical field.

3. The internal physical field in the rock sample after its excavation from its rock massif differs essentially from its external physical field in the surrounding atmosphere.

4. After excavation of the sample its internal physical field conforms for some time to its average external physical field, which is expressed by the sample strength value increase, i.e. hardening, which begins first of all with some elastic decompression.

5. The state of the external physical field of a sample oscillates about its average state, what causes some oscillations of its internal physical field and later on - its mechanical fatigue by periodic and quasiperiodic compression and decompression, what becomes obvious after hardening.

6. Hardening and increase with time of mechanical
fatigue of rock samples begin and develop simultaneously from the moment of the sample excavation from the rock massif.

7. Four stages of hardening can be distinguished in each rock sample initial, main, late and final ones, after which the rock sample strength values decrease, manifesting development of detectable mechanical fatigue process.

8. The atmospheric pressure oscillations are one of weathering factors (as the temperature and humidity), that cause (some) mechanical fatigue of rock in the top soil in the atmospheric air.

9. Many misunderstandings in engineering-geological elaborations done for building and mining purposes are results of a wrong assumption, which is strength values of the same rock (the same mineral content, same structure and texture) are always constant in each occurrence conditions, so they are the same or almost the same inside a rock massif and in various weathering zones.

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Consolidation et imperméabilisation des sols argileux par électro-injection du silicate de sodium

Consolidation and watertightness of clay soils by electro-injection of sodium silicate

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ABSTRACT: In this paper a technique of electro-injection, to strengthen and to render more impervious clay soils is described; it exploits the electrokinetic properties which clay soils are practically the only ones to possess. The electro-injection consists of introducing a solution of sodium silicate, which may be polymerised, into soils by electro-osmosis. The silicate polymerisation phenomenon within a clay similar to clays found in disposal sites has been studied. The relevant clay parameters such as mineralogical and physico-chemical aspects, soil strength, hydraulic permeability, are studied. Conditions which favour electro-injection have also been studied. The results show that this process may be applied to clays used for disposal.

RESUMÉ: Cette communication décrit un procédé de consolidation et d'imperméabilisation des sols argileux en mettant à profit les propriétés électrocinétiques que les argiles sont pratiquement seules à posséder. La technique dite d'électro-injection, consiste à introduire dans le sol par effet électro-osmose une solution de silicate de sodium polymérisable. Le phénomène de polymérisation du silicate au sein d'une argile comparable aux argiles qu'on trouve dans les sites de confinement des déchets a été étudié. Nous avons mis en évidence l'influence des paramètres pertinents tels que minéralogie, physico-chimie... ainsi que les modifications des caractéristiques (résistance mécanique, perméabilité hydraulique...). Nous avons également étudié les conditions favorables à l'électro-injection. Les résultats ont montré que le procédé peut être appliqué aux argiles de confinement.

1 INTRODUCTION

Le procédé de consolidation des sols par le silicate de sodium est décrit dans la bibliographie et parfois utilisé avec des résultats encourageants [9]. La plupart des applications concernent des sols relativement perméables dans lesquels il est possible d'injecter le silicate sous pression (sables, roches fissurées...). Cette technique d'injection, bien entendu, n'est guère applicable dans les sols argileux qui ont en général une très faible perméabilité.

Le procédé électrochimique dit d'électro-injection de silicate exposé dans cette communication, utilise les propriétés électrocinétiques de surfaces, que les argiles sont pratiquement seules à posséder.

Dans le domaine des stockages de déchets, ce procédé pourrait permettre :
- de consolider l’argile pour y fonder des structures industrielles,
- d’imperméabiliser les parois des puits pour éviter les risques de lixiviation des déchets...

2 ELECTRO-OSMOSE

Les surfaces développées des particules argileuses sont chargées négativement, quand elles sont mises en présence de l’eau. Il se forme alors une double couche électrique (couche fixe et couche diffuse) à l'interface particule-eau où règne un potentiel électrocinétique, dit potentiel Zétal. Quand on applique à ce milieu un champ électrique E, l'expérience montre qu'il peut se produire deux phénomènes :

- soit un flux d'eau dit flux électro-osmose si la paroi solide est fixe ; les ions chargés positivement dans la phase liquide se déplacent dans le sens du champ.
- soit un mouvement de particules électronégatives si la suspension est suffisamment diluée ; c'est l'électrophorèse.

\[
\begin{align*}
\text{Sens du champ} & \quad \text{Déplacement du liquide} \\
\text{Solide} & \quad \text{couche fixe} \\
\text{couche diffuse} & 
\end{align*}
\]

Fig 1
2.1 Principe

L’électro-osmose est une méthode connue de drainage de l’eau d’un milieu poreux à l’aide d’un champ électrique.[9]

En effet, si on applique un courant continu entre deux électrodes enfoncées dans un sol l’eau se déplace de l’anode vers la cathode (figure 2).

![diagram](image)

Fig 2

L.Casagrande a établi une loi simple donnant l’expression du débit d’eau écoulé par électro-osmose.

\[
Qe = Ke \cdot S \cdot \frac{\Delta V}{L}
\]

\(Qe\) : Débit d’eau (m³/seconde)

\(S\) : Section de l’échantillon (m²)

\(\Delta V\) : La différence de potentiel (volt)

\(L\) : La longueur de l’échantillon (m)

\(Ke\) : Coefficient de perméabilité électro-osmotique (m²/V.S)

Dans les sables, la perméabilité électro-osmotique \(Ke\) est négligeable par rapport à la perméabilité hydraulique. Cette méthode est donc exclusivement réservée aux terrains fins (sols argileux) en partie. La consolidation et le drainage par électro-osmose ont l’inconvénient d’être réversible ; si l’on ne change pas l’environnement hydrique de la structure traitée, elle peut revenir avec le temps à la situation initiale. C’est pourquoi on introduit dans l’eau un produit polymérisable pour une consolidation durable ; c’est l’électro-injection.

2.2 L’électro-injection

Le principe de l’électro-injection consiste donc à introduire le produit consolidant au sein du sol argileux à l’aide d’un flux hydrique d’origine électro-osmotique. Le produit (silicate de sodium), dans des conditions de pH et de concentration adéquates, s’y polymérisera en consolidant le sol et en l’imperméabilisant par remplissage des pores.

Les principales conditions pour la réussite de ce procédé sont :
- Le produit doit être soluble dans l’eau. La solution de silicate utilisée a les caractéristiques suivantes :

* TNL

Formule : Na₂O-Si₃O₁₆ ; densité : 1.365 ; Rapport SiO₂/Na₂O=3,3 ; concentration en silicate : 36%.

- Le potentiel électrocinétique Zéta des particules solides du sol doit être suffisamment élevé ; c’est le cas des sols suffisamment riches en argiles. Nous avons particulièrement étudié une argile (argile A) provenant d’un site profond du bassin parisien destiné à un éventuel stockage de déchets radioactifs.

3 ESSAIS D’IDENTIFICATION

Trois échantillons d’argile A dont les caractéristiques sont données ci-dessous, ont été étudiés.

3.1 Teneur en eau à l’état naturel

<table>
<thead>
<tr>
<th>Tableau 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>ECHANTILLON (REFERENCE)</td>
</tr>
<tr>
<td>A 1</td>
</tr>
<tr>
<td>A 2</td>
</tr>
<tr>
<td>A 3</td>
</tr>
</tbody>
</table>

3.2 Valeur de bleu de méthylène (11)

La capacité d’échange est un critère qui permet de savoir si un sol est susceptible d’être traité par ce procédé. Elle conditionne en effet le rendement du procédé (surface spécifique, potentiel électrocinétique). Elle peut être mesurée par l’essai au bleu de méthylène, mis au point par l’un d’entre nous* au Laboratoire Central des Ponts et Chaussées. Il est l’objet d’une norme Afnor au niveau français et bientôt au niveau européen en vue de la classification des sols et des granulats de génie civil.

Le but de l’essai est de caractériser la fraction argileuse d’un sol d’une façon globale, c’est-à-dire que le résultat de l’essai dépend directement à la fois de la quantité et de la nature de l’argile de cette fraction. Il permet de mesurer la capacité d’adsorption ionique des sols à l’aide du bleu de méthylène. Les résultats de l’essai s’expriment en Valeur de Bleu (VB).

<table>
<thead>
<tr>
<th>Tableau 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Echantillon</td>
</tr>
<tr>
<td>A 1</td>
</tr>
<tr>
<td>A 2</td>
</tr>
<tr>
<td>A 3</td>
</tr>
<tr>
<td>KAOLIN</td>
</tr>
<tr>
<td>BENTONITE</td>
</tr>
</tbody>
</table>
3.3 Limites d’Atterberg

Le fait que les teneurs en eau naturelles, les valeurs de bleu et les Limites d’Atterberg sont pratiquement les mêmes pour les trois échantillons montrent que ce massif argileux est relativement homogène. D’après la classification française des sols et des terrassement (G.T.R), le massif se situe dans la classe des sols plastiques (A3 ou A4).[12]

4 PARTIE EXPERIMENTALE

Elle comporte l’étude des points suivants :
- la polymérisation du silicate au sein de l’argile, le silicate étant alors simplement mélangé au sol,
- l’électro-injection du silicate de sodium sur un échantillon reconstitué,
- l’électro-injection d’un échantillon non remanié.

4.1 Etude de la polymérisation

Trois mélanges ont été réalisés aux dosages pondéraux respectifs suivants : 0%, 7% et 20%. Pendant la polymérisation on effectue des mesures des résistances électrique (1000 Hz) et mécanique par un pénétromètre à ressort.

Les courbes des figures (3 et 4) traduisent la loi d’ Archie, qui indique que la résistance électrique apparente d’un sol est proportionnelle à la résistivité de la solution d’imprégnation (solution de silicate).[13]. La résistivité électrique est un indicateur sensible de la présence du silicate de sodium et sera utilisée pour suivre la progression du front de silicate. Le fait que la résistance électrique augmente à partir de 40 jours peut être expliqué par le développement d’un nouveau facteur de formation (coefficient de proportionnalité de la loi d’ Archie) et par la diminution de la mobilité ionique dus à la polymérisation.

On observe sur la figure 4 que :
- la résistance mécanique est multipliée respectivement par 4 et par 8 pour les dosages de 7 et de 20%.
- 80 % de la résistance mécanique est atteinte au bout de 10 jours. Cette durée peut être considérée comme le temps de prise.

On peut affirmer que le silicate de sodium permet effectivement de consolider cette argile. D’autres résultats qui ne sont pas exposés dans cette communication, ont montré cependant que nous avons obtenu une consolidation nettement supérieure.
pour la kaolinite et pour la montmorillonite-Ca. Le phénomène de consolidation semble donc dépendre notablement de la minéralogie et de la physico-chimie des sols (pH, cations échangeables...).

4.2 Étude de l'électro-injection

Avec l'échantillon N° A 1 on a fabriqué avec de l'eau déminéralisée un mélange pratiquement liquide, c'est à dire à une teneur en eau égale à trois fois sa Limite de Liquidité.
La cellule de la figure 6 étant remplie d'eau, on introduit la boue par le tube central et on la laisse se décanter. De cette façon, le remplissage s'effectue sans poche d'air.
Le sédiment est ensuite consolidé en chargeant le piston du tube central.

![Fig 6](image)

Taille de l'échantillon :
section : 40 cm² - longueur : 20 cm

À la fin du tassement, la teneur en eau pondérable du matériau est de 63%, la masse volumique calculée est de 1,85 g/cm³ qui est une valeur proche des densités des sols en place.

4.2.1 Mesure de la perméabilité hydraulique

Les perméabilités hydrauliques (moyenne de 5 mesures) déterminées à l'aide de cette cellule sont données dans le tableau ci-dessous en appliquant la loi de Darcy.

![Fig 7](image)
Les valeurs expérimentales trouvées (3.10^-9 m/s par v/m) sont comparables aux valeurs trouvées pour plusieurs autres argiles par L.Casagrande (5.10^-9 m/s par v/m) [3]. On constate sur la courbe figure (7) que le débit d'eau, en accord avec la loi de Casagrande, augmente linéairement avec la densité de courant jusqu'à environ 10 A/m². Au-delà de cette valeur, l'énergie électrique sert plus à l'échauffement de l'échantillon qu'aux transports électro-osmotiques proprement dits. Cela pourrait s'expliquer par une augmentation de la résistance électrique du matériau suite au départ d'eau.

De même, la figure (8) montre par la proportionnalité entre le champ électrique et la densité de courant, que la loi d'Ohm est également valable jusqu'à 10 A/m².

4.2.3 L'électro-injection du silicate de sodium

On remplace, dans le compartiment anodique l'eau par une solution de silicate de sodium où on immérite un anode en nickel. Deux installations hydraulique permettent de mettre en circulation les solutions de chaque compartiment et le maintien des pH à des valeurs de consigne:
- pH = 11 pour l'anode
- pH = 7 pour la cathode.

Ce pH évolue en effet en fonction de diverses réactions électro-chimiques qui ont lieu pendant l'injection.

Réaction anodique:
2OH- <---> 1/2 O₂ + H₂O + 2e-

Réaction cathodique:
H₂O + e- <---> 1/2 H₂ + OH⁻

4.2.4 Suivi de l'effet du silicate par mesure de résistance électrique

Deux électrodes impérissables Ag/AgCl (E1 et E2) sont implantées le long de la cellule et permettent, avec l'anode et la cathode de suivre le front de silicate par les variations de résistances. La figure 9 indique le profil de résistance en cours d'injection où l'on observe une nette diminution des résistances électriques dans la cellule. Au bout de 50 h d'injection, la résistance est devenue très faible partout dans la cellule. Ce fait indique que la totalité de l'échantillon est effectivement traité et que le front de produit a atteint la cathode où l'on observe l'apparition de gel de silicate. En fin d'injection, le bilan de l'électro-injection est le suivant :
- Concentration pondérale du silicate dans le matériau : 14%.
- Consommation en énergie électrique : 800 Kwh/m³ de sol

4.2.5 Mesure des coefficients de Darcy et de Casagrande

Nous avons mesuré les perméabilités hydraulique Kh et électro-osmotique Ke du sol traité.

<table>
<thead>
<tr>
<th>Tableau 6</th>
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<tbody>
<tr>
<td></td>
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<tr>
<td>Mesure 1</td>
</tr>
<tr>
<td>Mesure 2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tableau 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Densités de courant (A/m²)</td>
</tr>
<tr>
<td>Mesure 1</td>
</tr>
<tr>
<td>Mesure 2</td>
</tr>
<tr>
<td>Mesure 3</td>
</tr>
</tbody>
</table>

La perméabilité de Darcy a été réduite d'un facteur de 100 par le traitement. Les observations au M.E.B confirment que le silicate, en polymérisant, emplit et bouche les pores. Par ailleurs, les mesures aux tensionnètres (pression de l'eau interstitielle) que nous ne présentons pas ici, indiquent que la polymérisation se traduit également par un changement de l'état libre de l'eau à un état lié.
La réduction de la teneur en eau libre peut contribuer également à cette diminution de la perméabilité. On n'a pas constaté de modification semblable en ce qui concerne les perméabilités électro-osmotiques.

4.2.6 Résistances mécaniques

<table>
<thead>
<tr>
<th>Echantillons</th>
<th>Résistance de poissonnement (KPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A 01208</td>
<td>400 *</td>
</tr>
<tr>
<td>Kaolinit</td>
<td>1300 *</td>
</tr>
<tr>
<td>Bentonite sodique</td>
<td>600 *</td>
</tr>
<tr>
<td>Bentonite Calcique</td>
<td>800 **</td>
</tr>
</tbody>
</table>

* après 270 jours  
** après 150 jours

Pour améliorer le niveau de résistance mécanique de l'argile A traitée, qui est comparativement faible (tableau 4) on a effectué, sur le sol déjà "électro-injecté de silicate" une électro-injection d'un sel acide divalent (CaCl2) dont on connaît le rôle accélérateur de polymérisation [4].

4.3 Electro-injection du CaCl2

On remplace dans le compartiment anodique la solution de silicate par une solution de CaCl2.
Le rôle de ce sel acide divalent est double :
- Accélérer la polymérisation du silicate de sodium par abaissement de pH,
- Favoriser la formation du silicate de calcium qui est insoluble et de ce fait est plus résistant que le silicate de sodium. Nous rappelons que le silicate de calcium est l'un des constituants de base du béton.

4.3.1 Suivi de l'effet du CaCl2 par la mesure de la résistance électrique

On a implanté dans l'échantillon des électrodes E1 E2 en Ag/AgCl, indiquées sur la figure (10).

L'électro-injection du chlorure de calcium (CaCl2) se traduit par une augmentation de la résistance électrique de l'argile A silicatée, confirmant l'accélération de la prise.

4.3.2 Résistance mécanique

Après l'électro-injection du chlorure de calcium, la carotte a été retirée de la cellule et on en a établi le profil de résistance à l'aide du pénétromètre à ressort.
L'élimination qui sont faciles à réaliser à forte teneur en eau, peuvent s'effectuer néanmoins sur un terrain à faible teneur en eau (10%).

5.1 L'électro-osmose

La progression de l'eau dans la cellule est suivie à l'aide d'électrodes en acier B1, E2, E3 et E4 implantées dans la carotte (fig. 14). La figure 15 indique que, au cours de l'électro-osmose, les résistances décroissent de très fortes valeurs initiales, typiquement de 15 kΩ, à moins de 1 kΩ du fait du départ d'eau.

5.2 Electro-injection

Dans le compartiment anodique, on a remplacé l'eau par une solution de silicate de sodium. Au cours de l'électro-injection, les résistances, tout le long de la carotte, diminuent de façon importante (d'un facteur de 5) traduisant la pénétration du silicate.

L'ensemble de ces résultats montre que malgré une teneur en eau relativement faible, on a pu réaliser l'électro-injection de silicate. Si l'on se réfère aux mesures de résistances électriques, le traitement ainsi obtenu apparaît homogène.
6 OBSERVATIONS au M.E.B

On a observé une nette différence de texture entre les sols traités et non traités (photos 1 et 2), les plaquettes hexagonales de la kaolinite semblent être soudées par le silicate de sodium. Dans l'argile, on a plutôt constaté un emplissage de pores par le produit polymérisé. (photos 3 et 4.)

Le renforcement des pics Na et Ca dans les spectres de rayonnements issus du MEB obtenus sur un échantillon traité confirme la présence des produits électro-injectés.

D'une façon inexplicable, on n'a pas observé de différence significative dans le cas de la carotte non remaniée.

7 CONCLUSION

On peut déduire de l'ensemble des résultats de cette expérience en laboratoire les points suivants :

- On a pu réaliser par électro-injection une diffusion homogène de silicate dans tout l'échantillon.

- Les résultats du traitement dépendent de la nature minéralogique et physico-chimique. Pour l'argile A étudiée, les expériences préliminaires ont montré que le silicate de sodium permet de la consolider sans atteindre le niveau obtenu pour certaines argiles. L'addition de CaCl₂ a amélioré très nettement les résultats.
- Le traitement au silicate réduit d'un facteur de 100 la perméabilité hydraulique.

- Les résultats sur carotte non remaniée montrent que malgré une très faible teneur en eau, on a pu réaliser l'électro-injection d'une façon satisfaisante.

- Ce procédé de consolidation et d'imperméabilisation semble donc applicable aux sols argileux.

REFERENCES


3 / BONNEMAY M. et ROYON J. : Technique de l'ingénieur section électrochimie, électro-osmose


Changes in engineering properties of peats due to consolidation
Changement des propriétés des tourbes avec la consolidation

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Agriculture Academy at Olsztyn, Poland

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Gdańsk Technical University, Faculty of Hydroengineering, Geotechnical Department, Poland

ABSTRACT: The statistical tests conducted for the peat samples taken from one profile and different depth are shown. Changes of the filtration coefficient with the void ratio and time are approximate by homograph function. The practical application of the shown relations is more accurate prediction of behaviour for improved soil bed during the construction of design embankment.

RESUMÉ: Les résultats statistiques des recherches concernant des propriétés de la tourbe provenant de différentes profondeurs du même profil sont présentés. Les changements du coefficient de filtration avec le changement du coefficient de porosité et du temps sont approximés avec une fonction homographique et une application pratique des relations établies forme une prognoxe plus précise du comportement du sous-sol amélioré en train de la construction d'un remblai projeté.

1 INTRODUCTION

The area of the northern Poland is covered with thick (more than 100 meter) layer of glacial deposits. As far as geomorphology is concerned this area is very varied. Together with wide plain the ranges of terminal moraines hills exist. We meet few thousand lakes on this area. Some part of them in result of overgrowing change in swamp and marsh which are 7 - 10 % of the area of Northern Regions of Poland. Typical are Holocene organic sediments made as peat, gyttja, mud's and other kinds of organic materials sometimes to the large thickness in range of tenth meters.

These soils give specific problems with foundation for engineering structures.

Presented research results for peat are from the territory of Olsztyn City obtained during the construction of a new highway.

2 GENERAL DATA ABOUT THE SITE AND THE SOIL IMPROVEMENT

The design highway embankment crosses the marsh with quickly changing thickness on the way of 100 m. The width of embankment was 50 m. maximum thickness of peat layers was close to 12 m. under the peat are Pleistocene loam and sands and clays. Typical cross section profile is shown in Fig. 1.

For choosing inexpensive and effective optimal technology of soil improvement the extensive researches of the parameters of existing weak layers were conducted. Among others dynamic consolidation, blasting method, lime and stone columns, overloading embankment and the vertical sand drains were considered. The economic raisons and the localisation of the site in town close to the urban area decided that the consolidation with vertical drains and partial peat exchange was chosen. Precise prediction of behaviour for highly organic soil medium, like the peat, strengthened by consolidation methods using vertical drains (sandy piles, geodrains, stone columns etc.) requires apart from the initial parameters in deposit also the evaluation changes of these parameters in the process.
of consolidation. The basic equation for one dimensional consolidation:

\[
\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}
\]

(1)

Where:
- \( u \) pore pressure,
- \( c_v \) coefficient of vertical consolidation,
- \( t \) time,
- \( z \) depths,

usually is solved with very simplified assumptions. Among others, it is the assumption of constant value of compressibility coefficient and filtration coefficient during the consolidation process. These assumptions rise many doubts. Especially in case of peat where the consolidation process is much more complicated. Together with primary consolidation, the secondary consolidation is connected with decomposition of the organic mass.

In case of vertical sand drains the consolidation equation could be written as follows:

\[
\frac{\partial u}{\partial t} = c_v \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) - c_v \frac{\partial^2 u}{\partial z^2}
\]

(2)

Where:
- \( c_h \) coefficient of horizontal consolidation,
- \( r \) the radial coordinate,
- \( z \) the depths.

are situation is similar, the existing solutions based on the same simplifications. Deep simplification of solutions for presented equations, neglect determination of soil parameters, incorrect taking samples, cause that the results which we got are very often far from reality. Avoidance of these errors requires wide knowledge not of only the initial parameters occurring in site, but the knowledge of the changes of these parameters in the process of consolidation is necessary. Precise estimation of value and time of the settlement for the design road embankment need a lot of the time consuming field and laboratory test. During these tests we observed some regularities which could be interesting in similar cases. Interests in the changes of the parameters in the process of consolidation rise due to fact that nowadays buildings are more often situated on the areas of the weak ground.

In this case the observed regularities helped us to predict the behaviour of the bad during the stage construction process of embankment.

3 LABORATORY TESTS

The tests were made on undisturbed samples taken from monoliths. The analysed monoliths in dimension of 0.2 * 0.2 * 0.2 m were taken as the block from the depths of 2.0, 3.0, 4.0, 4.5 and 5.0 m below the surface. The scope of the test results for the basic physical parameters is shown in table 1.

The compressibility of the peats were tested in oedometric conditions as primary compression of peats in terms of compression index \( C_C \) where the change in the void ratio \( \Delta e \) due to primary compression as the result of increasing vertical effective stress from \( \delta_i \) to \( \delta_f \) is given by the familiar relationship:

\[
\Delta e = -C_c \cdot \log \frac{\delta_f}{\delta_i}
\]

(3)

\( C_c \) is the compression index.

The applied load were 12.5, 50, 100, 200, 400 kPa. Observation were conducted to stabilisation of the settlement with accuracy 10^-5 m. The elapse time for one load interval varied from 30 to 90 days. The secondary compression in terms of a coefficient \( \alpha \) where the change in void ratio \( \Delta e \) due to secondary compression over the time between \( t \) and \( t_i \) is given by relationship:

\[
\Delta e = -\alpha \cdot \log \frac{t_i}{t_f}
\]

(4)

Where: \( \alpha \) is known as the secondary compression index.

Because of the large spatial variation in compressibility of a given organic deposits it is sufficiently accurate to apply the constrained modules

<table>
<thead>
<tr>
<th>No</th>
<th>Depth [m]</th>
<th>( w ) [%]</th>
<th>( \rho_s ) [g/cm³]</th>
<th>( \rho ) [g/cm³]</th>
<th>( \rho_d ) [g/cm³]</th>
<th>( e_0 ) [-]</th>
<th>( I_{om} ) [%]</th>
<th>R [%]</th>
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<tbody>
<tr>
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<td>0.093</td>
<td>16.2</td>
<td>91.7</td>
<td>25</td>
</tr>
</tbody>
</table>

\( w \) water content, \( \rho_s \) specific densities, \( \rho \) bulk densities, \( \rho_d \) dry densities, \( e_0 \) initial void ratio, \( I_{om} \) organic content, R degree of the peat decomposition according to Van Post scale.
D over the relatively wide range of stress encountered at the site.

At the given stress level the primary compression of the peat can be also described by a constrained modules D:

\[ D = \frac{\delta_f - \delta_i}{S_p} \cdot H \]  

(5)

Where: \( S_p \) is the primary compression of the layer of thickness H.

The values of oedometric modules for particular range of loading in the function of some physical properties are shown in table 2. The final value of the water contents and void ratios are also shown.

The filtration parameters during the consolidation in long term measurements were conducted in the oedometers with the use of procedure of decreasing water head according to the standard ONORM B 4422.

The tests were conducted for seven samples with vertical flow to the natural position in site and three samples for horizontal placement were the sample were parallel to the natural layer. The tests results are shown in table 3.

### 4 STATISTICAL ANALYSES OF THE FILTRATION COEFFICIENTS

In order to estimate changes of the filtration coefficient in the process of consolidation, the changes of that parameter of filtration \( k = f(t) \) in time and depending on the void ratio \( k = f(e) \) were checked. The changes of these parameters occurring with loading at range of 12.5 + 400 kPa were presented as relationships between the filtration factor and coefficient of porosity, the contents of organic parts, changes of the filtration factor with duration of time and others.

The experiment was based on the curve fitting method with looking after the regression equation with minimal value of the correlation coefficient R. In tests different types of function \( k = f(x) \) were taken. The best fitting and the highest coefficient correlation’s were achieved for the homographic function as follows:

\[ y = \frac{b_1 + b_2 \cdot x}{1 + b_1 \cdot x} \]  

(6)

Also, we show the results for the logarithmic

### Table 2. Oedometric Modules for peats

<table>
<thead>
<tr>
<th>No</th>
<th>Depth [m]</th>
<th>Wf [%]</th>
<th>( e_r )</th>
<th>Modules [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Load range in [kPa]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0+</td>
</tr>
<tr>
<td>2/1</td>
<td>2</td>
<td>283.03</td>
<td>69.3</td>
<td>137.7</td>
</tr>
<tr>
<td>3/1</td>
<td>3</td>
<td>257.22</td>
<td>81.9</td>
<td>144.5</td>
</tr>
<tr>
<td>4/1</td>
<td>4</td>
<td>251.24</td>
<td>58.6</td>
<td>103.5</td>
</tr>
<tr>
<td>4/2</td>
<td>4.5</td>
<td>250.26</td>
<td>63.7</td>
<td>188.2</td>
</tr>
<tr>
<td>5/1</td>
<td>5</td>
<td>379.49</td>
<td>75.8</td>
<td>127.0</td>
</tr>
<tr>
<td>2/2</td>
<td>2</td>
<td>247.18</td>
<td>14.6</td>
<td>140.5</td>
</tr>
<tr>
<td>3/2</td>
<td>3</td>
<td>228.17</td>
<td>93.9</td>
<td>99.3</td>
</tr>
<tr>
<td>4/2</td>
<td>4</td>
<td>250.27</td>
<td>98.2</td>
<td>118.2</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>269.32</td>
<td>97.1</td>
<td>128.1</td>
</tr>
</tbody>
</table>

### Table 3. Coefficients of filtration for the peats tested

<table>
<thead>
<tr>
<th>No</th>
<th>Depth [m]</th>
<th>Flow Direction</th>
<th>Coefficient of filtration ( k [cm/s] )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Applied load range ( \Delta ) [kPa]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>3/1</td>
<td>3</td>
<td>↓</td>
<td>1.75</td>
</tr>
<tr>
<td>4/1</td>
<td>4</td>
<td>→</td>
<td>4.53</td>
</tr>
<tr>
<td>4/1</td>
<td>4.5</td>
<td>↓</td>
<td>2.03</td>
</tr>
<tr>
<td>5/1</td>
<td>5</td>
<td>→</td>
<td>1.53</td>
</tr>
<tr>
<td>5/1</td>
<td>5</td>
<td>↓</td>
<td>5.36</td>
</tr>
<tr>
<td>2/2</td>
<td>2</td>
<td>↓</td>
<td>3.38</td>
</tr>
<tr>
<td>3/2</td>
<td>3</td>
<td>↓</td>
<td>3.71</td>
</tr>
<tr>
<td>4/2</td>
<td>4</td>
<td>↓</td>
<td>4.26</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>↓</td>
<td>2.4</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>↓</td>
<td>3.48</td>
</tr>
</tbody>
</table>

### Table 4. Parameters of Equation tested

<table>
<thead>
<tr>
<th>Type of Load Equation</th>
<th>Parameters of equation</th>
<th>Load ( \Delta ) [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>( k = \frac{b_1 + b_2 \cdot x}{1 + b_3 \cdot x} \cdot 10^{-4} )</td>
<td>b1</td>
<td>5360</td>
</tr>
<tr>
<td>b2</td>
<td>374</td>
<td>856</td>
</tr>
<tr>
<td>b3</td>
<td>296</td>
<td>200</td>
</tr>
<tr>
<td>Coefficient of Regression</td>
<td>R</td>
<td>0.999</td>
</tr>
<tr>
<td>( k = \frac{b_1 + b_3 \cdot x^{-0.4}}{1 + b_1 \cdot x^{-0.4}} \cdot 10^{-4} )</td>
<td>b1</td>
<td>-1870</td>
</tr>
<tr>
<td>b2</td>
<td>131</td>
<td>0.96</td>
</tr>
<tr>
<td>b3</td>
<td>5210</td>
<td>-44.90</td>
</tr>
<tr>
<td>Coefficient of Regression</td>
<td>R</td>
<td>0.999</td>
</tr>
<tr>
<td>( k = [\frac{b_1 + b_2 \cdot x}{1 + b_3 \cdot x}] \cdot 10^{-4} )</td>
<td>b1</td>
<td>-7370</td>
</tr>
<tr>
<td>b2</td>
<td>2750</td>
<td>34.2</td>
</tr>
<tr>
<td>Coefficient of Regression</td>
<td>R</td>
<td>0.993</td>
</tr>
</tbody>
</table>
relation:

\[ y = b_1 + b_2 \cdot \ln x \]  \hspace{1cm} (7) 

As the result of the calculated tests for each load step and for each case 58 regression equation were obtained for regarding relation \( k = f(t) \) and \( k = f(c) \). The results of one series of calculations for sample no 5/1 are shown in table 4. The diagrams are shown in Fig. 2. The comparison of the parameters of equation tested for different samples for full range

---

Fig. 2. Parameters of equations tested.
of load ($\Delta \delta = 12.5 \div 400$ kPa) is shown in Table 5.

5 CONCLUDING REMARKS

The statistical tests conducted for the samples taken from the same monoliths from one profile and different depth shown large nonhomogeneity of the values of peat parameters. One of the parameters of the peat which change so much during the consolidation is the coefficient of filtration. These changes are very dependent on the load level. For the small stress range $0 \div 100$ kPa the changes are similar and could be approximated by linear relation. It is well seen on Fig. 2. Changes of the filtration coefficients with time for larger loads than 100 kPa are much smaller and the graphic for this values are different.

Changes of the filtration coefficient with the void ratio and time are quite good approximate by homographic function (6).

It is well seen in results of particular load steps and coefficients of the equations.

For the loads 12.5; 50; 100 kPa the relation is linear and correlation coefficients are very high. For the load steps 200; 400 the relation is non-linear and obtained coefficients of correlation’s are very low. It is very difficult to find general correlation between the coefficients of this equation and initial parameters of soil. It need a large amount of data. This observation gives us more general conclusion, changes in filtration conductivity for the organic materials are very quick for one step loading due to primary consolidation. For multi steps loading with consolidation for each step the rate of the decreasing filtration coefficient rapidly changes for loads above the critical value of load. This critical value of load is dependant on the deformability of organic components of the soil, their age and decomposition. In our case these critical stress was close to 100 kPa. Above this value the rate of the decreasing permeability is much smaller. It means that for practical purposes it is possible to use the pair of homographic equations, one for the load to critical value and second for above. The critical value can be estimated from oedometric tests.

The practical application of the shown relations is more accurate prediction of behaviour for improved soil bed during the construction of design embankment.

Table 5. Comparison of the parameters of equation tested for different samples for full range of load ($\Delta \delta = 12.5 + 400$ kPa).

<table>
<thead>
<tr>
<th>Sample No</th>
<th>Depth [m]</th>
<th>Flow Direction</th>
<th>Coefficients</th>
<th>Regression</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$b_1$</td>
<td>$b_2$</td>
</tr>
<tr>
<td>3/4</td>
<td>3</td>
<td>$\downarrow$</td>
<td>-1.10</td>
<td>0.18</td>
</tr>
<tr>
<td>4/4</td>
<td>4</td>
<td>$\rightarrow$</td>
<td>-0.55</td>
<td>0.12</td>
</tr>
<tr>
<td>4.5/1</td>
<td>4.5</td>
<td>$\downarrow$</td>
<td>-4.56</td>
<td>0.61</td>
</tr>
<tr>
<td>5/5</td>
<td>5</td>
<td>$\rightarrow$</td>
<td>-4.40</td>
<td>0.50</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>$\downarrow$</td>
<td>-3.27</td>
<td>0.44</td>
</tr>
<tr>
<td>2/2</td>
<td>2</td>
<td>$\downarrow$</td>
<td>-11.70</td>
<td>2.79</td>
</tr>
<tr>
<td>3/2</td>
<td>3</td>
<td>$\downarrow$</td>
<td>-14.40</td>
<td>2.57</td>
</tr>
<tr>
<td>4/2</td>
<td>4</td>
<td>$\downarrow$</td>
<td>-19.70</td>
<td>2.97</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>$\rightarrow$</td>
<td>-112.0</td>
<td>2.46</td>
</tr>
<tr>
<td>5/2</td>
<td>5</td>
<td>$\downarrow$</td>
<td>-143.0</td>
<td>2.41</td>
</tr>
</tbody>
</table>

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Standard ÖNORM B 4422 Erd und Grünbau. Untersuchung von Bodenproben Bestimmung der Wasserdrückfähigkeit. Laborprüfung n...
Free swell of bentonites from Radzjonków (Poland)
Gonflement libre des bentonites de Radzjonków (Pologne)

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Institute of Hydrogeology & Engineering Geology Faculty of Geology Warsaw University, Poland

ABSTRACT: Bentonites from Radzjonków (Poland) are represented by two lithologic types – A and B. Such a classification is derived from differences in granulometric and mineral compositions and physical properties. Laboratory tests showed that the values of free swell differ and are related to the particular method of testing. Lithology of the bentonite and chemical composition of pore water also exert the influence on the above parameter. The bentonite tested have been recognized as "model" on the basis of extremally high values of free swell.

RESUME: Les bentonites de Radzjonków (Pologne) sont représentées par deux types lithologiques A et B. Une telle division ressort des différences de la composition granulométrique, minéralogique et des propriétés physiques. Il a été montré à la base des études laboratoires que les valeurs du gonflement libre sont différenciées et elles mettent en rapport avec une technique des mesures convenables. Ces valeurs sont fonction aussi de la lithologie des bentonites, de même que du chimisme des eaux saturantes. Les bentonites étudiées ont été avouées comme "modèle" à cause de leurs extrêmement hauts valeurs du gonflement libre.

1 LITHOLOGY AND PHYSICAL PROPERTIES

Recognized as "model" and selected to be tested bentonite of carboniferous age from western Poland are represented by the following clay minerals: smectite, mixed - layer minerals: smectite/illite and by minor amount of illite and kaolinite.
The content of these clay minerals is changing vertically along the profile. This implies the variability of exchangeable cations content and of physical properties. Taking into account the above two lithological types have been distinguished in the vertical profile:

Type A - represented mainly by mixed - layer smectite/illite (roof and floor parts of the profile).
Type B - represented mainly by smectite (middle part of the profile).
The characteristic properties of the bentonites together with the composition of the exchangeable cations are given in table 1. Those results show that smectite and smectite/illite which are the main clay minerals in the bentonites have sodium type of sorption complex and amount of exchangeable sodium is higher in type B. This will be reflected in the free swell values and expansiveness.

2 METHODS OF TESTING IN DETERMINING FREE SWELL VALUES

Free swell have been determined following two procedures:
- by Holtz & Gibbs (1956)
- by ASTM D 4546-90
The samples of the soil had been prepared in the same way that is oven dried and passed through a 425 μm sieve. Initial dry bulk density of all the samples was 0.8 Mg/m³. Free swell ($FS_{90}$) according to the description by Head (1992) is
Table 1. Physical and physical-chemical properties of the bentonites from Radzionków

<table>
<thead>
<tr>
<th>Physical properties and exchangeable cations</th>
<th>Lithological type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>D_{&lt;2mm} (%)</td>
<td>22 - 29</td>
</tr>
<tr>
<td>ρ (Mg/m³)</td>
<td>2.09 - 2.32</td>
</tr>
<tr>
<td>w_L (%)</td>
<td>109 - 150</td>
</tr>
<tr>
<td>w_p (%)</td>
<td>21 - 40</td>
</tr>
<tr>
<td>I_p (%)</td>
<td>70 - 111</td>
</tr>
<tr>
<td>w (%)</td>
<td>6.2 - 7.4</td>
</tr>
<tr>
<td>Na⁺ (mval/100g)</td>
<td>25.7 - 39.1</td>
</tr>
<tr>
<td>K⁺ (mval/100g)</td>
<td>2.2 - 3.6</td>
</tr>
<tr>
<td>Ca^{2+} (mval/100g)</td>
<td>9.4 - 14.0</td>
</tr>
<tr>
<td>Mg^{2+} (mval/100g)</td>
<td>2.8 - 8.4</td>
</tr>
</tbody>
</table>

defined as the increase in volume of the soil from the loose powder form when it is poured (submerged) into water, expressed as percent of the original volume if the original volume was 10 ml. Free swell is calculated from the equation (1).

\[ FS_{NO} = [(V - 10)/10]*100\% \] (1)

Free swell (FS) according to ASTM D4546-90 is the percent heave following the sorption of water at the seating pressure \( σ_w \) at least 1 kPa. It is calculated from the equation (2).

\[ FS = (Δh/h)*100\% \] (2)

The results of the tests carried out following the above methods are given in table 2.

The analysis of the results presented in table 2 indicate that the values of free swell for both lithological types acc. to Holtz & Gibbs are 2 - 3 times higher than the values of free swell acc. to ASTM.

Head (1992) states that soils reaching values of 100% or more are associated with clays which should swell considerably when wetted, especially under light loadings.

Table 2. Free swell of the bentonites acc. to Holtz & Gibbs (FS_{NO}) method and acc. to ASTM D4546-90 (FS) method

<table>
<thead>
<tr>
<th>Lithological type</th>
<th>FS_{NO} %</th>
<th>FS %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>140 - 270</td>
<td>47 - 81</td>
</tr>
<tr>
<td>B</td>
<td>170 - 500</td>
<td>88 - 186</td>
</tr>
</tbody>
</table>

Basing on the comparative tests carried out according to the methods cited above one finds the free swell measurement acc. to Holtz and Gibbs may be made only for loose fills which have not been subjected to high loading. This is because even relatively small pressure which in this case equals to 1 kPa (acc. to ASTM D4546-90), reduces the values of free swell obtained acc. to Holtz & Gibbs (1956) by 2 - 3 times.

Free swell value is influenced, apart from the method of testing, also by other factors. The chemical composition which can be characterized by pH is one of the factors. The samples of the bentonites in question have been saturated with water of variable pH.
Table 3. Free swell acc. to ASTM D4546-90 at variable pH of water.

<table>
<thead>
<tr>
<th>Lithological type</th>
<th>ρ₄ [Mg/m³]</th>
<th>FS %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>pH 2</td>
<td>7</td>
</tr>
<tr>
<td>A</td>
<td>1.50</td>
<td>23 - 33</td>
</tr>
<tr>
<td>B</td>
<td>1.50</td>
<td>33 - 95</td>
</tr>
</tbody>
</table>

(pH = 2, 7, 13). All the samples had the same value of ρ₄ = 1.5 Mg/m³ (table 3).

Obtained results confirmed the fact that the value of free swell at pH value of water of 7 (which corresponds to pure distilled water) is approximately average for FSₚ₄=1 and FSₚ₄=13.

It should be noted that values of FS for pH = 7 (which are comparable with values of FS in table 2) are generally lower. This is caused by different ρ₄ values in both cases.

The theoretical basis explaining the mechanism of different water sorption capacity of clays in the environment of variable pH values are the subject of numerous research papers. However the attention should be paid to the fact that dynamics of deformation changes which take place in "in situ" conditions where soils are saturated with water of chemical composition influenced by man will certainly differ from the laboratory character when samples are usually saturated with pure water. Therefore water of chemical composition corresponding to the "in situ" conditions and an adequate method of testing should be used. Taking into account the above statement it should be noted that the same clay may heave or settle.

3 LINEAR SHRINAGE AND EXPANSIVENESS

The deformation changes of the bentonite relying on the measurement of its shrinkage were investigated according to British Standard (BS 1377; part 2: 1990: 6.5) (Head 1992).

The measurement of the shrinkage was begun at the moisture content of the soil paste corresponding to the liquid limit. The results are given in table 4.

Table 4. Linear shrinkage of the bentonites (acc. to BS 1377: Part 2: 1990: 6.5)

<table>
<thead>
<tr>
<th>Lithological type</th>
<th>LS %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>24 - 32</td>
</tr>
<tr>
<td>B</td>
<td>25 - 61</td>
</tr>
</tbody>
</table>

At the present stage of the research it is difficult to state any firm correlation between the value of free swell and linear shrinkage. Obtained results allow only for statement that bentonite of type A (relatively poorly swelling) have lower value of linear shrinkage (up to approximately 30%) while intensively swelling type B has that value higher (to approximately 60%).

The comparison of the results given in tables 2, 3 and 4 will be acceptable when shrinkage of clay is expressed in volumetric percentage. This is the subject of current and future author’s research.

On the basis of using empiric Van Der Merwe’s chart (1964) the following have been stated:
- bentonite of type A is of high and very high potential expansiveness (PE),
- bentonite of type B are of very
high potential expansiveness. Taking into account Seed's chart
(Seed et al. 1962) bentonites have
very high value of the degree of
expansion (DE) and swelling
potential $S > 25\%$.

4 CONCLUSIONS

1. The tested bentonites are of
variable lithology and are
represented by two types A and B.
Type A is mainly of smectite/illite
composition, type B — mainly of
smectite. Sodium cation constitutes
a predominant exchangeable cation in
the both types.

Taking into account the higher
percentage content of clay fraction
in type B and its smectite
character Atterberg limits,
plasticity index, moisture content
are of higher values in that type
then in type A. The range of
activity values (A) is higher too
then in the bentonite of type A
(table 1).

2. Free swell value for type B is
approximately twice higher than
for type A irrespectively of testing
method applied.

3. Free swell (acc. to Holtz &
Gibbs) for both lithological types
of bentonites is 2 — 3 times higher
than the values of free swell (acc.
to ASTM D 4546-90). This is caused
by relatively small seating
pressure $\sigma_0 = 1$ kPa applied acc. to
ASTM.

4. Free swell values differ while
soil is subjected to the saturation
with water of variable pH. Increase
in FS value together with increase
in pH value have been observed
(table 3).

5. The value of the linear
shrinkage for type B is
approximately twice as high as for
that value for type A (table 4).
This results from the differences
in granulometric and mineral
compositions.

6. The bentonites of type A and B
reach extremely high free swell
values and outstand any others
clays in Poland in this field. For
that reason they have been termed
as "model".

7. Both types are positioned on
van der Merwe's chart in a way
which allows for describing their
potential expansiveness (PE) as
high and very high. They are also

very active, A > 2.0 (acc. to Head,
1992). Acc. to Seed et al. (1962)
they are characterized by very high
degree of expansion (DE) and
swelling potential $S > 25\%$.

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Weatherability and alteration studies of some shales and siltstones from the Recôncavo sedimentary basin – Northeast Brazil
Météorisation et altération de quelques schistes argileux et siltites du bassin sédimentaire de Recôncavo, Nord-est du Brésil

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Federal University of Rio de Janeiro, Brazil

E.A. Vargas Jr
Federal University of Rio de Janeiro & Catholic University, Brazil

ABSTRACT: In spite of the great area covered by sedimentary weak rocks (uniaxial compressive strength lower than 25 MPa - as suggested by International Society for Rock Mechanics) in Brazil, and the many engineering constructions installed or to be installed over these rocks, little is known about their geotechnical, physical, chemical, mineralogical and textural characteristics and the mechanisms related to their alteration processes. The present work is an attempt to identify the processes responsible by the weathering and expansion of some sedimentary rocks from Recôncavo sedimentary basin (Bahia- Northeast Brazil) through a wide laboratory procedure involving basic characterization (chemical and mineralogical composition, texture, index properties, etc.) and weatherability of these materials. The results obtained lead to some conclusions that are presented in the end of this paper.

RESUME: Malgré la grande surface couverte par roches sédimentaires tendres (résistance à la compression uniaxial jusqu'à 25 MPa) au Brésil, et des nombreuses oeuvres de la Génie Civil installées ou à être installées sur ces roches, on sait peu sur ses caractéristiques physiques, chimiques, minéralogiques et texturales et sur les mécanismes mis en rapport avec le procès d'alteration. Ce travail est une tentative d'identifier les proces responsables pour l'alteration et l'expansion de quelques roches sédimentaires du bassin sédimentaire du Recôncavo (Bahia/Nord-est du Brésil), à travers une vaste campagne d'essais de laboratoire, en couvant la caractérisation de base et l'alterabilité de ces roches. Les résultats obtenus permettent d'atteindre quelques conclusions, présentées dans la partie final du travail.

1 INTRODUCTION

The occurrence of weatherability and alteration processes in sedimentary weak rocks, has been reasonably studied in recent years (Olivier, 1990; Taylor & Smith, 1986; Seedsman, 1986; among others), with significant progress related to the possible alteration mechanisms of chemical, physical and/or physical-chemical nature (Taylor & Smith, op. cit. and Taylor, 1988).

Sedimentary weak rocks (shales, siltstones and mudstones) have a widespread occurrence in the Recôncavo Sedimentary Basin (Northeast Brazil, Figure 1 - Milani, 1987), present in several Cretaceous units of this basin. These rocks have, as a geotechnical common characteristic, a high sensitivity to variation of moisture that causes its swelling/shrinking and strength reduction and big deformations as a consequence (Simões, 1986). These volume changes can be related to shrinking and are difficult to control.
Slope stability and foundations problems are often noticed to have been caused by alteration and weatherability of these weak rocks (Simões, 1987). In order to characterize such materials, a wide laboratory program involving chemical, physical, mineralogical and geotechnical tests, in addition to field observation, was carried out. Results obtained are presented in this paper, that is part of a wide research project on the weatherability of Argillaceous Rocks in Brazil.

2 TYPICAL WEATHERING PROFILE

The typical weathering profile of the sedimentary weak rocks of the Recôncavo Basin is similar to that presented in Dobereiner et al. (1990) for argillaceous rocks, although small differences due to lithology and rock moisture, produce considerable changes in the thickness of these profiles. The development of such profiles are limited in depth by the water table position. Below it, because the rock is permanently saturated, weathering doesn't occur. Variation in moisture in the capillary zone allows the weathering processes to occur moderately. Above the capillary zone, because of the great variations in moisture, weathering processes increase, causing an intense fracturing and a strength reduction of rock mass.

At the surface a 0.50 - 1.00 m thick layer of "tablet" like material (Figure 2), is formed by the fluctuation of moisture due to cycles of wet and dry periods. Apparently this layer acts as a natural protection against the development of the weathering process, and being removed, the process is reactivated, originating a new layer.

The morphology of the profile can be modified if sandstone layers are interbedded with the argillaceous rocks, as those rocks have a high strength to weathering and because its stiffness, they may fracture, creating unstable blocks.
3 MINERALOGICAL CHARACTERIZATION AND INDEX PROPERTIES

3.1 Mineralogical characterization

Differential Thermal Analysis, Thermal Gravimetric Analysis and X-ray Diffraction gave an overall view of the mineralogy of silstones and shales studied. Table 1 to 3 shows the mineral composition of such rocks, and it can be seen that samples 1, 2 and 5 do have some minerals that can cause swelling and shrinking (montmorillonite and illite). The calcite and dolomite present in the samples 1, 2 and 5 aren't disseminated as a cement and can't cause shrinking.

Table 1. DTA results

<table>
<thead>
<tr>
<th></th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Sample 5</th>
<th>Sample 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montmorillonite</td>
<td>107.6</td>
<td>105.0-704.8</td>
<td>-----</td>
<td>137.9-810.4</td>
<td>-----</td>
</tr>
<tr>
<td>Palygorskite</td>
<td>329.8</td>
<td>332.5</td>
<td>333.9</td>
<td>351.6</td>
<td>332.8</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>-----</td>
<td>-----</td>
<td>124.9-398.5</td>
<td>-----</td>
<td>131.3</td>
</tr>
<tr>
<td>Quartz</td>
<td>575.6</td>
<td>572.7</td>
<td>576.6</td>
<td>566.6</td>
<td>-----</td>
</tr>
<tr>
<td>Siderite</td>
<td>-----</td>
<td>-----</td>
<td>746.6 (?)</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Dolomite</td>
<td>758.9-885.7</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Chlorite</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Calcite</td>
<td>-----</td>
<td>809.3</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
</tbody>
</table>

Obs.: Peak Maximum temperatures (°C), DTA tests realized with air atmosphere.

Table 2. TG results

<table>
<thead>
<tr>
<th></th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Sample 5</th>
<th>Sample 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Temperature</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water (T&lt;400°C)</td>
<td>7.13 %</td>
<td>6.27 %</td>
<td>4.47 %</td>
<td>7.28 %</td>
<td>9.12 %</td>
</tr>
<tr>
<td>Hydroxiles</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water (400°C&gt; T&gt;600°C)</td>
<td>2.94 %</td>
<td>1.70 %</td>
<td>4.83 %</td>
<td>5.13 %</td>
<td>5.42 %</td>
</tr>
<tr>
<td>High Temperature</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water (T&gt;600°C)</td>
<td>0.08 %</td>
<td>1.83 %</td>
<td>0.40 %</td>
<td>2.36 %</td>
<td>0.43 %</td>
</tr>
<tr>
<td>Estimated CO₂</td>
<td>14.50 %</td>
<td>1.85 %</td>
<td>0.36 %</td>
<td>2.09 %</td>
<td>0.00 %</td>
</tr>
<tr>
<td>from TG Curve</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Lost to</td>
<td>24.65 %</td>
<td>11.65 %</td>
<td>10.06 %</td>
<td>16.86 %</td>
<td>14.97 %</td>
</tr>
<tr>
<td>Fire at 1080°C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Estimated CO₂</td>
<td>4.05 %</td>
<td>1.85 %</td>
<td>0.00 %</td>
<td>1.03 %</td>
<td>0.00 %</td>
</tr>
<tr>
<td>from CaCO₃</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Estimated CO₂</td>
<td>10.45 %</td>
<td>0.00 %</td>
<td>0.00 %</td>
<td>1.06 %</td>
<td>0.00 %</td>
</tr>
<tr>
<td>from MgCO₃</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Estimated CO₂</td>
<td>0.00 %</td>
<td>0.00 %</td>
<td>0.36 %</td>
<td>0.00 %</td>
<td>0.00 %</td>
</tr>
<tr>
<td>from FeCO₃</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Obs.: TG tests realized with air atmosphere.
Table 3 - DR-X results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Sample 5</th>
<th>Sample 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anatase-Rutile</td>
<td>0.54</td>
<td>0.65</td>
<td>0.56</td>
<td>0.57</td>
<td>0.64</td>
</tr>
<tr>
<td>Siderose</td>
<td>0.00</td>
<td>0.00</td>
<td>9.12</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Gypsum</td>
<td>0.00</td>
<td>0.00</td>
<td>0.13</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Dolomite</td>
<td>21.28</td>
<td>6.02</td>
<td>0.00</td>
<td>2.20</td>
<td>0.00</td>
</tr>
<tr>
<td>Calcite</td>
<td>9.23</td>
<td>5.32</td>
<td>0.00</td>
<td>2.36</td>
<td>0.00</td>
</tr>
<tr>
<td>Biotite (Mg)</td>
<td>5.91</td>
<td>4.54</td>
<td>4.52</td>
<td>9.18</td>
<td>0.00</td>
</tr>
<tr>
<td>Biotite (Fe)</td>
<td>2.13</td>
<td>6.66</td>
<td>0.23</td>
<td>0.00</td>
<td>7.00</td>
</tr>
<tr>
<td>Palygorskite</td>
<td>0.70 (27.00)</td>
<td>5.35 (0.00)</td>
<td>1.41 (30.00)</td>
<td>0.00 (0.00)</td>
<td>13.25 (-----)</td>
</tr>
<tr>
<td>Muscovite</td>
<td>15.16 (58.00)</td>
<td>4.23 (0.00)</td>
<td>6.19 (44.00)</td>
<td>22.80 (67.00)</td>
<td>25.40 (-----)</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>1.93</td>
<td>4.23</td>
<td>18.62</td>
<td>0.00</td>
<td>1.05</td>
</tr>
<tr>
<td>Albite</td>
<td>4.23</td>
<td>8.54</td>
<td>2.29</td>
<td>5.58</td>
<td>0.00</td>
</tr>
<tr>
<td>Anorthite</td>
<td>2.98</td>
<td>5.15</td>
<td>1.24</td>
<td>5.96</td>
<td>0.00</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>0.00 (0.00)</td>
<td>5.38 (11.00)</td>
<td>4.92 (16.00)</td>
<td>2.50 (0.00)</td>
<td>19.27 (-----)</td>
</tr>
<tr>
<td>Chlorite</td>
<td>0.00 (9.00)</td>
<td>0.00 (0.00)</td>
<td>15.13 (10.00)</td>
<td>4.24 (14.00)</td>
<td>0.00 (-----)</td>
</tr>
<tr>
<td>Nontronite</td>
<td>9.63</td>
<td>0.00</td>
<td>0.00</td>
<td>10.70</td>
<td>0.00</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>0.96 (6.00)</td>
<td>0.00 (39.00)</td>
<td>0.00 (0.00)</td>
<td>0.00 (19.00)</td>
<td>0.00 (-----)</td>
</tr>
<tr>
<td>Quartz</td>
<td>15.76</td>
<td>37.15</td>
<td>29.96</td>
<td>20.40</td>
<td>23.92</td>
</tr>
<tr>
<td>Low Temp. H₂O</td>
<td>1.54</td>
<td>6.27</td>
<td>2.67</td>
<td>6.85</td>
<td>6.86</td>
</tr>
<tr>
<td>High Temp. H₂O</td>
<td>7.01</td>
<td>0.86</td>
<td>0.06</td>
<td>3.91</td>
<td>1.48</td>
</tr>
</tbody>
</table>

Obs.: (1) Percentage values;
(2) The values between parenthesis gives the percentual of each mineral in clay fraction;
(3) The values between parenthesis related to muscovite refer to the micaceous of clay fraction.

In order to analyze the influence of the fabric in the development of weathering and alteration processes, Backscatter Electron Image (B.E.I.) and Scattered Electronic Microscope (S.E.M.) were used to obtain images of the texture and mineralogy, shown in the photos 1 to 10, presented at the end of this paper.

3.2 Index properties

The following index properties have been determined for the rocks under study: porosity, dry density, absorption, porosimetry, slake durability and wet and dry oven cycling.

Table 4. Porosity, dry density and absorption results

<table>
<thead>
<tr>
<th></th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Sample 5</th>
<th>Sample 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity (%)</td>
<td>26.12</td>
<td>22.69</td>
<td>35.90</td>
<td>37.11</td>
<td>51.91</td>
</tr>
<tr>
<td>Dry density (g/cm³)</td>
<td>2.051</td>
<td>2.089</td>
<td>1.778</td>
<td>1.601</td>
<td>1.333</td>
</tr>
<tr>
<td>Absorption (%)</td>
<td>12.91</td>
<td>10.91</td>
<td>20.00</td>
<td>23.20</td>
<td>40.67</td>
</tr>
</tbody>
</table>

Table 4 shows the results obtained for porosity, dry density and absorption. It can be seen that all rocks studied present very high values of porosity and absorption, increased by microfissuring.

Figure 3 (a-e) shows the porosimetry values
and pores diameter distribution from porosimetry analysis for each sample. Lower the pores' diameters are, higher are the negative pore pressures that can be developed during wetting and drying cycles, causing the shrinking of the samples and the improvement of the weathering processes. Samples 1, 5 and 6 have a concentration of their pores (over 50%) under 200 Å (diameter), which suggest that high negative pore pressures may develop during wetting and drying cycles. Sample 2 shows a better graded pore distribution and, finally, sample 3 has a concentration between 3.000 Å and 18.000 Å (diameter).

(a) Sample 1  
(b) Sample 2

(c) Sample 3
Fig. 3 (a-e) - Porosimetry values of the rocks studied

The results of slake durability test (table 5) show some influence of the initial moisture over the disaggregation of these shales and siltstones. It appears that the higher the initial moisture, the higher is the strength to wetting and drying cycles. It may also be seen that the lower the moisture content gets to the field moisture content at sampling, the higher the strength to weathering. Figure 4 shows a comparison (just for sample 3) between a slake durability test using water against a slake using ethylene glycol as an immersion fluid. The result shows a higher strength of samples in the test with ethylene glycol. This behavior could be explained in two ways: The influence of the clay minerals in the weathering of the sample could be small or otherwise the ethylene glycol is too viscous to penetrate into the pores of the specimens.

Table 5. Sake durability test results

<table>
<thead>
<tr>
<th>Initial moisture content (%)</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Sample 5</th>
<th>Sample 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>3.6</td>
<td>4.8</td>
<td>0.0</td>
<td>2.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Idp (%)</td>
<td>63.2</td>
<td>95.8</td>
<td>74.5</td>
<td>63.3</td>
<td>82.9</td>
</tr>
</tbody>
</table>

Fig. 4 - Slake durability with ethylene glycol X slake durability with water
The same behavior observed in the slake durability tests (Table 5) were observed in the wet and dry oven cycling of sample 3: the higher values of initial moisture gave the highest strength to wetting and drying cycles (Table 6), in different sizes of samples.

The results from the slake durability and wet and dry oven cycling tests show that the samples studied shrink a lot when submitted to wetting and drying cycles.

Table 6 - Wet and dry oven cycling test results

<table>
<thead>
<tr>
<th>Initial Fragments Size</th>
<th>$\phi &gt; 2''$</th>
<th>$2'' &gt; \phi &gt; 1''$</th>
<th>$1'' &gt; \phi &gt; 1/2''$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content (%)</td>
<td>1.72</td>
<td>3.65</td>
<td>8.96</td>
</tr>
<tr>
<td>% Retained (*)</td>
<td>100</td>
<td>70</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>1.72</td>
<td>3.65</td>
<td>8.96</td>
</tr>
<tr>
<td></td>
<td>94</td>
<td>43</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>1.72</td>
<td>3.65</td>
<td>8.96</td>
</tr>
<tr>
<td></td>
<td>57</td>
<td>33</td>
<td>12</td>
</tr>
</tbody>
</table>

(*) The % retained indicates the percentual of original material that continue to have the same size after the test.

Photo 1 - Well-formed calcite/dolomite grains in an argillaceous matrix. No mineral preferred orientation can be noted. It is possible to observe a "honey comb" structure formed by clay minerals (S.E.M., sample 1)

Photo 2 - Calcite and dolomite grains in argillaceous matrix. Small and scattered pores can be seen. No weathering or shrinking observed (B.E.I., sample 1)
Photo 3 - Quartz, feldspar, iron hydroxide and mica in an argillaceous matrix. No preferred orientation of grains is noted (S.E.M., sample 2)

Photo 5 - Sharped quartz and feldspar grains, involved in an argillaceous matrix with mica and without a preferred mineral orientation (S.E.M., sample 3)

Photo 4 - Irregular microfracturing around various sizes and composition grains (B.E.I., sample 2)

Photo 6 - Silt and clay particles predominant and few crystals. Many microfissures presents, some of than produced during thin section preparation (B.E.I., sample 3)
Photo 7 - Well-formed calcite/dolomite crystals and quartz grains in an argillaceous matrix. The rock is almost completely formed by silt and clay particles. No preferred mineral orientation noted. (S.E.M., sample 5)

Photo 9 - This rock is almost completely formed by clay minerals with some quartz and mica grains. The clay minerals are clearly oriented (S.E.M., sample 6)

Photo 8 - Plentiful argillaceous matrix involving small dolomite/calcite crystals. The fissures show a preferred orientation (B.E.I., sample 5)

Photo 10 - Preferred orientation is given by clay minerals. A great number of fissures are present (B.E.I., sample 6)

4 CONCLUSIONS

The following conclusions can be drawn from the present state of the investigation on weatherability of argillaceous rocks from the Recôncavo sedimentary basin - Brazil:

- the mineralogical characterization is important to identify the presence of expansible clay minerals or carbonatic cement or any other substance that can lead rocks to swelling and shrinking;

- the samples 1, 2 and 5 have some minerals (montmorillonite and illite) that can expand, causing the shrinking of these rocks;

- porosimeter analysis showed that samples 1, 5 and 6 have a great part of its pores under 200 Å diameter, which may cause high negative pore pressures to be developed during wetting and drying cycles;

- the skake durability test and the wet and dry oven cycling have shown to be a simple and reliable way to establish the degree of
weathering of the rocks under study;
- weathering of argillaceous rocks from the Recôncavo basin seems to show a predominance of physical rather than chemical processes in the breakage of these rocks.

5 - REFERENCES


Relationships between quantitative microstructure parameters and properties of chosen clay soil

Dépendences entre les paramètres quantitatifs de la microstructure et les propriétés du sol argileux

Robert Czajka
Institute of Hydrogeology & Engineering Geology Faculty of Geology Warsaw University Poland

ABSTRACT: The article presents quantitative and qualitative SEM Microstructural Analysis of glacilacustrine clays from South Baltic Coast in Poland (Quaternary - North-Polish Glaciation-Vistulian). The results of the analysis have enabled to determine the main types of microstructure of clays and their quantitative microstructural parameters. Experimental investigations have been made including: quantitative RTG analysis of the mineral composition of clays and analysis of physical and deformability parameters. The quantitative description of the soil parameters has been made with the application of statistical computer analysis. Interrelations between microstructure, mineral composition, physical and deformability parameters are presented. The results should be useful in preliminary assessment and the introduction to the possibility prognosis of properties of the soil with the use of SEM analysis.


1. INTRODUCTION

The microstructure (prefix "micro" refers here to SEM microscopical analysis - Grabowska-Olszewska 1984) of clay soils has been defined as a spatial system of respective microstructural elements (particle, aggregate, microaggregate or granular), which are characterized by quantitative ratio of microstructural elements, its mutual influences (interparticle forces) and morphological, geometrical, physical and other properties. The microstructure reflects genesis and geological history of soils and determines its physical and mechanical properties.

The quantitative and qualitative microstructural SEM analysis (Smart 1973, Grabowska-Olszewska 1984, Love 1985, Tovey 1981, 1982, Osipov 1989, Czajka 1992) of chosen clay soil (glacilacustrine clays) from: South Baltic cliffs (Ustka-Orzechowa, Podąbie, Dębina, Jastrzębia Góra, Rzucewo, Osionino) were made with application of the system SEM Hitachi S-800. Also the STIMAN (ver. 2.05) program was applied to the processing of SEM data (Osipov 1989, Sokolov 1990).

2. MINERALOGICAL COMPOSITION

The mineralogical composition of glacilacustrine clays from South Baltic Coast was described by the
same analytical methods: X-Ray Quantitative Analysis (Drits 1976, Reynolds 1980) and in the same laboratory.

The general mineralogical composition of the clays in all places can be characterized as follows: carbonates 7.3 - 32.2 % (calcite, dolomite), zeolites 0.2 - 4.3 % (heulandite), quartz 35.4 - 76.1 %, feldspars 8.4 - 23.7 % (microcline, plagioclase), clay minerals 7.4 - 28.9 % (illite, chlorite, kaolinite, smectite) and other minerals 0.1 - 3.8 % (pyrite, hornblende, siderite, gypsum, goethite).

The mineralogical composition of clay fraction of the investigated soil is characterized mainly by total contents of illite and smectite (more than 70 %).

3. PHYSICAL AND DEFORMABILITY PARAMETERS

The physical properties (Czajka 1991, 1993) can be characterized as follows: grain size analysis shows that this soil is a typical clay (65.4 %) silty clay (19.4 %), sandy clay (0.4 %), clay loam (7.2 %) and sandy clay loam (7.6 %). The silt fraction content varies 13.0 - 77.8 % and clay fraction content 18.0 - 65.0 %. The value of water content is 17.0 - 40.2 %. Analyzing the properties describing the condensation of the soil (density and porosity) a slight compaction is stated which is expressed by bulk density (in the range 1.64 - 2.06 Kgm/m$^3$ x 10$^3$), small value of bulk density of soil skeleton (1.26 - 1.70 Kgm/m$^3$ x 10$^3$), high value of porosity (0.37 - 0.54) and void ratio (0.58 - 1.20). The soil state variables can be described as hard plastic/plastic with plasticity index (average value) from 18.3 % to 38.4 %, the range average value of liquidity index changes between -0.11 - 0.20, the Skempton activity values (0.45 - 1.16) lead to a conclusion about average activity and high hydrofiliity (0.76 - 2.38) of the investigated soil.

The laboratory testing of the glacialacustrine clays in the uniaxial state of deformation was made using the consolidometer for the intervals of pressure: 50 - 100 kPa, 100 - 200 kPa, 200 - 400 kPa. The compressibility modulus for the pressure ranging 100 - 200 kPa is with in the limits 4.4 - 14.8 MPa and for the range 200 - 400 kPa changes between 8.6 - 20.9 MPa. The consolidation coefficient is within limits for the range of pressure 100 - 200 kPa from 0.16 to 0.72 x 10$^3$ m$^2$/s; 200 - 400 kPa from 0.22 to 0.82 x 10$^3$ m$^2$/s. The value of coefficient of consolidation varies in the range of pressure 100 - 200 kPa 1.16 - 4.81 x 10$^4$ 1/kPa and in the range of pressure 200 - 400 kPa 0.82 - 2.22 x 10$^4$ 1/kPa. These parameters lead to a conclusion about variability of deformability of glacialacustrine clays.

4. QUANTITATIVE SEM ANALYSIS

The main qualitative type of microstructure of glacialacustrine clays from South Baltic Coast is determined as matrix-laminar or honey-comb matrix-laminar. The matrix-laminar type differs from the typical laminar microstructure by the presence of matrix microstructure at local sites (Grabowska-Olszewska 1984).

The results of SEM analysis (about 50 samples) have enabled to determine microstructural elements of pore space: number of pores in the range 18.5 - 63.3 x 10$^3$, porosity value is 0.36 - 0.46, total area of pores (from 7.5 x 10$^3$ to 9.5 x 10$^3$ um$^2$), total perimeter of pores (changes between 37.2 - 98.8 x 10$^3$ um), average diameter of pores values 0.177 - 0.258 um, average area of pores values 0.145 - 0.503 um$^2$, average perimeter of pores values 1.369 - 2.330 um and specific area values changes from 1.201 to 3.189 1/um. The analysis of distribution according to equivalent diameters leads to the conclusion about quantitative differentiation of pore kinds: ultra (diameter < 0.1 um), micro (0.1 um < diameter < 10 um), mezo (10 um < diameter < 100 um) and absence of macropores in the investigated samples of the soil. The typical dominating range of equivalent diameters for glacialacustrine clays is 0.6-0.10 um. Form index (F) describes the morphological types of pores: isometric (F>0.66), anisometric and fissure-like (F<0.10). We can confirm quantitative differentiation of isometric and anisometric pores, for example in glacialacustrine clays from Jastrze-
bía Góra (76.6%), Ustka-Orzechowo (83.5%), Podgąbie (78.6%), Deblina (82.3%) and Ruzewo (83.3%) anisometric pores dominate, but in the samples from Osolinowo isometric ones (63.4%) prevail. The fissure-like pores are present (maximum of content 2.5% for clays from Jastrzębia Góra, minimum of content 0.1% for Osolinowo); it is typical and very characteristic for the investigated soil (Czałka 1992, 1994).

The obtained values of Ka coefficient (anisotropy coefficient of orientation of microstructural elements) confirm: high anisotropy of structural elements (Ka>22% Sokolov 1990) for clays from Ruzewo (28.3% average value), Jastrzębia Góra (42.3%); medium anisotropy of structural elements (7%<Ka<22% - Sokolov 1990) for clays from Podgąbie (7.4-19%), Deblina (12%), Osolinowo (14.3%). The clays from Ustka-Orzechowo are characterized by the biggest variability of values of Ka coefficient from 3.1% to 20.2%. This confirms low (Ka<7% - Sokolov 1990) or medium anisotropy of structural elements.

5. REGRESSION ANALYSIS

The types of microstructure are formed in close connection with clay mineral accumulation, as well as subsequent compaction and other processes during lithogenesis. Hence, there is a close interrelation between microstructure (microstructural parameters) and the origin (mineralogical composition parameters) and degree of lithification (physical and deformability parameters) of the soil.

The regression and correlation analysis between investigated soil parameters (microstructures, physical, deformability, mineral composition) has been made with the application of the STATGRAPH (ver. 6.0 plus) program. The table 1 presents the results of regression analysis between microstructural parameters and mineralogical composition of glacialicustrine clays (samples 22, correlation coefficient value higher than /0.700/, significance level 0.05, confidence limits 95.0%, prediction limits 95.0%). It is very characteristic that the microstructural parameters (total pore area, total pore perimeter, specific area of pores) and relative contents of calcite, quartz and smectite have participated in the relationships.

The figure 1 presents some dependencies between anisotropy coefficient of microstructural elements and mineralogical composition (calcite, quartz, chlorite contents). The mathematical models of these relationships is the form y=exp(a+bx).

The table 2 presents the results of regression analysis between microstructural elements and physical properties of glacialicustrine clays. The silt fraction content, clay fraction content, bulk density (physical properties) have strong participation in these relationships. The set of microstructural parameters is the same with the parameters in the relations described earlier (between microstructural parameters and mineralogical composition). The dependence between anisotropy coefficient and sand fraction content or clay fraction content is shown at the figure 2. The relations with sand fraction content have mathematical form: y=exp(a+bx) (this is the same with the mathematical model in the regression analysis between anisotropy coefficient and mineralogical composition). The dependence with clay fraction content have the mathematical model y=ax^a. The

<table>
<thead>
<tr>
<th>Relationship</th>
<th>Model</th>
<th>Values</th>
<th>Correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>total pore area / calcite content</td>
<td>y=a+bx</td>
<td>-0.184, 1.891</td>
<td>0.703</td>
</tr>
<tr>
<td>total pore area / quartz content</td>
<td>y=exp(a+bx)</td>
<td>9.583, -0.011</td>
<td>-0.701</td>
</tr>
<tr>
<td>total pore area / smectite content</td>
<td>y=a+bx</td>
<td>2.087, 1.854</td>
<td>0.704</td>
</tr>
<tr>
<td>total pore area / illite content</td>
<td>y=a+bx</td>
<td>3.053, 1.957</td>
<td>-0.724</td>
</tr>
<tr>
<td>total pore perimeter / quartz content</td>
<td>y=exp(a+bx)</td>
<td>12.992, -0.043</td>
<td>-0.720</td>
</tr>
<tr>
<td>specific area of pores / quartz content</td>
<td>y=exp(a+bx)</td>
<td>-3.010, -0.054</td>
<td>-0.743</td>
</tr>
<tr>
<td>specific area of pores / smectite content</td>
<td>y=ax^a</td>
<td>0.874, 0.273</td>
<td>0.724</td>
</tr>
</tbody>
</table>
Fig. 1 The relationships between anisotropy coefficient and mineralogical composition on the example of calcite, quartz and chlorite contents.

Table 2 The results of regression analysis between microstructural parameters and physical properties of glacial lacustrine clays.

<table>
<thead>
<tr>
<th>Relationship</th>
<th>Model</th>
<th>Values</th>
<th>Correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total pore area / silt fraction content</td>
<td>y=a+bx</td>
<td>2.195</td>
<td>-1.637, 0.765</td>
</tr>
<tr>
<td>Total pore area / clay fraction content</td>
<td>y=a+bx</td>
<td>-2.304</td>
<td>0.982, -0.675</td>
</tr>
<tr>
<td>Average pore area / bulk density</td>
<td>y=ax+bx</td>
<td>-1.592</td>
<td>1.735, -0.754</td>
</tr>
<tr>
<td>Average perimeter of pores / bulk density</td>
<td>y=exp(a+bx)</td>
<td>1.735</td>
<td>-1.562, -0.716</td>
</tr>
<tr>
<td>Form index / bulk density</td>
<td>y=ax</td>
<td>1.192</td>
<td>-2.254, -0.777</td>
</tr>
<tr>
<td>Anisotropy coefficient / bulk density</td>
<td>y=ax+bx</td>
<td>1.924</td>
<td>1.587, -0.692</td>
</tr>
</tbody>
</table>

Fig. 2 The dependences between anisotropy coefficient and physical properties on the examples of sand and clay fraction content.

Fig. 3 The relations between form index of pores and compressibility coefficient and consolidation coefficient.

relationships between microstructural parameters (form index, total area of pores) and deformability parameters (consolidation and compressibility coefficients) are presented at the figure 3. The
the mathematical model of relations between form index and compressibility coefficient (for interval pressure 50–100 kPa) have form \( y = a + bx \) with the correlation coefficient \( R = 0.734 \), but in the relations with consolidation coefficient have form: \( y = ax^2 \) with correlation coefficient \( R = -0.727 \). The relationship between total pore area and preconsolidation pressure is presented at the figure 4. This relation has linear mathematical model.

Fig. 4 The relationship between total pore area and preconsolidation pressure.

6. CONCLUSIONS

The set of the results of the research describes the origin of clays (during lithogenesis and their geological history) and clay minerals accumulation by quantitative and qualitative SEM analysis (determined type of microstructure, microstructural parameters, distributions of diameters of pores, morphological types of pores, orientation of microstructural elements of pore space).

In addition, regularities indicate interrelations between microstructure (microstructure elements) and mineralogical composition and physical and deformability properties. These investigations are significant as they are reliable evaluation of parameters needed for forming mathematical models and making adequate engineering-geological or geotechnical calculations.

This is the first step to prognosis of properties of clay soils by the way of SEM analysis. The results of the investigations allow to make proper estimation of the physical and mechanical properties of clay soil.

7. ACKNOWLEDGEMENTS

This research was supported by the Committee of Scientific Investigation of Poland, Department of Research, Section No. 5.6.2., under Grant No. 9 9224 92 03. The author would like to express his gratitude for their support.

The author also wishes to thanks Dr. V.N Sokolov (Moscow State University, Faculty of Geology), as his help in SEM laboratory investigation.

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Assessing the roughness of rock discontinuities using fractal techniques
Évaluation de la rugosité des fissures rocheuses en utilisant des techniques fractales

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ABSTRACT: The roughness of joint surfaces has a marked influence on the shear strength, permeability and deformation behaviour of rock masses. The joint roughness may be estimated directly from asperity angle measurements, analysis of joint surface profiles or the back analysis of direct shear tests on joints. Fractal geometry may also be used to assess the deviation of the surface profile from a smooth line or surface of Euclidean space. The determination of the fractal dimension (D) of standard roughness profiles and their correlation to the Joint Roughness Coefficient (JRC) provides the following empirical relationship:

\[ JRC = -85899.4536 + (169183.9214 \times D) - (83285.0820 \times D) \times D \]

RÉSUMÉ: La dureté de la surface des diagnostiques a une influence marquée sur la résistance de la fissure, la permeabilité et la déformation des masses rocheuses. La dureté de la diagnostique peut être évaluée directement par les dimensions de l’angle d’asperité, par l’analyse des profils de la surface de la diagnostic ou par analyse des tests de cisaillement sur les diagnostiques. On peut utiliser la géométrie fractale pour estimer la déviation de la surface du profil en partant d’une ligne régulière ou de la surface de l’espace euclidien. La détermination de la dimension fractale (D) des profils standard de dureté et leur corrélation avec le coefficient de dureté de la diagnostic représente le rapport empirique suivant:

\[ JRC = -85899.4536 + (169183.9214 \times D) - (83285.0820 \times D) \times D \]

1 INTRODUCTION

Approximate estimates of the roughness of rock surfaces may be obtained by recording their profiles and visually comparing them with the standard profiles (Fig. 1) presented by Barton and Choubey (1977). This is a subjective method since it is based on a visual examination alone. Barton and Choubey (1977) derived an empirical law based on a number of laboratory shear tests on rock joints:

\[ JRC = \frac{\arctan\left(\frac{r}{\sigma_{N}}\right) - \phi_{b}}{\log_{10}\left(\frac{JCS}{\sigma_{N}}\right)} \]

(1)

where JRC is the joint roughness coefficient, τ is the shear stress, σN is the effective normal stress, Φb is the basic friction angle and JCS is the joint wall compressive strength. Barton and Choubey then assigned JRC values determined from equation 1 to each of the profiles in Fig. 1, allowing an estimate of JRC to be determined from surface profiles.
In recent years, a number of researchers have applied fractal geometry to the description of rock discontinuity surfaces (e.g., Carr and Warriner, 1987; Turk et al., 1987; Lee et al., 1990; Sakellariou et al., 1991). Fractal geometry, introduced by Mandelbrot (1967), allows the description of irregular forms which are more complex than standard Euclidean shapes, where a perfectly straight line is a one dimensional feature, and an ideal plane is a 2-D feature, and an ideal sphere is a 3-D feature. Fractal or fractional geometry allows the deviation from a smooth line or surface to be determined. Thus, a cross section through topographic relief, or discontinuity roughness profile has a fractal dimension between 1 and 2, while the fractal dimension of a surface lies between 2 and 3. An important property of some fractals is that they are self-similar; that is, the visual appearance and statistical properties of the surface are similar at all scales of magnification. Mandelbrot (1985) implies that vertical cross sections of relief (such as the roughness profile of a rock discontinuity) are self-affine; that is, they remain statistically similar only if they are scaled differently in different directions. The fractal dimension D of such fractals has a value between the topological dimensions of 1 and 2, with values of 1.0 - 1.5 commonly obtained from rock fractures (McWilliams, et al., 1990).

2 THE FRACTAL DIMENSION

Consider the length of the coastline of Great Britain; suppose we wanted to measure the distance between two points on the west coast of Britain. Using a map, we could measure the distance by stepping a pair of dividers of arbitrary spacing along the coast between the two points and multiplying the divider spacing r by the number N of steps. The process is then repeated using the divider set to smaller spacings, and it is found that
where $D$ is the fractal dimension, $r$ is the divider spacing, and $N$ is the number of divider steps required to measure the distance between the two points (Mandelbrot, 1985). If $\log_{10} r$ is plotted against $\log_{10} N$, a straight line with a negative slope is produced. The fractal dimension $D$ is equal to the value of the slope. Fig. 2 shows the fractal dimension of a number of coastlines.

3 FRAC TAL DIMENSION DETERMINATION

The standard roughness profiles shown in Fig. 1 were enlarged to the scale defined by Barton and Choabey (1977) and the $y$-coordinate digitized at a constant $x$-coordinate interval of 0.5mm. To ensure that there were no errors in the digitizing process, the digitized profiles were replotted on a plotter and compared with the original roughness profiles. If any difference was noted the profile was re-digitized until the difference disappeared.

A computer program was written to read the digitized profiles and determine the fractal dimension by calculating the length of each profile, using sampling steps between 1 and 10mm. The data for each profile was plotted as a log - log plot,

- $D = \frac{\log N}{\log r(N)}$ (4)

the measured distance between the two points increases as the divider spacing is reduced. The smaller divider spacing is able to sample the smaller bays and inlets that the larger spacing could not. The relationship between the total length and the divider spacing can be expressed as:

![Image](image_url)  

Fig. 3. Relationship between fractal dimension and JRC value
Table II Summary of results of previous workers

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>1.00064</td>
<td>1.0000</td>
<td>1.000446</td>
<td>1.0003</td>
</tr>
<tr>
<td>2-4</td>
<td>1.00126</td>
<td>1.0019</td>
<td>1.001687</td>
<td>1.0009</td>
</tr>
<tr>
<td>4-6</td>
<td>1.00215</td>
<td>1.0027</td>
<td>1.002805</td>
<td>1.0013</td>
</tr>
<tr>
<td>6-8</td>
<td>1.00432</td>
<td>1.0049</td>
<td>1.003974</td>
<td>1.0032</td>
</tr>
<tr>
<td>8-10</td>
<td>1.00438</td>
<td>1.0054</td>
<td>1.004413</td>
<td>1.0032</td>
</tr>
<tr>
<td>10-12</td>
<td>1.00483</td>
<td>1.0045</td>
<td>1.005641</td>
<td>1.0042</td>
</tr>
<tr>
<td>12-14</td>
<td>1.00700</td>
<td>1.0077</td>
<td>1.007109</td>
<td>1.0059</td>
</tr>
<tr>
<td>14-16</td>
<td>1.00767</td>
<td>1.0070</td>
<td>1.008055</td>
<td>1.0081</td>
</tr>
<tr>
<td>16-18</td>
<td>1.00875</td>
<td>1.0104</td>
<td>1.009584</td>
<td>1.0074</td>
</tr>
<tr>
<td>18-20</td>
<td>1.01318</td>
<td>1.0170</td>
<td>1.013435</td>
<td>1.0123</td>
</tr>
</tbody>
</table>

and the slope or fractal dimension determined. In each case a straight line could be fitted to the data, with a correlation coefficient in excess of 0.95. As can be seen from Table I the smoother profiles have low values of D, and as the roughness of the profile increases, so does the fractal dimension. A plot of fractal dimension versus JRC range is presented in Fig. 3. Because each of the standard roughness profiles represented in Fig. 1 correspond to a range of JRC values, the graph can only be accurate to ± 1 JRC value.

Table II and Fig. 4 present the results of other researchers who have carried out similar work. Turk et al., (1987) determined their relationship manually, using a pair of dividers on the enlarged roughness profiles. The studies by Lee et al., (1990) and Sakellariou et al., (1991) used computer programs to determine the fractal dimensions for each of the profiles. There is excellent consistency between the findings of each of these researchers and this study.

The data from Table II was input into a curve fitting program, and in each case an equation of the following form gave the best correlation coefficient.

\[ JRC = A + (B \times D) + (C \times D) \times D \]  \hspace{1cm} (5)

where A, B and C are the curve fitting coefficients and D is the fractal dimension. Table III presents the equation coefficients for the curves plotted in Fig. 4.

Fig. 4. Comparison of relationships between fractal dimension and JRC value, derived by a number of authors
Table III Results of curve fitting

<table>
<thead>
<tr>
<th></th>
<th>Coefficient A</th>
<th>Coefficient B</th>
<th>Coefficient C</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>This study</td>
<td>-85899.4536</td>
<td>169183.9214</td>
<td>-83285.0820</td>
<td>0.9698</td>
</tr>
<tr>
<td>Turk et al., (1987)</td>
<td>-71279.3177</td>
<td>140242.0841</td>
<td>-68962.9438</td>
<td>0.9160</td>
</tr>
<tr>
<td>Lee et al., (1990)</td>
<td>-77171.6136</td>
<td>151830.2085</td>
<td>-74659.4676</td>
<td>0.9894</td>
</tr>
<tr>
<td>Sakellariou et al., (1991)</td>
<td>-108596.5239</td>
<td>214404.7148</td>
<td>-105807.5727</td>
<td>0.9692</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

Because the method of estimating the JRC value of a discontinuity surface is subjective, fractal geometry has been used to quantify surface roughness. The fractal dimension has been found to be directly proportional to surface roughness, with a polynomial equation being fitted to the data to provide an empirical relationship. This relationship is in agreement with the findings of other researchers in the field.

REFERENCES


Some effects of nitrates on the properties of clay
Quelques effets des nitrates sur les propriétés de l’argile

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ABSTRACT: The effects of several inorganic electrolytes on the engineering properties of a remoulded glacio-lacustrine clay from the North East of England were investigated. A range of tests were carried out to ascertain some of the more common engineering properties of the clay and how these were affected by the addition of electrolytes. This was done by using the nitrate anion together with four cations, magnesium, sodium, potassium and ammonium in an attempt to isolate the effects of specific ions on the behaviour of the clay.

Tests revealed that the liquid limit, as determined by the cone-penetrometer could be significantly affected by changes in electrolyte type. As the liquid limit test is a form of strength test this infers changes in the strength properties of the soil.

RÉSUMÉ: Les effets de plusieurs électrolytes minéraux sur les propriétés techniques d’une argile glacio-lacustre remoulée, provenant du nord-est de l’Angleterre, ont été étudiés.
Plusieurs essais ont été menés pour déterminer les propriétés techniques générales de l’argile et comment celles-ci sont affectées par l’introduction d’électrolytes. L’anion nitrate, ainsi que quatre cations, le magnésium, le sodium, le potassium et l’ammonium ont été utilisés afin d’identifier les effets d’ions spécifiques sur le comportement de l’argile.

Les essais ont révélé que la limite liquide, déterminée par pénétromètre conique, peut être grandement affectée par des changements d’électrolytes. La limite liquide étant l’équivalent d’un essai de résistance, ceci implique des changements dans les propriétés de résistance du sol.

1. INTRODUCTION

1.1 Superficial geological deposits in the Tyne and Wear district.
The clay which was investigated was from Birtley, in the county of Tyne and Wear which is in the North East of England. In its natural state it shows clear laminations each being limited to a few millimetres in thickness. Sheets 20 and 21, Geological Survey of Great Britain (1978) and Taylor, Land, Mills, Smith and Warren (1971) show the clay to overlie Westphalian middle coal measures.

The origin of the clays in Tyne and Wear has been the subject of investigations for over a century and in summarising this work Beaumont (1968) confirmed that the laminated deposits of the area are true lacustrine clays. The bounds of the meltwater lake (Lake Wear) which led to the formation of many of the clays in the region were proposed by Merrick (1909) and further modified by Beaumont (see Figure 1). It was suggested (Smith, 1981) that the drift in the region of Lake Wear was late Devensian probably formed 20,000 to 13,000 years ago.

1.2 Some properties of Birtley clay

Tests were carried out in accordance with BS 1377, Methods of test for soils for civil engineering purposes (1990) to establish some of the properties of the clay. The results of the tests are shown in Table 1 and have been confirmed by Common (1987).
analysis carried out by the authors yielded qualitative evidence in support of that of Taylor et al.

1.3 Preparation of samples

The clay was made ready for testing by first oven drying it for 24 hours and then powdered by passing it through a cross-beater mill fitted with a 0.2 mm sieve. All samples were, therefore, in a remoulded reconstituted state.

The nitrate anion was common sodium, potassium and ammonium. All electrolytes to all of the tests carried out, the range of cations used in the test included magnesium, were prepared at concentrations of 0.1, 0.5 and 1.0 normal to the cation; electrolyte at the appropriate concentration was present as the pore-fluid in all cases.

A range of standard and near standard tests was carried out on the electrolytically treated clay samples. The testing regime carried out was aimed at establishing the change in the civil engineering properties caused by the electrolytes. Initial tests were carried out to establish the effects of the electrolytes on the index properties of the clay. Subsequent testing investigated the effects of the electrolytes on the consolidation and shear strength characteristics of the clay, but these will not be discussed in this paper.

2. THE EFFECTS OF SELECTED ELECTROLYTES ON THE INDEX PROPERTIES

2.1 Testing to determine the plastic limit

Tests to ascertain the plastic limit of electrolytically treated Birtley Clay were not carried out. It was only feasible to determine this property using untreated clay. Any result obtained from treated soil would be of limited use since the concentration of the electrolyte at the plastic limit would not be known. This concentration of electrolyte could not easily be calculated, since the nature of the plastic limit test required thin threads of clay to be rolled, which resulted in unknown moisture losses without proportional chemical losses. Where the overall moisture content had to be reduced further, the threads of clay were re-joined to the mass of clay. On the periphery of this clay, there may be high localised chemical concentration where moisture evaporation has taken place.
2.2 Testing to determine the liquid limit

This test was carried out in accordance with BS 1377 Method 4 using an 80 g 30° cone. The liquid limit is determined from a plot of penetration versus moisture content and is defined as the moisture content at which the cone penetrates the soil to a depth of 20 mm.

2.3 Analysis and discussion of results

The effects of electrolytes on the behaviour of clays in the index and shrinkage tests is often regarded as being indicative of their behaviour in consolidation and triaxial testing. It is widely accepted that many properties of a soil may be derived from the simple index tests (Wroth and Wood, 1978). These tests therefore provide an obvious starting point and a good basis for all subsequent testing. Throughout liquid limit testing of Birtley clay the concentration of all electrolytes could be kept constant by keeping the sample in a sealed container to prevent loss of water and hence change in concentration. The time the sample spent in the open was kept to the minimum and any loss of moisture due to evaporation was regarded as being negligible.

Many researchers have concerned themselves with investigations into the nature of the liquid limit of soils but few with the effects of electrolytes on this commonly measured parameter. The cone penetrometer as used in BS 1377 (1990) for determination of the liquid limit is basically a fall-cone apparatus which has been used in Scandinavia since the beginning of the century to determine strength properties of soils. Hansbo (1957) provides a detailed study of several different cone formats, varying its mass and tip angle. Whereas previously test data had been obtained from largely empirical formulae, Hansbo developed a more theoretical analysis. He proposed from dimensional analysis.

\[ c_u = \frac{KW}{d^2} \]  

(Eqn 1)

where \( c_u \) is the undrained shear strength of the soil.
K is a constant for the cone.
W is the mass of the cone.
d is the depth of penetration of the cone.

It was shown that the constant K for a particular cone was largely dependent on only the cone’s tip angle. The form of the proposed equation indicates that the undrained shear strength of the soil is proportional to the cone’s mass and inversely proportional to its depth of penetration. Hansbo lists \( c_u \) values for various cone masses and tip angles. Interpolation of these lists gives an undrained shear strength of 0.202 t/m² (1.98 kPa) for a British Standard 30°, 80 g cone at a penetration of 20 mm. Other research has revealed measured undrained shear strengths between 1.3 and 2.4 kPa.

One of the most comprehensive reports of an investigation into the variation of undrained shear strength with moisture content is by Youssef, el Ranli and el Demery (1965), who tested a wide range of clays. The liquid limit and change in undrained shear strength with moisture content was measured for 29 clay soils from the United Arab Republic. Liquid limits were shown to vary from 34% to almost 190%. The liquid limit of the soils was measured using a Casagrande cup and the shear strength versus moisture content, which they called the \( w_L \) line, reproduced in Fig. 2. It can be seen that there is a distinct trend showing a reduction in shear strength with increase in liquid limit from 2.4 kPa to 1.3 kPa.

![Figure 2](image_url)  

**Figure 2** Relationship between moisture content and undrained shear strength (after Youssef et al, 1965)

2.3.1 The effects of selected nitrates on the liquid limit of Birtley clay

It is known that many properties of clays are affected by the thickness of the double layers surrounding the clay particles.
The thickness of this double layer is dependent on the type of adsorbed cations on the negative faces of the clay particles. Metallic cations may be arranged in the Hofmeister (or Lyotropic) series thus:

$$\text{Al}^{+++} \text{Ba}^{++} \text{Sr}^{++} \text{Ca}^{++} \text{Mg}^{++} \text{NH}_4^+ \text{K}^+ \text{Na}^+ \text{Li}^+$$

The ions to the left of this list tend to have less thick double layers than those to the right. It seems reasonable to suppose that the increased thickness of the double layers around particles leads to the possibility of increased mutual repulsion of the like charges on the outside of the double layers.

Lambe (1960) proposed an equation which considered the attractive and repulsive forces which exist in soils on a microscopic level. The equation was of the form

$$\sigma = \sigma_m + u + R - A \quad \text{(Eqn 2)}$$

where

- $\sigma$ = total stress
- $\sigma_m$ = mineral to mineral contact stress
- $u$ = equivalent pore pressure
- $R$ = total interparticle repulsion divided by total interparticle area
- $A$ = total interparticle attraction divided by total interparticle area

Sridharan and Venkatappa Rao (1973) presented this equation in the form

$$\sigma = \sigma_m = \sigma - u_w - u_a - R + A \quad \text{(Eqn 3)}$$

where

- $\sigma$ = effective contact stress
- $u_w$ = effective pore air pressure
- $u_a$ = effective pore air pressure

The last algebraic term ($-R + A$) gives the net electrical attractive force. It can be seen from the previous equation that if the repulsive component of the net electrical attractive force is reduced then an increase in soil strength will result. A reduction in the double layer thickness, which occurs in the case of some of the electrolytes, effects such a change.

Since the liquid limit test is essentially a test which measures the moisture content at which the soil has a particular strength, it would be expected that soils with less thick double layers would be able to maintain this strength at higher moisture contents than those with thicker double layers.

The behaviour of clay soil is not only dependent on the long range electrical forces described above but also on the shearing resistance at or near the contact points within the soil mass and the elastic bending of clay plates. From the work of Sridharan and Venkatappa Rao (1973) it seems that kaolinite behaviour relies mainly on long range electrical forces. However, typical soils are dependent on a combination of these forces as they are generally never completely made up of clay minerals in the pure form.

### Table 2 The effects of selected nitrates on the liquid limit (percentage) of Birtle Clay

<table>
<thead>
<tr>
<th>Concentration/Electrolyte</th>
<th>0.1 N</th>
<th>0.5 N</th>
<th>1.0 N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magnesium Nitrate</td>
<td>53.5</td>
<td>52.5</td>
<td>51.8</td>
</tr>
<tr>
<td>Sodium Nitrate</td>
<td>52.5</td>
<td>51.0</td>
<td>50.5</td>
</tr>
<tr>
<td>Potassium Nitrate</td>
<td>55.5</td>
<td>62.1</td>
<td>60.0</td>
</tr>
<tr>
<td>Ammonium Nitrate</td>
<td>56.5</td>
<td>69.3</td>
<td>69.0</td>
</tr>
</tbody>
</table>

All four of the nitrates at all of the concentrations tested gave an increased value of the liquid limit compared to that of Birtle clay treated with deionised water (see Table 2). Ammonium nitrate increased the liquid limit by the greatest amount followed by potassium nitrate, magnesium nitrate and finally sodium nitrate. The order of effects is the same at all three concentrations tested the greatest change in liquid limit occurred at 0.5N concentration. With the exception of magnesium nitrate, the nitrates show a change in liquid limit in accordance with their position in the Hofmeister series i.e. those electrolytes causing a thinner double layer to be formed around the clay particles cause it to have a higher liquid limit. The results of all but magnesium nitrate confirm that the soil's behaviour can be qualitatively described by equations 2 and 3. According to the Hofmeister series magnesium nitrate should cause a thinner double layer than the other three electrolytes tested. This would cause the liquid limit of the soil treated with magnesium nitrate to be greater than if treated with other nitrates. This was not observed, the liquid limit showed only a slight increase at all concentrations.
Figure 3  The effects of selected nitrates on the undrained shear strength of Birtley clay in the vicinity of the liquid limit as measured by the cone penetrometer.
2.3.2 The effects of selected nitrates on the undrained shear strength measured at the liquid limit of Birtley clay as measured by the cone penetrometer.

By measuring the penetration of a 80 g, 30° British Standard cone into Birtley clay prepared at different moisture contents it was possible to assess its undrained shear strength and predict the value of this at the liquid limit. The strength of the clay was determined using Hansbo's formula

\[
c_u = \frac{KW}{d^2} \quad \text{(Eqn 1 bis)}
\]

An estimated value of shear strength of 2.05 kPa at the liquid limit was used which was taken from the work of Youssef et al (1965) for a soil with a liquid limit of 48.5% as shown in Fig. 2. This value was inserted into equation 1 to give a K value of 1.045 for the penetrometer. This value of K, approximately equal to one, is supported by Hansbo’s (1957) charts for \( c_u = 2.05 \) kPa and was subsequently confirmed by Wood (1983).

From the data obtained from liquid limit the undrained shear strength of the clay at several points around the liquid limit was calculated. Fig. 3 shows the variation in strength with moisture content for the selected nitrates. On each curve is marked the liquid limit of the appropriate electrolyte. The liquid limit points have been linked together by the best straight line marked \( w_L \). The moisture content - strength line for Birtley clay treated with de-ionised water has been added. All nitrates tested at all concentrations show an increase in the measured moisture content for any particular strength value compared with de-ionised water. From extrapolated lines it can be seen that for most nitrates there exists a point at lower moisture content values where the same strength may be obtained by treatment with de-ionised water. If the moisture content is further reduced, extrapolated strengths of some electrolytically treated samples are lower than those with de-ionised water. It is appreciated however that these extrapolated points need determining experimentally to confirm their existence. This could easily be done by recording soil strength as measured by the cone penetrometer over a wide range of moisture contents for the various nitrates. The interpolated values of strength at the liquid limit values all fall on a near straight line \( w_L \) which corresponds to a value of around 2 kPa. This shows that there is a unique undrained shear strength at the liquid limit for Birtley clay treated with nitrates. Unlike the results of Youssef et al (1965) shown in Fig. 2, the variation in strength at liquid limit moisture contents is small. This can be attributed to the effects of soil self-weight. The Casagrande cup apparatus used by Youssef et al to determine the liquid limit is particularly susceptible to the effects of soil self-weight (Bloomer and Coupe), whereas the cone penetrometer is affected to a much lesser degree (Houlby, 1982).

REFERENCES


The influence of length of joints on solubility of carbonaceous rocks
L’influence de la longueur des fissures sur la solubilité des roches carbonatées

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H. Jalaly
Ab-Niru Consulting Engineers

ABSTRACT: This paper is concerned with the geotechnical assessment of the influence of the length of joints on the solubility of carbonaceous rocks. Solubility of carbonaceous rocks play a great role in engineering behaviour of rock masses, particularly in relation to the engineering constructions in karstic areas. The method of assessment of solubility was based on laboratory works on the core samples of carbonaceous rocks from Alisadr area in Iran. This paper introduces a consideration of influence of length of discontinuities on solubility of carbonaceous rocks. This approach requires the laboratory determination of the effect of length of joints on solubility of carbonaceous rocks. This research presents the application of the new method for the solubility measurement of the carbonaceous rocks in concerned with the soluble rocks. The analysis of the laboratory works indicated that the increase of length of joint will cause an increase in the diffusion phenomenon and consequently the amount of discharge in joints will be reduced with time.

RESUME: Cet article porte sur l’évaluation géotechnique de l’influence de la longueur des joints sur la solubilité des roches carbonatées. La solubilité des roches carbonatées joue un rôle très important dans le comportement mécanique des masses rocheuses, notamment par rapport aux constructions mécaniques dans les régions karstiques. La méthode d’évaluation de la solubilité était basée sur les travaux laboratoires sur échantillons des roches carbonatées de la région d’Alisar en Iran. Cet article présente l’importance de l’influence de la longueur des discontinuities sur la solubilité des roches carbonatées. Cette approche exige que le laboratoire détermine l’influence de la longueur des joints sur la solubilité des roches carbonatées. Cette recherche présente l’application de la nouvelle méthode; l’analyse de ces travaux en laboratoire indique que l’augmentation de la longueur des joints engendre une augmentation du phénomène de diffusion, et par conséquent, de la quantité de décharge dans les joints diminuera avec le temps.

1. INTRODUCTION:

It is widely acknowledged today the engineering geology problems are a necessary adjunct to assess engineering properties of rock for engineering purpose, particularly in karstic areas. Because of solution to every problem created by ground conditions is more or less unique, the study of individual case histories is obviously important (Bell, 1980).

Solubility of rocks has a great influence on the engineering behaviour of rock masses. Therefore it is one of the major geotechnical problems in karstic areas, particularly in relation to dam construction and tunnelling. In this respect it is very important to investigate the effect of the degree of solubility of rocks in karstic areas. Solution of underlaying rocks is one of the most important geotechnical problems, which needs great attention during site investigation. Gruner (1962) listed the causes of failure dams.
According to his report 40% of total failure dams is attributed to foundation problems. The influence of solubility on engineering constructions has been considered by many authors (Beggs & Ruth, 1984; Bell, 1980; Fookeys & Hawkins, 1987; Grunner, 1963; Jalal, 1987; James & Kirkpatrick, 1980; Zhag & Fang, 1983).

The study described in this paper is concerned with the geotechnical assessment of the solubility of carbonaceous rocks which is mainly governed by systematic joint sets throughout the rock mass. Alisadr area of Iran where this study was carried out is underlain with carbonaceous rocks of the Jurassic period. In this area there are some large and very beautiful underground caverns filled with water. In fact Alisadr area is a karstic area and therefore it was chosen for this study.

2. RESEARCH OBJECTIVE AND PROGRAM:

The research approach for consideration of the effect of length of discontinuities on the solubility evaluation of carbonaceous rocks was based on the following discrete steps;

1- Field investigation
2- Laboratory evaluation of solubility characteristics of jointed rock samples.
3- Theoretical solubility analysis.

The field study was aimed at the acquisition of the geological and geotechnical data and collection of representative samples for laboratory studies. This study enables an assessment of effect of the length of joints on the solubility of carbonaceous rocks which is a very important geotechnical factor on dam construction and tunnelling.

3. GEOLOGICAL DATA COLLECTION AND PRESENTATION:

the foundation but they also govern the material available for construction (Bell, 1980). Geology is an essential study for anyone entering the professions of civil and geotechnical engineering.

Of the various natural factors which directly influence the design of dams none is more important than the geological, not only do they control the character of
The Alisadr area is located in latitudes 48°, 17' and longitudes 35°, 20', which is approximately more than 300 Km south west of Tehran (capital of Iran).

Figure 1 shows the location of the site study.
In order to study the effect of length of joints on solubility of carbonaceous rocks, an extensive field and laboratory works have been carried out. During the site investigation for recognition of the relation between joint sets orientations and orientation of cavern, a joint survey programme was carried out.

For this purpose a magnetic clinometer (Clar compass) was used in the measurement of dip and dip-direction of the discontinuity planes. It is recommended for geotechnical purposes by some authors (Hoek & Bray, 1981).

Figure: 1 Location map of the study area

In this field investigation around 530 reading were taken and some additional data which are necessary for the evaluation of the engineering geology characteristics were also obtained from the existing rock exposures. Contour diagram plotted of discontinuities of area is shown in figure 2.
on the surface and underground, characterised by the presence of several large caverns such as Alisadr and Sarab caverns.

4. LABORATORY DETERMINATIONS OF SOLUBILITY PROPERTIES OF CARBONACEOUS ROCKS:

In order to assess the solubility of rock masses, it is necessary to know not only the geological characteristics of the rock masses, but also the engineering properties. For this purpose the cylindrical rock specimens were obtained from the block rock samples by the core drilling machine in the laboratory in a suitable length and diameters. The specimens were prepared by cutting the cores into required sizes using disc saws.

In this research in order to consider the effect of length of joints on solubility of carbonaceous rocks artificial joints of 1 mm width were made from core samples of 10, 15, and 25 cm length respectively. Water was circulated through the artificial joints under 5 Psi pressure in a closed cycle. The circulation arrangement and apparatus are shown in fig 3.

Before starting the test the weight of the specimens and the initial width of the them was carefully measured. Table 1 shows the weight variation of the core samples during the test period. Testing the core samples with different size has been carried out for three continuous months and every 10 days amount of discharge from each specimen has been measured.

Table: 1 Variation of weight of core samples during the 90 days.

<table>
<thead>
<tr>
<th>Sample No</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial weight (gr)</td>
<td>513</td>
<td>820</td>
<td>1090</td>
</tr>
<tr>
<td>After 30 days</td>
<td>511</td>
<td>816</td>
<td>1086</td>
</tr>
<tr>
<td>After 45 days</td>
<td>509</td>
<td>814</td>
<td>1083</td>
</tr>
<tr>
<td>After 60 days</td>
<td>507</td>
<td>812</td>
<td>1077</td>
</tr>
<tr>
<td>After 75 days</td>
<td>506</td>
<td>811</td>
<td>1074</td>
</tr>
<tr>
<td>After 90 days</td>
<td>505</td>
<td>810</td>
<td>1072</td>
</tr>
<tr>
<td>Variation (%)</td>
<td>1.6 %</td>
<td>1.22 %</td>
<td>1.38 %</td>
</tr>
</tbody>
</table>
During 90 days test period, all above condition were fixed. The water pressure in the circulation apparatus was maintained around 5 Psi during the testing period. The specification of core samples and variation of the discharge of them are shown in Tables 2 and 3 respectively.

Table 2 presents the variation of the discharge of the jointed rock samples during the 90 days period. Test condition was fixed through out the test period. Water temperature was maintained between 0-5°C and the water acidity (PH) was between 5.5 - 6.

Analysis of data showed that the increase of length of joints will cause an increase in the diffusion phenomenon. This process will cause more sedimentation at the end of longer specimen (25 cm length). As a result the amount of discharge in a joint will be reduced. Figure 4 shows the results of the test.

**Figure: 3** Circulation apparatus used to assess the solubility of carbonaceous rocks.

**Table: 2 Variation of the discharge of jointed rock samples during 90 days period.**

<table>
<thead>
<tr>
<th>Sample No</th>
<th>Q average Cm³/sec At the first time</th>
<th>Q average Cm³/sec After 10 days</th>
<th>Q average Cm³/sec After 20 days</th>
<th>Q average Cm³/sec After 30 days</th>
<th>Q average Cm³/sec After 40 days</th>
<th>Q average Cm³/sec After 50 days</th>
<th>Q average Cm³/sec After 60 days</th>
<th>Q average Cm³/sec After 70 days</th>
<th>Q average Cm³/sec After 80 days</th>
<th>Q average Cm³/sec After 90 days</th>
<th>Q/Qo%</th>
<th>+ve %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>51</td>
<td>52.1</td>
<td>52.5</td>
<td>55.8</td>
<td>56.7</td>
<td>57.2</td>
<td>57.8</td>
<td>58.1</td>
<td>58.8</td>
<td>59</td>
<td>15.7</td>
<td>5.2</td>
</tr>
<tr>
<td>2</td>
<td>48.3</td>
<td>49</td>
<td>50.1</td>
<td>50.6</td>
<td>51.2</td>
<td>52.5</td>
<td>52.75</td>
<td>52.90</td>
<td>53.2</td>
<td>53.35</td>
<td>10.1</td>
<td>3.42</td>
</tr>
<tr>
<td>3</td>
<td>45</td>
<td>46.8</td>
<td>47.3</td>
<td>50</td>
<td>49.1</td>
<td>47.5</td>
<td>45.6</td>
<td>43.8</td>
<td>42.4</td>
<td>41</td>
<td>-8.9</td>
<td>2.96</td>
</tr>
</tbody>
</table>

During 90 days test period, all above condition were fixed. The water pressure in the circulation apparatus was maintained around 5 Psi during the testing period. The specification of core samples and variation of the discharge of them are shown in Tables 2 and 3 respectively.

Table 2 presents the variation of the discharge of the jointed rock samples during the 90 days period. Test condition was fixed through out the test period. Water temperature was maintained between 0-5°C and the water acidity (PH) was between 5.5 - 6.

Analysis of data showed that the increase of length of joints will cause an increase in the diffusion phenomenon. This process will cause more sedimentation at the end of longer specimen (25 cm length). As a result the amount of discharge in a joint will be reduced. Figure 4 shows the results of the test.

**Table: 3 Specifications of core samples used in research**

<table>
<thead>
<tr>
<th>Sample no</th>
<th>Length of sample, cm</th>
<th>Diameter of sample, mm</th>
<th>Width of joint, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>54</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>54</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>54</td>
<td>1</td>
</tr>
</tbody>
</table>

This figure shows that approximately after 30 days amount of discharge of the sample 3 started to reduce and it has continued until end of test period (90 days), but in sample 1 and 2 amount of discharge has increased during the test period.

5. DISCUSSION AND CONCLUSIONS:

The purpose of this paper was to study the solubility problems of jointed rock samples associated with carbonaceous rocks. The
solubility process can be considered from different approaches.

In this research, efforts have been made to consider the effect of length of joints on the solubility of carbonaceous rocks of Jurassic period from Alisadr area in Iran. During this investigation geological and geotechnical data were obtained from representative samples prepared from the blocky rock samples of the area.

According to the result of this experimental work it has been found that the length of the joints controls the amount of discharge of the joints. Detailed geological and geotechnical investigations are therefore necessary to evaluate engineering behaviour of a jointed rock mass undergoing karstification process. Solubility evaluation of the carbonaceous rocks have shown that the increase of length of joints will cause an increase in the diffusion phenomenon and consequently the amount of discharge through the joint will be reduced.

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Figure 4 Relationship between discharge (Q) and time (t)
Geotechnical parameters characteristic for some organic soils from Masurian Lake District

Paramètres géotechniques caractéristiques pour quelques sols organiques du district du lac Masurian

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ABSTRACT: In the paper authors present classifications, contents of various kinds of matter, and main physical parameters of gyltjas. Article presents the calculated values of oedometer modulus, consolidation coefficient, and also their relationship to permeability coefficient measured by variable head method. Settlement of gyltja specimen in oedometer long duration test was described and the probable interpretation of of these was given.

RÉSUMÉ: Dans cet article les auteurs présente les classifications, la composition variable et les propriétés principales de "gyltja". On présente des valeurs calculées du module édémétrique, du coefficient de consolidation et aussi leurs relations avec la perméabilité déterminée par le méthode de charge variable. On rapporte l’exécution et l’interprétation d’un essai édémétrique de longue durée sur un échantillon de "gyltja".

1 INTRODUCTION

Area of northern Poland is covered by thick layer of Quaternary deposits. Sands and various kinds of clays are Pleistocene ones. Peats, sills and gyltjas are Holocene sediments which are connected to water environment. Gyltjas are characteristic lacustrine deposits found in some countries close to Baltic Sea. Swampy areas cover 75% of north–east part of Poland. In these areas gyltja can be found either directly on the ground surface or under a layer of peat. These soils make the problems when hydraulic engineering, road etc. objects are to be designed and built in those areas. Physical and mechanical properties of peats are relative well known. There are not numerous investigations aimed to gyltjas properties. In this paper laboratory tests results of gyltja properties are shown. Samples of gyltja origin from Masurian Lake District.

2 Forming of gyltja

Forming of Holocene lacustrine deposits has been lasting since continental glacier receded (far11–12 thousands years). Lokes are gradually more and more shallow and finally they disappear as a result of accumulation of these deposits. In Poland about 67.4% of glacial–origin lakes have disappeared in that way. Gyltjas and peats are situated in the places of that lakes. Now as gyltja we can regard all organic lacustrine deposits situated under a layer of peat (in natural conditions). Gyltjas consist of various kind of matter such as:

– organic one;
– carbonate one;
– mineral one.

Organic matter is formed in oxygen–free conditions from atrophied plankton and disintegrated and metamorphosed residuum of plants. Organic matter can also be formed from some water–soluble parts of humus. At the bottom of a lake process of mineralization of organic matter takes place as a result of a biochemical and chemical decomposition of this organic matter. Carbonate matter (CaCO3) existing in the gyltjas is brought into the lakes by the
precipitation water, ground water, flows. Calcium acid carbonate brought into the lake precipitates as calcium carbonate in the way shown by following equation:

$$\text{Ca(HCO}_3\text{)}_2 = \text{CaCO}_3 + \text{CO}_2 + \text{H}_2\text{O}$$

Lacustrine deposits seldom are composed of only one kind of matter. Usually carbonate–organic–mineral sediments exists. Sometimes we can find a pure carbonate sediment like lacustrine chalk which can have up to 98% of CaCO₃, or organic gyttja with its ash contents lower than 3.1%. It should be emphasized that lacustrine deposits are both biological and chemical origin and almost always contain non-carbonate mineral matter, mostly clay, brought into a lake by water and wind from surroundings. Average depth of lacustrine deposits is equal about several meters. Sometimes it is greater than 20 m. Usually lacustrine deposits are covered by 2–4 m thick layer of peat formed in result of making lake shallow and expansion of swampy plants.

3. CLASSIFICATION OF GYTTJA

The most significant classifications of gyttja were made for agricultural and pedological needs. Markowski and Okruszko were two of many who made those classifications.

Table 1. Okruszko's proposal of gyttja classification

<table>
<thead>
<tr>
<th>Type of gyttja</th>
<th>Kind of gyttja</th>
<th>Matter contents in %</th>
<th>organ. CaCO₃</th>
<th>clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Organic</td>
<td>1. detritus</td>
<td>60</td>
<td>20–40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>algal</td>
<td>60</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>II. Miscellaneous mineral</td>
<td>organic</td>
<td>60</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>clayey</td>
<td>40–60</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>organic</td>
<td>40</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>III. Mineral</td>
<td>4. clayey–organic</td>
<td>20–40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>colcareous</td>
<td>40</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>5. colcareous</td>
<td>40</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>6. clay</td>
<td>40</td>
<td>20</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>IV. Mineral homogeneous</td>
<td>7. lacustrine</td>
<td>20</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>chalk</td>
<td>20</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>8. lacustrine</td>
<td>clay</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

Similar proposal of gyttja classification is given by Markowski. He divided gyttja into three groups:
- organic sediments;
- carbonate sediments;
- mineral sediments.

Using these proposals for geotechnical needs is very difficult but they can be a base for making engineering geological classification of gyttja. Being based on works by Amoryon, Helenelud and Radforth we can say that organic soils should be qualified by:
- structure;
- contents and quality of water;
- basic physical and mechanical properties.

These last directions can be regarded as proper also for making engineering geological classification of organic sediments. Trial of making such classification is presented by M. Dlugaszek. He divided gyttja into three types and described some of characteristic physical and mechanical parameters.

Table 2. Base parameters list of separated out gyttja types. Extremal values given by Dlugaszek.

<table>
<thead>
<tr>
<th>Sediment type</th>
<th>ion</th>
<th>Wn</th>
<th>$S_1$</th>
<th>$S_5$</th>
<th>Mo</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>%</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kPa</td>
</tr>
<tr>
<td>mineral</td>
<td></td>
<td></td>
<td>$10^{-3}$</td>
<td>$10^{-3}$</td>
<td></td>
</tr>
<tr>
<td>gyttjas</td>
<td>2–10</td>
<td>117.4</td>
<td>2.03</td>
<td>2.72</td>
<td>3700</td>
</tr>
<tr>
<td>gyttjas</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mineral
- organic: 10–167.2 1.09–2.02–275–
- gyttjas: 30–479 1.29 2.39 770
- organic: 353–1.00 1.42–250–
- gyttjas: >30 1630 1.09 1.98 400

(Explanations of used symbols in the text below)

Oedometer modulus of compressibility Mo given in the Table 2 was qualified for load 0–200 kPa.

Proposal of classification shown by Dlugaszek doesn't answer for all requirements that must be answered by engineering geological classifications. This classification does not qualify strength parameters and parameters characterizing consolidation.

4. PHYSICAL AND MECHANICAL PROPERTIES OF GYTTJA

To qualify physical and mechanical properties of gyttja samples from seven areas (which will be called
Table 3.

<table>
<thead>
<tr>
<th>Object number</th>
<th>Wn</th>
<th>$\mathcal{Q}$</th>
<th>$\mathcal{Q}_s$</th>
<th>e</th>
<th>Iom</th>
<th>CaCO$_3$</th>
<th>Sediment type by Dlugosz and Zielke</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 185–496</td>
<td>312</td>
<td>1.13–1.26</td>
<td>2.40–2.52</td>
<td>4.73–11.84</td>
<td>4.55–7.14</td>
<td>50.0–68.7</td>
<td>mineral</td>
</tr>
<tr>
<td>3 180–191</td>
<td>182</td>
<td>1.24–1.27</td>
<td>2.34–2.43</td>
<td>4.27–4.65</td>
<td>0.72–1.76</td>
<td>91.3–95.58</td>
<td>mineral</td>
</tr>
<tr>
<td>4 220–275</td>
<td>246</td>
<td>1.14–1.22</td>
<td>2.38–2.39</td>
<td>5.80–7.00</td>
<td>12.0–15.93</td>
<td>39.0–60.89</td>
<td>mineral–organic</td>
</tr>
<tr>
<td>5 238–309</td>
<td>274</td>
<td>1.15–1.18</td>
<td>2.33–2.46</td>
<td>5.66–7.79</td>
<td>8.1–16.96</td>
<td>47.15–67.7</td>
<td>mineral–organic</td>
</tr>
<tr>
<td>6 169–427</td>
<td>261</td>
<td>1.07–1.20</td>
<td>2.08–2.39</td>
<td>4.07–9.15</td>
<td>18.8–37.9</td>
<td>5.0–9.03</td>
<td>mineral–organic</td>
</tr>
<tr>
<td>7 55–102</td>
<td>74</td>
<td>1.36–1.72</td>
<td>2.42–2.49</td>
<td>1.59–2.59</td>
<td>3.24–5.12</td>
<td>79.6–88.59</td>
<td>mineral</td>
</tr>
</tbody>
</table>

Minimal, maximal and average values are given.

"objects" in the following text) close to Olszyn were investigated. Undisturbed structure samples were taken from depth 0.5 – 5.5 m under ground surface. In laboratory tests following parameters were obtained:
- natural humidity (Wn);
- volume density of soil ($\mathcal{Q}$);
- density of solid particles ($\mathcal{Q}_s$);
- organic matter contents (Iom);
- CaCO$_3$ contents.

Organic matter contents was qualified using Turin's method. Results of investigations are shown in the Table 3. Investigations by Skempton show that method usually used to obtain organic matter contents (ignition method) gives incorrect results in the case of some organic soils. That's why chemical Turin's method was used to obtain Iom. Samples taken from objects 1–5 were investigated to compare both methods. Results of the investigations were shown in the Table 4.

In the engineering practice we need to know characteristic parameters of compressibility and consolidation to safely build all kinds of engineering objects based on soil grounds. Three sequential stages of oedometer tests were performed to investigate these parameters. First stage includes fifteen samples of gyttja from two objects. Samples from object 6 are of mineral organic gyttja and they were taken from the same depth (0.5 m). Samples from object 7 are of mineral gyttja and they were taken from the depth of 1 m. Performing the oedometer tests we use five incremental loading procedure with doubling the load for each increment. Taking results of these tests as a base we calculate:
- $\text{Mo}$ - oedometric modulus of compressibility;
- $\text{Cv}$ - consolidation coefficient by both Taylor’s and Cassagrande’s methods.

No significant correlations were noticed between calculated $\text{Cv}$ values (using both methods) and oedometric modulus of compressibility values. Calculating $\text{Cv}$ we don’t consider specimen settlement

Table 4. Various kinds of matter contents for particular objects.

<table>
<thead>
<tr>
<th>Object number</th>
<th>Iom</th>
<th>Ac</th>
<th>S</th>
<th>CaCO$_3$</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 6.06</td>
<td>67.20</td>
<td>32.80</td>
<td>61.40</td>
<td>32.58</td>
<td></td>
</tr>
<tr>
<td>2 27.10</td>
<td>63.80</td>
<td>38.18</td>
<td>6.91</td>
<td>71.99</td>
<td></td>
</tr>
<tr>
<td>3 1.32</td>
<td>68.36</td>
<td>28.80</td>
<td>97.77</td>
<td>4.92</td>
<td></td>
</tr>
<tr>
<td>4 13.72</td>
<td>61.43</td>
<td>38.57</td>
<td>47.86</td>
<td>38.42</td>
<td></td>
</tr>
<tr>
<td>5 12.52</td>
<td>60.33</td>
<td>39.67</td>
<td>57.44</td>
<td>30.05</td>
<td></td>
</tr>
</tbody>
</table>

Average values are given.

Iom - organic matter
Ac - ash contents
S - ignition loss
M - non-carbonate mineral matter

from object 7 are of mineral gyttja and they were taken from the depth of 1 m. Performing the oedometer tests we use five incremental loading procedure with doubling the load for each increment. Taking results of these tests as a base we calculate:
- $\text{Mo}$ - oedometric modulus of compressibility;
- $\text{Cv}$ - consolidation coefficient by both Taylor’s and Cassagrande’s methods.

No significant correlations were noticed between calculated $\text{Cv}$ values (using both methods) and oedometric modulus of compressibility values. Calculating $\text{Cv}$ we don’t consider specimen settlement
Table 5. Characterizations of compressibility and consolidation.

<table>
<thead>
<tr>
<th>Spec. number</th>
<th>1 increment 0.0–12.5 kPa</th>
<th>II increment 12.5–25 kPa</th>
<th>III increment 25–50 kPa</th>
<th>IV increment 50–100 kPa</th>
<th>V increment 100–200 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mo</td>
<td>kPa</td>
<td>Mo'</td>
<td>kPa</td>
<td>Mo''</td>
<td>kPa</td>
</tr>
<tr>
<td>Cv</td>
<td>m²/s/10⁻¹⁰</td>
<td>Cv'</td>
<td>m²/s/10⁻¹⁰</td>
<td>Cv''</td>
<td>m²/s/10⁻¹⁰</td>
</tr>
<tr>
<td>S</td>
<td>10⁻¹⁰</td>
<td>S'</td>
<td>10⁻¹⁰</td>
<td>S''</td>
<td>10⁻¹⁰</td>
</tr>
<tr>
<td>1</td>
<td>246 3.93 2080 216 0.59 622 249 1.14 1101</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>153 5.31 631 121 0.15 192 242 0.70 305 397 0.17 1121 769 0.41 2117</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>177 5.54 1565 147 0.54 427 179 0.83 667 385 0.45 975 492 0.71 742</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>385 6.17 2030 274 0.77 754 305 0.55 1000 400 0.55 712 757 2.15 2597</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>231 4.11 1893 189 0.17 401 325 0.18 1036 1131 0.39 2800 1290 0.63 3320</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Mo - oedometer modulus of compressibility (for total settlement of specimen)
Mo' - oedometer modulus of compressibility in the range of primary consolidation settlement of specimen (using h0 and h100 obtained by Taylor's method)
Cv - consolidation coefficient (by Taylor's method; this coefficient was calculated only using Taylor's method according to Maczkowski, who show it as more proper for organic soils)

of initial compression phase (which have various quantities for each of samples during the same load increment). That is probably why we don't notice significant correlations between Mo and Cv. Values of oedometer modulus of compressibility were calculated however for total settlement of the specimen during one load increment.

In the second stage long term oedometer tests were performed. Five samples obtained from various depth (0.5 – 3.5 m) from object 1 were investigated. Duration of one load increment was six month. Loading procedure was the same as in the first stage. Basing on the results of investigations we calculate parameters show in Table 5.

Comparing Mo – Cv, Mo' – Cv values from the second stage also doesn't show significant correlation. In the case of Mo – Cv correlation from the second stage considerable in the long term test settlements of secondary compression are additionally reason of absent of significant correlations. In the case of Mo' – Cv correlation from the second stage absent of significant correlations can be caused by probable great variation of permeability coefficient (k) of investigated samples of gyttja. The third stage of investigations preformed to check how permeability coefficient influences correlations Mo – Cv and Mo' – Cv. This stage includes:

- long duration one load increment oedometer test (0.0 – 12.5 kPa). Thirteen samples of various class of gyttja obtained from objects 2, 3, 4, 5 from the depth range of 0.5 – 5.5 m.

During the tests, which were lasting over seven month each, repeated investigations of permeability coefficient were performed using variable head test. Irregular decrease of permeability coefficient was noticed.

![Figure 1. Course of specimen settlement and measured k-values during the long term oedometer tests.](image)

Basing on curves: height of specimen – time of testing, obtained as results of the tests, Mo, Mo', Cv were calculated in the same way as before – in the second stage tests. These values were related to permeability coefficients k obtained from the tests: k\text{max} – maximal (usually initial value of permeability coefficient)
<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Mo (kPa)</th>
<th>Cv (m²/s)</th>
<th>Mo' (kPa)</th>
<th>Ksr (m/s)</th>
<th>kmax (m/s)</th>
<th>kmin (m/s)</th>
<th>ko1 (m/s)</th>
<th>ko2 (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75</td>
<td>2.5E-9</td>
<td>967</td>
<td>3.18E-7</td>
<td>2.16E-7</td>
<td>4.34E-8</td>
<td>2.58E-8</td>
<td>3.33E-9</td>
</tr>
<tr>
<td>2</td>
<td>56</td>
<td>1.87E-10</td>
<td>95</td>
<td>3.43E-9</td>
<td>2.43E-8</td>
<td>5.17E-10</td>
<td>1.97E-8</td>
<td>3.34E-7</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
<td>2.17E-9</td>
<td>55</td>
<td>4.04E-9</td>
<td>6.94E-8</td>
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<td>2.90E-7</td>
<td>6.78E-7</td>
</tr>
<tr>
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<td>47</td>
<td>2.8E-9</td>
<td>75</td>
<td>6.0E-9</td>
<td>7.73E-8</td>
<td>7.73E-11</td>
<td>2.93E-7</td>
<td>9.65E-7</td>
</tr>
<tr>
<td>5</td>
<td>228</td>
<td>2.19E-8</td>
<td>938</td>
<td>1.45E-7</td>
<td>6.89E-7</td>
<td>2.14E-9</td>
<td>2.33E-7</td>
<td>9.65E-7</td>
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<tr>
<td>6</td>
<td>171</td>
<td>9.3E-9</td>
<td>675</td>
<td>3.49E-8</td>
<td>6.3E-7</td>
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<td>1.38E-7</td>
<td>5.44E-7</td>
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<tr>
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<td>167</td>
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<td>4.53E-8</td>
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<td>8</td>
<td>168</td>
<td>2.12E-9</td>
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<td>1.32E-8</td>
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<td>1.72E-7</td>
<td>7.72E-7</td>
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<tr>
<td>10</td>
<td>159</td>
<td>6.25E-9</td>
<td>320</td>
<td>7.26E-9</td>
<td>3.07E-8</td>
<td>1.35E-9</td>
<td>1.95E-7</td>
<td>3.93E-7</td>
</tr>
<tr>
<td>11</td>
<td>154</td>
<td>8.1E-10</td>
<td>311</td>
<td>2.6E-9</td>
<td>6.16E-9</td>
<td>8.75E-10</td>
<td>2.6E-8</td>
<td>5.26E-8</td>
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<tr>
<td>12</td>
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<td>732</td>
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<td>7.64E-8</td>
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<td>3.09E-7</td>
<td>1.39E-6</td>
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<tr>
<td>13</td>
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<td>8.72E-11</td>
<td>864</td>
<td>3.69E-8</td>
<td>7.5E-8</td>
<td>9.97E-9</td>
<td>7.54E-8</td>
<td>4.82E-8</td>
</tr>
</tbody>
</table>

ksr - average value of permeability coefficient
kmin - minimal value of permeability coefficient

This relating was made in the following way:
1. ko1 coefficient was calculated as a parameter satisfying the equation:
   \[ \text{Cv} = (\text{Mo'ko1})/\text{T_w} \]
2. ko2 coefficient was calculated as parameter satisfying the equation:
   \[ \text{Cv} = (\text{Mo'ko2})/\text{T_w} \]
what is shown in Table 6.
3. Calculated ko1 and ko2 were compared to kmax, ksr and kmin obtained permeability tests. In the most of cases both ko1 and ko2 were the nearest kmax. Calculated ko1 values were usually nearer kmax than ko2 values.

However there are so significant differences between kmax and ko1 that we can not say that they satisfy equation:
\[ (\text{Mo'kmax})\text{T_w} = (\text{Mo'ko1})\text{T_w} \]
for the the gyttjas investigated in such a way and for Mo' and Cv calculated using Taylor’s method.
The reason of these discrepancies is probably standard Taylor’s method used to calculation. There is significant probability of incorrect determining of both consolidation time and values of consolidation settlements using Taylor’s method to long duration oedometer tests. The reason of this fact may be significantly more irregular settlements during long duration oedometer tests of gyttjas than in the case of mineral soils which are subjects of Taylor’s method. Figure 1 gives an example of course of specimen settlements of the oedometer tests and permeability coefficients k measured at variable head permeability tests. Such a course of specimen settlement suggests that initial settlements, primary consolidation ones and secondary compression ones can repeatedly happen during one load increment. It probably results from gyttja structure reaction caused by effective stress increment. During initial and primary consolidation specimen settlements pore water takes over part of total stress increment. In the secondary compression phase excess of pore pressure is almost equal 0, and soil skeleton takes over almost total stress increment. In the case of gyttja soil skeleton exists as a specific structure.

Figure 2. Relationship Cv-ko1, ko2 and kmax

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which influenced by effective stress probably becomes fatigue and after any long time can be broken. It causes repeating of whole process:
1. settlement of initial compression phase (when excess of pore pressure is generated)
2. settlements of primary consolidation phase (when excess of pore pressure is dissipated and structure of soil skeleton is strengthened)
3. settlements of secondary compression phase (when excess of pore pressure is almost equal 0).
This cycle repeats until gyttja skeleton structure becomes enough strengthened for given effective stress increment. To check this suggestion up we should perform long term oedometer tests (like described above) with pore pressure measurement what we are going to do.

5 CONCLUSIONS

1 We can’t estimate parameters of compressibility and consolidation by used so for classifications of gyttja.
2 Ignition method used to qualify organic matter contents in the case of carbonate gyttja gives to high values. In such cases we suggest to use Tiurin’s method.
3 Values of Cv coefficient obtained by standard methods (Taylor’s and Cassagrande’s), which are based on oedometric test are various for these two methods. In the case of long duration oedometric test on gyttja these methods give often incorrect results because they don’t considerate significant irregularity of specimen settlement during the test.
4 Permeability coefficient qualified during long duration oedometer tests shows irregular declinging trend.
5 Significant differences between calculated permeability coefficients k01 and measured permeability coefficients kmax suggest that duration of primary consolidation phase and values of that phase settlement are qualified incorrect.
6. Course of specimen settlements in long duration one load increment oedometer tests suggests that cyclic sequence of initial compression settlements, primary consolidation settlements and secondary compression settlements often happens until structure of gyttja skeleton becomes enough strengthened for given load.

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Volume changes of soils and their effects in Natal, South Africa

Contraction des terrains et ses conséquences géotechniques au Natal, Sud-Afrique

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ABSTRACT: The occurrence of potentially expansive soils in Natal, has long been recognised. Over the past two years because of the drought conditions in Natal low-rise buildings have suffered structural damage because of soil shrinkage. This has resulted in numerous insurance claims in respect of damage to structures. In the case of shallow founded structures, shrinkage of soil due to desiccation exterior to the structures gives rise to an 'edges-down' effect which has the same structural consequences as expansive soil beneath the structure in that the pattern of cracks developed is similar. Desiccation shrinkage of embankment fills has also occurred with the development of systems of tensile cracks therein parallel to the margins of the fill which superficially can be mistaken for those arising from slope instability. Shrinkage settlement of the surfaces of the fills has also occurred to the detriment of roads. The paper describes the occurrence, nature and geotechnical properties of such soils and a number of case histories illustrative of the problem.

RESUME: La présence au Sud Afrique de terrains potentiellement instables (expansifs) a été depuis longtemps reconnue. Dû à la sécheresse prévalente durant ces deux dernières années au Natal, des dommages aux structures des bâtiments ont eu lieu. Ceci a été la cause de nombreuses demandes de compensation d'assurance. Dans le cas de bâtiments à fondations peu profondes la contraction du terrain environnement causée par la sécheresse, produit un effet "edge down" qui a le même résultat sur les structures qu'un terrain expansif sous les structures, produisant un système de fissures de déformation typiquement associé à des conditions d'expansion. La contraction dûe à la sécheresse du remplissage des grands et petits terrassements a aussi été constaté et développement une série de fissures parallèlement à la bordure du remplissage et qui pourrait être mèfroit comme provenant d'une instabilité d'inclinaison

Le tassement après contraction des surfaces du remplissage se produit au détriment des routes au autre structures construites sur les terrassements. Ce document décrit la présence, la nature et les propriétés géotechniques de ces terrains et un nombre de cas illustrant le problème.

1 INTRODUCTION

One of the most notable characteristics of some clay soils from the engineering point of view is their susceptibility to slow volume changes due to swelling or shrinkage. Such volume changes can give rise to ground movements which may result in severe damage to buildings. Low-rise buildings are especially vulnerable to ground movements as they usually do not possess the weight or strength to resist. Differences in the period and magnitude of precipitation and evapotranspiration are the major factors influencing the swell-shrink response of a clay soil beneath a building. Poor surface drainage or leakage from underground pipes can produce concentrations of moisture in clay. Trees with high water demand and uninsulated hot process foundations may dry out clay causing shrinkage. Cold stores also may cause desiccation of clay soil. The depth of the active zone in expansive clays (i.e. the zone in which swelling and shrinkage occurs in wet and dry seasons respectively) varies.

There are two modes of swelling in clay soils, namely, intercrystalline and intracrystalline swelling. Intercrystalline swelling takes place in any type of clay deposit irrespective of its mineralogical composition, and the process is reversible. In relatively dry clays the particles are held together by relict water under tension from capillary forces. On wetting the capillary force declines and the clay expands. In other words, intercrystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and the void spaces between the crystals. Intracrystalline swelling, on the other hand, is characteristic of expansive clay minerals, of montmorillonite in particular. The individual molecular layers which make up a crystal of montmorillonite are weakly bonded so that on wetting water enters not only between the crystals but also between the unit layers which comprise the crystals. Swelling in Na montmorillonite is the most notable and can amount to 200% of the original volume, the clay then having formed a gel. Generally kaolinite has the smallest swelling capacity of the clay minerals and nearly all of this is of the interparticle type. Illite may swell by up to 15% but intermixed illite and montmorillonite may swell some 60 to 100%. Swelling of Ca montmorillonite is very much less than in the Na variety, it ranges from about 50 to 100%. The large swelling capacity of montmorillonites means that they give the most trouble in
terms of building construction.

The swelling potential of clay soil depends upon its initial moisture content, initial density or void ratio, its microstructure and the vertical stress, as well as the type and amount of clay minerals present. Cemented and undisturbed expansive clay soils often have a high resistance to deformation and may be able to absorb significant amounts of swelling pressure. As noted, expansive clay minerals take water into their lattice structure. In loose dense soils they tend to expand initially into zones of looser soil before volume increase occurs. However, in densely packed soil with low void space the soil mass has to swell more or less instantly to accommodate the volume change of the expansive clays. As expansive clays tend to possess extremely low permeabilities, moisture movement is slow and an appreciable period of time may be involved in the swelling/shrinking process. Consequently moderately expansive clays with a smaller potential to swell but with higher permeabilities than clays having a greater swell potential may swell more during a single wet season than more expansive clays.

Expansive clays are often heavily fissured due to seasonal changes in volume which produce shrinkage cracks and shear surfaces. Consequently near vertical fissures are frequently found at shallow depth with diagonal fissures at greater depth. Sometimes the soil is so desiccated that the fissures are wide open and the soil is shattered or micro-shattered.

A site which is to be developed requires an initial visit to examine the ground, and to assess how well drained it is and the type and amount of vegetation cover. In addition, a study of the recent history of the site may reveal that trees have been removed within the past few years. Aerial photographs may be of value in this respect.

If the site is going to be developed for low-rise buildings, then the subsequent ground investigation cannot afford to be over elaborated. As such, the simplest method of examining the soil is to excavate trenches or pits. Detailed soil profiles should be recorded on diagrammatic logs or photographed. Samples of soil can be taken where required for testing. Drhoe holes allow the soil to be examined and sampled at greater depths, and borehole logs of the soil then are produced with all the relevant data recorded on them. A stand-pipe should be placed in a borehole so that measurements of water level can be taken over a few months.

Settlement points can be installed at different depths in the ground using sleeved rods to measure the seasonal movements of the ground in conjunction with moisture content. These measurements provide direct evidence of potential shrinkage and swelling movements. However, such measurements are time consuming and Williams and Pidgeon (1983) pointed out that the measurement of seasonal ground movements under natural conditions may give an appreciable underestimate of potential total or differential movement under a building, particularly when the desiccation extends to some depth.

2 MECHANICS OF VOLUME CHANGES

The surfaces of the clay particles are negatively charged and so attract positively charged ions in the pore water of soil. However, the ability of clay minerals to attract positive ions varies significantly. When water is freely available clay particles attract layers of bound water around them, referred to as 'diffuse double layers', and the spacing between the particles increases. If a suction is applied to the water, or the clay particles are compressed by an external force, the bound layers of water are reduced in thickness, some water is expelled, provided drainage conditions allow, and the clay particles move closer together. The suction, or externally applied pressures, required to bring the particles closer together can be very large (i.e. over 10 000 kPa). This is why clay soils harden when dried and why when wetted they can undergo large changes in volume or, if this is inhibited, exert high swelling pressures.

The swell-shrink behaviour of a clay soil under a given state of applied stress in the ground is controlled by changes in soil moisture suction. The relationship between pore water suction and water content depends on the proportion and type of clay minerals present, their microstructural arrangement and the chemistry of the pore water. Changes in soil suction are brought about by moisture movement through the soil due to evaporation from its surface in dry weather, by transpiration from plants, or alternatively by recharge consequent upon precipitation. The climate governs the amount of moisture available and is counteracted by that which is removed by evapotranspiration (i.e. the soil moisture deficit). In semi-arid climates there are long periods of high soil moisture deficit alternating with short periods when precipitation balances or exceeds evapotranspiration.

The volume changes which occur due to evapotranspiration from clay soils can be conservatively predicted by assuming the lower limit of the soil moisture content to be the shrinkage limit. Desiccation beyond this value cannot bring about further volume change.

Transpiration from vegetative cover is a major cause of water loss from soils in semi-arid regions. Indeed the distribution of soil suction in soil is primarily controlled by transpiration from vegetation and represents one of the most significant changes made in loading (i.e. to the state of stress in a soil). The behaviour of root systems is exceedingly complex and is a major factor in the intractability of swelling and shrinking problems. The spread of root systems depends on the type of vegetation, the soil type and groundwater conditions. The suction induced by the withdrawal of water fluctuates with the seasons, reflecting the growth of the vegetation and probably varies between 100 and 1 000 kPa (equivalent to pF values 3 and 4 respectively). The complete depth of active clay profiles usually does not become fully saturated during the wet season in semi-arid regions.

The moisture characteristic (moisture content v. soil suction) of a soil provides valuable data concerning the moisture contents corresponding to the field capacity (defined in terms of soil suction this is a pF value of about 2.0) and the permanent wilting point (the level at which moisture is no longer available to plants which corresponds to a pF value of about 4.2), as well as the rate at which changes in soil suction take place with variations in moisture content. This enables an assessment to be made of the range of soil suction and moisture content which is likely to occur in the zone affected by seasonal changes in climate.

The extent to which the vegetation is able to increase the suction to the level associated with the shrinkage limit is obviously important. In fact the moisture content at the wilting point exceeds that of the shrinkage limit in soils with a high content of clay and is less in those possessing low clay contents. This explains why settlement resulting from the desiccating effects of trees is more notable in low to
moderately expansive soils than in expansive ones.

When vegetation is cleared from a site, its desiccating effect is also removed. Hence the subsequent regain of moisture by clay soils leads to them swelling. Swelling movements on expansive clays in South Africa, associated with the removal of vegetation and subsequent erection of buildings, in many areas have amounted to about 150 mm. The largest upward movement recorded so far is 374 mm (Williams and Pidgeon, 1983).

The suction pressure associated with the onset of cracking is approximately pF 4.6. The presence of desiccation cracks enhances evaporation from soil. Such cracks lead to a variable development of suction pressure, the highest suction occurring nearest the cracks.

3 LABORATORY ASSESSMENT OF SWELLING AND SHRINKAGE POTENTIAL

An important aspect in the design, especially of light structures, is to assess the potential for volume change of a soil when the various geological, environmental and climatic factors give rise to significant changes in moisture content. The volume change potential does not represent a prediction of behaviour but the potential for such behaviour to occur if the appropriate conditions exist.

Methods of predicting volume changes soils can be grouped into empirical methods, soil suction methods and oedometer methods. Empirical methods make use of the swelling potential as determined from liquid and plastic limits, and activity. For example, Driscoll (1983) proposed that the moisture content (m) at the onset of desiccation (pF = 2) and when it becomes significant (pF = 3) could be approximately related to the liquid limit (LL), in the first instance m = 0.5 LL and in the second m = 0.4 LL. Anon (1980) suggested that the plasticity index provided an indication of the swelling and shrinkage potential as follows:

<table>
<thead>
<tr>
<th>Plasticity index (%)</th>
<th>Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over 35</td>
<td>Very high</td>
</tr>
<tr>
<td>22-48</td>
<td>High</td>
</tr>
<tr>
<td>12-32</td>
<td>Medium</td>
</tr>
<tr>
<td>Less than 18</td>
<td>Low</td>
</tr>
</tbody>
</table>

A degree of overlap was allowed. It should be borne in mind that because the determination of plasticity is carried out on remoulded soil, then these grades of volume change potential do not consider the influence of soil texture, moisture content, soil suction or pore water chemistry.

Vijayvergiya and Ghazzaly (1973) developed a method of estimating the amount of potential swell of clay soil from its natural moisture content and liquid limit (Fig. 1). Brackley (1975) and Weston (1980) also developed empirical relationships for swelling which involved the initial void ratio of the soil, its initial moisture content, its plasticity index or liquid limit and the external load. Brackley's method involves the following expression:

$$\text{Swell} = (5.3-147c-\log_{10}(p)) \times (0.525 + 4.1 \times 0.85 G)$$  \hspace{1cm} (2)

where $c$ is the original void ratio; $p$ is the external load (kPa); $G$ is the initial moisture content (%); and PI is the plasticity index. The correlation derived by Weston is similar to that of Brackley and is as follows:

$$\text{Swell} = 4.11 \times 10^{-4} (\text{LL}_a)^{0.17} \times p^{-0.508} \times w^{-2.23}$$  \hspace{1cm} (3)

where $\text{LL}_a$ is the LL(% < 500 μm/100) in the external load (kPa); and $w$ is the moisture content (%). The data used by Brackley to develop his equation was based on soils compacted at optimum moisture content. This condition is different from that of undisturbed soil. The expression is sensitive to small changes in plasticity index, for example, 1% change in the plasticity index results in almost a 15% change in the predicted heave.

One of the most widely used properties to predict the potential expansiveness of clay soils is their activity and the method developed by Van der Merwe (1964) is that most frequently used in Natal. It is based on plotting plasticity against percentage clay fraction (Fig. 2a). Van der Merwe derived the following expression for obtaining the amount of expansion:

$$\text{Heave (mm)} = F \exp \left(-0.377/D\right) \times \left(\exp(-0.377T)-1\right)$$  \hspace{1cm} (1)

where $D$ is the thickness (m) of an overlying non-expansive layer, if present; $T$ is the thickness (m) of the expansive layer; $F$ is a factor which depends the degree of expansion obtained from Figure 2a which includes four classes of potential expansiveness as follows:

<table>
<thead>
<tr>
<th>Potential expansiveness</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0</td>
</tr>
<tr>
<td>Medium</td>
<td>0.055</td>
</tr>
<tr>
<td>High</td>
<td>0.11</td>
</tr>
<tr>
<td>Very high</td>
<td>0.222</td>
</tr>
</tbody>
</table>

Alternatively the amount of potential expansion can be
suction is the stress which, when removed allows the soil to swell. In other words, the value of soil suction in a saturated, fully-swollen soil is zero. O’Neill and Poormoayed (1980) quoted the United States Army Engineers Waterways Experimental Station (USACEWES) classification of potential swell (Table 1) which is based on the liquid limit, plasticity index and initial (in situ) suction. The latter is measured in the field by a psychrometer.

Brackley (1980) also incorporated suction into his assessment of swell potential. He suggested that the maximum movement due to swelling beneath a building founded on expansive clay could be obtained from the expression:

\[
\text{Swell \%} = 0.1(\pi - 10) \log_{10} (S/p)
\]  

where \(\pi\) is the plasticity index, \(S\) is the soil suction at the time of construction and \(p\) is the overburden plus foundation pressure acting on each layer of soil.

Soil suction is not easy to measure accurately. Filter paper has been used for this purpose (McQueen and Miller, 1968). This method involves placing a piece of filter paper in close contact with the soil sample whose suction is to be measured. The sample and filter paper are carefully sealed in a plastic bag to prevent evaporation. The sample is left for seven days until the moisture content of the filter paper has been wetted up to an equilibrium value. The equilibrium water content of the filter paper depends directly on the pore water suction in the soil sample. Whatman’s No. 42 filter paper has been calibrated against soil samples of known suction; and the results using the filter paper method have proven to provide surprisingly accurate estimates of soil suction. This technique may be used over a range of soil suction from almost zero to 5 MPa. Chandler and Gutierrez (1986) showed that for suction in the range 80 to 6 000 kPa, for which the filter paper water content \(w_f\) was less than 47%, the relation between the soil suction (kPa) and \(w_f(\%)\) is given by:

\[
\log_{10} S = 4.48 - 0.0622w_f
\]

where \(S\) is the soil suction. For lower soil suction measurements and filter paper water content greater than 47% the relationship is:

\[
\log_{10} S = 6.05 - 2.48 \log_{10} w_f
\]

According to Chandler et al (1992) measurements of soil suction obtained by the filter paper method compare favourably with measurements obtained using psychrometers or pressure plates. Nonetheless there are a few factors which could significantly affect the results. Firstly, the filter paper must be in close contact with the soil sample. Secondly, the pressure of the soil on the filter paper must not be allowed to cause the paper to become compressed and so expel moisture. The third factor is to ensure that the sample is completely closed off to the atmosphere by placing it in not less than two well sealed plastic bags. Fourthly, the filter paper is very sensitive to moisture in the atmosphere, therefore weighing of the oven dried filter paper must be completed quickly and efficiently to avoid absorbing moisture.

The oedometer methods of determining the potential expansiveness of clay soils represent more direct methods. In the oedometer methods undisturbed samples are placed obtained from Figure 2b. Van der Merwe’s method does not take into account the initial moisture content or void ratio of the soil, two important parameters influencing soil heave. These methods are useful provided they are recognized as a simple swelling indicator methods and nothing more. The complexities of expansive soils cannot be adequately accounted for in simple empirical equations. The assumption is made that the soil is homogeneous or that it has well defined layers with distinct physical properties. In practice, this is seldom or never the case.

Soil suction methods use the change in suction from initial to final conditions to obtain the degree of swelling. Soil
Table 1. USAEWES classification of swell potential (From O'Neill and Poorymood, 1980)

<table>
<thead>
<tr>
<th></th>
<th>Liquid limit</th>
<th>Plastic limit</th>
<th>Initial (in situ)</th>
<th>Potential swell</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(%)</td>
<td>(%)</td>
<td>suction (kPa)</td>
<td>(%)</td>
<td></td>
</tr>
<tr>
<td>Less than 50</td>
<td>Less than 25</td>
<td>Less than 145</td>
<td></td>
<td>Less than 0.5</td>
<td>Low</td>
</tr>
<tr>
<td>50-60</td>
<td>25-35</td>
<td>145-385</td>
<td></td>
<td>0.5-1.5</td>
<td>Marginal</td>
</tr>
<tr>
<td>Over 60</td>
<td>Over 35</td>
<td>Over 385</td>
<td></td>
<td>Over 1.5</td>
<td>High</td>
</tr>
</tbody>
</table>

in the oedometer and a wide range of testing procedures are used to estimate the likely vertical strain due to wetting under vertical applied pressures, which may be equated to overburden pressure and that of the structure which is to be erected. Jennings and Knight (1957) devised the double oedometer test in which one sample of soil is tested at natural moisture content and another is tested under saturated conditions. Both specimens are held under a small load (1 kPa) for 30 mins. The specimen at natural moisture content is then consolidated, and the test cell containing the specimen to be tested under saturated conditions is inundated and the soil allowed to swell. Consolidation of the specimen is carried out once swelling ceases. A comparison of the two consolidation curves provides an indication of swelling potential. However, this test has been shown to underestimate the amount of expansion likely to occur in a clay soil (Burland, 1984). Hence Jennings et al (1975) proposed a single oedometer test in which the soil is placed in the oedometer and loaded at natural moisture content to 1 kPa. The sample is then inundated and allowed to swell until movement ceases. A normal consolidation test is then carried out. A measure of the potential swell is obtained from the change in void ratio between the original state and the value corresponding to the overburden pressure. This method has been shown to overestimate the amount of potential swell. Another approach is to load the specimen at its natural moisture content up to the applied overburden pressure plus structural load, submerge the sample and keep adding load so as to prevent swelling. The sample then comes to equilibrium at its swelling pressure. The sample is unloaded in increments to derive a swelling curve (Frydman and Calabresi, 1987).

A problem with oedometer testing can arise from presence of any small inclusions in the test specimen since they could inhibit proper consolidation of the sample. In oedometer testing one-dimensional compression/swelling occurs. However, in reality most expansive clays are fissured which means that lateral and vertical strains develop locally within the ground. Even when the soil is intact, swelling or shrinkage is not truly one-dimensional. The effect of imposing zero lateral strain in the oedometer is likely to give rise to over predictions of heave and the greater the degree of fissuring the greater the over prediction. The values of heave predicted using oedometer methods correspond with specific values of natural moisture content and void ratio of the sample. Therefore any change in these affect the amount of heave predicted.

The results obtained from the abovementioned tests can show a wide variation between one test method and another. This has been highlighted by Bull et al (1993) for expansive soils from Ladiesmith, Natal.

4 FOUNDATIONS FOR LOW-RISE BUILDINGS ON EXPANSIVE SOILS

Effective and economic foundations for low-rise buildings on swelling and shrinking soils have proved difficult to achieve. This is partly because the cost margins on individual buildings are low. Obviously detailed site investigation and soil testing are out of the question for individual dwellings. Similarly many foundation solutions which are appropriate for major structures are too costly for small buildings. Nonetheless the choice of foundation is influenced by the subsoil and site conditions, estimates of the amount of ground movement and the cost of alternative designs. Different building materials have different tolerances to deflections (Burland and Wroth, 1974). Hence materials which are more flexible can be used to reduce potential damage due to differential movement of the structure. The following are some examples of allowable deflection ratios (i.e. the maximum displacement (Δ) relative to a straight line between two points divided by the distance (l) separating the points):

<table>
<thead>
<tr>
<th>Material</th>
<th>Δ/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid brickwork</td>
<td>1/2000</td>
</tr>
<tr>
<td>Articulated brick</td>
<td>1/800</td>
</tr>
<tr>
<td>Brick veneer</td>
<td>1/500</td>
</tr>
<tr>
<td>Articulated brick veneer</td>
<td>1/300</td>
</tr>
<tr>
<td>Timber frame</td>
<td>1/200</td>
</tr>
</tbody>
</table>

These methods can be adopted when choosing a design solution for building on expansive soils, namely, provide a foundation and structure which can tolerate movements without unacceptable damage; isolate the foundation and structure from the effects of the soil; or alter or control the ground conditions. The following precautions may be taken where differential movements up to 25 mm are likely to occur:

(i) Deep strip foundations - preferably with nominal reinforcement.
(ii) Short, small diameter bored cast-in-place piles with suspended reinforced ground beams where swelling is anticipated.
(iii) A light stiffened raft. The design of the slab is dependent on assumed allowable relative deflections and it is usually necessary to incorporate certain anti-cracking features in the superstructure (Pidgeon, 1980).
(iv) The provision of movement joints in the superstructure.
(v) The use of soft cement mortar for all masonry.

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(vi) Floating floor slab with total separation from foundations and outer walls.
(vii) Flexible support of interior walls on floor slab.
Alternatively support the interior walls on separate deep strip or piled foundations.

In addition to these construction details moisture control measures should be adopted as far as possible. The isolation of foundation and structure has been widely adopted for 'severe' (25-50 mm differential movement) and 'very severe' (over 50 mm differential movement) ground conditions. Straight-shafted bored piles can be used in conjunction with suspended floors for severe conditions. The piles are sleeved over the upper part and provided with reinforcement. For severe conditions it may be necessary to place piles at appreciable depth (i.e. below the level of fluctuation of natural moisture content) and/or use underreams to resist the pull-out forces.

5 GROUND TREATMENT

Moisture control is perhaps the most important single factor in the success of foundations on shrinking and swelling clays. The aim is to maintain stable moisture conditions with minimum moisture content or suction gradients. The loss of moisture around the edges of a building which leads to the moisture content of the soil under the centre of the building being higher gives rise to differential heave. In order to control this an attempt should be made to maintain the same moisture content beneath a building. This can be achieved by the use of horizontal and vertical moisture barriers around the perimeter of the building, drainage systems and control of vegetation coverage. Common sources of moisture are broken pipes and poor drainage. Hence all water supply pipes and waste water pipes should have flexible connections and couplings; all rainwater pipes should be ducted well away from the foundations; storage tanks and septic tanks should be reinforced to minimize cracking and have flexible water-proofing; and the ground should be sloped away from the building to convey run-off away. Large trees and bushes should be situated at a distance of at least one and a half times their mature height away from foundations. Paving (approximately 2 m wide) around the perimeter of the building helps to reduce moisture fluctuations in the vicinity of the foundations. It also helps to reduce differential movements across the building. Ideally this paving should be laid on thick polythene or PVC.

A simple method of reducing or eliminating ground movements due to expansive soil is to replace or partially replace them with non-expansive soils. There is no requirement for the thickness of the replacement material but a minimum of 1 m has been suggested by Chen (1988). The material should be granular but it should not allow surface water to travel freely through the soil so that it wets any swelling soils in lower horizons. Hence the presence of a fine fraction is required to reduce permeability. If enough of this type of material cannot be obtained it may be possible to blend on-site soils with some imported granular soil.

If expansive soil is allowed to swell by wetting prior to construction and if the soil moisture content is then maintained, the soil volume should remain relatively constant and no heave take place. Pendion is the most common method of wetting. This may take several months to increase the water content to the required depth, notably in areas with deep groundwater surfaces. The presence of fissures may aid in the process, although vertical wells can be installed to facilitate flooding and thus decrease the time necessary to adjust the moisture content of the soil. Williams (1980) described a case where severe damage due to swelling was corrected by controlled wetting. The 'drip irrigation system' involves wetting clay soils beneath the foundations of structures. The method is cheaper than conventional underpinning methods. It involves installing a trench around the perimeter of a building and boring holes from the base of the trench at 1.5 m centres. The holes then are filled with coarse sand and a plastic pipe with nozzles located at the sand-filled holes is laid in a sand blanket at the base of the trench. The trench is backfilled and the pipe is gravity fed with water from a supply pipe. The inflow of water into the ground is regulated by a tap at the head of the line.

The amount of heave of expansive soils is reduced significantly when compacted to low densities at high moisture contents. In fact, Okura and Lee (1965) showed that expansive soils compacted above optimum moisture content underwent negligible swell for any degree of compaction. On the other hand, compaction below optimum resulted in excessive swell.

Many attempts have been made to reduce the expansiveness of clay soil by chemical stabilization. For example, lime stabilization of expansive soils, prior to construction, can minimize the amount of shrinkage and swelling they undergo. In the case of light structures, lime stabilization may be applied immediately below strip footings. However, the treatment is better applied as a layer beneath a raft so as to overcome differential movement. The lime stabilized layer is formed by mixing 4 to 6% lime with the soil. A compacted layer, 150 mm in thickness, usually gives satisfactory performance. Furthermore the lime stabilized layer redistributes unequal moisture stresses in the subsoil so minimizing the risk of cracking in the structure above, as well as reducing water penetration beneath the raft. One of the problems with lime stabilization is the ability to thoroughly mix the lime with the soil. If this does not occur, then the method is not effective. Premix or mix-in-place methods can be used (Bell, 1988). Alternatively lime treatment can be used to form a vertical cut-off wall at or near the footings in order to minimize movement of moisture.

The lime slurry pressure injection method has also been used to minimize differential movements beneath structures, although it is more expensive. The method involves pumping hydrated lime slurry under pressure into soil, the points of injection being spaced about 1.5 m apart. The lime slurry forms a network of horizontal sheets, often interconnected by vertical veins. Injection can be used to form a seam line around the perimeter of a building to provide a barrier to moisture movement, the seams extending below the critical zone of change in moisture content.

Lime columns, installed by a tool like a giant "eggbeater", have been used instead of piles as foundations for light structures. The columns reduce total and differential movements and may be placed in a square pattern with a concentration beneath loaded walls. The load of the structure can be distributed to the lime columns by way of a concrete slab.

Cement stabilization has much the same effect on expansive soils as lime treatment, although the dosage of cement needs to be greater for heavy expansive clays. Alternatively they
can be pretreated with lime.

6 CASE HISTORIES

6.1 Finch Road, Empangeni

A one-storey building had been constructed on a deep layer of black clay. The building was founded on shallow strip footings. Due to the recent drought, desiccation of the clay had led to severe cracking in the building. The pattern of the cracks indicated that the structure had suffered an "edges down" effect because of shrinkage of the clay. In other words, differential settlement had occurred, the edges of the building settling more than the centre because the clay dries out more at the edges. Also there were cracks up to 30 mm wide in the grounds around the building.

It was decided that before any damage to the structure was repaired that the clay ground should be artificially wetted. This was accomplished, in a phased manner, by creating a series of low earth bunds around the building at 5 m distance from the paved surround. The areas between the bund and the paved surround were then filled with water to a depth of 150 mm and the water level in the bund was maintained for 72 hours.

After wetting, the subsoil was maintained in a moist condition by placing a plastic sheet 300 mm below the existing ground level over most of the wetted area. At the edge of the wetted area the plastic sheet was taken to a depth of 1 m in a trench. The sheeting was then covered with earth made up to ground level. A permanent drip irrigation system was installed in a sand-filled trench located 0.5 m from the paved surround.

Augered piles were placed beneath the footings. Flexing joints were incorporated at appropriate locations in the walls and floor of the building to accommodate any future movements. The cracks were stitched and then plastered over.

6.2 Edinburgh Road, Westville

Severe cracking occurred in a low-rise building at the above location. The cracks ran from roof to floor and occurred above doors, and cornices had pulled away from walls. The building had been founded on mini-piles, 250 mm in diameter and extending to a depth of 1 to 1.5 m, supporting a ground beam. Nonetheless the building underwent differential settlement because of shrinkage of the subsoil.

Investigation of the ground by means of exploratory pits revealed up to 4 m of silty clay fill overlying residual clay and completely weathered dolerite. Boreholes indicated that the latter extended to a depth of some 9.5 m below ground level. A number of dynamic cone penetrometer tests were carried out to determine the consistency of the subsoil. Triaxial testing of soil samples gave an average angle of friction (ϕ) of 27° and of cohesion (c) of 39 kPa. When saturated the average values of ϕ and c were 23° and 6 kPa respectively. The calculated allowable load on each pile was 14 kN yet the actual load imposed was 64 kN. Hence the pile sizes were inadequate for the load carried and the situation was aggravated by the shrinkage of the clay material.

It was recommended that the new pile lengths should extend to 6.5 m below ground level. These were 114 mm diameter steel encased concrete jackpiles reinforced with steel bar. The jackpiles were installed by jacking against the underside of the footings until refusal or the load capacity was satisfied or the building started to lift. On completion the head of each pile was removed and encased in a concrete cap which was cast against the underside of the footings. The cracks were repaired after the underpinning was complete.

Concrete paving which had been badly cracked was removed and the ground beneath excavated to a depth of 300 mm and filled with inert backfill. The fill was compacted at 93% Modified AASHTO maximum dry density. New paving was then laid.

6.3 Knoll Road, St Winifredes

Another example of typical "edges down" cracking occurred in a low-rise building at the above location. Cracks were over 5 mm in width. Exploratory pits revealed 400 mm of brown sandy soil overlying 200 mm of brown clay. Beneath this was a pebble bed some 30 mm in thickness, below which was 600 mm of highly weathered shaley clay resting on shales.

Again underpinning was recommended. Reinforced augered piles were located at 1.5 m centres and taken into the shales. The cracks in the building were repaired by stitching, that is, 8 mm diameter steel bars, 600 mm in length, were placed in grooves running vertically across the cracks. The bars were grouted into place with epoxy resin. The surface of the walls concerned was then replastered.

Because the clay soil had a medium expansivity, it was decided that storm-water should be conveyed from the building in sealed conduits. The latter led into spreader boxes, the purpose of which was to allow evaporation of discharge. These basically are boxes lined with plastic sheeting and filled with gravel.

6.4 Northcliff Drive, Westville

The building concerned was located on moderately active clayey soil which overlies the Natal Group Sandstone. The foundations were strip footings which extended to a depth of between 250 mm and 1100 mm below ground level. These foundations proved too shallow and unsuitable. Because of the recent drought conditions, shrinkage of the soil led to differential settlement resulting in severe vertical and diagonal cracking of walls in the building. These cracks were several millimetres wide and a section of wall below the damp course had rotated outwards giving rise to a misalignment of the external brick face. Some of the corners of the brickwork had become separated from those above.

Inspection pits showed that the building was underlain by 0.7 m of colluvium, then fine silty sand to 2.4 m, above silty clay. The latter had a linear shrinkage of 10%, a liquid limit of 62%, a plasticity index of 37% and a clay fraction of 58%.

Again underpinning in the form of mini-piles was recommended. The cracks in the building were then stitched and filled.
6.5. Town Hill, Pietermaritzburg

A section of the major road between Durban and Johannesburg, at Town Hill near Pietermaritzburg, developed significant cracks in the pavement surface, as well as along the crests of embankments. In this particular location the road is laid over the Vryheid Formation which consists primarily of sandstones, shales and mudstones. However, the latter Formation is largely covered by a mantle of Recent deposits of colluvial origin consisting of a matrix of reddish brown silty clay containing cobbles and boulders of dolerite and occasional sandstone and shale fragments. Dolerite sills have intruded the Vryheid Formation. Steep crossovers on the road required the construction of fills and the fill material generally was obtained from the cuttings for the road, or where additional material was required, from borrow pits. Cracking occurred due to desiccation and shrinkage of the colluvial clays and fills during the recent drought. This was particularly the case where deep accumulations of such clays occupied palaeovalleys or gullies in the Vryheid Formation. These cracks extended laterally, coalescing with adjacent cracks as well as new cracks which developed into the fast lane of the freeway. A survey showed that 30 to 40 mm of vertical movement had occurred in this section of the road over a period of 14 months, with most occurring in the latter 3 or 4 months. The situation was further complicated by the fact that creep movements have occurred on steep crossovers. In other words gradual downslope movement was taking place which was aggravated by the formation of shrinkage cracks which would allow the ready entry of water into the ground, so facilitating movement. In fact the outer shoulder of the road already had experienced slip failure in this section and it was only a matter of time before the next slip, defined by the shrinkage cracks, occurred.

It therefore was recommended that the existing road should be realigned, that is, that it should be cut more deeply into the slope and thereby avoid, where possible, location on the fill embankments and colluvium. A monitoring system was to be installed to record any movements which took place during the period prior to the implementation of the remedial works.

7 CONCLUSIONS

Some clay soils are characterized by their ability to undergo significant volume changes when they are wetted or they dry out. Such ground movements can adversely affect buildings, especially low-rise buildings, leading to the development within them of significant cracking. These ground movements have an edges down effect upon the overlying structure. Roadways can also be subjected to cracking by volume changes which occur in the subsoils beneath.

Swelling and shrinking are associated especially with expansive clays. The latter usually contain clay minerals belonging to the smectite family. The swell-shrink behaviour of a clay soil is governed by soil suction. Changes in soil suction come about by loss of soil moisture due to evaporation, transpiration or groundwater recharge.

The laboratory assessment of volume changes in clay soils can be undertaken by a number of methods, which can be grouped into empirical methods, soil suction methods and oedometer methods. Empirical methods assess volume changes in terms of simple soil properties such as liquid limit, plasticity index, natural moisture content, void ratio or activity or some combination thereof. Although these methods are useful, they must be accepted as simple indicator methods and no more. Soil suction methods make use of the change in suction from initial to final conditions so as to obtain the amount of volume change. Oedometer methods represent more direct methods of assessing volume changes. Nevertheless they can give overestimates or underestimates of the likely amount of volume change. Indeed the results obtained from all the tests referred to can vary widely one with another.

In particular, low-rise buildings founded on expansive soils have been damaged by ground movements due to volume changes. Generally low-rise buildings are associated with low cost margins and as such extensive site investigations and soil testing are economically out of the question. Even so certain precautions can be taken such as using reinforced deep strip footings as foundations or founding of piles sunk to an adequate depth (ideally below the level of moisture change in the soil). Alternatively the ground can be treated to minimize changes in moisture content.

A number of case histories have been outlined in which severe cracking of buildings has been brought about by volume changes in subsoils. This has meant that the buildings had to be underpinned by piling and that the cracks had to be repaired by metal stitching and replastering. A major road which was affected by cracking attributable to soil shrinkage had to be relocated off susceptible colluvial clays and fills. This involved excavation into hilltops so that the new road rested on stiff cemented shales.

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Propriétés géotechniques des marnes de la région d’Alger
Geotechnical properties of marls in the Algiers region

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ABSTRACT: Compilation of geotechnical data and laboratory tests have been completed to determine the main properties of marls in the region of Algiers. Instabilities (landslides, swelling) are observed in these marls which can be explained by the geotechnical properties.

RESUME: Une compilation de données géotechniques ainsi que des essais de laboratoire ont été réalisés pour la détermination des propriétés géotechniques des marnes de la région d’Alger. Ces marnes sont le siège de nombreuses instabilités (glissements, gonflement) qui peuvent être expliquées par ces paramètres géotechniques.

1 INTRODUCTION
Les marnes occupent une grande partie des sols de la région d’Alger. Elles sont le siège de nombreuses instabilités dues à une intense urbanisation de la ville. Une connaissance détaillée des propriétés géotechniques est nécessaire pour la conception des ouvrages et l’aménagement urbain. On trouve dans la littérature scientifique peu d’études sur ces sols malgré que de nombreux projets de Genie civil aient été réalisés et de nombreux phénomènes observés.

Les propriétés géotechniques sont présentées à partir d’une compilation de données d’archives de différents organismes de travaux publics et quelques essais de laboratoire.

2 GEOLOGIE ET MINERALOGIE
On distingue les marnes du plaisancien qui affleurent dans le Sahel et les marnes d’el harrach (figure 1). Les marnes du plaisancien sont des formations tertiaires d’origine marine résultant de la sédimentation de sols à grains fins en milieu marin alors que les marnes d’el harrach sont des formations quaternaires résultant du remplissage du bassin de la mitidja. L’histoire géologique et l’origine de la formation de ces dépôts de sols a une grande influence sur les paramètres géotechniques.

Des analyses minéralogiques aux rayons X sur des échantillons prélevés à différents sites nous ont permis de déterminer la composition minéralogique de ces marnes (Tableau 1). Ce sont des sols composés d’un mélange de minéraux argileux et de carbonates. D’après les résultats de minéralogie, on voit la prédominance des minéraux interstratifiés de type illite-montmorillonite (30 à 45 %), la teneur élevée de l’illite (30 à 40 %) et la faible teneur en kaolinite (13 à 17 %) et en chlorite (5 à 8 %). Pour les minéraux non argileux la teneur en quartz est de 10 à 15 % et la teneur en carbonates de 10 à 20 %, ce qui fait classer ces sols plus dans la catégorie des argiles marneuses que dans celle des marnes. La difficulté de classification réside dans l’hétérogénéité de ces formations et le phénomène d’altération. Ce phénomène est plus important dans le cas des marnes du plaisancien qui sont affleurantes et où on distingue les marnes altérées des marnes saines.

3 PROPRIETES GEOTECHNIQUES
Les paramètres géotechniques ont été déterminés par une compilation de données géotechniques (Tableaux 2 et 3). Une étude statistique a été réalisée en prenant pour chaque couche les valeurs moyennes. La
Fig. 1 Carte géologique d'Alger (Benallal, K. et Ourabia, K.)

Tableau 1. Composition minéralogique des marnes du plaisancien et d'el harrach

<table>
<thead>
<tr>
<th>Illite</th>
<th>Kaolinite</th>
<th>Chlorite</th>
<th>Interstratifié illite montmorillonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td>%</td>
<td>%</td>
</tr>
<tr>
<td>Marnes alterées du plaisancien</td>
<td>20.6</td>
<td>14.2</td>
<td>5.8</td>
</tr>
<tr>
<td>Marnes saines du plaisancien</td>
<td>40.8</td>
<td>13.3</td>
<td>8.3</td>
</tr>
<tr>
<td>Marnes d'el harrach</td>
<td>32.5</td>
<td>17.5</td>
<td>trace</td>
</tr>
</tbody>
</table>
### Tableau 2 Propriétés géotechniques des marnes du plaisanaïen

<table>
<thead>
<tr>
<th></th>
<th>Marnes altières</th>
<th>Marnes découpées</th>
<th>Marnes calcaire</th>
</tr>
</thead>
<tbody>
<tr>
<td>Teneur en eau résiduée (w %)</td>
<td>23.48</td>
<td>19.73</td>
<td>10.06</td>
</tr>
<tr>
<td>Limite de liquéfaction (wil %)</td>
<td>61.47</td>
<td>56.28</td>
<td>51.5</td>
</tr>
<tr>
<td>Indice de plasticité (Ip X)</td>
<td>32.16</td>
<td>29.4</td>
<td>25.34</td>
</tr>
<tr>
<td>Densité sèche (Gd g/cm³)</td>
<td>1.62</td>
<td>1.37</td>
<td>1.81</td>
</tr>
<tr>
<td>Teneur en argile (1% Zn %)</td>
<td>45.2</td>
<td>45.16</td>
<td>46.33</td>
</tr>
<tr>
<td>Indice de compression (Cc X)</td>
<td>18.0</td>
<td>13.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Coefficient de gonflement (Cg X)</td>
<td>8.0</td>
<td>5.0</td>
<td>3.8</td>
</tr>
<tr>
<td>Pression de préconsolidation (Pc bar)</td>
<td>2.41</td>
<td>2.95</td>
<td>3.61</td>
</tr>
<tr>
<td>Cohésion (Ccu bar)</td>
<td>0.72</td>
<td>0.92</td>
<td>0.99</td>
</tr>
<tr>
<td>Angle de frottement interne (φcu °)</td>
<td>14.5</td>
<td>14.63</td>
<td>11.34</td>
</tr>
<tr>
<td>Cohésion résiduelle (Ccr bar)</td>
<td>0.1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Angle de frottement résiduel (φr °)</td>
<td>7.0</td>
<td>12.0</td>
<td>-</td>
</tr>
</tbody>
</table>

### Tableau 3 Propriétés géotechniques des marnes d’el harrach

<table>
<thead>
<tr>
<th></th>
<th>0 a m</th>
<th>5 10 m</th>
<th>10 15 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Teneur en eau résiduée (w %)</td>
<td>18.1</td>
<td>17.35</td>
<td>17.20</td>
</tr>
<tr>
<td>Limite de liquéfaction (wil %)</td>
<td>18.5</td>
<td>18.38</td>
<td>18.5</td>
</tr>
<tr>
<td>Indice de plasticité (Ip X)</td>
<td>23.57</td>
<td>23.45</td>
<td>23.78</td>
</tr>
<tr>
<td>Densité sèche (Gd g/cm³)</td>
<td>1.76</td>
<td>1.77</td>
<td>1.77</td>
</tr>
<tr>
<td>Teneur en argile (1% Zn %)</td>
<td>43.1</td>
<td>43.05</td>
<td>41.32</td>
</tr>
<tr>
<td>Indice de compression (Cc X)</td>
<td>12.4</td>
<td>11.0</td>
<td>11.5</td>
</tr>
<tr>
<td>Coefficient de gonflement (Cg X)</td>
<td>4.0</td>
<td>3.7</td>
<td>4.8</td>
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<tr>
<td>Pression de préconsolidation (Pc bar)</td>
<td>2.3</td>
<td>2.55</td>
<td>2.81</td>
</tr>
<tr>
<td>Cohésion (Ccu bar)</td>
<td>0.77</td>
<td>1.01</td>
<td>1.18</td>
</tr>
<tr>
<td>Angle de frottement interne (φcu °)</td>
<td>10.85</td>
<td>15.92</td>
<td>13.66</td>
</tr>
<tr>
<td>Cohésion résiduelle (Ccr bar)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Angle de frottement interne résiduel (φr °)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

783
décomposition par couches se fait suivant le degré d’altération et la variation des propriétés géotechniques.

3.1 Propriétés physiques

Les résultats montrent que les marnes de la région d’Alger sont des sols fins avec un fort pourcentage de fines (10 à 45 %), denses et de consistance raide. On voit d’après les diagrammes de plasticité (figure 2) la plasticité élevée de ces argiles. Les marnes du plaisancien sont plus plastiques (Ip de l’ordre de 30) que les marnes d’el harrach (Ip de l’ordre de 25) et ont un pourcentage de fines plus grand.

3.2 Propriétés mécaniques

3.2.1 Compressibilité

La variation de l’indice de compression Cc et de la pression de préconsolidation Pc (figure 3) déterminés à partir d’essais oedométriques conventionnels nous donne une idée sur la compressibilité de ces sols. Ce sont des argiles peu à moyennement compressibles (Cc de 0.08 à 0.19) et surconsolidées. D’après le coefficient de gonflement Cg les marnes altérées ont un fort potentiel de gonflement (Cg=0.08) alors que les marnes saines et les marnes d’el harrach sont peu gonflantes ce qui est observable sur le terrain.

3.2.2 Cisaillement

La résistance au cisaillement des marnes a été déterminée en considérant la résistance de pic et la résistance résiduelle (figure 4).

- Résistance de pic

Les résultats sont donnés à partir des essais classiques de cisaillement (essai de cisaillement direct, essai triaxial) de type consolidé non drainé CU. Les paramètres de résistance montrent une résistance au cisaillement moyenne à élevée pour les marnes saines du plaisancien et les marnes d’el harrach, ce qui explique leur bonne portance. Par contre les marnes altérées du plaisancien ont une résistance plus faible due à leur sensibilité à l’eau.

- Résistance résiduelle

On a réalisé des essais de cisaillement annulaires pour la détermination des paramètres résiduels Cr et 9r. Ces essais nous permettent de simuler de larges déplacements qui correspondent généralement à des glissements anciens réactivés. Ce sont des essais dont la durée est longue et qui sont réalisés sur des échantillons remaniés. Les résultats montrent pour les marnes du plaisancien une cohésion résiduelle nulle Cr=0 et des angles de frottement résiduels 9r variant de 7° à 12°, ces paramètres nous permettent une meilleure évaluation de la résistance résiduelle de ces argiles et son utilisation dans les calculs de stabilité des glissements renouvelés.

1. PROBLEMES GEOTECHNIQUES

De nombreuses instabilités sont observées au niveau des marnes de la région d’Alger. La forte plasticité et la surconsolidation ainsi que la minéralogie des marnes altérées du plaisancien montrent une forte tendance au gonflement qui s’observe sur le terrain par des désordres au niveau des constructions légères (figures). Ces désordres sont causés par les soulèvements et laissements des structures fondées sur ce type de sol et ont un impact sur le plan économique.

On observe aussi des mouvements de terrain (glissement de terrain, flange). Les glissements de terrain (glissement du telemly, glissement d’el achour) sont des glissements anciens réactivés dont la surface de rupture est généralement au contact marnes altérée - marnes saine qui est une zone critique comme il a été montré par les paramètres géotechniques. Le calcul de stabilité de ces glissements nécessitent l’utilisation de la résistance résiduelle pour éviter des ruptures catastrophiques et prévoir ainsi des solutions adéquates.

Les mouvements de flange dans ces marnes se manifestent par une inclinaison des obstacles naturels précédant généralement les glissements de terrain et constituent un danger à long terme.

5. CONCLUSION

La compilation des différentes données géotechniques nous a permis de faire une synthèse sur le comportement des marnes de la région d’Alger.

On distingue les marnes du plaisancien et les marnes d’el harrach. Les marnes saines du plaisancien sont de bonne portance et résistantes mais dès qu’elles sont altérées elles sont le siège de
Fig. 2 Diagrammes de plasticité

nombreuses instabilités. Les marnes d'el Harrach ont de meilleures propriétés.
Une meilleure approche du comportement de ces argiles est nécessaire par des études détaillées du gonflement et des mouvements de terrain et la recherche d'un modèle de comportement qui simulerait le mieux le comportement de ces argiles.

REFERENCES:


Fig. 3 Variation de la pression de préconsolidation en fonction de la profondeur

Fig. 4 Enveloppes de rupture de Mohr Coulomb

Marnes du plaisancien prof. 5 à 7 m
Marnes du plaisancien prof. 9 à 12 m
Relationships between joint density and P-wave velocity in rock units of the Cantabrian Zone (NW Spain)
Relations entre la densité des diaclases et la vitesse des ondes P dans des massifs rocheux de la Zone Cantabrique (NW d’Espagne)

M. Gutiérrez-Claverol, L. Rodríguez-Bouzo, B. Sánchez-Fernández & M. Torres-Alonso
ETS de Ingenieros de Minas, Universidad de Oviedo, Spain

ABSTRACT: Determination of the joint density index of different Paleozoic formations has been possible through the study of jointing in these lithostratigraphic units. This has been accomplished for different rocks and the relation between joint density and P-wave velocity has been determined by realisation of seismic profiles.

From the relationships between seismic parameters and fracture density different particularities may be inferred. Two groups are distinguished in the limestone depending on the velocity of primary waves, but both of them show an inverse relationship between propagation velocity and joint density index -when the index decreases the velocity increases-. The silicilastic rocks show less slope and correlation with carbonated rocks.

RÉSUMÉ: Il s’agit d’une analyse de la fracturation qui affecte diverses formations paléozoïques et du calcul d’un index de densité de diaclases qui présente un grand intérêt pour la caractérisation géomécanique de ces massifs. De plus, plusieurs profils sismiques ont été tracés à fin de caractériser les matériaux rocheux, en analysant le rapport entre la fracturation et la vitesse de propagation des ondes P.

En ce qui concerne la relation entre la vitesse des ondes P et l’index de densité de fracturation, plusieurs particularités sont relevées. Parmi les roches calcaires l’on remarque l’existence de deux familles distinctes suivant les hautes ou basses vitesses de propagation des ondes, mais dans chacune d’elles la relation avec le taux de fracturation présente la même tendance inverse -plus la vitesse est grande, plus le taux est bas-. Dans le cas des roches détritiques, cette relation présente un écart moins prononcé et un degré de corrélation inférieur.

1 INTRODUCTION

The work has been carried out in the complete Paleozoic succession cropping out along the valley of the Bernesga river (N of León) from Cambrian to Carboniferous periods, about which only scarce bibliographical references, concerning mechanical properties can be found. The two aspects considered: joint density and primary seismic wave velocity, have been dealt with in articles by Rodríguez Bouzo et al. (1992) and by Sánchez Fernández et al. (1992), which analyse some of the materials studied here. From the same area comes the essay by Carbó Gorosabel (1984) in which geotechnical parameters are obtained by means of Down-Hole tests.

The analysis of rock mass jointing is frequently used to determine some of the indexes (RSR, RMR, IF and Q) on which geomechanical classifications are based, jointing orientation, spacing and degree of aperture playing the main role in them. The measured fracturation affects 16 lithostratigraphic formations, grouped in 27 measurement stations, the situation of which is shown in Fig. 1. The best studied lithology has been carbonates; due to its pelitic nature, no measurements were possible on the Formigoso, Fuego and Vegamión Formations, and La Vid shales and Alba shales.

Field geophysical measurements were also made in order to determine, at rock mass scale, the variations in propagation velocity which longitudinal elastic waves (P-waves) present when moving across it. Analysis of these variations allows to determine features such as degree of weathering, lithological changes, fracturing degree, discontinuities, presence of fluids, etc., which facilitates a better knowledge of material behaviour as far as geotechnical effects are concerned. At the same time, wave propagation was also measured on some laboratory samples, in order to evaluate these formations at intact rock scale.
Fig. 1. Geological sketch-map and location of measuring stations. Legend: (1) Cambrian, (2) Ordovician, (3) Silurian y Devonian, (4) Carboniferous and (5) Stephanian.

2 GEOLOGICAL ENVIRONMENT OF THE STUDIED AREA

The area of study is located in the Variscan Belt of the Iberian Peninsula, and within it, in the so-called Cantabrian Zone. More specifically, it includes two geological units: Sobia-Bodón and Somiedo-Correcilla (Fig. 1). Paleozoic stratigraphic succession is complete, formed by alternating carbonate and sandstone formations, representative of the Cambrian (Herrera, Láncara and Ovile Formations), Ordovician (Barrios Fm.), Silurian (Formigoso and San Pedro Fms.), Devonian (La Vid, Santa Lucía, Huergas, Portilla, Nocedo, Fuego and La Ermita Fms.) and Carboniferous (Bañales, Vegamán, Alba, Barcaliente, Valdejela and San Emilián Fms.). The dominant formations are those of carbonated character: Láncara, La Vid (in part), Santa Lucía, Portilla, Nocedo (in part), Bañales, Alba, Barcaliente, Valdejela and, partially, San Emilián.

In the northern sector of the La Sobia-Bodón unit, there is an important stratigraphical break, which covers the greater part of the Middle and Upper Devonian (absence of the Huergas, Portilla, Nocedo and Fuego Fms). The structures present a quite generalised E-W orientation, locally modified (for example in the Busdongo area) by N-S folds formed by lateral ramps.

The Somiedo-Correcilla unit, constitutes the allochton of a higher unit, the basal nappe just S of Villamanín, where it is overlain by synorogenic materials of the San Emilián-Villamanín basin (Lower Bashkirian-Moscovian). The stratigraphic sequence becomes more complete towards the south.

It is worth mentioning the existence of the León Fault to the N and the Sabero-Gordón Fault to the S which laterally makes up the southern limit of the Cifera-Matlallana Stephanian basin. Transversal faults are also present, with a general NE-SW trend, despite a much lesser importance.

The Stephanian is found unconformably overlying the nappe surfaces, and appears folded along E-W axes. Alonso et al. (1990) concluded that the folds related with the nappes were produced in the Stephanian during this period, reorienting their course and acquiring the present S vergent.

3 METHODOLOGY

The study of joint density was made applying a method proposed by Davis (1984) and recently modified by Gutiérrez Claverol et al. (1991). Basically, consists in measuring all joins, independently of their size or any other features, included in the circle of reference of the pre-established area (circle-inventory method) being later computer processed, in order to obtain rose and density diagrams (Fig. 2), together with joint density index.

![Fig. 2. Representative examples of rose diagram and density diagram of the measuring stations.](image)

Once all data from one station have been collected, the fracture density is quantified, using the relationship: \( p_j = \frac{L}{\pi r^2} \), where \( p_j \) represents joint density, \( L \)
the accumulated length of all joints, and \( r \) the radius of the inventory circle. The length of joints is normally measured in cm, and consequently, as joint density is a rate length/area, the density index is expressed in cm/cm² (cm⁻¹). In the density diagram, concentration maxima can be observed in characteristic points of the sets of sub parallel joints, which define the different existing families, and on these maxima direction an average dip of bedding can be measured.

For the study of P-wave propagation velocity, seismic refraction method was chosen, as it was considered the most interesting and practicable one for the determination of the aforementioned feature, this being one of the parameters which is involved in numerous correlations with geomechanical factors. The knowledge of seismic waves velocity, when they travel across rock materials, allows the evaluation of their degree of weathering and fracturing.

The field tests consisted in the elaboration of 22 geophysical profiles. The waves were created artificially, either by means of the impact of a 10 kg thumper with free fall from a 3 meters height, generating an energy of 30 kpm, or with a sledge hammer. Two equipments were used. The first one was the ABEM TRIO, type 5352, which works with a maximum of 12 seismic traces. Recording velocity varied between 50, 100 y 200 cm/s, with a separation across the time lines of 2 milliseconds. The other equipment used was a Geometrics digital seismograph, model ES-2401, with vertical geophones, which offers simplicity and reliability, both in wave generation, and in record logging, together with a greater measurement accuracy, with 9 to 12 geophones.

For the interpretation of data obtained in these tests, and as the rock masses present a subvertical bedding, it was chosen to measure the propagation velocity of surface and shallow-travelling waves, measuring the time it takes the wave to travel along the outcrop surface from the shooting point to the first geophone and from this to the second one and so on. Knowing the time and the distances determined during the realisation of each test, the velocity at which the direct wave crosses each different lithological medium is calculated. In order to be able to relate the velocity to given domains within the same formation, the geophones were placed in the inter-phases or contacts between different levels.

Normally, for each of the studied profiles, several shots were carried out from the opposite ends, and the corresponding dromochrones were set up (Fig. 3). It was observed that when one of the slices presents surface weathering, occasionally, the velocities obtained for each of the opposing shots are not identical, offering very different values, due to the interference of surface waves close to the source of vibrations. This seems to be due to the surface weathering that exists in the rock masses, in such a way that the area with greater velocity corresponds to the deepest ones in the rock mass, which are fresher, and consequently, the ones which present a smaller degree of weathering.

In the laboratory tests, the P-wave propagation velocity was measured in dry samples from carbonate Carboniferous rocks. The method used was the "ultrasonic impulse" with an Oyo Corporation equipment, model 5217A New Sonicview, which consists of a 63 kHz transmitter, and a P.A.C. receiver, with a band jump from 20 to 100 kHz. The pulse frequency was 128 Hz, sampling interval 400 nanoseconds, and transmitter excitation voltage 400 volts. The test consisted in applying the sample (cylindrical or cubic test-tubes) a vibration, and measuring the time the wave needed to propagate across it between two parallel faces, one in contact with the transmitter, and the other with the receiver.

4 JOINTING GEOMECHANICAL FEATURES

Jointing shows a dominant trend between NW-SE and NNE-SSW with bedding dipping variably, generally accompanied -locally- by another the E-W set, with S dipping bedding. The detected sets, together with their jointing index are shown in Tables I and II, where besides the orientation and the percentage of the different families present in each measurement
TABLE I. Joint orientation and joint density index of the Cambrian, Ordovician, Silurian and Devonian formations.

<table>
<thead>
<tr>
<th>GEOLOGICAL TIME</th>
<th>FORMATION</th>
<th>STATION</th>
<th>BEDDING</th>
<th>STRIKE AND DIP OF JOINTING (%)</th>
<th>( \rho_j )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Devonian</td>
<td>La Ermita</td>
<td>J-18</td>
<td>352/87</td>
<td>194/20 (6) 77/75 (15) 296/80 (29) 291/58 (19) 90/60 (10)</td>
<td>0/0 (21) 0.13</td>
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<tr>
<td></td>
<td>Noceda</td>
<td>J-17</td>
<td>187/74</td>
<td>40/62 (4) 82/72 (11) 116/79 (31) 516/31 (47)</td>
<td>0/0 (17) 0.33</td>
</tr>
<tr>
<td></td>
<td>J-16</td>
<td>277/60</td>
<td>39/60 (46) 164/79 (56)</td>
<td></td>
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<tr>
<td>Ordovician</td>
<td>Portilla</td>
<td>J-15</td>
<td>345/85</td>
<td>236/93 (46) 72/60 (11) 230/35 (5) 72/22 (5)</td>
<td>0/0 (30) 0.22</td>
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<td></td>
<td>Huergas</td>
<td>J-14</td>
<td>260/60</td>
<td>33/85 (23) 125/57 (18) 180/75 (53)</td>
<td>0/0 (4) 0.22</td>
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<tr>
<td></td>
<td>J-13</td>
<td>0/72</td>
<td>82/98 (43) 182/19 (37)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silurian</td>
<td>Santa Lucia</td>
<td>J-12</td>
<td>20/70</td>
<td>265/53 (6) 116/72 (55) 137/14 (16)</td>
<td>282/21 (23)</td>
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<tr>
<td></td>
<td>La Vid</td>
<td>J-11</td>
<td>13/67</td>
<td>246/26 (58) 275/65 (3) 112/85 (52) 99/64 (27)</td>
<td></td>
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<tr>
<td></td>
<td>San Pedro</td>
<td>J-10</td>
<td>34/67</td>
<td>230/16 (67) 298/74 (22) 329/87 (31)</td>
<td>332/9 (2) 0.07</td>
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<td></td>
<td>Pedro</td>
<td>J-9</td>
<td>0/72</td>
<td>200/31 (65) 270/87 (55)</td>
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<tr>
<td>Ordovician</td>
<td>Barrios</td>
<td>J-8</td>
<td>330/66</td>
<td>74/60 (60) 250/65 (15) 85/41 (10)</td>
<td>180/20 (16)</td>
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<td></td>
<td>J-7</td>
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<td>274/65 (12) 119/67 (10) 132/60 (26)</td>
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<td>0.34</td>
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<td>Cambrian</td>
<td>Oville</td>
<td>J-6</td>
<td>342/87</td>
<td>246/72 (59) 60/56 (14)</td>
<td>156/21 (27)</td>
</tr>
<tr>
<td></td>
<td>J-5</td>
<td>194/43</td>
<td>79/52 (35) 71/68 (38)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Láncara</td>
<td>J-4</td>
<td>341/83</td>
<td>237/79 (11) 90/11 (32) 271/62 (21)</td>
<td>269/55 (27)</td>
</tr>
<tr>
<td></td>
<td>J-3</td>
<td>166/88</td>
<td>45/28 (65) 91/52 (45) 279/93 (30)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Herrera</td>
<td>J-2</td>
<td>162/90</td>
<td>77/78 (10) 103/89 (20) 101/53 (6) 288/15 (6)</td>
<td>0/0 (49) 0.20</td>
</tr>
<tr>
<td></td>
<td>J-1</td>
<td>169/90</td>
<td>60/49 (36) 236/88 (3) 62/75 (2) 229/40 (51)</td>
<td></td>
<td>0.39</td>
</tr>
<tr>
<td>GEOLOGICAL TIME</td>
<td>FORMATION</td>
<td>STATION</td>
<td>BEDDING</td>
<td>STRIKE AND DIP OF JOSTING (%)</td>
<td>( \rho_j )</td>
</tr>
<tr>
<td>-----------------</td>
<td>-----------</td>
<td>---------</td>
<td>---------</td>
<td>-------------------------------</td>
<td>-----------</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>WNW</td>
<td>NW</td>
<td>NNW</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>San</td>
<td>J-31</td>
<td>312/68</td>
<td>17/81 (8)</td>
<td>210/61 (19)</td>
</tr>
<tr>
<td></td>
<td>Emiliano</td>
<td>J-30</td>
<td>329/81</td>
<td>206/21 (7)</td>
<td>238/60 (11)</td>
</tr>
<tr>
<td>A</td>
<td>Valdeteja</td>
<td>J-29</td>
<td>338/89</td>
<td>197/68 (20)</td>
<td>59/67 (10)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J-28</td>
<td>37/90</td>
<td>275/10 (10)</td>
<td>293/86 (21)</td>
</tr>
<tr>
<td>R</td>
<td></td>
<td>J-27</td>
<td>307/58</td>
<td>200/32 (26)</td>
<td>220/87 (11)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Barcaliente</td>
<td>J-26</td>
<td>328/76</td>
<td>206/66 (13)</td>
<td>215/79 (13)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J-25</td>
<td>197/84</td>
<td>270/70 (39)</td>
<td>277/26 (17)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J-24</td>
<td>333/75</td>
<td>237/79 (30)</td>
<td>76/63 (26)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J-23</td>
<td>179/27</td>
<td>226/63 (26)</td>
<td>271/19 (2)</td>
</tr>
<tr>
<td>F</td>
<td></td>
<td>J-22</td>
<td>37/28</td>
<td>195/40 (19)</td>
<td>45/60 (14)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>Alba</td>
<td>J-21</td>
<td>18/85</td>
<td>63/74 (4)</td>
<td>13/76 (23)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J-20</td>
<td>308/65</td>
<td>109/31 (11)</td>
<td>213/70 (7)</td>
</tr>
<tr>
<td>G</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Balceas</td>
<td>J-19</td>
<td>6/76</td>
<td>76/80 (33)</td>
<td>221/32 (58)</td>
</tr>
</tbody>
</table>
station, the density index ($p_j$) which varies from 0.06 to 0.57 cm$^{-1}$ is represented.

Plotting the jointing index against the thickness of the strata on which the measurements were made, a certain linear relationship appears. In the case of stratified carbonate rocks, and alternating sandstones and shales, the relationship between both parameters is of the inverse logarithmic type (Figs. 4 y 5).

On the contrary, quartzite rocks present a direct relationship (Fig. 6). No dependence has been found between both variables in the massive carbonated rocks, concluding thus that after a certain bed thickness (normally above 40 cm) $p_j$ is independent of thickness, which agrees with the conclusions of Ladeira y Price (1981).

There seems to be an anomalous behaviour in the formations characterised by alternating competent and incompetent beds, and in those that present a great heterogeneity in the bed thickness (i.e., Oville, San Pedro, Huergas and Nocedal Fms.).

5 MEASUREMENT OF SEISMIC VELOCITIES

Starting from the dromochrones, before calculating the slopes of the different resulting slices, the velocities for each material were obtained, and the slices with different velocities and its value being deducted. In some profiles, the existence of several zones was detected. These were mainly two, which presented different values for the velocity parameter. The ones with a greater slope, belonging to a smaller velocity, corresponds to the surface zone, and therefore, to the one with greater weathering.

Table III presents the data obtained for the Paleozoic formations of the Cambrian and Devonian periods, whereas Table IV synthesises the velocity values for P-waves measured, in the rock masses and in the laboratory, for rocks of the Carboniferous period.

The fact that the surface zone gives lower velocity values is due principally to weathering processes, decompression of the rock mass, fracturing, etc. which diminish as we go deeper. The results obtained in the laboratory are in general, higher than the field ones, as they were carried out on the rock matrix, and
therefore, free of the discontinuities which affect rock mass.

However, there is a certain dispersion of results due to the texture heterogeneity typical of sedimentary rocks. In the cubic samples, the velocity measured perpendicularly to stratification is significantly smaller than parallel to it, which can be accounted for by, apart from the very effect of bedding, the existence of thin sedimentary lamination.

### TABLE III. P-wave propagation velocity in Cambrian, Ordovician, Silurian and Devonian formations.

<table>
<thead>
<tr>
<th>GEOLOGICAL TIME</th>
<th>FORMATION</th>
<th>STATION</th>
<th>VELOCITY OF P WAVE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Depth &gt;3m</td>
</tr>
<tr>
<td>D</td>
<td>Ermite</td>
<td>S-24</td>
<td>4100</td>
</tr>
<tr>
<td></td>
<td>Nocedo</td>
<td>S-22</td>
<td>2850 (4500)</td>
</tr>
<tr>
<td></td>
<td>Portilla</td>
<td>S-21</td>
<td>2750</td>
</tr>
<tr>
<td></td>
<td>Hueras</td>
<td>S-19</td>
<td>3500</td>
</tr>
<tr>
<td></td>
<td>Santa</td>
<td>Llocta</td>
<td>S-18</td>
</tr>
<tr>
<td></td>
<td>La</td>
<td>S-17</td>
<td>3300</td>
</tr>
<tr>
<td></td>
<td>Vid</td>
<td>S-16</td>
<td>3650</td>
</tr>
<tr>
<td>SILEURIAN</td>
<td>San</td>
<td>S-15</td>
<td>2500</td>
</tr>
<tr>
<td></td>
<td>Pedro</td>
<td>S-14</td>
<td>5500</td>
</tr>
<tr>
<td></td>
<td>S-13</td>
<td>3750</td>
<td>1050</td>
</tr>
<tr>
<td>ORDовичIAN</td>
<td>Barrios</td>
<td>S-12</td>
<td>2450</td>
</tr>
<tr>
<td></td>
<td>Ovitte</td>
<td>S-11</td>
<td>7000 (4350)</td>
</tr>
<tr>
<td></td>
<td>S-10</td>
<td>3550</td>
<td>1150</td>
</tr>
<tr>
<td></td>
<td>A</td>
<td>S-9</td>
<td>4650</td>
</tr>
<tr>
<td></td>
<td>S-8</td>
<td>3350</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td>S-7</td>
<td>4500</td>
<td>2000</td>
</tr>
<tr>
<td></td>
<td>S-6</td>
<td>2300</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>2900</td>
<td>6000</td>
<td>850</td>
</tr>
<tr>
<td></td>
<td>6700 (2)</td>
<td>4000 (3)</td>
<td>1000 (3)</td>
</tr>
<tr>
<td></td>
<td>3600</td>
<td>1650</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>S-2</td>
<td>3950</td>
<td>950</td>
</tr>
<tr>
<td></td>
<td>S-1</td>
<td>3750</td>
<td>550</td>
</tr>
</tbody>
</table>

**NOTES: (1) Oolite intercalated zone. (2) Thick beds zone. (3) Stratified zone.**

### 6 CONCLUSIONS

#### 6.1 Jointing

* Jointing orientation allows to distinguish several families trending from NW-SE to NE-SW, with predominant WNW and NW directions. Occasionally, and specially related to longitudinal folds, E-W trending sets appear.

* In general, dominant join families are suborthogonal to stratification, being evident certain differences related with folds (limb and hinge).

* Joint density index varies depending on:
  - Space location. It is greater in the La Sobia-Bodón unit.
  - Lithology. Carbonate rocks generally have a somewhat greater than siliciclastic ones.
  - Bed thickness. Bedded carbonate rocks and alternating sandstones and shales have an inverse logarthmic relationship between the bed thickness and fracturation index, in such a way that the smaller values for the latter parameter appear in greater bed thickness. In the case of quartzitic rocks the tendency is opposite, presenting a direct thickness relationship. Massive carbonate rocks do not show any kind of correlation.

* The geometrical figure resulting from the join intersections is generally of the rhomboidal or trapezoidal shape.

* The lithostratigraphic units in which jointing presents a greater geomechanical importance are Herrerfa, Lánca, Barrios and Nocedo Fms. and above all, the Carboniferous limestones.

#### 6.2 P-waves velocity

* The P-wave velocity data separates two zones in the rock masses, a shallow one, with low velocity values, which indicates a high degree of weathering, and includes, approximately, the first three metres and a deeper one, not weathered, with higher velocity.

* Discontinuities in the rock mass, derived both from bedding surfaces (the greater velocities are associated to massive outcrops) and fracturation ones, greatly reduce wave propagation velocity.

* Velocities obtained in the laboratory show a clear relationship with the structures present in the test-tubes. Smaller velocities are associated to samples that show a greater number of structures (predominantly sedimentary laminations, porosity, small fractures), and especially when these are perpendicularly to wave propagation.
TABLE IV. P-wave propagation velocity in Carboniferous formations.

<table>
<thead>
<tr>
<th>GEOLOGICAL TIME</th>
<th>FORMATION</th>
<th>STATION</th>
<th>VELOCITY OF 'P' WAVES</th>
<th>DINAMIC YOUNG'S MODULUS (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>IN SITU</td>
<td>LABORATORY</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Depth &gt;3m</td>
<td>Depth &lt;3m</td>
</tr>
<tr>
<td>C</td>
<td>San Emiliano</td>
<td>S-37</td>
<td>-</td>
<td>6500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-36</td>
<td>5150</td>
<td>6900</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-35</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Valdeteja</td>
<td>S-34</td>
<td>5200</td>
<td>4000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-33</td>
<td>4550</td>
<td>5700</td>
</tr>
<tr>
<td></td>
<td>Barcallente</td>
<td>S-32</td>
<td>-</td>
<td>6000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-31</td>
<td>-</td>
<td>2700</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-30</td>
<td>3800</td>
<td>4800</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-29</td>
<td>5450</td>
<td>5300</td>
</tr>
<tr>
<td></td>
<td>Limestones</td>
<td>S-28</td>
<td>5800</td>
<td>3800</td>
</tr>
<tr>
<td></td>
<td>Shales</td>
<td>-</td>
<td>2100</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Limestones</td>
<td>S-27</td>
<td>5800</td>
<td>5700-6800 (5)</td>
</tr>
<tr>
<td></td>
<td>Shales</td>
<td>-</td>
<td>2100</td>
<td>1200</td>
</tr>
<tr>
<td></td>
<td>Limestones</td>
<td>S-26</td>
<td>-</td>
<td>2500</td>
</tr>
<tr>
<td></td>
<td>Shales</td>
<td>-</td>
<td>2200</td>
<td>1100</td>
</tr>
<tr>
<td></td>
<td>Bales</td>
<td>S-25</td>
<td>-</td>
<td>8400</td>
</tr>
</tbody>
</table>

NOTES: (1) The measurement was parallel to the stratification. (2) Very surface measurement. (3) Stratified zone. (4) Zone with stratified intercalations. (5) Measurements on cubic samples, parallel and perpendicular to the structures.
6.3 Relationship between $V_p$ and $\rho_j$

* Several peculiarities appear as regards the relationship between $V_p$ and joint density ($\rho_j$) (Fig. 7):

- Within the carbonate rocks two different families can be found, one with high, and another with low wave propagation velocities; but in both, the relationship with the joint index shows the same inverse tendency (the greater the velocity the smaller the index). The relationship straight lines are parallel between themselves, and have similar correlation degrees (0.63 and 0.66 respectively).

- In the case of siliciclastic rocks, the relationship straight line presents a lesser slope, with a lower correlation degree.
Fine grained soils and the Atterberg limits of sand/clay mineral mixes
Sols fins et limites d’Atterberg des mélanges sables/argiles

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Engineering Geology Research Unit, University of Bristol, UK

ABSTRACT: The paper briefly reviews the confusion in the description of engineering soils and stresses the importance of recording the classification used and the laboratory procedure followed in order to ensure the verbal description and the figures produced can be correlated by others. The paper then presents in both tabular and graphical form, the Atterberg limits obtained on various mixes of sand with differing proportions of the clay minerals kaolinite, illite and calcium montmorillonite.

RESUME: Cette étude examine la confusion dans la description des sols pour souligner l’importance de noter la classification usée et le procédé de laboratoire suivi pour assurer que la description verbale et les chiffres produits peuvent être en corrélation par des autres. Puis l’étude présente, en la forme de tables et en la forme graphique, les limites Atterberg qui ont été obtenues avec des doses différentes de la sable et des minérales d’argille kaolinite, illite et calcium montmorillonite.

INTRODUCTION

Engineering soils are the un lithified detrital particles which, when cemented, make up the argillaceous, arenaceous and rudaceous rocks typically referred to as mudrocks, sandstones and conglomerates/breccias. Engineering soils are characteristically divided into two groups - the cohesive and non-cohesive - depending on the nature of any bonding forces and the mode of behaviour of the samples. A convenient way of separating the cohesive and non-cohesive soils was the plastic limit test introduced by Atterberg (1911) for pedological soil analysis. The work of Atterberg has been greatly extended by the geotechnical fraternity who appreciated the way in which these very simple index tests indicated both the proportions of cohesive/non-cohesive materials and the nature of the clay minerals.

For many years there has been confusion between geologists and engineers as to the appropriate boundaries of the particle size classes. Terzaghi and Peck (1948) produced a table which drew attention to the boundary used by the Bureau of Soils between silt and clay content of 5μm (0.005 mm), compared with that of Atterberg (1911) and MIT (1931) which used 2μm (0.002 mm). The British engineering classification follows the MIT divisions but it should be noted that many geologists do not use these boundaries but follow those of Udden and Wentworth (see Tucker, 1991). The various boundaries are shown in Table 1.

<table>
<thead>
<tr>
<th>Table 1: Particle size boundaries</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pedologists</strong></td>
</tr>
<tr>
<td>British Standard 1981 2000 μm 60 μm 2 μm</td>
</tr>
<tr>
<td>ASTM 1992 4750 μm 75 μm Not given</td>
</tr>
</tbody>
</table>

It is of note that ASTM appreciating the difficulty of separating clays and silts and the problem of precisely where to put the clay/silt boundary, have chosen to use the different boundaries given in the Unified Soil Classification. In ASTM (1992), a CLAY is defined as

"soil passing a No. 200 (75-μm) sieve that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dry. For classification, a clay is a fine-grained soil, or the fine-grained portion of a soil, with a plasticity index
equal to or greater than 4, and the plot of plasticity index versus liquid limit falls on or above the "A" line.

and a SILT as

"soil passing a No.200 (75-μm) sieve that is non-plastic or very slightly plastic and that exhibits little or no strength when air dry. For classification, a silt is a fine-grained soil, or the fine-grained portion of a soil, with a plasticity index less than 4, or the plot of plasticity index versus liquid limit falls below the "A" line."

With regard to particle size, it is regrettable that ASTM has chosen to confuse further an already confusing subject. The system has merits, however, in that by defining clay and silt as it does, it is closer to separating potential claystones and siltstones as engineering materials. In the former, there are insufficient voids and permeability for much cementation to develop hence it remains a weak rock which will absorb water and expand. In the case of a siltstone, however, because of its greater inherent pore size and permeability, it will frequently be well cemented and hence strong and will not have sufficient clay minerals for any measurable change in water content or strength on weathering. A similar argument is used by Hawkins and Pinches (1992) in their proposed description of mudrocks.

It should be noted that the size of the particles as measured will depend partly on their shape. When sieving is used to separate the particles coarser than approximately 60μm, a fibrous shaped particle may pass through the mesh while if a particle of the same volume was spherical in shape, it may be retained on a larger mesh. Similarly, when using pipette or hydrometer methods, it is assumed that the particles will fall in accordance with Stokes' Law which assumes the particles to be round and of a constant density so that they will behave in a similar manner. Even with careful testing, including the elutriation particle counter method (see Singer et al., 1988) it is only with the more spherical grains that it is possible to obtain an accurate indication of the particle size.

With cohesive soils it is necessary to break down the bonding in order to determine the grain size of the material. If grain size distribution charts are to be compared, it is of paramount importance that the nature of the dispersant used, the length of time over which the disintegration takes place and whether or not the sample is agitated during this period are standardised and recorded. It is well known that some materials form clay aggregates which unless carefully dispersed, will be indicated as silt grade material. However, it is not always appreciated that an increase in the dispersant used or a more vicious chemical can result in much higher clay fraction percentages being obtained.

As Atterberg only undertook tests on fractions passing the 425 μm sieve, it is essential to record the percentage of original material on which the Atterberg limits are obtained. Tested samples may represent only a small proportion of the general material mass and hence care must be taken when correlating the other parameters with only a limited proportion of the material.

This paper describes the results of a number of experiments undertaken using various mixtures of sand and material consisting almost exclusively of the clay minerals kaolinite, illite and calcium montmorillonite. This forms part of a large study in which various percentages of these minerals have been assessed for both volumetric and linear shrinkage; the latter work will be published in a separate paper.

THE MATERIALS

The sand fraction used was obtained commercially as a sand of less than 425 μm in size with particle size analysis indicating 70% between 60 and 150 μm and less than 10% finer than 2μm. This was examined in the laboratory and the sand confirmed to be a pure quartz sand.

In order to obtain as pure a kaolinite as possible, the material was obtained from the ECLP Research Laboratories at St Austell. X-ray analysis confirmed it consisted almost exclusively of kaolinite.

It is not possible to obtain pure illite. However Srodon et al (1986) recorded a quarry in which very highly illitic materials occurred in bands in a Silurian rock sequence. Samples were collected from this quarry and again the X-ray diffractogram indicated almost pure illite although some smectite/illite mixed layer clays were present.

The calcium montmorillonite sample was obtained from a pit in the Lower Greensand near Reigate. The X-ray diffraction indicated the material used was as pure as could reasonably be expected.

TEST PROCEDURE

The samples were dried at 105°C, the standard temperature used in geotechnical soil testing.

Various mixes of the dried material were prepared, using weight percentages. It is clear that such a process will slightly under-estimate the volume of sand present because of its higher density. However it is a practical method and relevant to that of particle size analysis, while the volume percentage would be difficult to prepare and hence liable to inconsistencies.

After the various mixes were prepared, they were stored in airtight polythene bottles until required for sample preparation. Sample preparation was carried out following precisely the recommendations in
British Standard BS 1377 (1991). This requires the sample to be placed in a mixing dish where distilled or de-ionised water is progressively added in order to obtain a consistency similar to that which will yield the liquid or plastic limit. When this consistency has been established, the sample is left to stand for at least 24 hours during which the physicochemical properties of the sample can be re-established such that an Atterberg test can be carried out.

### Table 2: Atterberg limits of the various materials used in the mixes

<table>
<thead>
<tr>
<th>Material</th>
<th>$W_l$ (%)</th>
<th>$W_p$ (%)</th>
<th>$I_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>57</td>
<td>28</td>
<td>29</td>
</tr>
<tr>
<td>Illite</td>
<td>72</td>
<td>32</td>
<td>40</td>
</tr>
<tr>
<td>Ca Montmorillonite</td>
<td>102</td>
<td>45</td>
<td>57</td>
</tr>
</tbody>
</table>

The same operator undertook all the tests as it has been shown by Sherwood (1970) that various operators can produce a 3% difference in the plastic limit. In the experiments reported here, the fall cone method described in British Standard BS 1377 was used in preference to the Casagrande cup as the authors are of the opinion that this test provides a greater repeatability than that originally introduced by Casagrande. The plastic limit tests were undertaken in accordance with British Standard.

### RESULTS

Table 2 gives the Atterberg limits (index properties) for the individual components used in this analysis; the results of the liquid limits and plasticity indexes of the different mixes are recorded in Tables 3 and 4 and in graphical form as Figure 1.

In Figure 1 where only 20% of the sample consists of clay minerals, it can be seen that when kaolinite is

### Table 3: Liquid limit of mixtures of sand and varying proportions of two clay minerals

<table>
<thead>
<tr>
<th>Proportion of each mineral (%)</th>
<th>Kaolinite / Illite</th>
<th>Kaolinite / Ca Montmorillonite</th>
<th>Illite / Ca Montmorillonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>20/0</td>
<td>24</td>
<td>24</td>
<td>31</td>
</tr>
<tr>
<td>10/10</td>
<td>28</td>
<td>30</td>
<td>34</td>
</tr>
<tr>
<td>0/20</td>
<td>30</td>
<td>39</td>
<td>39</td>
</tr>
<tr>
<td>40/0</td>
<td>28</td>
<td>28</td>
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</tr>
<tr>
<td>30/10</td>
<td>29</td>
<td>33</td>
<td>41</td>
</tr>
<tr>
<td>20/20</td>
<td>32</td>
<td>40</td>
<td>48</td>
</tr>
<tr>
<td>10/30</td>
<td>33</td>
<td>47</td>
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</tr>
<tr>
<td>0/40</td>
<td>38</td>
<td>53</td>
<td>53</td>
</tr>
<tr>
<td>60/0</td>
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</tr>
<tr>
<td>50/10</td>
<td>38</td>
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</tr>
<tr>
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</tr>
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<td>42</td>
<td>53</td>
<td>62</td>
</tr>
<tr>
<td>10/50</td>
<td>44</td>
<td>60</td>
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</tr>
<tr>
<td>0/60</td>
<td>46</td>
<td>68</td>
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</tbody>
</table>

### Table 4: Plasticity index of mixtures of sand and varying proportions of two clay minerals

<table>
<thead>
<tr>
<th>Proportion of each mineral (%)</th>
<th>Kaolinite / Illite</th>
<th>Kaolinite / Ca Montmorillonite</th>
<th>Illite / Ca Montmorillonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>20/0</td>
<td>8</td>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>10/10</td>
<td>9</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>0/20</td>
<td>10</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>40/0</td>
<td>11</td>
<td>11</td>
<td>16</td>
</tr>
<tr>
<td>30/10</td>
<td>12</td>
<td>13</td>
<td>19</td>
</tr>
<tr>
<td>20/20</td>
<td>14</td>
<td>19</td>
<td>26</td>
</tr>
<tr>
<td>10/30</td>
<td>15</td>
<td>26</td>
<td>31</td>
</tr>
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<td>32</td>
</tr>
<tr>
<td>60/0</td>
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</tr>
<tr>
<td>50/10</td>
<td>19</td>
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<tr>
<td>40/20</td>
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<td>22</td>
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</tr>
<tr>
<td>20/40</td>
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</tr>
<tr>
<td>0/60</td>
<td>22</td>
<td>37</td>
<td>37</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Proportion of each mineral (%)</th>
<th>Kaolinite / Illite</th>
<th>Kaolinite / Ca Montmorillonite</th>
<th>Illite / Ca Montmorillonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>80/0</td>
<td>21</td>
<td>21</td>
<td>31</td>
</tr>
<tr>
<td>70/10</td>
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<td>32</td>
</tr>
<tr>
<td>50/30</td>
<td>23</td>
<td>28</td>
<td>34</td>
</tr>
<tr>
<td>40/40</td>
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<td>32</td>
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</tr>
<tr>
<td>30/50</td>
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<td>35</td>
<td>39</td>
</tr>
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<td>20/60</td>
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<td>36</td>
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</tr>
<tr>
<td>10/70</td>
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<td>46</td>
</tr>
<tr>
<td>0/80</td>
<td>31</td>
<td>47</td>
<td>47</td>
</tr>
</tbody>
</table>

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Figure 1. Liquid limit and plasticity index for clay mixes with given sand content (triangle = illite/Ca montmorillonite; circle = kaolinite/Ca montmorillonite; square = kaolinite/illite)
present, the liquid limit is relatively low, varying between 24 and 30% for sand with mixtures of kaolinite/ilite. Thus, with the increasing proportion of illite, the liquid limit of this mixture rose by 7% while with the addition of calcium montmorillonite to the kaolinite/sand mix, the liquid limit rose to 39%, clearly indicating the greater effect of the calcium expanding lattice clay mineral. Interestingly, the illite/calcium montmorillonite/sand mixture came to the same liquid limit, again emphasising the overriding effect of the calcium montmorillonite with the illite mix as well as with the kaolinite mix.

Figures showing 40, 60 and 80% clay fraction show similar trends with the liquid limit rising as the proportion of clay minerals to sand fraction increases. It is noted, for instance, that the liquid limit with mixtures of sand and kaolinite rise from 24% through 28, 37 and 47% for the 20 to 80% variation in proportion of kaolinite. In the case of the sand/illite mix, the rise in liquid limit is from 30 to 58%, indicating the higher liquid limit of the illite clay mineral compared with kaolinite. The rise, however, is significantly more pronounced when calcium montmorillonite is present in the mix. In this case, both the kaolinite/calcium montmorillonite and illite/calcium montmorillonite have the same liquid limit, emphasising the dominant effect of the calcium montmorillonite. It is noted that when this mineral is present, the liquid limits rise from 39% when only 20% clay mineral is present to 84% when 80% of the mix consists of calcium montmorillonite. When the plasticity index is plotted (Figure 1), it can be seen that a 20% kaolinite/sand mix would have a plasticity index of 8%, an illite/sand mixtures of 8% and a calcium montmorillonite/sand mixture of 10%. With an increasing percentage of clay minerals in the sand mixes, the plasticity index for the kaolinite-rich soils rises from 8 to 21%, for the illite-rich mixtures from 8 to 21% and for the calcium montmorillonite-rich from 10 to 31%.

From the data presented in Figure 1, it is interesting to note that by the ASTM classification, a material with 80% sand and 20% kaolinite ($w_s = 24$, $w_p = 16$ and $I_p = 8$), is recorded as a clay as it has an $I_p$ of $> 4\%$ and plots above the Casagrande A line.

Whilst to improve our understanding of real soil behaviour it is necessary to examine the components individually and as mixes, it must be borne in mind that the true soils are more complex and that the purpose of the tests is to gain an appreciation of that soil's characteristics. It is noted, for instance, that Hawkins et al (1991) have drawn attention to the variability of the index properties when the tests are carried out by the same operator (a) as per British Standard, (b) using the full soil mix and (c) when using a soil mixture not previously desiccated as required in British Standard. It can be seen that the Atterberg limits vary by as much as 30% between the various methods.

CONCLUSIONS

Too frequently young geotechnical engineers describe materials as if their classification is simple and distinct and then undertake mathematical calculations using precise figures. This paper has highlighted:

a) the confusion of terminology used by engineers within individual countries and worldwide and between engineers and geologists.

b) the necessity to undertake the tests on samples prepared in a consistent manner if correlations are to be made between engineering soils.

c) the variation in Atterberg limits obtained on samples tested under different conditions.

The variation in the Atterberg limits depends entirely on the proportion of clay minerals and the nature of the clay minerals present. The mixtures between sand and varying proportions of clay minerals has emphasised the controlling effect that the montmorillonite/smectite minerals have on the Atterberg limits. It is also clear that, even for the same percentage of clay minerals, the proportion of sand makes a considerable difference such that for the sand/calcium montmorillonite samples, the liquid limit changes from 39 to 84% with a proportion of 80 to 20% sand.

The paper draws attention to the care with which index tests on actual soils should be treated. If consistency is to be obtained such that correlations can be made worldwide, it is essential that a worldwide standard is produced and accepted. However, such a standard may not necessarily represent the true characteristics of the individual soils which do not necessarily respond in the manner they would in the field when they have experienced the drying procedure recommended and/or the removal of materials greater than 425 μm.

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Atterberg, A. 1911. On the investigation of the physical properties of soils and on the plasticity of clays (in German) Int. Mitt. fur Bodenkunde 1, 10-43.


Tests and limits for fines in sands
Essais et limites pour les fines des sables

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Green Land Reclamation Ltd, Maidenhead, UK

ABSTRACT: When considering specification limits for the fines content of sands to be used in concrete or in mortar, the nature of the fines is more important than their quantity. Methylene blue adsorption appears to be the best available method for assessing the nature of fines in sands and the optimum conditions for a standard test are discussed. Specification limits for both quantity and quality of the fines in sands, which will recognise existing National practices, are also discussed. Setting such limits requires the consideration of variations arising from geology, production and testing, as well as the performance of the end-product.

RESUMÉ: En prenant en consideration les limites de spécification du contenu des fines du sable à utiliser pour le béton ou pour le mortier, la nature des fines importe plus que sa quantité. L’adsorption du bleu de méthylène semble être le meilleur moyen d’évaluation de la nature des fines dans le sable et les meilleures conditions de l’épreuve normalisée sont discutées. Les limites de spécification de la quantité, ainsi que de la qualité des fines dans le sable reconnaissant des pratiques nationales, sont également discutées. L’imposition des limites pareilles nécessite la prise en consideration des variants provenant de la géologie, la fabrication et des essais, ainsi que du rendement du produit final.

1. INTRODUCTION

The properties of concrete and mortar are very much dependent on the quantity of water present in unit volume of the mix.

Water is necessary to hydrate the cement and to provide the mix with "workability". The latter term is difficult to define because it combines the effects of fluidity and cohesiveness, among others, and local traditions lead to different perceptions of "good" workability.

The quantity of water necessary in a unit volume of concrete or mortar mix depends almost totally on the aggregate in the mix. Because strength and durability of the hardened mix are related to the ratio of mass of free water to mass of cement, the more water required by the aggregate, the more cement must be added and this can increase the temperature rise and the drying shrinkage of the concrete.

The properties of aggregate which influence water demand include those which affect packing of particles (e.g. particle shape and size), amount and rate of water absorption and surface area.

Obviously, the fines fraction of an aggregate can dominate surface area; this paper considers the relevant issues.

2. THE IMPORTANCE OF FINES

2.1 Definition of fines

Fines are generally defined as "that fraction of the aggregate which passes a (stated) sieve". The UK has used 75 μm and other countries have used 63 μm or 80 μm. The organisation (CEN) which prepares European Standards has a Technical Committee for aggregates (TC154) which has agreed, in principle,
that 63 μm will be used to define fines in its standards.

2.2 A simple model for surface area

Consider the following concrete which is a typical mix supplied to meet a specification for concrete required to have a compressive strength of 30 to 40 MPa.

Table 1. Model concrete

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Mass (kg)</th>
<th>Solid volume (m³)</th>
<th>Surface area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>300</td>
<td>0.095</td>
<td>-</td>
</tr>
<tr>
<td>Aggregate</td>
<td>1280</td>
<td>0.483</td>
<td>360</td>
</tr>
<tr>
<td>20-44 mm</td>
<td>640</td>
<td>0.242</td>
<td>3350</td>
</tr>
<tr>
<td>Sand</td>
<td>180</td>
<td>0.180</td>
<td>-</td>
</tr>
</tbody>
</table>

The surface area of cement has been omitted from Table 1 although it is very large, because cement is reactive and does not play the same role as aggregate in the water/workability relationship.

The surface area of the sand has been calculated on the assumption that it contains 3 per cent passing 63 μm, but the contribution of these fines to the area is not included. Three per cent fines has been chosen because it was, for many years, the limit for sands in the British Standard for concrete aggregates, BS 882.

Now consider the additional surface arising from the 3 per cent fines for different mean sizes of the fines particles (see Table 2).

Table 2. Additional surface area of the 3% fines, to be added to the 3710 m² of Table 1.

<table>
<thead>
<tr>
<th>Diameter (μm)</th>
<th>Surface area (m²)</th>
<th>Surface area % of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>138</td>
<td>3.5</td>
</tr>
<tr>
<td>25</td>
<td>172</td>
<td>4.4</td>
</tr>
<tr>
<td>10</td>
<td>278</td>
<td>7.0</td>
</tr>
<tr>
<td>5</td>
<td>451</td>
<td>10.8</td>
</tr>
<tr>
<td>1</td>
<td>1843</td>
<td>33.2</td>
</tr>
</tbody>
</table>

2.3 The nature of fines

Table 2 shows that the coarser fines, having particle diameters of more than 10 μm, contribute less than 7 per cent of the total surface area of the aggregate (for this model concrete). These fines might increase water demand a little but this will be offset by the improved workability and cohesiveness of the mix and the particles may act as nucleating sites in promoting cement hydration. Siliceous and calcareous particles are chemically compatible with cement hydrate.

In contrast, very fine particles will contribute a significant proportion of the area of the aggregate and, in particular, clay minerals with diameters of 1 μm or less can have a dramatic effect on water demand.

It is necessary, therefore, in considering fines in aggregates for concrete or mortar to take account not only of their amount but also their composition.

The fines which have an especially pronounced effect are the smectites or swelling clays which not only have a large surface area arising from the small size of the particles but also have a very large internal surface. As a result, smectites increase water demand greatly. The effects discussed above are illustrated in Table 3 (after Unikowski, 1982), which gives the quantity of water, in grams, required by a standard mortar (EN 196-1) to give a defined flowability, when varying amounts of different fine powders are added. Table 3 gives the results for the addition of 4 per cent powder by mass of the sand. The properties of the powders used are shown in Table 4.

Table 3. Water content of standard mortar for additions of 4% fines by mass of sand.

<table>
<thead>
<tr>
<th>Powder</th>
<th>Water (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>240</td>
</tr>
<tr>
<td>Siliceous</td>
<td>243</td>
</tr>
<tr>
<td>Calcareous</td>
<td>230</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>285</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>340</td>
</tr>
</tbody>
</table>

Table 4. Powders used

<table>
<thead>
<tr>
<th>Powder</th>
<th>Median Diam (μm)</th>
<th>Area (BET) (m²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous</td>
<td>13</td>
<td>3.3</td>
</tr>
<tr>
<td>Calcareous</td>
<td>22</td>
<td>1.7</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>7</td>
<td>11.8</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>&lt;1</td>
<td>42.6</td>
</tr>
</tbody>
</table>

Note: Median diameter taken from Sedigraph results.
3. TEST METHODS

3.1 Measuring fines content

For many years, the British Standard for testing aggregates, BS 812, included a decantation method for measuring what was called "clay, silt and dust". This was done separately from the sieving method, which included the requirement that "...when the aggregate contains material which causes agglomeration of particles..." the sample should be washed by decantation, dried and sieved. Because the decision to include the pre-washing step was often left to the testers' discretion, many chose not to follow that time-consuming procedure and, as a result, there was a great deal of confusion about gradings, particularly at the fine end. For example, Pike and Limbrick (1981) show a sand with 11 per cent passing 150 μm when dry-sieved but 25 per cent when washed before sieving.

The British Standard was revised in 1985, as BS 812: Part 103, to make washing-and-dry-sieving the standard method, with simple dry-sieving being permitted only when it can be demonstrated that it gives the same result as the standard method. The precision of the standard method was shown to be satisfactory. For example, 11 laboratories tested 11 different sands and the calculated precision values are shown in Table 5.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Fines as % of dry sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>5.5</td>
</tr>
<tr>
<td>Repeatability, r</td>
<td>0.8</td>
</tr>
<tr>
<td>Reproducibility, R</td>
<td>1.5</td>
</tr>
</tbody>
</table>

This method now includes the fines as part of the whole grading, as it should, and the separate decantation method is no longer used.

Wet-sieving was also examined but it gave anomalous results and caused increased wear and tear on the test sieves. The finest woven-wire sieve specified in standards has apertures of 38 μm and this was also examined. However it was found to be very fragile and gave unreliable results. When the 38μm sieves were collected from the participants at the end of the survey carried out by Pike and Limbrick and sent to the manufacturer for calibration, seven of the eleven sieves did not comply with the appropriate standard (BS 410).

BS 812 also contains a sedimentation method which gives results for 20 μm, 10 μm and 6 μm Stokes' diameter particles. This is of limited value because the characteristics of fine particles are not dependent only on size and the method is not widely used.

The Sand Equivalent test (SE) is used in some countries as a field test for assessing the quantity of fines. In this test the sand test portion is shaken with a flocculating solution (calcium chloride) in a cylinder and then allowed to stand. The relative heights, and thus volumes, of rapidly sedimented sand and of the flocculated fines are observed. A variation of this test was removed from BS 812 in 1985 because of poor results.

The CEN Technical Committee dealing with aggregates, TC154, has accepted BS 812: Part 103 as the basis for their standard method for sieve grading. The SE test has also been proposed but its status is not yet clear.

3.2 Assessing the nature of fines

Many industries use methylene blue (MB) adsorption as a measure of clay content and the test is specified in some National Standards for aggregates. France and The Netherlands, in particular, use the test to define limits for aggregates although the details of the methods are quite different in each country. It is believed that some concrete producers in The United Kingdom use dye adsorption methods for internal quality control purposes but details are not available.

The method has been extensively studied in the UK (Pike,1992) and several rounds of inter-laboratory trials have been carried out. This work shows that five points must be satisfied to give the most meaningful results from the MB test on sands used in concretes and mortars.
1. The test must be carried out on dried test portions of the whole sand (that is, not on fractions extracted by washing or sieving).
2. The sand and the dye solution must be properly mixed; use of a paddle stirrer is not sufficient if the sand contains particles larger than about 2 mm.
3. Although methylene blue is somewhat unstable, no satisfactory alternative has yet been found. The dye solution must be standardised in use.
4. Results must be expressed in terms of grams anhydrous dye per kg of dried sand. Other expressions such as grams dye [unspecified hydration] per mass of fines must not be accepted.

5. Tests should be carried out on reference materials. Kaolinite can be used, but the provision of calibrated reference sands would be better.

A precision experiment, with 16 participants who all used a method following the above precepts, gave the results shown in Table 6.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>g dye (anhyd) /kg sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>5.50</td>
</tr>
<tr>
<td>Repeatability, r</td>
<td>0.08</td>
</tr>
<tr>
<td>Reproducibility, R</td>
<td>0.30</td>
</tr>
</tbody>
</table>

This precision data is very much better than that obtained from a previous experiment, using a method not following the precepts for an ideal method shown above.

A draft MB test method is being developed within CEN/TC 154 but it does not follow the rules that are essential for obtaining the most meaningful results.

Suggestions have been made that the SE test gives an assessment of fines quality. This cannot be so and it is only suitable as a semi-quantitative measure of fines quantity.

4. LIMITS AND STANDARDS

There are a number of strategies which may be considered and these are discussed in turn.

1. Impose very low limits for quantity of fines so that, even if they are all harmful clays, there will not be sufficient to cause problems for the user. This would not be acceptable to the UK where the local tradition for concrete and mortar users prefers some fines to increase the cohesion of the mix and to give better working properties. Un-washed, dry-screened sands are widely used for masonry and rendering mortars and imposition of low limits would not only cause technical problems but would also lead to unnecessary expense.

In The Netherlands, where low limits are the norm, it is the practice for users to add fines, typically pulverised fuel ash or lime, to some of their mortar mixes to improve workability.

2. Allow higher limits for fines content but require that, when the quantity is above a defined "trigger" limit, a test to measure clay contents, such as methylene blue adsorption, be done. Some producers of crushed rock sands, claim their products show high MB values but do not contain clay. It is possible that the dye is adsorbed by clay minerals within aggregate particles as well as clay around or between particles. Pike (1992) gave a methodology for separately measuring clays in these forms.

This strategy is that being proposed in the drafts for aggregates from TC 154 although the actual values of the limits are not yet agreed.

3. Have no limits at all for the sand but require a performance specification for the concrete or mortar made from it. This may be argued to give the most efficient use of natural resources because it is the final product, the concrete or mortar, that matters. However, the purchaser of the mix is as much concerned about long-term durability and stability as about short-term properties, such as setting times and compressive strength, which are easily measured.

Reliable tests for long-term effects are not yet sufficiently developed to give complete confidence in their assessments. The draft CEN standards for aggregates for concrete and mortar have gone some way along this route by incorporating a clause that reads "...where there is experience that no problems have been encountered and there is evidence of satisfactory use..." the sand can be used without further testing. However there is no definition of "experience" (whose and how long?), "problem" (what sort of problem?) and "satisfactory" (satisfactory to whom?) and it is possible that this kind of clause will provide income only for lawyers.

The choice of strategy and the values for limits have to recognise the properties of sands that are being produced and, apparently, successfully used and to equate these with the desired properties of the concretes and mortars made with the sands.

Catalogues of the properties of the wide variety of sands produced in Britain are being compiled, but these usually consider only results obtained on single samples. It is essential that the variability of a product, arising from the geology of the source and the
method used to produce and test it, be known. A novel methodology for assessing the variability of aggregate from a source has been described by Pike (1992) and work using this method is in progress.

The work being carried out by the authors to study the variability of the properties of commercial sands needs to be extended to other European countries as a matter of some urgency.

Correlations between fines content, expressed as quantity and quality, and compressive strength have been made by Unikowski (1982). This, and other, work confirm that some fines, those with low MB values, have little effect on strength while those with high MB values markedly affect strength unless the water content is kept constant. However, if this leads to low workability, the ease of compaction is reduced with adverse effects on the strength of cast specimens. The reported work has usually been carried out using artificially produced sands with a range of properties. The effects of correcting the increased water demand by adding cement or water-reducing admixtures have not been reported. Drying shrinkage may be increased and, with brick-laying or rendering mortars, this could have adverse effects on, for example, rain penetration of masonry.

Work to study the practical effects of fines in sands on concretes and mortars is urgently required.

Studies on the durability of brick-laying mortars, made with a range of sands, have been made by Harrison (1986) but no attempt was made to characterise the fines in the sands he used. The results do not show any correlation with fines quantity.

A program of work to study the correlation of mortar durability with the fully characterised fines content of the sand in the mortar has been started by the authors.

5. CONCLUSIONS

The quality and quantity of the fines in sands, that is the fraction of the sand passing a 63 μm sieve, is of the greatest importance for the properties of the concretes or mortars made from them. It will not be possible to set limits, on a truly scientific basis, for the fines in sands until more information about the quantitative aspects of their influence on the properties of cement based composites is available. Characterisation of sands in commercial use should be carried out as a matter of some urgency. Of particular importance is the variability of sand from a source.

More work on studying the influence of the fines in sands on the properties, particularly long-term durability and drying shrinkage, of concrete and mortar should be carried out.

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Pore pressure propagation during consolidation of fine grained soils
Propagation des pressions interstitielles pendant la consolidation des sols fins

F.A. K. Hassona
Civil Engineering Department, Minia University, Egypt

ABSTRACT: Consolidation of fine grained soils is a major behaviour when exposed to external loads. During the process of consolidation, pore pressure build up is expected directly after loading as the permeability of such soils is very low. The propagation of pore water pressure during consolidation through soil layer is the aim of this investigation. Measurements of pore water pressure are made in a 250 mm diameter hydraulic consolidation cell at three different locations (25, 70, 112.5mm from sample center). Two different loading conditions are employed. These types are free straining and equal straining. Four types of drainage condition are considered, top, bottom, middle (inward), and side (outward) drainages. These tests are conducted on reconstitute samples of clayey silt prepared at water content equal to twice its liquid limit. Comparisons and discussions of the test results are made in relation to soil characteristics.

RÉSUMÉ: La consolidation des sols fins est essentiel quand ils sont exposés à des forces extérieures. Pendant la consolidation, l’augmentation de la préssion interstitielle se manifeste tout de suite après la compression, vue que la perméabilité du sol est très petite. La propagation des pressions interstitielles pendant la consolidation a travers la couche de sol, est le but de cette recherche. Des mesures de pressions interstitielles sont faites à l’aide d’une cellule de consolidation hydraulique à 250 mm de diamètre. Deux conditions de chargement différents ont été employées. Drainage a été considéré à quatre niveaux.

1. INTRODUCTION:

The use of hydraulic consolidation cells similar to that developed by Rowe and published by Rowe and Barde (1966) considerably increases the scope of laboratory testing of soils compared with conventional oedometer test. In particular it allows for the testing of larger samples under varying conditions of drainage. The advantages of the hydraulic consolidation cells are described in details by Head (1985). Early attempts to measure pore water pressures in consolidation tests were confined to research experiments in the U.S.A. Following the introduction of the pore pressure measuring equipment developed by Bishop and Henkel in 1962, consolidation with pore pressure measurement in triaxial cell on samples up to 100 mm diameter became an established procedure all over the world. Pore pressure measurements were made by Leonards and Girault (1961) in a conventional type of oedometer cell 112 mm diameter, using a manually controlled mercury/water pore pressure device. A major step forward was the application of an electrical pressure transducer to the measurement of pore water pressure by Whitman, Richardson and Healy (1961). They described the advantages of this device and investigated the compliance and response time of the system. Pore pressure transducers and associated monitoring systems were subsequently improved and adapted to soil testing requirements enabling Rowe and Barden to fit them to their cells from the outside. Measurements of pore water pressure in undrained tests is usually made through a drainage path at the center of the soil sample. It was felt, at this point, that a need for monitoring pore water pressure at different locations of cell base under different drainage conditions and different types of loading is necessary specially for larger samples. This is the aim of the present investigation.
2 EQUIPMENTS AND MATERIAL

A standard 250 mm diameter hydraulic consolidation cell was employed in the present study. This cell accommodates a samples of 90 mm in height. Two independently controlled pressure systems, giving maximum pressure up to 1000 kPa, one for loading the diaphragm and the other for providing the back pressure. A motorized air/water system is used to provide the pressure required during testing which is controlled by a pressure regulators and pressure gauges reading to 10 kPa. Measurements of pore water pressure were made through the base of the cell, at three different locations using an electric pressure transducers with a maximum capacity of 1000 kPa. The photo shown in Fig. 1 shows the general arrangements of testing equipments.

The material used to form the samples was silty clay soil. To achieve a repeatable soil samples, a soil blocks were dried, pulverized and thoroughly mixed to produce a homogenous soil batch. Several tests were carried out to classify this soil by determining its properties. These tests were carried out according to the B.S. specifications. The constituents of this soil were 57% clay, 38% silt and 5% fine sand. Its physical properties were 74.6% liquid limit, 31.7% plastic limit and specific gravity of 2.73.

![Fig. 1. The general arrangements of testing equipments](image1)

3 TESTING PROGRAM AND SAMPLE PREPARATION

Three different parameters were thought in to be investigated. One of them the effect of pore pressure measurement location on the value of pore water pressure readings in undrained tests. The second is the effect of the direction of drainage path on these measurements. The third is to study the effect of type of loading condition. To study the first parameter, measurements of pore water pressure were made at three different locations at the base of the cell. These were as shown in Figure 2. Three different drainage paths were adopted which were bottom, top and the outer perimeter surface of the sample.

The effect of loading condition was investigated using two methods of loading, flexible loading (free straining) through the rubber diaphragm included inside the cell and rigid loading (equal straining) using a rigid plate between the rubber diaphragm and the surface of the soil sample. Schematic diagrams of different tests carried out under different conditions are shown in Fig. 3.

Fully saturated samples were achieved by mixing soil to a slurry with distilled water. A motorized rotary mixer was used to mix dry soil with an amount of water equal to twice the liquid limit. That gives a viscosity low enough to allow removal of air by the application of vacuum. The slurry was poured into the cell to a certain precalculated height. An initial vertical stress of 55 kPa was firstly applied on top of the sample. This stress was left for 24 hours while the drainage paths were open. At the end of such time the drainage paths were closed and the test begins.

![Fig. 2. Pore pressure transducer locations](image2)
4. TEST RESULTS AND DISCUSSION

The results of the tests carried out in this investigation can be explained clearly with the help of the schematic diagrams shown in Fig.3. This figure shows the types of loading, the position of different transducers and the direction of drainage paths.

Each test was carried out under the application of three different stress values. These values were 75, 150 and 300 kPa. The reason for using different values of stress is to check the repeatability of the test results. Fig. 3.a. shows the conditions of conducting the first test, the drainage path is downward and the type of loading is flexible loading. Typical results are shown in Figs 4, 5 and 6. A repeatable conclusion can be drawn from these figures. At a certain time, the generation of pore water pressure is higher at the transducer located at position (2) while the transducer located at position (3) being the least. One can explain this result by the fact that the surface of the sample, being loaded with flexible diaphragm, took the bowl shape. That means more deformations at the middle of the sample than at the edges. This leads to a more pore water pressure generation from the middle towards the edges. Equalization of pore water pressure at different points takes an appreciable time as the permeability of such soil is very low.

Fig. 4. Variations of pore water pressure with transducer location (test 3.a)

While this explanation seems logic, the results of the second test with the conditions shown in Fig. 3.b. (the drainage path is downward and the type of loading is rigid loading), gave similar trend although the sample was loaded uniformly. The results of this are shown in Fig. 7. At this point, one can conclude that higher values of pore water pressure at the middle of the soil sample in raw consolidation cell is expected. This value decreases towards the edges of the specimen. It appears that during loading it is easier for the soil near the cell walls to lose water horizontally towards the cell walls than that at the middle. So that pore water pressure readings near the
cell walls are less than those at the middle.
In the third test, the pours plate was put on top of the sample as shown in Fig. 3.c. That means the drainage path was upward and the sample was loaded using the flexible diaphragm. The results of this test is shown in Fig. 8. Contradictory to the results of the previous test, the transducer located at the middle of the specimen showed lower values of pore water pressure than those towards the edge. These results are expected since the flexible loading tends to deform the plastic pours disc like the bowel shape.

In this case the drainage path is shorter at the middle than at the edges. So it is easy for the water to dissipate upward leading to lower values of pore water pressure.
When a rigid loading plate is used on top of the plastic pours plate as shown in Fig. 3.d, the original trend was observed as shown in Fig. 9. Higher values of pore water pressure at the middle than at the edges were observed.
In the last test, flexible loading with the plastic pours
plate around the circumferential of the soil sample were adopted (Fig. 3 e). This gives the condition of a horizontal drainage path and confirm the results of the previous tests. Confirmation of the results shown in the previous tests is shown in Fig. 10. As the pressure transducer is located near a drainage path, lower values of pore water pressure is expected. The differences in pressure between the "inside" and "outside" transducers are much pronounced because of the presence of the plastic pours plate.

Fig. 9 Variations of pore water pressure with transducer location (test 3.d)

Fig. 10. Variations of pore water pressure with transducer location (test 3.e)

**CONCLUSIONS**

In this work a study has been made of the variations of pore water pressure with transducer location, type of loading and drainage path. The following conclusions can be drawn from the results of these tests:

1. When measuring pore water pressure in Rowe consolidation cell, the location of measuring device at the cell base, type of loading and drainage path should be clearly specified.

2. When flexible loading (free draining) is adopted and the drainage path is downward, pore water pressure values decreases from the center radially towards the outer surface of the sample.

3. Similar trend was observed when rigid loading (equal straining) is employed.

4. When flexible loading is used and the drainage path is upward, quite opposite results were observed. Pore water pressure values decreases from the outer surface of the sample inward toward the center.

5. The original trend was observed when rigid loading is applied and the drainage path is upward.

6. Pore water pressure values near the drainage path are less than those near the center of the sample when the drainage path is radial (pours disc around the circumference of the soil sample)

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Effect of particle properties on the limiting void ratio test results

Effet des propriétés des particules sur les résultats des essais pour la détermination des indices de vides

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ABSTRACT: The state of packing of a soil may be expressed by a density or void ratio parameter. Different packing states may be achieved by varying the compaction procedure or subjecting the soil to different stress histories. Tests are carried out on two different materials with different particle properties. These materials are Leighton Buzzard sand and glass beads. The effect of particle properties such as size and shape on the limiting void ratio values as well as the effect of grading are investigated and reviewed. Statistical analysis of the test results is carried out. It is found that the measurement of maximum and minimum void ratios, upon which much of the interpretation of soil strength has been made, a statistical approach is needed if results of sufficient validity are to be obtained. All other things being equal, the limiting ratios would not be expected to depend on absolute particle size. However it was found that the maximum void ratio of glass beads increased when the particle size decreased; the minimum void ratio of the glass beads was sensibly constant.

RÉSUMÉ : L’état d’entassement d’un sol peut être exprimé par un paramètre de densité au rapport de “vide”. Différents états d’entassement peuvent être réalisés en faisant varier la procédure d’entassement ou en soumettant le sol à des programmes de contraintes. Des essais ont été effectués sur deux matériaux différents. Ces matériaux sont le sable de Leighton Buzzard et des grains de verre. L’effet des propriétés des particules comme la taille ou la forme sur les valeurs du rapport de vides, ainsi que l’effet du calibrage sont étudiés et critiqués. L’analyse statistique des résultats des essais est effectuée. On trouve que les mesures maximum des rapports de vide sur lesquels l’interprétation de la intensité du sol à été faite. Une approche statistique est nécessaire si des résultats suffisamment valables doivent être obtenus. Toute autre chose étant égale par ailleurs, les rapports limites ne devient pas dépendre de la forme absolue des particules. Cependant, on a trouvé que le rapport de vide maximum des grains de verre augmente quand la taille de la particules diminue, le rapport de vide minimum des grains de verre est sensiblement constant.

1 INTRODUCTION

It is generally understood that the void ratio of a natural cohesionless soil depends on the sizes, shapes, and surface conditions of the grains, as well as the manner of deposition. The surface condition of a grain depends on the mineral content and includes the surface texture, density of electrostatic charges and so on. The reason why the maximum and minimum void ratios of cohesionless materials sometimes vary with the nominal grain size may be due to particle shape. It was observed by Trowenhofel (1950) that in natural sands there is a correlation between particle size and particle shape; the larger the size of the particle, the more rounded it tends to have become during transportation by water, although the roundness is also influenced by the distance travelled and particle hardness. A similar observation was made by Dickin (1973). It may therefore be easier to fit the coarser grains together than the smaller ones. However non-geometrical factors may also be involved such as electrostatic charges and the momentum of the grains during particle deposition. The effect of particle shape has been investigated by Holzboc and D’Appolonia (1973). Their results lead to the conclusion that, other things being equal, angular grains tend to possess a higher void ratio than rounded ones and this is more noticeable for the maximum void ratio. Similar observations were made by Dickin (1973) who noted that both maximum and minimum void ratios decrease with increasing sphericity. The effect of grading has been discussed
by Johnston (1973). He analyzed the results of tests on subangular to rounded granular soils having all material coarser than 0.075 mm and specific gravities ranging from 2.65 to 2.89. A trend of decreasing maximum and minimum densities, with increasing uniformity coefficient was observed. In the present work, the effect of absolute particle size on the values of limiting void ratios is investigated.

2 MATERIALS

The materials used in this investigation are Leighton Buzzard sand and glass ballast (the word ballastini is used to denote small, usually clear, glass spheres). Leighton Buzzard sand is a uniform, pale yellow, medium quartz sand with subrounded to subangular grains. Its specific gravity, uniformity coefficient, and D10 are 2.65, 1.41, and 0.34 respectively. Grades 4, 8, 12, and 15 of glass ballastini with a specific gravity of 2.95 were used in this investigation. Specifications of glass ballastini are given in Table 1.

<table>
<thead>
<tr>
<th>Grade No.</th>
<th>Nominal grain size (mm)</th>
<th>Pass sieve size (um)</th>
<th>Retained on sieve size (um)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1.015</td>
<td>1180</td>
<td>850</td>
</tr>
<tr>
<td>7</td>
<td>0.55</td>
<td>600</td>
<td>500</td>
</tr>
<tr>
<td>8</td>
<td>0.46</td>
<td>500</td>
<td>425</td>
</tr>
<tr>
<td>9</td>
<td>0.33</td>
<td>355</td>
<td>300</td>
</tr>
<tr>
<td>11</td>
<td>0.22</td>
<td>250</td>
<td>180</td>
</tr>
<tr>
<td>12</td>
<td>0.15</td>
<td>180</td>
<td>125</td>
</tr>
<tr>
<td>15</td>
<td>0.077</td>
<td>90</td>
<td>65</td>
</tr>
</tbody>
</table>

3 LIMITING VOID RATIO TESTS

Various methods have been proposed for the determination of the limiting packing states of granular soils. In general these methods prescribe some form of deposition for the maximum void ratio, and some form of compaction for the minimum void ratio. Different methods of testing were discussed in details by Hassona (1986). He mentioned that the values of maximum and minimum void ratios at which a material do not necessarily represent the maximum and minimum void ratios at which a material can exist in nature, but instead reflect the laboratory procedures used. In the present investigation, maximum void ratio values are obtained by method described by Burminister (1970).

It involves pouring the material inside a standard compaction mould using a funnel with 5.0 mm outlet diameter and a fall height of 5.0 mm. Knowing the volume and weight of material used, void ratio was calculated. Minimum void ratio was determined by two methods. For Leighton Buzzard sand, the method described by Head (1980) was used. This method involved vibrating sand with a hammer. The sand was vibrated in a standard compaction mould in three layers. At the end of compaction, the volume and weight were measured and void ratio is calculated. For glass ballastini, this method could not be used since crushing of particles is observed. The method described by Yoshimi and Tohno (1973) and Kirkpatrick (1963) was employed. It involved placing the material in three layers in a compaction mould. Each layer was subjected to three minutes of vibration on a vibrating table. Again, knowing the volume and weight of the sample, void ratio is calculated.

4. STATISTICAL ANALYSIS OF TEST RESULTS

Statistical methods were used to assess the variability of the results and also to predict the minimum number of tests required to achieve a given accuracy. In practice, most frequency curves dealing the characteristics of soil tend to exhibit some skew. However it is generally agreed that, provided the coefficient of variation is not greater than 30%, a skewed distribution can be assumed to be normal for the most engineering purposes, Smith and Pole (1980). The coefficient of variation \( C = \frac{S}{X} \) where \( S \) is the standard deviation and \( X \) is the mean. Histograms of the test results were drawn and the coefficient of variation was calculated and found to be much less than 30%. Hence the distributions can be assumed to be normal and the number of times a measurement of maximum and minimum void ratio required to achieve a given accuracy with a certain degree of confidence can be determined. In the present work the level of confidence and precision were chosen as 95% and 1.0% respectively. It was found that the test methods adopted and described above should be carried out at least four times and the mean of the results is recorded.

5 TEST RESULTS AND DISCUSSION

A review of maximum and minimum void ratio test results carried out on Leighton Buzzard sand by others was made and presented in Figure 1 along.
with the results of the tests carried out in the present investigation on the same soil. The experimental variation of the maximum and minimum void ratios of Leighton Buzzard sand with grain size is demonstrated. Although, the results presented in Figure 1 were determined by different methods and different persons, a fact which is likely to increase systematic errors, for both the maximum and minimum void ratios there is some evidence of an increase in the value as the particle size reduces (the results of Kirkpatrick (1963) appears exceptionally uniform).

Table 2. Limiting void ratios for different sizes of glass beads.

<table>
<thead>
<tr>
<th>Grade No.</th>
<th>Diameter (mm)</th>
<th>e max</th>
<th>e min</th>
<th>Max. Density (t/m³)</th>
<th>Min. Density (t/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1.015</td>
<td>0.699</td>
<td>0.401</td>
<td>2.105</td>
<td>1.737</td>
</tr>
<tr>
<td>7</td>
<td>0.55</td>
<td>0.722</td>
<td>0.396</td>
<td>2.133</td>
<td>1.713</td>
</tr>
<tr>
<td>8</td>
<td>0.46</td>
<td>0.720</td>
<td>0.390</td>
<td>2.122</td>
<td>1.715</td>
</tr>
<tr>
<td>9</td>
<td>0.33</td>
<td>0.735</td>
<td>0.410</td>
<td>2.092</td>
<td>1.700</td>
</tr>
<tr>
<td>11</td>
<td>0.22</td>
<td>0.750</td>
<td>0.394</td>
<td>2.116</td>
<td>1.685</td>
</tr>
<tr>
<td>12</td>
<td>0.15</td>
<td>0.774</td>
<td>0.395</td>
<td>2.114</td>
<td>1.662</td>
</tr>
<tr>
<td>15</td>
<td>0.077</td>
<td>0.801</td>
<td>0.391</td>
<td>2.120</td>
<td>1.637</td>
</tr>
</tbody>
</table>

In order to investigate the effect of grain size alone on the limiting void ratios (i.e. to eliminate any other factors such as changes of particle geometry or surface texture) glass beads of seven different sizes were used as shown in Table 1. There was complete geometrical similarity of the grading curves of different sizes. The maximum and minimum void ratios were determined the funnel pouring method and the vibrating table method described briefly above. The results of these tests are summarized in Table 2. and shown in Figure 2 along with the theoretical values.

It is well demonstrated that the maximum void ratio increases as the particle size decreases but that the minimum void ratio remains constant. In Figure 2 data from earlier studies by Kolbuszewski and Frederick (1963) and Ishihara and Watanabe (1976) are consistent with the first of these conclusions. the results of the tests carried out on glass beads support the trend observed in Figure 1 for Leighton Buzzard sand in the case of maximum void ratio but not in that of the minimum void ratio. Therefore it is likely that the change of the minimum void ratio of change in particle shape or texture, such changes being far from obvious.

6 CONCLUSIONS

In the measurement of maximum and minimum void ratios, upon which much of the interpretation of soil behaviour and strength, a statistical approach is needed if results of sufficient validity are to be obtained. All other things being equal, the limiting void ratios would not be expected to depend on absolute particle size. However it was found that the maximum void ratio of glass beads increased when the particle size decreased; the minimum void ratio of glass beads was sensibly constant.

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Dickin, E. A. 1973. The influence of grain shape and size upon the limiting porosities of sand. Evaluation of relative density and its role in geotechnical projects involving cohesionless soils. ASTM STP 523


The mechanical properties of a physical model mélange
Les propriétés mécaniques d’un modèle physique de mélange

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ABSTRACT: Physical model melanges made up of stronger and stiffer blocks in a weaker and softer matrix were tested in triaxial compression to study the effect block proportion and orientation have on mélange strength and stiffness. The results of these tests showed that increasing the block proportion generally decreased the cohesion, increased the angle of internal friction and increased the modulus of deformation. The highest block proportion specimens had a cohesion as low as half of that of the matrix and an angle of internal friction as much as 16.5° higher than that of the matrix. Block orientation also affected the strength, most notably the cohesion. The modulus of deformation was also affected by block orientation. The modulus increased with increasing block proportion and this increase was most pronounced for the models with blocks oriented parallel to the axial loading direction and least pronounced for those with blocks oriented perpendicular to the axial loading direction.

RÉSUMÉ: Pour comprendre l’effet de la proportion des blocs et leurs orientations sur la résistance et la rigidité du mélange, les essais de compression triaxiaux ont été effectués sur les modèles physiques. Ces modèles ont été formés avec des blocs solides et rigides dans une matrice molle et faible. Les résultats des essais ont montré qu’une augmentation dans la proportion des blocs a eu généralement pour effet la diminution de la cohésion, l’augmentation de l’angle de frottement interne et du module de déformation. Les échantillons comprenant la plus grande proportion de blocs étaient caractérisés par une cohésion diminuée de moitié et un angle de frottement interne augmenté de 16.5° par rapport à la matrice. L’orientation des blocs influence la résistance, le plus notamment la cohésion. Le module de déformation est également influencé par l’orientation des blocs. La croissance de ce dernier est plus prononcée avec des blocs orientés parallèlement à la direction de la charge axiale qu’avec des blocs perpendiculaires à cette même charge.

1 INTRODUCTION

Mélange is the French word for “mixture” and is defined in the Dictionary of Geological Terms (Bates and Jackson, 1984) as, “a mappable body of rock that includes blocks of all sizes, both exotic and native, embedded in a fragmented and generally sheared matrix.” In essence, therefore, a mélange is a rock body made up of stronger blocks in a weaker matrix. Medley (1994) has coined the “bimrock” to describe such block-in-matrix rocks. Other examples of bimrocks are conglomerate, breccia and sheared serpentinite.

Due to their complex nature, melanges are difficult geotechnical materials with which to work. For example, D’Elia, et al. (1984) found that it was not possible to determine the mass mechanical properties of a volume of the "Argille Scaglissi" (an Italian mélange unit) by testing samples in the laboratory or in-situ. The mass was too heterogeneous to be captured in a relatively small sample. This fact has been realized by many engineers and geologists working with melanges all over the world. The reasons for this difficulty include:

1. Obtaining "undisturbed" samples of a mixture of harder blocks in a weaker matrix by
coring is quite difficult. The drilling resistance of the harder and softer materials are sufficiently different that the harder materials may gouge into the weaker materials resulting in significant sample disturbance, or even a complete loss of the weaker material.

2. Even if one were able to recover an undisturbed sample, it is improbable that the sample would be representative of the melange mass of interest. Large in-situ test samples are perhaps the most promising approach to finding and testing a representative sample, but at his time the possibility for successful demonstration of this approach seems remote.

For these reasons, a different approach for determining the mechanical properties of a melange needed to be developed.

It has been found that it is possible to recover and test samples of pure matrix and pure block. Realistically the mechanical properties of the block and matrix components at any site can therefore be determined. Common engineering practice is to design for the strength and deformation properties of the weak matrix. This approach neglects the possibility that the blocks strengthen and stiffen a melange mass, therefore it may be an overconservative practice. A less conservative alternative was proposed by Volpe et al. (1991), who suggested that the strength of a melange mass could be represented by the weighted average of the strengths of the weaker matrix and stronger blocks based on their volumetric proportions. Unfortunately, no theoretical basis was provided for this strength model.

Physical model melanges were fabricated and tested in triaxial compression in an effort to either substantiate the strength model proposed by Volpe et al. (1991) or determine a different relationship between the volumetric proportion of blocks and the mechanical properties of a melange.

2 PHYSICAL MODEL MELANGES
Melanges tend to have a fabric of weaknesses resulting from both block alignment and prevalent shearing around the blocks. This is an important structural characteristic because structurally anisotropic rocks are known to have anisotropic strength characteristics (Jaeger, 1960 and Donath, 1964). Oriented blocks and mock shearing were therefore incorporated in the models in an effort to mimic this fabric.

The model block shapes and block size distributions were based on observations of melanges studied along California's north coast and a variety of photographs and descriptions of melanges from around the world. The block shapes ranged from lenticular to tabular. The ratio of maximum axial dimension to minimum axial dimension for all block sizes was 2 to 3. Because blocks in a melange can be quite large relative to the volume of interest, the maximum block size in the models was allowed to be quite large as well (i.e. a maximum axial dimension of up to little more than one half the specimen diameter).

Cemented soils were chosen to model both the block and matrix components. A sandportland cement-fly ash mixture was used for the blocks and a bentonite-portland cement mixture was used for the matrix. Thin layers of wax coated with talcum powder were used to model matrix shearing.

Rectangular blocks of model melange were cast and then cored. Six-inch diameter cylindrical specimens of the model melange were tested in triaxial compression to determine their Mohr-Coulomb strength parameters and stress-strain behavior. Models with four different block orientations, each with three block proportions, were created. Figure 1 schematically shows the different model types. The arrows in the diagram indicate the axial loading direction, and the angle indicated (0°, 30°, 60° or 90°) is that between the axial loading direction and the orientation in which the major axes of the blocks are aligned.

Five specimens of each type shown in Figure 1, along with seven 6-inch diameter pure matrix and ten 2-inch diameter pure block specimens were created. This means a total of sixty-seven 6-inch diameter specimens and ten 2-inch diameter specimens were tested for this study. Including all the trial mixes used to finalize the model materials and specimen preparation methods, over one hundred 6-inch diameter and eighty 2-inch diameter specimens were tested.
3 TEST RESULTS

3.1 Effect of block proportion and orientation on strength

Shear strength is represented by the Mohr-Coulomb strength parameters, cohesion (c) and angle of internal friction (ϕ). Table 1 presents the cohesion values for all the specimen types tested.

The effect of block proportion on cohesion is summarized in Figure 2. This plot shows that increasing the block proportion generally decreased the cohesion. In fact, the specimens with high block proportions (approximately 70 percent) had a cohesion only about half that of the matrix alone.

As discussed previously, fabric anisotropy often results in strength anisotropy in rocks. When planes of weakness (e.g., cleavage or shears) are oriented "adversely" with respect to the loading, the result is a significantly lower strength. The block-in-matrix models used in this study were anisotropic due to the orientation of the larger blocks in the specimens. Although the fabric created by orienting the blocks was not nearly as pronounced as that formed, by say, slaty cleavage, it still resulted in at least minimal strength anisotropy. Note that in all cases the specimens with blocks oriented at 30° to the axial loading direction (the most "adverse" orientation tested) had the lowest cohesion, and that the medium and high block proportion specimens with 0° and 90° orientations had higher cohesions than those at 30° and 60°.

<table>
<thead>
<tr>
<th>Block orientation</th>
<th>Average block proportion</th>
<th>c (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/A</td>
<td>0%</td>
<td>2.28</td>
</tr>
<tr>
<td>N/A</td>
<td>100%</td>
<td>3.07</td>
</tr>
<tr>
<td>0°</td>
<td>29%</td>
<td>2.38</td>
</tr>
<tr>
<td>30°</td>
<td>31%</td>
<td>1.72</td>
</tr>
<tr>
<td>60°</td>
<td>33%</td>
<td>2.60</td>
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<tr>
<td>90°</td>
<td>29%</td>
<td>1.90</td>
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<td>0°</td>
<td>50%</td>
<td>1.61</td>
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<td>30°</td>
<td>53%</td>
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<td>0°</td>
<td>72%</td>
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</tr>
<tr>
<td>30°</td>
<td>74%</td>
<td>1.12</td>
</tr>
<tr>
<td>60°</td>
<td>73%</td>
<td>1.24</td>
</tr>
<tr>
<td>90°</td>
<td>71%</td>
<td>2.08</td>
</tr>
</tbody>
</table>
Figure 2. Cohesion versus volumetric block proportion

The trends between cohesion and orientation are similar, although less pronounced, to those reported by Jaeger (1960) and Donath (1964).

Table 2 presents the angle of internal friction values for the models, and Figure 3 shows these values graphically. Figure 3 indicates that the angle of internal friction generally increased with increasing block proportion. This increase was as much as 16.5° for the 60° orientation high block proportion specimens over the pure matrix specimens.

The values of angle of internal friction for the medium and high block proportion specimens did not vary very significantly with block orientation. It is typical that the angle of internal friction varies less than the cohesion due to fabric anisotropy (Goodman, 1989), therefore this was not an unexpected result.

The low block proportion specimens showed evidence of a threshold block proportion below which the presence of blocks had little effect on strength. Note that the cohesion and angle of internal friction for the 0° and 60° specimens were very close to those of the matrix material, while the 30° and 90°
specimens had much lower cohesions and higher angles of internal friction. These data suggest that there was a threshold block proportion at approximately 30%, below which the strength parameters were little affected. Many researchers have reported similar thresholds for heterogeneous soils. Heterogeneous soils are mixtures of coarser particles in a finer grained matrix. Because heterogeneous soils consist of stronger "blocks" in a weaker matrix, they are somewhat analogous to bimrock. For instance, based on a review of the heterogeneous soil literature and some of their own laboratory work, Irfan and Tang (1993) proposed a threshold volumetric block proportion for heterogeneous soils of 25%, a proportion quite close to the 30% suggested by these models.

An inspection of the failed specimens provided an explanation of how the block proportion affected the cohesion and angle of internal friction. The failures tended to form along the block-matrix contacts rather than through the blocks even though the blocks were only about twice as strong as the matrix in unconfined compression. Figure 4 is an unrolled tracing of the cylindrical surface of one of the failed high block proportion specimens with blocks oriented at 60°. It is quite clear that the shear failure closely followed the block-matrix contacts.

The block-matrix contacts were surfaces of weak response just as they are in real melange. Additional testing demonstrated that the Mohr-Coulomb strength parameters for a planar block-matrix contact were a cohesion of 1.03 MPa and an angle of internal friction of 30°. The wax "shears" were also tested and proved to have a cohesion of 0.34 MPa and an angle of internal friction of 12°. Also, as the block proportion was increased, the wax layers became better aligned with the edges of the larger blocks creating more throughgoing "shear" surfaces. The specimens with a higher block proportion therefore, not only had a higher density of weakness surfaces (more blocks), but these weakness surfaces were more continuous. The result of these factors was a lower cohesion.

![Figure 4. Tracing of a failed specimen—high block content, 60° block orientation](image-url)
On the other hand, the angle of internal friction increased because the failure surfaces became more tortuous or "rougher" (they had to fail around more blocks) at higher block proportions. An increase in tortuosity of the failure surface due to the presence of blocks in bimocks has been hypothesized, but never demonstrated, by D'Elia, et al. (1988) for melange and Savely (1990) for boulder conglomerate.

As stated previously, the failure surfaces through the models only rarely passed through the blocks. For this reason, it does not appear that the strength (cohesion and angle of internal friction) of the blocks in a bimock play a role in the strength of the mass so long as the blocks are stronger than the matrix. The strength contrast between the blocks and matrix required to prevent failure through the blocks is not known, but these test results indicate that the strength difference need only be modest.

3.2 Effect of block proportion and orientation on the modulus of deformation

The modulus of deformation is defined in this study as the slope of a line drawn from the origin on the stress-strain plot to the point on the virgin loading curve corresponding to 40 percent of the maximum stress difference \((\sigma_1 - \sigma_3)_{\text{max}}\). This measure is typically called the secant modulus. The term modulus of deformation is used rather than the modulus of elasticity because during virgin loading the model specimens underwent both recoverable (elastic) and nonrecoverable (plastic) deformations.

Figure 5 shows that increasing the block proportion generally increased the modulus of deformation of the models. Each plot is for a different block orientation as indicated by the schematic specimen sketch below each plot. The increase in modulus was expected because as the block proportion was increased, the volume of stiffer inclusions in the softer matrix was increased, resulting in a stiffer model. Normal strength concrete can be viewed as an "artificial bimock" (i.e. it is made up of stronger and stiffer aggregate in a weaker and softer matrix), and similar increases in modulus have been reported in the concrete literature for many years.

Figure 5 also shows that the rate of increase in modulus with block proportion decreased as the block orientation was varied from vertical to horizontal. This result was also not unexpected. Simple analytical models, such as the one proposed by Hirsch (1962) for concrete, can be used to show that when the blocks were vertically inclined they attracted more of the axial stress. The result was a stiffer model. The increase in modulus was smaller for the specimens with horizontally oriented blocks due to wax layer alignment as well. Aligning the blocks tended to align the wax "shears" resulting in subparallel layers of wax in the specimen. The wax used to model the "shears" had a modulus that was approximately one tenth the modulus of the matrix material; hence, when the layers were closer to horizontal they significantly reduced the stiffness of the model.

Note that the moduli for the medium (approximately 50 percent) block proportion specimens with 0° and 30° block orientations fell below the trend set by the low and high proportion specimens. This seemingly anomalous behavior has not proven easy to explain, but one possible solution is based on the alignment of the wax "shears" in the specimens. Inspections of the 0° and 30° medium block proportion specimens provided an indication that for some as yet undetermined reason an unusually large proportion of the wax "shears" were aligned horizontally even though the blocks were more steeply inclined. An unusually large number of horizontal wax "shears" might explain this anomalous behavior.

4 SUMMARY AND CONCLUSIONS

The results of triaxial tests on physical model melanges indicated that the strength and deformation properties of a physical model melange were significantly affected by block proportion and fabric orientation.

The test results showed that the cohesion decreased and the angle of internal friction increased with increasing block proportion, and that the strength, particularly the cohesion, decreased as the block orientation became more adverse. It is postulated that the strength parameters were affected in these ways because the failures formed along the block-
Figure 5. Modulus of deformation versus volumetric block proportion (a) 0° (b) 30° (c) 60° (d) 90°
matrix contacts.

The test results also indicated that the modulus of deformation increased with increasing block proportion and this increase was greatest for specimens with blocks aligned parallel to the direction of axial loading (0°) and smallest for specimens with blocks aligned perpendicular to the direction of axial loading (90°).

Given this information, it is clear that the block proportion is an important value to determine for melanges in the field. Some ideas on how this value can be determined were given by Medley and Goodman (1994). It must be remembered, though, that melange in the field will always be significantly more complex than the models used in this study, and many factors beyond the block proportion and fabric orientation may be important to consider in engineering design work. For example, one might find major zones of sheared material that could play an important role in the field behavior. Even so, it is believed that the basic behavior exhibited by these models is indicative of that which will be found in actual melange.

5 ACKNOWLEDGMENTS

Lab testing was out with the aid of research assistants Dimitrios Protopirou and Jason Choi. Funding for this research was provided by the Pacific Gas and Electric Company, San Francisco. Their support is gratefully acknowledged. Thanks also go to my colleague Ed Medley for all of his thought provoking comments and ideas. Finally, I would like to acknowledge my mentor and thesis advisor Professor Richard Goodman for giving me the chance to tackle this challenging project.

REFERENCES


Behaviour of soils of Vinh-Phuc serie near Hanoi and varved clays from Warsaw under influence of changing water and salt content

Comportement des sols de la série de Vinh-Phuc aux environs de Hanoi et des argiles varves de Varsovie sous l'influence de changements du contenu en eau et sel

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R. Kaczyński
Warsaw University, Poland

ABSTRACT: The results of the laboratory investigations (vane test, triaxial compression, consolidometer) testing changes of water content, salt content and particles orientation influence on strength and deformability parameters are presented in the paper. Tested soils were the cohesive ones of Pleistocene origin which had been collected near Hanoi and Warsaw. The assessment of the very precise parameter influence was described by exponential function. The functional formulas were determined by testing the model soil, which has been accordingly prepared of the natural soils.


1 INTRODUCTION

There are two kinds of states of limiting equilibrium which generally are checked up during designing engineering objects. Using the first one (of ultimate limit state) we can make sure if the planning object’s load doesn’t exceed the soil strength. The second state of equilibrium (serviceability limit state) is for checking if analytical object settlement isn’t greater than permissible deformation for the concrete kind of the building. The solution of problems connected with the object stability requires the knowledge of all parameters which are necessary for checking the above-mentioned states. There are the most fundamental soil characteristics such as shear strength (angle of internal friction, cohesion) and deformability parameters (compressibility modulus, coefficient of consolidation). The above-mentioned parameters are functions of many variables such as:
- soil structure (anisotropy, surfaces of discontinuity),
- changes of water content (effective loading, saturation ratio),
- salt content.

The work is devoted to ascertain influence of the above-mentioned factors on the behaviour of the soils from Warsaw and from Hanoi. The selection of the factors has been dictated by natural conditions. Considering the remarkable heterogeneity of the tested soils in the natural conditions the laboratory investigations were realized on the homogeneous model soil which had been prepared in the laboratory.
2 CHARACTERISTICS OF THE TESTED SOILS

The soils of Vinh-Phuc (V-P) series occur in Red River delta - one of the depressions of Neogenic - Quaternary origin, which are very common in the region of southeastern Asia. Thickness and depth of the soils increase from the delta beginnings towards the sea. The total thickness of the series ranges from 5 to 30 meters. The cohesive soils of V-P series were formed in the conditions of calm sedimentation; they are relatively young, maritime deposits, probably of Pleistocene origin, which in their history - because of the lack of any further extra loadings - haven't undergone any larger consolidation. Presently these soils remain in the state plastic (I_L=0.25-0.50) or hard-plastic (I_L=0.0-0.25). They are laminar deposits consisting of clay layers and silt-clay layers. The brown-red or grey-green colours are their characteristic feature. Concerning their granulometric composition, the cohesive soils of V-P series are clays - 40 % and loams - about 60 %. They are characterized with rather homogeneous mineral composition. The main minerals are: clay minerals which make up to 70 % (illite:kaolinite = 3:2) and quartz, rarely iron hydroxides, chlorite, heavy minerals and organic particles. The structural SEM tests indicate that in undisturbed state V-P soils are characterized with honeycomb microstructure, occasionally matrix one. Salt content in groundwaters is unstable, usually not exceeding 1.0 g/dm³ NaCl, sometimes up to 3.0 g/dm³ NaCl.

The varved clays from Warsaw region are lacustrine clays connected with Middle Polish Glaciation. They were deposited in stagnant waters, generally in basin without outflow. In these lakes the materials from melting glacier were sedimented. The quantity of unfractional material was not always equal, it have led to the stratification of the sediments. It has been stated, that in the warmer period, when the melting of ice was intensified, the flow of thicker material was observed, then the light beds were formed. On the other hand during the colder period the clayey material created the dark beds. The thickness of varved clays changes in the range 2-10 m. These soils are of Pleistocene origin deposited in water conditions with salt content below 0.1 %. In dark varves one can distinguish: quartz, hydrocarbons, illite (as main minerals) and kaolinite, chlorite, smectite (as auxiliary ones). In light varves quartz and illite are main minerals and hydrocarbons and other as auxiliary ones. Microstructure in natural state is skeletal, medium-dispersed (dark varves) and matrix-skeletal and coarsely-dispersed (light varves).

The physical-mechanical parameters of the tested soils are presented below:

<table>
<thead>
<tr>
<th>PARAMETERS</th>
<th>UNITS</th>
<th>The Vinh Phuc series from Hanoi</th>
<th>The varved clays from Warsaw</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay fraction content [%]</td>
<td>10-70</td>
<td>15-75</td>
<td></td>
</tr>
<tr>
<td>specific density [Mg/m³]</td>
<td>2.68-2.73</td>
<td>2.69-2.76</td>
<td></td>
</tr>
<tr>
<td>volume density [Mg/m³]</td>
<td>1.81-2.13</td>
<td>1.80-2.05</td>
<td></td>
</tr>
<tr>
<td>porosity [%]</td>
<td>30-54</td>
<td>35-50</td>
<td></td>
</tr>
<tr>
<td>natural water content [%]</td>
<td>12-43</td>
<td>20-40</td>
<td></td>
</tr>
<tr>
<td>liquid limit [%]</td>
<td>22-63</td>
<td>30-70</td>
<td></td>
</tr>
<tr>
<td>plastic limit [%]</td>
<td>13-28</td>
<td>20-35</td>
<td></td>
</tr>
<tr>
<td>plasticity index [%]</td>
<td>7-30</td>
<td>10-40</td>
<td></td>
</tr>
<tr>
<td>sketpton activity [l]</td>
<td>0.3-1.5</td>
<td>0.5-1.2</td>
<td></td>
</tr>
<tr>
<td>angle of internal friction [°]</td>
<td>3.5-4</td>
<td>10-25</td>
<td></td>
</tr>
<tr>
<td>cohesion [kPa]</td>
<td>10-90</td>
<td>30-100</td>
<td></td>
</tr>
<tr>
<td>compressibility modulus [MPa]</td>
<td>2-100</td>
<td>5-50</td>
<td></td>
</tr>
</tbody>
</table>

3 METHODS AND THE RESULTS OF INVESTIGATIONS

To eliminate the changeability of granulometric composition and to ensure independence of the conditions of the soils origin, the decision was made to carry out tests on the model soil with constant grain and mineral composition specially prepared in the laboratory. The model soils were characterized with the following parameters:
The samples of the model soil were formed as follows. Mass of dry soil with grain size below 0.05 mm with the proper quantity of water (sometimes salted) was stirred for 30 minutes. After 48 hours when the water content was balanced the sample was statically compacted in a special cylindrical steel container. The velocity of exerting the loadings was 400 kPa per hour. The obtained mass had various orientation of particles (H - horizontal, V - vertical), final compaction: 200, 300, 1200 kPa, salt content: 0, 3, 10 mg/dm³ and water content in the range 5-35%. These samples were tested in the following instruments:

- vane test (Wykeham Farrance Engineering Ltd)
- triaxial apparatus
- consolidometer (oedometer)

The qualitative evaluation of the influence of changes of water content, salt content and particles' orientation appears as follows:

1. Increase of water content (in %) causes decrease of shear strength in model soils independently of the applied apparatus. This relation is curvilinear, exponential (all relations were determined for the correlation coefficient R>0.85 and significance level = 0.01), for:

   - vane test apparatus (τᵥ, max. shear strength, MPa; VP - V-P soils; VW - varried clays from Warsaw): τᵥ = e^αᵥwbᵥ, where: aᵥ=4.90, bᵥ=0.36 and aᵥ=3.80; bᵥ=0.28,

   - triaxial shear apparatus (σᵥ, deviator stress at the moment of failure, MPa): σᵥ = e^αᵥwbᵥ, where: aᵥ=1.30; bᵥ=-0.14 and aᵥ=1.40; bᵥ=-0.18.

   In consolidometric tests the dependance of compressibility modulus Mᵥ (MPa) on water content (%) was established: Mᵥ = aᵥwbᵥ, where: aᵥ=14.67-54.05; bᵥ=-0.56 to -0.20 and aᵥ=20.70-62.10; bᵥ=-0.58 to -0.23, as well as coefficient of consolidation cᵥ (10⁴ cm²/s) and water content (%): cᵥ=αᵥwbᵥ, where: aᵥ=19.6 to -6.8; bᵥ=250-1062 and aᵥ=40.5 to -7.1; bᵥ=257-1105.

2. The influence of NaCl content on shear strength is heterogeneous and depends on the loading and concentration of solution (0, 3, 10 - contents of NaCl in g/dm³; tanβ - slope of path of stress). The coefficients characterizing this influence in the tests:

   - vane test: Zᵥ = τᵥ / (10): τᵥ (3): τᵥ (0) = 0.80-1.18,
   - triaxial compression apparatus: τᵥ = tanβ (10): tanβ (3): tanβ (0) = 0.87-1.19,
   - consolidometer: Zᵥ(NaCl) = Mᵥ(10): Mᵥ(3): Mᵥ(0) = 0.84-2.15.

   The above relations indicate weak reaction of V-P soils on the changes of salt content.

The respective coefficients for varred clays from Warsaw appear as follows: Zᵥ(NaCl) = 1.10-2.77 and Zᵥ(NaCl) = 0.84-2.32.

3. The influence of the orientation of the particles on strength and compressibility parameters varies concerning intensity. In V-P soils one determines relatively little anisotropy, type H>V. It means that the strength of horizontal samples is bigger than of vertical ones. It doesn’t depend on the applied apparatus. Anisotropy coefficient for shear strength equals 0.99-1.12, for compressibility modulus equals 0.83-1.35.

Varred clays from Warsaw indicate also anisotropy type H>V. Anisotropy coefficient for shear strength equals 1.06-1.44, for compressibility modulus equals 0.96-1.35. In the clays from Warsaw with the increase of salt content one determines increase of anisotropy up to 1.55.

4 CONCLUSIONS

The soils of V-P series from the region of Hanoi and the varred clays from the region of Warsaw have been deposited in water conditions (first ones in a shallow sea, second in a large stagnant basin with little salt concentration) in Pleistocene. They are characterized with similar mineral composition and similar
consolidation and almost the same colloidal activity. The test made
with the application of unified method reveal that:

1. In the range of applied changes of water content the shear strength
decreases with the increase of water content. This follows independently of the apparatus used as well as the kind of soil. Of course, susceptibility of the soils on deformation increase proportionally to increasing water content. These interdependences are expotentially curvilinear.

2. Weak reaction of the V-P soils on the changes of salt content are
caused by the sorptive complex (containing ions Na⁺) connected with the primeval conditions of sedimentation. Clear influence of salt content on shear strength and deformability of the clays from Warsaw should be linked with the lack of primeval salt content in the sedimentary basin.

3. In the tested soils the type H>V of anisotropy is observed, independently of the applied apparatus, this concerns both shear strength and compressibility modulus. In the V-P soils anisotropy coefficients are relatively small. In the varved clays from Warsaw anisotropy is more evident; there is clear influence of salt content on increase of anisotropy coefficient in Warsaw clays.

REFERENCES

Geotechnical aspects of three volume change soil types
Aspects géotechniques de trois types de changement de volume en sols

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ABSTRACT: Part one introduces (i) physiography and geology of the Cape Peninsula, (ii) location and insitu profile for each site, and (iii) consistency limits and basic geotechnical properties for each soil type. Part two deals with observed changes in volume and mass of initially undisturbed soil samples, as the water content is altered, while the sample itself is either free, partially or fully restrained from changing its volume. Observations are presented for three conditions: Free shrinkage and free swell paths; Expansion or collapse under known vertical pressure while being laterally confined; Swell pressure for controlled strain conditions.

RESUMÉ: La première partie décrit (i) la physiographie et la géologie de la Péninsule du Cap, (ii) montre la localisation des profils géologiques pour chaque site, et (iii) présente les limites de consistance et autres caractéristiques géotechniques principales pour chaque type de sol. La deuxième partie présente les variations de volume et de masse d'échantillons intacts en fonction de la variation de la teneur en eau, pour des conditions de non confinement latéral de l'échantillon et avec contrôle partiel ou total du volume. Des observations sont faites pour trois conditions: Retraction et expansion libres, Gonflement ou collapse sous pression verticale contrôlée et déformation latérale empêchée; Pression de gonflement pour une déformation imposée.

1 PHYSIOGRAPHY AND GEOLOGY OF THE CAPE PENINSULA

1.1 Introduction to the area and its physiography

The Cape Peninsula is densely populated, except in the mountainous areas and hills near Tygerberg and Durbanville. It has a Mediterranean climate with hot dry summers and cool, wet winters (Capetour 1985).

Mean annual rainfall is 550 mm. At Table Mountain the Peninsula reaches a maximum altitude of 1086 m. It is bordered on the east by the Cape Flats on which the highest dunes are 65 m above Sea Level. Farther east the Tygerberg Ridge is 456 m high at Kanokop.

Upon weathering, the resistant sandstones (of Table Mountain Group) have given rise to steep cliffs, as compared with rolling hilly landscape characterizing granite and Malmesbury rock terrain. The Diep, Mosselbank, Elsieskraal, Liesbeek, Salt and Kuils Rivers constitute the major drainage systems, with a few smaller streams in the mountainous Peninsula. They all are influenced by wind blown sand.

An aerial view of the Cape Peninsula area is shown in Figure 1.

1.2 Geological formations present in the area (Theron 1984)

Malmesbury Group (oldest rocks in local sequence): It was deformed under influence of pressure from the east into narrow parallel synclines/anticlines (strike is north, north-west).

The Tygerberg formation consists predominantly of irregular alternations of grey to green phyllitic shale, siltstone and massively bedded medium fine grained quartzitic greywacke. They are found over a large area, mostly covered by surficial sediments, and often deeply weathered to a red brown or yellow clay.
Cape Granite Suite (next oldest rocks): The Cape Peninsula pluton consists mostly of biotite granite characterized by large feldspar phenocrysts. In addition to metamorphosed Malmesbury inclusions, tourmaline rich clots are present. The grey coarse grained biotite granite gives rise to massive round blocks characterized by prominently weathering and twinned orthoclase phenocrysts. These rocks occur south & west of the contact with Malmesbury rocks. The Kuils River-Helderberg pluton consists mainly of leucocratic, fine to medium grained quartz feldspar granite. Agglomerate and Basaltic rocks are a mixed mass of dark green to grey with red brown specks.

Cape Supergroup: The Graafwater formation rests on granite and other pre-Cape rocks. A weathered zone (few millimetres to several metres thick) characterizes the granite at contact. Thickness ranges from 65 m at Llandudno-Karbonkelberg to 25 m at Simonstown. It comprises alternations of red to purple siltstone, sandstone, shale, and mica rich sandy shale together with thin bedded pink to white quartztitic sandstone. Thickness of units varies between 20 mm and 6 m. Lowest formation in series, basal shales differ from the sandstone (overlying it) by thinner bedding, red to maroon colour and fine micaceous siltstone layers.

The Peninsula formation consists of a uniform light grey, well bedded, medium coarse grained quartztitic sandstone. Massive thinly bedded units alternate with one another. It is found on Table Mountain (reaches 600 m thick). Outcrops of Parkhuis formation (sandy tillite) are confined to area on summit of mountain.

Dykes of Dolerite (Carboniferous to Jurassic): Dolerite dykes are intrusive into Malmesbury rocks and granite, have a north westerly strike, a steep dip, and outcrops confined to beach zones because of scree slopes. Hornblende-lamprophyre dykes occur north of Bellville. The rock consists of brown to green hornblende phenocrysts. Trachyte dykes weather to grey green dense hard rocks with pale feldspar and dark mica phenocrysts.

Surface Deposits (youngest in local sequence): Alluvial deposits, consisting of dark coloured organic sand, border the larger river courses and gradually merge into the adjoining grey sandy soil. Scree deposits consist of angular sandstone, in blocks of several centimeters to a few metres in diameter, with interstitial gravel and sand. Table Mountain sandstone, granite and Malmesbury rocks are covered by this layer. Along the coast and over large portions of Cape Flats, shell bearing light grey sands occur (up to 50 m thick). It varies from fine to coarse grained, and with highly rounded silica grains.
Inland, grey to pale red sandy soil underlies large areas of Cape Town's southern suburbs. It stretches to the slopes of Tygerberg and Bottelaryberg hills and upper reaches of the Kuils and Mosselbank Rivers (Kraaifontein-Durbanville area). The deposit consists of interwoven sandstone and thin clay layers. The predominant soil formed on Malmesbury rocks is yellow red or brown. It is clayey and contains small nodules of ferricrete, quartz fragments, and a variable amount of sand grains. Exposure of this loamy soil is confined to Tygerberg-Durbanville as a thin cover. It may rest directly on rock or on a partly cemented layer with ferricrete or a clay layer (thickness varies, grades into weathered rock). Raised beach terraces/deposits (former higher sea) are confined to the Peninsula.

Silcrete and ferricrete are widely distributed in isolated strips, specially north of Durbanville and near Phesantekraal. Silcrete outcrops consist of hard, well silicified rock that weathers into massive, smooth and partly rounded blocks with conchoidal fracture. Ferricrete either occurs in the soil as loose nodules (few millimetres to several centimetres in diameter), or as more compact zones of variable thickness.

Typical surface calcrete, composed of a hard layer up to a few metres thick and often covered by sand, constitutes most of the outcrops. Calcrete varies from typical massive, grey sandy surface limestone, to cemented well bedded sandy limestone, to friable partly cemented calcareous sand. The most important mineral resources of the area are glass sand, kaolin and construction materials.

2 LOCATION AND PROFILE FOR EACH SITE

2.1 Field investigations and location of sites

A number of sites were originally investigated in the Cape Peninsula (Vinagre 1978). The location of each site is shown in Figure 1. Trial holes were dug in areas previously identified from geological maps, exposed shales were inspected in road cuts, and commercial sources of clay were visited. Several soil samples were obtained from each potential site.

2.2 Insitu profiles for each site

Site 11, Rosebank: Sampling holes were excavated behind a building near the Rondebosch Common. Undisturbed samples were carefully cut and removed from the sides of the hole. They had a dark reddish brown colour, a plastic feel, and were found to be considerably moist, appearing saturated. The insitu soil profile is shown in Figure 2(i).

Site 36, Rondebosch: Block samples were cut from a trench near UCT's School of Music. The colour was a mixture of light orange and red brown. The soil was slightly moist and felt plastic and soft (when dry it was sandy soft). The profile is shown in Figure 2(ii).

Site 30, Hout Bay: Samples were taken from a road cut near Princess Road. They were slightly moist and yellow/orange in colour. The profile is in Figure 2(iii).

Figure 2: Insitu soil profiles for sites (from left) - (i) 11 Rosebank, (ii) 36 Rondebosch, and (iii) 30 Hout Bay.
3 CONSISTENCY LIMITS AND BASIC GEOTECHNICAL PROPERTIES

3.1 Observed limits and properties

Table 1 presents a summary of the measured limits and engineering properties. The consistency limits and particle size distribution for the soil types are presented in Figures 3 and 4 respectively.

Table 1: Soil types - summary of the measured consistency limits and basic engineering properties.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Location</th>
<th>w_1%</th>
<th>w_C</th>
<th>w_P</th>
<th>I_p</th>
<th>w_S</th>
<th>Clay</th>
<th>Silt</th>
<th>Sand</th>
<th>G_s</th>
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</thead>
<tbody>
<tr>
<td>11</td>
<td>Rosebank</td>
<td>11.4</td>
<td>69</td>
<td>31</td>
<td>23</td>
<td>29</td>
<td>12</td>
<td>2.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>Rondebosch</td>
<td>8.3</td>
<td>47</td>
<td>23</td>
<td>24</td>
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<td>49</td>
<td>2.69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Hout Bay</td>
<td>6.4</td>
<td>49</td>
<td>29</td>
<td>20</td>
<td>23</td>
<td>34</td>
<td>2.64</td>
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<td></td>
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</tbody>
</table>

3.2 Definition of soil types based on particle size

Based on particle size only, the soils can be defined: Site 11 - fine silty clay, with only a small amount of fine sand; Site 36 - clayey silt, with fine sand and a small amount of medium sand; Site 30 - fine / medium sandy silt with a small amount of coarse sand.

3.3 Soil types according to plasticity characteristics

With exception of site 30, plotted results of plasticity index against liquid limit place the soil types above the Casagrande A-line (separates inorganic clays and silts). According to plasticity characteristics the soils can be grouped: inorganic clay of high plasticity (S11 Rosebank), and medium plasticity (S36 Rondebosch); and low cohesion soils - inorganic silts of medium compressibility and organic silts (S30 Hout Bay).

4 FREE SHRINKAGE and FREE SWELL PATHS

4.1 Definition of free-shrinkage and free-swell

The word "free" will in this case mean that during the process of overall volume change, the sample is neither confined by externally applied stresses nor by a fixed volume container. In other words the sample is merely sitting on a flat smooth surface and is free to change its linear dimensions in all directions. The changes in the overall volume occur due to effective stresses between the grains, caused by changes in the tensions within the pore water pressure, and due to volumetric changes of the effective soil particles.
4.2 Free-shrinkage and free-swell paths

The change in void ratio which takes place between two positions on a free-swell or free-shrinkage path is mainly due to a change in the average effective stress in the soil. The change in volume of the actual soil grains will normally also cause a change in pore volume, but because these are both an increase or a decrease, the change in value of void ratio will be small due to this latter cause, and will be due mainly to the change in effective stresses between the grains. The free-swell or free-shrinkage path can therefore be regarded as evidence of isotropic consolidation, or swelling, due to effective stresses which are isotropic at each point in these unconfined samples.

Shrinkage path results from the plot of void ratio 'e' against water ratio 'wr' (defined by Sparks 1975, as water content x specific gravity) for each point of the air drying portion of the cycle, and the swelling path from the wetting portion. Typical free-shrinkage and free-swell paths (Vinagre 1978) for the three sites, are shown in figures 5(i), 5(ii) and 5(iii).

Figure 5(i): Site 11 Rosebank - free shrinkage (A to B) and free swell paths (B to C).

Figure 5(ii): Site 36 Rondebosch - free shrinkage (A to B) and free swell paths (B to C).
4.3 The Free Expansion Limit

The water content, above which the unrestrained soil sample will not swell any further, was observed. It has been expressed as water ratio, which takes into account the specific gravity of the soil particles.

The Free Expansion Limit was determined for each soil type - see Table 2.

Table 2: Natural soils [undisturbed] - Free Expansion Limit 'FEL' values.

<table>
<thead>
<tr>
<th>Site</th>
<th>Location</th>
<th>FEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Rosebank</td>
<td>1.25</td>
</tr>
<tr>
<td>36</td>
<td>Rondebosch</td>
<td>0.84</td>
</tr>
<tr>
<td>30</td>
<td>Hout Bay</td>
<td>0.85</td>
</tr>
</tbody>
</table>

The values obtained are shown in Figure 6 below.

Figure 6: Free expansion limits for the soil types.

5 EXPANSION OR COLLAPSE UNDER KNOWN VERTICAL PRESSURE WHILE BEING LATERALLY CONFINED

5.1 Introduction to the three types of expansion

The relationships between the void ratio 'e' and the water content 'wr' - includes the three types of expansion - are shown in Figure 7.

5.2 Type of expansive and collapsing behaviour (Sparks, Retief, Errera & Vinagre 1981)

The type of expansive or collapsing behaviour being investigated, under controlled wetting conditions, refers to soil samples which are confined in the horizontal direction and, while being subjected to a known vertical constraint, are allowed to increase (expand) or decrease (collapse) their volume along the vertical axis.

A standard consolidometer was used to apply the external vertical stress against which the sample expands or collapses upon being wetted. Samples were either permitted to lose or gain moisture until the required initial water content was reached.

The word Expansion refers to the increase in the overall volume of the sample caused by the interaction of water and the clay minerals in the soil sample. The addition of water causes an increase in the lattice spacing of the clay minerals which results in the overall volume increase of the sample.

The word Collapse refers to breakdown in soil structure resulting from the wetting up of the clay bridges which link the fine sand and silt grains, and result in loss of shear strength and decrease in overall volume of the sample.
The one-dimensional measurement of the change in volume of the sample, as the result of wetting and while being subjected to a known and constant stress, occurs under conditions in which drainage is permitted throughout.

Figure 8: Site 11 Rosebank [undisturbed, 100 Kpa during wetting] relationship between the void ratio 'e' and the water ratio 'wr'. (A to B = air drying; B to C = load; C to D = wetting; D to E = load; E to F = unload).
5.3 Test results for site 11, Rosebank (Figures 8 and 9)

The test results consist of measurements of vertical deflection observed in a sample which is confined in the horizontal axis. These changes in volume take place as a result of an increase (or decrease) in vertical pressure or water content.

Figure 9: Site 11, Rosebank [undisturbed, 100 Kpa during wetting] - Relationship between the void ratio 'e' and the effective vertical pressure (or stress) 'peV'. (A through to F as for Figure 8)

5.4 Site 11, Rosebank - plots of 'void ratio' versus 'effective vertical pressure' versus 'water ratio' (Figure 10)

6 SWELL PRESSURE FOR CONTROLLED STRAIN CONDITIONS

6.1 Swell Pressure

The word "swell" will in this case mean the outward expansion of the soil sample resulting from the interaction of the (added) water and its clay minerals.

The word "pressure" will in this case mean that during the process of overall volume change, the sample is prevented from increasing its volume by an externally applied stress (along the vertical axis) while being confined by a fixed consolidometer ring (horizontal plane).

6.2 Test results - Figure 11

In very brief terms, it consists of one dimensional measurement of pressure, resulting from controlled wetting, for no volume change.

Relationships of void ratio versus water ratio, for
the controlled wetting path, include the observed maximum and final values for swell pressure. It should be noted that the actual full test path also includes the free shrinkage and unloading paths.

pressure is reached where this volume change is zero. If the confining pressure is increased beyond this critical value the sample will decrease in volume when wetted.

7 PRELIMINARY CONCLUSIONS

In the case of expansive behaviour of the soil types, a number of preliminary conclusions can be made:

1. For the same initial void ratio and the same value of initial confining pressure, there appears to be a linear correlation between the initial water content (before wetting) and the percentage volume change resulting from wetting the sample.

2. The larger volume changes occur if the initial water content is low, and if the vertical confining pressure is also low.

3. As the effective vertical pressure restraining the sample increases, there is a gradual reduction in the amount of percentage volume change, until a certain
Figure 11: Site 11, Rosebank [undisturbed samples] - Void ratio 'e' versus water ratio 'wr', for the controlled wetting path (includes maximum and final pressures while confined)

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Laboratory characterization of consolidated ground around a land-sea junction for a strait crossing

Caractérisation en laboratoire d’un terrain consolidé autour d’une jonction terre-mer pour une traversée de détroit

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GEODATA SpA, Torino, Italy
A. Nosetto
AQUATER SpA, S. Lorenzo al Campo, Italy

ABSTRACT: In a proposed strait crossing scheme involving the submerged tunnel option, the land-sea junction required special attention to the strength and permeability of the loose ground (sand and gravel). A grouting system was proposed to improve the ground characteristics for guaranteeing stability of the tunnel and limiting the seepage of water. Laboratory tests were performed on both the natural and the grouted materials to quantify the design input data (strength, deformation modulus, and permeability). The results demonstrated the validity of the proposed grouting system for adequately improving the ground characteristics.


1. INTRODUCTION

The “Messina Strait Submarine Crossing” project was proposed by “Consorzio ENI per lo stretto di Messina” of Italy, and the feasibility study for this project was completed at the end of 1994. In the submarine crossing solution (as an alternative to the bridge crossing), there are many technical problems to be solved, in particular the construction of the land-sea junction tunnels.

The subsoils of both sides of the Messina Strait consist of gravelly sand deposits underlain by soft rocks. On the Sicilian shore, the sand and gravel deposits extend to a depth of more than 180m. The various formations at the site are described by Bosoni et al. (1994) together with a critical examination of the design and construction problems.

One of the major problems is related to the underground excavation in the gravelly sand deposits below water table, particularly for the land-sea junction tunnels. The alluvial deposits are completely incoherent, highly permeable, and lack a self-supporting capacity even for the short period required for installation of the initial or primary support. The proposed solution for reducing water inflow and ensuring stability of the excavation is to adopt an appropriate technique for pre-excavation ground improvement, for example, cement-grouting of a ring of the loose ground of certain thickness around to the excavation profile.

To provide the basis for implementing such a solution, parametric analysis of stress and deformation coupled with seepage were made for the excavation design of the tunnels using a finite element method. A detailed description of
the analysis and the results were reported by Grasso et al. (1994). According to the results of such analyses, the virgin material need to be subjected to a suitable ground improvement technique, in order to guarantee the stability of tunnel before installing the lining and to limit the amount of water inflow into the tunnel (to < 12 l/min).

The values of the geotechnical parameters of the virgin material and the corresponding values of the same material after improvement, as established by the analysis, are given in Table 1.

<table>
<thead>
<tr>
<th>Material type</th>
<th>γ (kN/m³)</th>
<th>c (kPa)</th>
<th>φ (°)</th>
<th>E (MPa)</th>
<th>K (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Virgin</td>
<td>19</td>
<td>15</td>
<td>35</td>
<td>100</td>
<td>5·10⁻⁴</td>
</tr>
<tr>
<td>Treated</td>
<td>21</td>
<td>500</td>
<td>40</td>
<td>500</td>
<td>1·10⁻⁴</td>
</tr>
</tbody>
</table>

The composition of the alluvial deposits is characterized by relatively coarse materials (coarse sand and gravels) besides medium sand, fine sand, and silt and clay. Knowing the penetration capacities and limits of the various available cementing agents, the optimal procedure for injecting this type of materials at low pressure is as follows:

1) A primary grouting is done with standard cement mortar to fill the relatively large voids in the ground,

2) An integrative injection is made using a microfine-cement based mix for treating the medium-to-fine portions of the ground.

3) If the result of the above treatments still cannot meet the design requirements, injection of silica or resin based mixes will have to be made to treat the remaining finer portions, mainly for "impermeabilization".

Grouting with ordinary cement mortar and injection of chemical solutions are both well-established ground-improvement techniques, whereas injection of microfine-cement based mixes is relatively new and only limited experience is available on its use. Therefore, a decision was made to conduct experimental research on consolidated samples of the virgin material to:

1) define the type of microfine cement suitable for the project,

2) determine the optimal ratios between the various components in a mix and the identification of suitable additives to guarantee workability and uniform penetration of the mix,

3) establish the most appropriate procedure for the execution of injecting microcement based mixes, and

4) verify the possibility of achieving the characteristics assumed for the virgin material after improvement.

2. PROGRAM OF LABORATORY STUDY AND SAMPLE PREPARATION

2.1 Program of investigation

To fulfill the objectives of this research, a laboratory investigation program was structured into the following three phases:

Phase 1: Geotechnical tests on reconstituted samples to determine the basic characteristics of the materials to be treated.

Phase 2: Study of the most appropriate injection mixes and performance of injectability and consolidation tests on the reconstituted materials.

Phase 3: Geomechanical tests on consolidated samples, and determination of mechanical and hydraulic characteristics of the consolidated materials as functions of the degree of cementation and the type of cement mix used.

2.2 Sampling and specimen preparation

Representative materials of certain quantity from 4 key boreholes along the land-sea junctions (Table 2) were selected for this laboratory study. The loose materials obtained from the boreholes were in a disturbed state and there was no alternative but to work with reconstituted samples.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Weight (kg)</th>
<th>Borehole No.</th>
<th>Side of the Strait</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>37.5</td>
<td>G2</td>
<td>Calabrian</td>
</tr>
<tr>
<td>2</td>
<td>24.5</td>
<td>B3</td>
<td>Calabrian</td>
</tr>
<tr>
<td>3</td>
<td>24.5</td>
<td>B3</td>
<td>Sicilian</td>
</tr>
<tr>
<td>4</td>
<td>29.5</td>
<td>G10</td>
<td>Sicilian</td>
</tr>
</tbody>
</table>

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The in-situ material has inherent variabilities not only in its grain size composition but also in its water content and density state. In order to have the reconstituted samples represent, as closely as possible, the virgin material, it was decided to single out three representative particle size distributions based on the results of statistical analysis of the many particle size analyses conducted for the feasibility. The grain size compositions of the representative distributions are given in Table 3. These typical distributions represent in effect three different types of material, which shall be referred to as material Types A, B, and C.

Table 3. Grain size composition of the three representative distributions

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt and clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>36</td>
<td>49</td>
<td>15</td>
</tr>
<tr>
<td>B</td>
<td>19</td>
<td>60</td>
<td>21</td>
</tr>
<tr>
<td>C</td>
<td>30</td>
<td>50</td>
<td>20</td>
</tr>
</tbody>
</table>

Particle size analyses by both wet and dry sieving were performed for the 4 samples from the key boreholes indicated in Table 2, to verify their representativeness and the feasibility of reconstituting the three types of materials shown in Table 3. According to the results of such analyses, Sample I from borehole G2 was used to reconstitute Type B material, Sample 2 from borehole B3 on the Calabrian side was used to make Type C material, and Samples 3 and 4 from the two boreholes on the Sicilian side were combined to form Type A material.

After reconstitution, further sieve analysis was made for each material type. Fig. 1 shows the particle size distributions of the three different types of reconstituted materials. The particle size distribution curve of a particular sand which represents the penetration limit between a standard cement mix and a microfine cement mix is also presented in Fig. 1 for comparison.

The coarser portions of the material are not expected to present any problem with respect to grouting by a conventional cement mix. Therefore, only the fraction between 0.075 and 4.0mm was selected from each material type for this study.

Finally, part of the reconstituted materials were allocated for the geotechnical tests of Phase 1 and the rest for Phase 2 study. The samples obtained by injection in Phase 2 were subsequently used for the geomechanical tests of Phase 3.

![Particle size distributions of the reconstituted materials](image)

Fig. 1 Particle size distributions of the reconstituted materials

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3. TESTING

The following subsections describe the various tests conducted for the three individual phases of this study.

3.1 Phase 1 tests

The geotechnical tests on the reconstituted loose materials corresponding to those dedicated for the injection tests, were made by PANGEA S.r.l of Milan, Italy, a commercial soil testing laboratory. The various tests conducted are summarized in Table 4.

<table>
<thead>
<tr>
<th>Type of test or measurement</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle size analysis</td>
<td>2</td>
</tr>
<tr>
<td>Apparent density</td>
<td>9</td>
</tr>
<tr>
<td>Water content</td>
<td>9</td>
</tr>
<tr>
<td>Porosity</td>
<td>9</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>9</td>
</tr>
<tr>
<td>Permeability</td>
<td>11</td>
</tr>
<tr>
<td>Triaxial compression test</td>
<td>11</td>
</tr>
<tr>
<td>Direct shear tests</td>
<td>18</td>
</tr>
</tbody>
</table>

Table 4. Summary of geotechnical tests

For each material type, 2 sets of direct shear tests (with 3 specimens in each set) were conducted to observe the influence of the degree of saturation (0% and 50%) on density and shear resistance. The condition of full saturation was verified by the triaxial tests. The specimens in each set were consolidated under normal stresses of 200, 400 and 600 kPa, respectively; and they were subsequently sheared under drained conditions.

For the triaxial tests, one specimen for each material type was reconstituted directly inside the membrane in saturated conditions. The procedure of multistage, isotropical consolidation, undrained shearing with measurement of pore pressure was adopted for testing.

The permeability for each type of reconstituted material was measured using the falling head test with a vertical stress (200 kPa) being applied to the specimen. Tests were conducted in modified oedometer cells. For each material, three specimens were reconstituted and tested at different degrees of saturation (0, 50, 100%), to evaluate the influence of saturation on permeability.

3.2 Phase 2 tests

The study of Phase 2 was mainly focused on the use of a particular type of microfine cement, allowing for eventually different cement concentrations in the mix and different injection procedures.

Microcement based mixes can have viscoelasticity near to that of water, high penetrability in porous matrix close to that of chemical grouts, and the advantages of ordinary cement based products such as high stability and considerable mechanical strength when cured.

The microcement trade name TREVICEM 0/3, was chosen for this study, which differs from ordinary cement only in the degree of fineness of the particles. Table 5 provides a comparison of the geometric characteristics between TREVICEM 0/3, other microcements, and ordinary cements. The particle size of microcements is normally measured accurately using the modern technology of laser diffraction.

Table 5. A geometric comparison between microcements and ordinary cement

<table>
<thead>
<tr>
<th>Geometric Characteristics</th>
<th>TREVICEM 0/3</th>
<th>Other Microcements</th>
<th>Ordinary cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>range of grain size (μm)</td>
<td>0.1-15</td>
<td>0.05-28</td>
<td>1-100</td>
</tr>
<tr>
<td>Blaine specific surface area (cm²/g)</td>
<td>15000</td>
<td>8000</td>
<td>2500</td>
</tr>
<tr>
<td>d50 (μm)</td>
<td>3</td>
<td>2-5</td>
<td>9-16</td>
</tr>
<tr>
<td>d98 (μm)</td>
<td>10</td>
<td>10-20</td>
<td>40-85</td>
</tr>
</tbody>
</table>

In particular, three kinds of mixes were defined for the current study on the basis of past experience. The composition and basic characteristics of the three mixes named M1, M2 and M3, are summarized in Table 6, where the same information regarding a typical ordinary cement (Portland 425) based mix (named OCM) with bentonite additives is also included in the last column for comparison.

The injection tests for this study were conducted on cylindrical columns, 46mm in diameter and 1000mm long, which held the reconstituted materials. The cylinder is made of a transparent material capable of resisting the
Table 6. Composition and basic characteristics of the mixes employed

<table>
<thead>
<tr>
<th>Composition and Characteristics</th>
<th>Mix Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M1</td>
</tr>
<tr>
<td>Water to cement ratio</td>
<td>2</td>
</tr>
<tr>
<td>Dosage per cubic meter</td>
<td></td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>428</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>856</td>
</tr>
<tr>
<td>additives (kg)</td>
<td>4.0</td>
</tr>
<tr>
<td>Basic properties</td>
<td></td>
</tr>
<tr>
<td>Unit weight (g/cm³)</td>
<td>1.2</td>
</tr>
<tr>
<td>Marsh viscosity</td>
<td>27</td>
</tr>
<tr>
<td>Volumetric yield</td>
<td>98</td>
</tr>
</tbody>
</table>

action of grout injection and is easy to break for extraction of the injected sample without disturbance. The whole test setup (Fig. 2) was realized by TREVI of Cesena, Italy. The limiting injection pressure of the system is 15 bars.

In order to simulate the field production grouting, the microcement-based mix was injected from the bottom of each column under low pressures regulated by compressed air. During each injection test, the volume of the mix injected and the injection pressure are continuously monitored.

The upward rate of advance of the mix in the cylinder is maintained as constant as possible (around 10 cm/min), while the injection pressure is kept at a value as low as possible. Each time a reduction in the advance rate under a given pressure becomes evident, the pressure is augmented gradually to a new level.

An injection test is considered as successfully completed if the mix overflows the top of the cylinder. In any case, the injection is interrupted if the maximum pressure of the system is reached.

Following the test procedure outlined above, injection tests on the reconstituted materials (Types A, B and C) were conducted using the three microcement based mixes defined in Table 6. In total, 12 column samples as indicated by the matrix shown in Table 7 were obtained and these samples were subsequently used for Phase 3 tests.

The numbers 1, 2 and 3 in Table 7 stand for both the mix type and the samples cured from the corresponding pure mixes, whereas A1...C3 represent samples of consolidated materials.

![Diagram](image)

1 - compressed air cylinder
2 - pressure regulator
3 - container of injection mix
4 - pressure gauge
5 - shaker
6 - column to be injected
7 - graded cylinder for collecting overflow-mix

Fig. 2 Sketch showing the laboratory injection test setup

Table 7. Samples consolidated for Phase 3 tests

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Material Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>1</td>
<td>A1</td>
</tr>
<tr>
<td>2</td>
<td>A2</td>
</tr>
<tr>
<td>3</td>
<td>A3</td>
</tr>
</tbody>
</table>

3.3 Phase 3 tests

Specimens were prepared (28 days after injection) from the 12 column samples shown in Table 7 for geomechanical characterization. The tests and measurements realized are summarized in Table 8. For quality control purposes, the relevant procedures described by Mahtab et al. (1994) were followed for the specimen preparation and testing.

Visual examinations of the 9 injected column samples were first made before dividing them up into specimens for testing. It was observed that the quality of consolidation of each column tends to decrease with the distance from the point of injection, since during injection the phenomenon of pressure filtration developed.
such that the mix deteriorated or gradually became poorer in the cement content as it traveled towards the top of the column. In the extreme cases of A3, B3 and C3 columns, incomplete penetration resulted in the upper part of the column where the materials is practically incoherent.

For the above reason, it was practically impossible to define a group of 4 to 5 specimens with similar characteristics from any single column sample for triaxial testing. Therefore, there was no choice but to adopt the multiple failure state triaxial testing procedure suggested by Kovari and Tisa (1975).

4. TEST RESULTS
The results of Phase 2 of the study are presented in Sec. 4.1. For the purpose of comparison, the results of Phase 1 are presented together with those of Phase 3 in Sec. 4.2.

4.1 Results of Phase 2 tests
For each material type, three 100cm-long columns were prepared and injected by three different mixes and, in total, nine column tests were made. The results of these column injection

---

Table 8. Summary of geomechanical test

<table>
<thead>
<tr>
<th>Type of test or measurement</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (air dried)</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>(saturated)</td>
</tr>
<tr>
<td>Sonic wave velocity (air dried)</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>(saturated)</td>
</tr>
<tr>
<td>Permeability</td>
<td>12</td>
</tr>
<tr>
<td>Unconfined Compression Test (air dried)</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>(saturated)</td>
</tr>
<tr>
<td>Triaxial Compression Test (air dried)</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>(saturated)</td>
</tr>
<tr>
<td>Brasilian Test (air dried)</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>(saturated)</td>
</tr>
<tr>
<td>Slake Durability Test</td>
<td>9</td>
</tr>
<tr>
<td>Tilt Test</td>
<td>23</td>
</tr>
<tr>
<td>Thin section analysis</td>
<td>9</td>
</tr>
</tbody>
</table>

However, due to the lack of a servo-controlled test system, it was possible to use only the stress-control in executing the multiple failure state triaxial tests. This procedure is justified because the consolidated material is weak and non-brittle. In addition the compression test machine used is relatively very stiff with respect to the samples tested. In fact, as shown in Fig. 3 post-peak stress -vs- strain curves were obtained for this type of material using the same test machine.

The permeability tests were conducted following the technique and procedure used for testing the reconstituted, but unconsolidated, materials, see Sec. 3.1. All other types of tests were standard tests, which are relatively straightforward.

---

Table 9. Summary of injection test results

<table>
<thead>
<tr>
<th>Column to be injected</th>
<th>Injection Results</th>
<th>Weight of mix injected (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material type</td>
<td>Mix type</td>
<td>Penetration length (cm)</td>
</tr>
<tr>
<td>Material weight (g)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>-------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>A 3250</td>
<td>M1</td>
<td>100</td>
</tr>
<tr>
<td>B 3100</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>C 3230</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>A 3200</td>
<td>M2</td>
<td>100</td>
</tr>
<tr>
<td>B 3070</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>C 3280</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>A 3230</td>
<td>M3</td>
<td>90</td>
</tr>
<tr>
<td>B 3070</td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>C 3400</td>
<td></td>
<td>90</td>
</tr>
</tbody>
</table>

* Case of incomplete penetration
tests are summarized in Table 9. Clearly, injections with M1 and M2 mixes for all three types of materials are all successful tests, whereas the three tests associated with the M3 mix all ended in incomplete penetration.

4.2 Results of Phase 1 and Phase 3 tests

The main results of these two phases are summarized in Tables 10, 11 and 12 for material Types, A, B, and C, respectively. In tables 10-12, the subscripts have the following meanings:

- d - air-dried condition
- s - saturated condition
- p - peak strength
- u - unconfined compression
- t - triaxial compression

The measured average values of unit weight and unconfined compressive strength are 12.7, 12.0, 12.2, and 5.8, 8.6, 3.8 for the mixes M1, M2 and M3, respectively.

Fig. 3 presents the results of the 4 unconfined compression tests on the specimens from the column sample B1.

**Table 10. Summary of Phase 1 and Phase 3 test results for Type A material**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Before injection</th>
<th>After injection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M1</td>
<td>M2</td>
</tr>
<tr>
<td>$\gamma_d$ (kN/m$^3$)</td>
<td>17.4</td>
<td>19.9</td>
</tr>
<tr>
<td>$\gamma_s$ (kN/m$^3$)</td>
<td>20.7</td>
<td>21.9</td>
</tr>
<tr>
<td>$\omega_b$ (%)</td>
<td>21.3</td>
<td>7.4</td>
</tr>
<tr>
<td>K (m/s)</td>
<td>1.4 $\times 10^5$</td>
<td>2.7 $\times 10^8$</td>
</tr>
<tr>
<td>$c_p$ (MPa)</td>
<td>0.9</td>
<td>2.4</td>
</tr>
<tr>
<td>$\phi_p$ (°)</td>
<td>36-39</td>
<td>38.5</td>
</tr>
<tr>
<td>$E_a$ (MPa)</td>
<td>770</td>
<td>2880</td>
</tr>
<tr>
<td>$E_t$ (MPa)</td>
<td>&lt;25</td>
<td>1220</td>
</tr>
<tr>
<td>$C_{tot}$ (MPa)</td>
<td>5.4</td>
<td>11.6</td>
</tr>
<tr>
<td>$C_{c}$ (MPa)</td>
<td>4.8</td>
<td>5.0</td>
</tr>
<tr>
<td>$T_o$ (MPa)</td>
<td>0.5</td>
<td>1.1</td>
</tr>
</tbody>
</table>

**Table 11. Summary of Phase 1 and Phase 3 test results for Type B material**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Before injection</th>
<th>After injection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M1</td>
<td>M2</td>
</tr>
<tr>
<td>$\gamma_d$ (kN/m$^3$)</td>
<td>16.0</td>
<td>18.6</td>
</tr>
<tr>
<td>$\gamma_s$ (kN/m$^3$)</td>
<td>19.3</td>
<td>21.0</td>
</tr>
<tr>
<td>$\omega_b$ (%)</td>
<td>25.2</td>
<td>7.6</td>
</tr>
<tr>
<td>K (m/s)</td>
<td>3 $\times 10^5$</td>
<td>1.3 $\times 10^8$</td>
</tr>
<tr>
<td>$c_p$ (MPa)</td>
<td>0-0.04</td>
<td>1.2</td>
</tr>
<tr>
<td>$\phi_p$ (°)</td>
<td>37</td>
<td>35</td>
</tr>
<tr>
<td>$E_a$ (MPa)</td>
<td>600</td>
<td>4320</td>
</tr>
<tr>
<td>$E_t$ (MPa)</td>
<td>30</td>
<td>990</td>
</tr>
<tr>
<td>$C_{tot}$ (MPa)</td>
<td>3.5</td>
<td>13.3</td>
</tr>
<tr>
<td>$C_{c}$ (MPa)</td>
<td>1.2</td>
<td>8.2</td>
</tr>
<tr>
<td>$T_o$ (MPa)</td>
<td>0.3</td>
<td>1.1</td>
</tr>
</tbody>
</table>
Table 12. Summary of Phase 1 and Phase 3 test results for Type C material

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Before injection</th>
<th>After injection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M1</td>
<td>M2</td>
</tr>
<tr>
<td></td>
<td>min.</td>
<td>max.</td>
</tr>
<tr>
<td>(\gamma_t) (kN/m²)</td>
<td>17.7</td>
<td>19.5</td>
</tr>
<tr>
<td>(\gamma) (kN/m²)</td>
<td>20.6</td>
<td>22.1</td>
</tr>
<tr>
<td>(\omega_\theta) (%)</td>
<td>18.6</td>
<td>4.8</td>
</tr>
<tr>
<td>(K) (m/s)</td>
<td>1.6 \times 10^3</td>
<td>3.9 \times 10^7</td>
</tr>
<tr>
<td>(c_p) (MPa)</td>
<td>0 – 0.02</td>
<td>0.9</td>
</tr>
<tr>
<td>(\phi_p) (°)</td>
<td>37 – 42</td>
<td>37</td>
</tr>
<tr>
<td>(E_a) (MPa)</td>
<td>240</td>
<td>1250</td>
</tr>
<tr>
<td>(E_t) (MPa)</td>
<td>510</td>
<td>960</td>
</tr>
<tr>
<td>(C_{\text{inl}}) (MPa)</td>
<td>3.2</td>
<td>4.8</td>
</tr>
<tr>
<td>(C_{\omega}) (MPa)</td>
<td>1.0</td>
<td>4.5</td>
</tr>
<tr>
<td>(T_o) (MPa)</td>
<td>0.1</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Fig. 4 presents a typical stress -vs- strain relationship of the multiple failure state traxial tests, and Fig. 5 shows a typical plot of the Mohr circles obtained from testing a single specimen. These results demonstrate that the triaxial testing procedure adapted for this research was satisfactory.

Finally, the relevant results obtained from thin section analysis of the reconstituted materials improved by injection are shown in Table 12, where the porosity data for each material type is the average value obtained from Phase 1 tests.

Fig. 3 The axial stress versus strain relationships of the 4 specimens from Sample B1

Table 13. Results of thin section analysis

<table>
<thead>
<tr>
<th>Material type</th>
<th>Parameters</th>
<th>Type of injection mix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Porosity (%)</td>
<td>33.5</td>
</tr>
<tr>
<td>A</td>
<td>Cement (%)</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>Bubbles (%)</td>
<td>&lt;1</td>
</tr>
<tr>
<td>B</td>
<td>Porosity (%)</td>
<td>39.6</td>
</tr>
<tr>
<td></td>
<td>Cement (%)</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Bubbles (%)</td>
<td>3–5</td>
</tr>
<tr>
<td>C</td>
<td>Porosity (%)</td>
<td>33.8</td>
</tr>
<tr>
<td></td>
<td>Cement (%)</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Bubbles (%)</td>
<td>5</td>
</tr>
</tbody>
</table>
5. ANALYSIS AND DISCUSSION OF RESULTS

5.1 Analysis of Phase 2 test results

Considering the results of the injection tests presented previously in Table 9, the following observations and comments can be made:

1) Reducing the cement content, such as the M3 mix with a water-to-cement ratio equal to 2.5, can give rise to the problem of pressure filtration and can lead to interruption of the penetration process. Due to separation of cement from water which tends to block the spaces between grains such that only water can penetrates, resulting in incomplete penetration.

2) Comparisons of the respective characteristics injection pressures for M1 and M2 show that, by increasing the cement content in the mix such as the M2 mix with a water cement ratio of 1.5, it is possible to inject each cylindrical column completely but it is necessary to raise the injection pressure, even to the limiting value (15 bars) of the test system.

3) Consequently, among all mixes tested, the M1 mix has the highest penetration capacity.

4) Evidently, to have good injection test results using microfine-cement based mixes, the water-to-cement ratio needs to be limited in the range of 2 to 1.5 in order to avoid the problem of pressure filtration and to keep the injection pressure low.

5) For all three types of materials treated, the amount of mix injected vary between 19 and 24% of the weight of the material to be treated, which is in agreement with the results of the petrographic analysis on thin sections shown in Table 13.

6) For any mix used, all the characteristic injection pressures related to Type B material...
are always higher than the corresponding values for the other two types of materials. This result that, in terms of injectability, Type B material is the most difficult, which is expected from comparing the particle size distribution curves of all 3 materials shown in Fig. 1.

Furthermore, the results of the thin section analysis (Table 13) show that the percentage of cement does not equal the total porosity because of the presence of bubbles and isolated pores. Also, the percentage of bubbles in Type C material is particularly high for all 3 mixes used. Apparently, this type of material contains more isolated or grout-impregnable voids due to its particle size distribution. On the other hand, by studying the injection test records (Table 9), it is noted that, for any of the 3 injection mixes used, the second step and the maximum injection pressures applied to Type C material are always the lowest of the three types of materials. This fact implies that the injection test procedure may be inadequate from the quality control point of view. Clearly, air-bubbles need to be expelled out of the system or forced into solution.

Nevertheless, the laboratory injection tests have demonstrated the feasibility of executing the ground improvement works for Types A, B, and C materials using microfine-cement based mixes.

5.2 Analysis of Phase 1 and Phase 3 test results

By examining carefully the results summarized in Tables 10, 11, and 12 the following observations can be made:

1) All the consolidated columns have demonstrated very clearly that the mechanical and hydraulic characteristics of the grouted material deteriorate gradually away from the point of injection, for example, see Fig. 3.

2) The unit weight of the injected materials increased with respect to that of the corresponding loose, reconstituted materials due to infilling of voids by the mix which caused a clear reduction (>50%) in the water absorption capacity of the treated materials.

3) The permeability coefficient (K) is also significantly reduced such that for the series of column samples A1, A2 and B1, B2, K is of the
order of 10E-8 m/s and meets the requirements of the design. However, for Type C material, the reduction is less significant due to the presence of air bubbles (cf. Table 13).

4) Shear tests on unconsolidated materials showed in some cases a small apparent cohesion, which is generally not justified in the case of loose materials and which is due to the use of the linear Mohr-Coulomb criterion. Nevertheless, such apparent cohesion values fall within the range of the values derived from in-situ tests.

5) On the other hand, all of the consolidated materials exhibited an effective cohesion whose value is about 1.0MPa, which is twice the value needed for the project design.

6) The friction angle of the consolidated materials, on the average, is also slightly increased except for Type C, which consists of relatively coarse particles and shows a reduction of dilation due to the introduction of microfine cements into its matrix.

7) For Types A and B materials, the design value of friction angle is met by injection of both M1 and M2 mixes.

8) The direct shear test results also confirm that, at the same level of normal stress, there is an increase in the shear resistance even when the cohesion of the consolidated material is exceeded.

9) The Young's modulus (E) of all samples treated with M1 and M2 mixes is always higher than the value supposed in Table 1 for the project design.

10) Under the conditions of lateral confinement (triaxial compression), it is possible to assess the improvement on the deformation modulus of the reconstituted materials before and after injection: at the same level of confining pressure (0.5MPa) E is increased by more than 10 times even for the relatively poor material, with the maximum increase being of the order of 100.

11) The unconfined compressive strength (Co) and tensile strength (To) are strongly dependent on the type of mix injected, being the highest for Type A material consolidated by M2 mix. The ratio of Co to To is on the average about 10, which falls within the range of its value for rock.

12) For each mix used, the value of Co for those specimens close to the injection point is noticeably higher than the average value obtained from the specimens of the cured, pure mix, confirming the positive effects of injection using microcement-based mixes for loose, sandy materials.

6. CONCLUSIONS

Based on the analyses and discussions presented in the preceding section, the following major conclusions can be drawn:

1) The TREVICEM 0/3 type of microfine cement is suitable for the project.

2) The optimum water-to-cement ratio of the corresponding injection mix should be in the range of 1.5 to 2.0, to ensure good results.

3) Improvement of Types A and B materials using M1 and M2 mixes can fulfil the specifications of the project design.

4) For Type C material, the improvement with all 3 mixes is less satisfactory, particularly in terms of the reduction in permeability. Thus, if strictly necessary, complementary injection of a non-toxic but extremely expensive chemical solution will have to be employed.

7. ACKNOWLEDGEMENT

The authors would like to thank “Consorzio ENI per lo Stretto di Messina” for allowing us to publish the data presented in this paper.

8. REFERENCES


Laboratory testing of deformability and seismo-acoustic properties of rocks

Essais en laboratoire de propriétés de déformabilité et séismo-acoustiques de roches

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Faculty of Mining and Geology, Belgrade, Yugoslavia

ABSTRACT: Within the framework of a wider program on investigation among some rock properties numerous types of rocks differ in physico-mechanical properties have been tested in the Laboratory for Rock Mechanics, Mining Division at the Faculty of Mining and Geology in Belgrade. Our attention was paid to the relationship between deformability and seismo-acoustic emission during the standard unconfined compression test. Particularly interesting and characteristic is the testing of salt rock. The samples were investigated according to two different testing programs. In the first series of tests, the samples of salt rock, the uniaxial compression strength (UCS) of which had previously been determined with sufficient reliability, were exposed to repeated loading and unloading cycles. The maximum load in the cycle was limited to about 70% of UCS. During testing, axial deformations and velocities of longitudinal elastic waves were measured, as well as the emission of seismo-acoustic signals. According to the second program of testing, the samples were subjected to gradual loading till exhaustion. Axial deformations and seismo-acoustic emission were monitored during this test, too. Results of all the above mentioned investigations are given in the form of tables and graphs. It can be concluded from the results that the transition point between elasto-plastic and fully plastic state of a specimen can be predicted in the function of stress-deformation and stress acoustic emission relationships.

RÉSUMÉ: Ont été faites simultanément les recherches de la déformabilité et des propriétés séismo-acoustiques des roches de diverses caractéristiques physico-mécaniques en Laboratoire de la Mécanique des roches du Département des Mines à la Faculté des Mines et de la Géologie à Belgrade. Les résultats obtenus par des essais faits sur les échantillons du sel gemme sont particulièrement caractéristiques. Les échantillons des roches dont l’axe de résistance à la pression est identifié, sont exposés aux charges et au décharge multiples, allant jusqu’à la charge la plus élevée qui ne dépasse pas 70% de l’axe de résistance. Lors des essais de charge et décharge sont mesurées les déformations verticales, puis la vitesse du passage des ondes élastiques longitudinales directement à la direction de la charge. En même temps est fait la registration de l’émission des impulsions séismo-acoustiques d’un échantillon de la roche. D’après un autre programme les échantillons du sel gemme sont exposé progressivement à la charge jusqu’au broyage. Dans ce cas sont suivies la déformation verticale et aussi l’émission des signaux séismo-acoustiques. Les résultats de toutes ces recherches sont présentés graphiquement et dans les tableaux. Après avoir fait l’analyse de ces résultats, on avait constaté en outre que le passage de l’état d’élasticité à l’état plastique pourrait être prévu à partir du rapport entre la contrainte-déformation et l’émission des ondes séismo-acoustiques.

1 INTRODUCTION

In the process of rock fracture, during formation of cracks, elastic variations that can be registered (detected) by apparatus, occurred. Accordingly, elastic waves are characterized by different frequencies. Mostly, the frequency of elastic waves registered during earthquakes does not exceed 10 Hz. The frequency of elastic waves occurring during rock crack down and fracture and rocks fracture "in situ" ranged from 100 to 1000 Hz value orders, while impulses detected during crack down of rock samples in laboratory are of far greater frequency.

Since recently, ever increasing attention has been paid to this field of science. It was proved that at the very moment of fracture of investigated sample
in the press, both frequency and energy of incited impulses increased.

In Russia, Vinogradov was a scientist who was deeply engaged in this problem and in Czechskivakin, J. Buben intensively studied the problems connected with this field. Apart from the mentioned authors, Broun, Gold, Godman, Mason, Rotter, Schofield, Suzuki and others contributed greatly to development of this scientific field.

Seismo-acoustic effects have been widely applied when investigating the samples and occurrences connected with rock bursts, stability of slopes at open pits and stability of underground facilities, particularly with interchannel columns. If number of impulses per a time unite at the boundary state of strength is known for a certain material, the time when interchannel column, a face or a slope will crack down or slide, can be determined with certain preciseness.

A critical value of the number of impulses per unit of time for a certain rock material can be determined in two manners:

1. by laboratory testing of rock samples on conditions that they are equal or almost equal to those "in situ",

2. on the basis of data collected within the same rock material "in situ" on condition that monitoring covers the complete process of change in stress state till the moment of collapse.

Having in mind all up-to-date investigation results obtained in this field, the aim of this paper was to confirm literature data through experiments on rock salt. The rock salt, regarding to its structuro-textural properties, has proved to be very interesting for this type of researches. If, apart from that, specific ways of rock salt exploitation by chamber methods imposing the problem of dimensioning both the chamber and columns, are taken into consideration, this way of study becomes even more justified and indispensable.

2 APPARATUS

With the aim to perform research work of the properties of rock salt under laboratory conditions, a particular apparatus, that completely fulfilled all the main conditions for this type of study, has been designed. On the basis of literature data it can be concluded that each and every author, engaged in this study, approached the problem of apparatus, according to his own idea and possibilities regarding to instruments. Different approaches to the same problem, resulted in creation of several types of apparatus for investigation of seismo-acoustic properties of rocks.

As each transformation of energy is followed by indispensable losses, then the effort for the measuring values to be in the first functional dependence and consequently be free from such defects, is reasonable. The essential conditions that have to be fulfilled for such a measuring apparatus, are the following:

- threshold of sensitivity to be sufficient for the acceptance of nothing but seismo-acoustic effects originating from the rock under investigation, by the selected receptor,
- selective intensifier of this signal to reach the level that is necessary for excitation of the system for counting, namely for measurement of frequency,
- the existence of suitable transformation unit for conversion of accepted discrete signals, in their original form, into impulses of the same shape,
- insertionless impulse counter,
- apparatus for registration that can reproduce the registered data as much as necessary and study the same on various bases,
- mechanical part of an apparatus for investigation should operate without any noise and should correctly register the real loading values. Hydraulic press should also produce neither noise nor sputter (fig. 1).

Figure 1. Schematic presentation of equipment for recording seismic-acoustic impulse emission during loading of the rock sample*

*UP - oil pump; HP - hydraulic press; UZ - rock specimen; PO - recording instrument; PP - pre-intensifier; P - intensifier; F - filter; DB - d-counter; M - tape recorder; OS - oscilloscope

The upper fixed plate of a press, against which the upper surface of investigated sample leans, has a hole in its central part for mounting of the acoustic sensor that should be symmetrically placed according to the place and size of investigated sample. The
thickness of the upper supporting plate can muffle the excited seismoacoustic impulses, what should certainly be taken into consideration. The guides of hydraulic press can also be used as a carrier for horizontally placed strain gauges, while vertically placed strain gauges are fixed to the upper supporting plate. Due to vertical and horizontal strain gauges, deformations of investigated sample in function of loading have been monitored, i.e. together with collection of acoustic impulses. The hydraulic press, designed by the author of this paper, is given in figure 2.

Figure 2. Hydraulic press*

*(1)steel plate; (2)specimen under examination; (3)vertically placed strain gauges; (4)guides; (5)pump pedestal

All the changes in sample volume cause acoustic impulses that are transformed into electric signals and are forwarded by an appropriate electric cable to the following stage in apparatus. In these experiments a ready-made sensor of KD2 type, production of Metra-Dresden, was used. It represents a piezoelectric donor having a total mass of 190 g. It can easily be mounted immediately on the upper supporting plate of hydraulic press. Its electric impedance is of capacity character of approximately. Within the frequency area from 6 Hz to 5 kHz its favourable property is defined. Its resonance frequency exceeds 15 KHz, thus avoiding this characteristic to influence infavourably on the result of measurement.

Preparation of signals coming from a piezocrystal, is done in preintensifier PP designed as a two-stage intensifier with HP transistors. As a steel plate is highly ohm, thus a co-axial electric write represents a link with a pre-intensifier. The pre-intensifier was temperature stabilized by the PNP transistors. It is supplied from dry batteries of 6 V voltage. The outcome impedance is adopted by transformers to the input into a linear intensifier of counter. The intensifier has its own step-like regulation of amplification and the complete experiment is performed within a cycle with counting of impulses during sample deformation. The intensifier is characterized by a stable electric supply through the implemented voltage stabilizer. The counter is mounted within the same block together with intensifier as a ring-counter, capacity of 0-999 999 what, in any case, satisfies the need of measurement.

During emission of seismo-acoustic signals, a spectrum of frequencies representing complex periodical functions of time, appeared. In order to determine characteristics \( n = f(\sigma) \), where \( n \)- number of impulses in time unit, and \( \sigma \)- stress, the filters were implemented. The filters enabled the impulses of a certainly determined frequency to be led to the counter. In spite of this, the property of the function is not altered as is evident from experiment. Thus, on the basis of one experiment, the characteristics corresponding to single frequencies can be confirmed and compared at the same diagram. The shape of separate impulses can be taken as photos when a signal from a type recorder is led to the entrance of oscilloscope. An electric photo, i.e. a record on the screen of oscilloscope corresponds to each seismoacoustic impulse (of sliding or fracture of elementary volume of investigated sample). It was also noticed that: these impulses appeared, as a rule, in a column or in "package" of signals, what means that mechanical deformation occurs discontinually with small time periods.

SAMPLES PREPARATION AND STUDY

One of the main conditions, the sample of rock under investigation should fulfill, understands its precise treatment. Namely, the condition of plan-parallel basis and that of perpendicularity of a vertical axis to bases, have to be fulfilled. Experiments were performed on the samples of rock salt, cylindric in shape, that were obtained "in situ". Dimensions of investigated samples were:
h = 50 mm and d = 50 mm, i.e. the ratio was h/a = 1.0.

During research work, the sample was exposed to attentive loading, strictly respecting the loading excess to be \( V = 15 \text{ daNm}^2 \text{m}^{-2} \). In the course of investigation, the increment in loading was followed by reading of a number of impulses every 60 sec. and a formed deformation was followed along the sample's height.

The whole spectrum of impulses, registered at the first research sample, for the value of loading till the fracture of sample, was divided, according to the height of amplitude, into six classes. According to such accepted criterion, the impulses on other samples were classified, as well accordingly, this means that number of impulses was both written each 60 seconds for every obtained loading and classified into classes according to the height of amplitude. In the last vertical column in the register, the total number of impulses was written. At the same time, deformation of samples per height was monitored.

Owing to this procedure, a great number of rock salt samples was studied. For example, Table 1 presents the collected data for sample U - 7. Graphic version of this example is given in Fig. 3.

Table 1.

<table>
<thead>
<tr>
<th>Ordinal number of measurement</th>
<th>Stress (daNm)</th>
<th>Time (sec)</th>
<th>Deformation (mm)</th>
<th>Frequency of impulses</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>20</td>
<td>1</td>
<td>0.120</td>
<td>6</td>
</tr>
<tr>
<td>2.</td>
<td>40</td>
<td>2</td>
<td>0.210</td>
<td>29</td>
</tr>
<tr>
<td>3.</td>
<td>60</td>
<td>3</td>
<td>0.675</td>
<td>64</td>
</tr>
<tr>
<td>4.</td>
<td>80</td>
<td>4</td>
<td>0.950</td>
<td>64</td>
</tr>
<tr>
<td>5.</td>
<td>100</td>
<td>5</td>
<td>0.725</td>
<td>79</td>
</tr>
<tr>
<td>6.</td>
<td>120</td>
<td>6</td>
<td>0.850</td>
<td>95</td>
</tr>
<tr>
<td>7.</td>
<td>140</td>
<td>7</td>
<td>1.050</td>
<td>112</td>
</tr>
<tr>
<td>8.</td>
<td>160</td>
<td>8</td>
<td>1.130</td>
<td>144</td>
</tr>
<tr>
<td>9.</td>
<td>180</td>
<td>9</td>
<td>1.325</td>
<td>188</td>
</tr>
<tr>
<td>10.</td>
<td>200</td>
<td>10</td>
<td>1.520</td>
<td>200</td>
</tr>
<tr>
<td>11.</td>
<td>220</td>
<td>11</td>
<td>1.780</td>
<td>300</td>
</tr>
<tr>
<td>12.</td>
<td>240</td>
<td>12</td>
<td>2.150</td>
<td>328</td>
</tr>
<tr>
<td>13.</td>
<td>252</td>
<td>13</td>
<td>2.700</td>
<td>403</td>
</tr>
<tr>
<td>14.</td>
<td>260</td>
<td>14</td>
<td>4.200</td>
<td>475</td>
</tr>
<tr>
<td>15.</td>
<td>fracture</td>
<td>15</td>
<td>6.600</td>
<td>530</td>
</tr>
</tbody>
</table>

\( \Sigma = 6600 \) 330 138 302 134 9 4602

**DATA ANALYSIS AND CONCLUSION**

When considering the diagram given in Fig. 4, as a final one for all investigated samples, the following conclusions can be made:

- diagram of deformation shows the course of elastoplastic and plastic deformations of salt samples,
- graphic presentation of seismic-acoustic impulses emission monitors the occurrence and development of sample deformation, first and foremost, in the part of the loading process, where elastic and elastoplastic deformation is defined; the graph representing the number of impulses is given by a straight line. However, any further increase in load causes abrupt increment in impulses of acoustic signal what is presented by a fracture at the graph. Inflection points in diagram of separate groups of impulses of the same amplitudes, have the mutual ascension, when the same sample is concerned, but are variously distributed along ordinate.
- it is evident, by analysing Fig. 4, that the diagram
of acoustic signals emission rapidly changes direction at the defined loading value (Point M). This point coincides with the point K at the diagram of deformation, that was not previously clearly defined.

The presented results of investigated samples of rock salt, refer to laboratory examinations, only. All the mentioned investigations, when located in the "in situ" conditions, would represent a source of even more thorough data. The very same investigations performed at the specimen of other rock will particularly contribute to confirmation of the above mentioned results.

Figure 4. A mutual diagram for frequence of acoustic signals and deformations for all salt samples

Consequently, the inflection point at the diagram of acoustic impulses shows, i.e. marks the load under which the change in materials quality occurs, what otherwise can hardly be noticed at the diagram of deformation.

It was established during experiment that emission of seismo-acoustic impulses does not exist when unloading is concerned. Only when in this new loading process the previous maximum of loading is reached, the emission of seismo-acoustic impulses will start again.

For the purpose of easier and better analysis of these experimental researches at rock salt, Table 2 presents all characteristic indices that were collected during experiment.

Table 2.

<table>
<thead>
<tr>
<th>mark of sample</th>
<th>one-axial pressure strength (daNm^{-2})</th>
<th>stress of inflection point (daNm^{-2})</th>
<th>k_{inf}</th>
<th>ah inflection</th>
<th>angle of inflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>U10</td>
<td>300.0</td>
<td>196.0</td>
<td>55.3%</td>
<td>3.320</td>
<td>1.100</td>
</tr>
<tr>
<td>U10</td>
<td>500.0</td>
<td>196.0</td>
<td>55.3%</td>
<td>3.320</td>
<td>1.300</td>
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<tr>
<td>U10</td>
<td>700.0</td>
<td>184.0</td>
<td>51.1%</td>
<td>2.270</td>
<td>0.400</td>
</tr>
<tr>
<td>U 7</td>
<td>300.0</td>
<td>184.0</td>
<td>51.1%</td>
<td>5.000</td>
<td>0.500</td>
</tr>
<tr>
<td>U 7</td>
<td>700.0</td>
<td>184.0</td>
<td>51.1%</td>
<td>4.200</td>
<td>0.700</td>
</tr>
<tr>
<td>U15</td>
<td>450.0</td>
<td>192.0</td>
<td>51.1%</td>
<td>4.480</td>
<td>0.600</td>
</tr>
<tr>
<td>U15</td>
<td>700.0</td>
<td>192.0</td>
<td>51.1%</td>
<td>2.000</td>
<td>0.200</td>
</tr>
<tr>
<td>E</td>
<td>220.0</td>
<td>137.0</td>
<td>43.6%</td>
<td>35.835</td>
<td>9.800</td>
</tr>
<tr>
<td>S1</td>
<td>318.5</td>
<td>195.2</td>
<td>69.1%</td>
<td>3.686</td>
<td>1.400</td>
</tr>
</tbody>
</table>

As is evident from Table 2, the average value of one-axial pressure strength of investigated samples is Q_{P}=318.5 daNm^{-2}. The inflection point at diagram of emission of seismo-acoustic impulses appears under load of Q_{P}=196.2 daNm^{-2} what represents 62% of pressure strength. Deformation established at the moment of fracture is h=3.686 mm, while loading deformation of the inflection point is h_{P}=1.4 mm. The average value of inflection angle is \beta=35.3\degree.

REFERENCES


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Influence of physical and chemical processes occurring in eluvium of the opole marls and their influence on the geotechnical parameters

L'influence des processus physico-chimiques qui se produisent dans l'éluvion des marnes d'opole sur les paramètres géotechniques

Jan Jaremski
Technical University of Opole, Poland

ABSTRACT: The paper presents the results of mineralogical and physicochemical investigations carried out with diffractometry and X-ray fluorescence. Microstructural examinations were made with a scanning electron microscope. A new method for determination of plasticity and liquid limits as well as swelling has been proposed. On the basis of various investigations a model of physicochemical phenomena occurring in weathering has been proposed. This permits to estimate the extreme values of geotechnical parameters of weathering. The physicochemical processes occurring during the formation of deposits in a sediments basin have been analysed. It has been found that they influenced geotechnical properties which as a result of weathering processes undergo the continual changes. The author has observed that marl preserves the "genetic memory" of physicochemical processes and has found that these processes are renewed.

RESUME: L'article contient les résultats des recherches minéralogiques et physico-chimiques réalisées grâce à l'aide de diffractométrie et de fluorescence de rayons X. Les recherches microstructurales, on les a exécutées au microscope à électronique balayé, et aussi les recherches de limite plasticité et de limite de liquidité, ainsi que la recherche de gonflement selon les méthodes rédigées par l'auteur. Sur la base de recherches multimode on a proposé le modèle d'effets physico-chimiques qui se produisent dans l'éluvion qui a permis de faire l'évaluation des valeurs extrêmes de paramètres géotechniques d'éluvion. On a analysé aussi les processus physico-chimiques qui se produisent dans le temps de la formation des prédépôts dans le bassin sédimentaire. On a constaté que ces processus influencent les valeurs géotechniques, ainsi qu'à cause de processus d'éluvion ils sont soumis aux changements constants. L'auteur a remarqué que le marne garde la mémoire génétique des processus physico-chimiques, et il a constaté que ces processus sont renouvelables.

1 INTRODUCTION

Marls belong to the soils which are subjected to the continual structural changes under the influence of external factors. It is difficult to classify soils of this type and define their geotechnical properties because there is no methodology of their investigations. In this paper the author has applied the results of his investigations conducted for many years in Opole, a city in south-west Poland. In Opole the eluvium of Cretaceous marl is situated under the humus layer. Fig.1. Thickness of this eluvium oscillates from 4.5 m to 6 m. The ground water level varies between 10 m and 6.0 m or even 7.0 m below the ground. The marl eluvium is closely connected with the aeration zone determined by the amplitude of changes of ground water level. For many years it was assumed that such eluvium was non-carrying subsoil. The multifarious investigations on geotechnical properties of more eluvia in Opole were of a great importance and they could influence development of the city. Typical laboratory tests of strength characteristics gave negative results. However, the investigations "in situ" (soil-wedge extrusion test, loads test, pressure-meter test) permitted to clas-

Figure 1. The part of the wall built of the Opole marl eluvium

sify weathering as a building subsoil. The author presented the problems in his previous papers (4),(5),(6),(10),(11),(12), deformability tests which were carried out during various degrees of humidity.

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The author has collected a lot of data for many years which enabled him to formulate a trial of description of physical and chemical processes occurring in the weathering during extreme wetting. Also the fact the eluvium of the Opole marls during the drought time is a typical stone debris is of a great importance. However, the sustained periods of moisture preceded by the drought make the same debris like a typical cohesive soil.

This separate structural character causes properties of the Opole marls to oscillate between the values typical for the incoshesive soils and the cohesive one. Owing to that, the applied investigation methods can give different results which can be even unreliable. Such unfavourable results may even occur when the analyses discussed above have given an information on the character of the processes occurring in the rock complex i.e., in the deposit. It has been stated that the Opole marls are built mainly of calcium carbonate which forms a structure filled with the minerals from the illite group, entirely resistant to the external factors. The illite minerals cause, due to their swelling ability, the process of compression of the fissures and empty spaces between rock chips in the marl eluvium with an increase in the water content. The strongly mineralized ground water, in its turn, creates the process of recrystallization of calcium carbonate in the material filling the fissures. Due to such processes the soil moisture increases up to the cohesive soil at the extreme saturation stage. On the basis of the research "in situ" and the laboratory model testing the author states that the illuvial eluvium is a typical phenomenon with moisture decreases its compressive strength within the range which does not disqualify it as a building foundation but is only a diminishing of strengthening. That is contradictory to the registered surface soaking of marl areas which convinces many practicians-explorers and ensures them about minimum compressive strength of eluvium.

These processes occur in gaps and spaces between rock pieces. The gaps and spaces are filled with a disintegrated material to a more or less extent. Swelling is a phenomena playing an important role here. From investigations it appears that swelling can be found only under circumstances, depending on a degree of disintegration, an amount of accumulated heat and initial humidity. Swelling finishes when the humidity, registered "in situ" approaches its maximum. Such conditions are fulfilled in case of the material filling the gaps and spaces. The material swells and this phenomenon determines all the processes mentioned above. On the other hand, swelling, protects the eluvium against unfavourable effects of changes of underground water level. These changes are caused by precipitation or its lack. The processes lead to filling in gaps and empty spaces, and permit to bind greater rock blocks or rock pieces and do not allow the weathering to smite and crush. The state of weathering, in which processes leading to swelling and other ones do not occur, was simulated during laboratory tests. The analysis of swelling by means of the triaxial compression apparatus, adequately adapted, made possible to analyse the samples with the internal diameter of 130 and length of 260 mm, corresponding to condensed and damped ones. In order to grasp the nature of this change, samples were subjected to creep tests under humidity of 2, 6, 11 and 19 per cent (the maximum humidity making the realization of tests possible). These tests permitted transition from short plastic to plastic states to be registered, and with humidity 19 per cent the progressive creep was recorded [9]. Under the natural conditions the natural water content 24 per cent was registered in which the marl eluvium kept its strength.

The increase in humidity, under the natural conditions, which is secured by the aeration zone is the consequence of the processes in weathering only. The reduction in numerical values of geotechnical characteristics was observed. This did not lead to the loss of strength, which resulted from the swelling tests, devoid of the described phenomena. The long-lasting research of the author has resulted in an application of the marl eluvium as the safe base for the building objects founded in new area of Opole city.

2 GEOLOGICAL SETTING

The Cretaceous deposits of the Opole region lie in the north-western slope of the Upper Silesian Anticlinorium. The Opole city itself is localized in the central part of the Cretaceous formation on the discrete Cretaceous horst. The Opole deposits display organogenic texture and disordered structure. The rock consists of calcite and clay minerals, which form a cryptocrystalline to micritic matrix. The illuvial eluvium occurring below the waste zone is the more or less fissured rock with vertical, horizontal or oblique cracks and forms in general one stratigraphically homogenous horizon differentiated only due to the progress of weathering. A following crosssection may be described in the Opole deposits starting from the top. The waste of thickness of 1 to 3 m, occasionally up to 5 m, occurring beneath the soil consists of blocks and rock chips with weathered surface disorderly mixed with silt fraction, formed as a result of weathering, Fig.2.

![Figure 2. The part of the wall quarry of the Opole marls](image-url)
The clayish-silty part content of the waste corresponds to 5±15 per cent in comparison to the marl. The marl eluvium described has been underlaid with admixtures-free waste which displays a horizontal layer of smaller amounts of debris. The stratum discussed lies on the nearly horizontal fractured and weathered marls which occur in small thicknesses up to the depth of 50 m. The thickness of this layer lies about 3 to 5 below the present surface in the examined places.

The fissures and individual cracks partly filled with weathering clay disposed in general an irregular shape and varied orientation, which has led to rock disintegration into blocks and smaller fragments. The dimension of these cracks remains under the influence of an amplitude of changes of ground water level, temperature changes, swelling degree and an occurrence of the fissures of another type.

Spatial differentiation of the structure of the rock complexes is usually connected with the changeability of the physical properties. The structural feature describes a general character of the rock complex and a significant influence on infiltration rate of liquids and gases and permeability of this complex as well.

3 HYDROGEOLOGICAL CONDITIONS AND CHEMICAL COMPOSITION OF WATER FILTERING IN MARLS

In the considered area 3 water-bearing levels occur at the free surface of water in the Turonian marls, the second one as subartesian in the Cenoman sands and the third one occurs in the Trias limestone. While conducting other studies the author found that there was a contact between particular water-bearing layers through comparatively numerous faults which was confirmed by the comparison of chemical composition of all types of water [4],[6]. The great permeability of faults exerts an influence on the phenomena occurring in weathering. Because in this area there are neither water-courses nor land-betterment ditches and the orderate of the level in the Turonian approximates to the ordinate of the Cenoman water, hence the retention of marls is limited and the variations of the underground water level occur in a comparatively short period of time. After the disappearance of rainfalls there occurs particularly fast drawdown of underground water in marls from 1 m to 6 m below the ground level which the author registered in the testing ground many times. This testifies the great conductance of the Turonian marls and what with little seepage points unfavourably the great conductance of faults and exerts a considerable influence on the composition of water in the marl eluvium.

The chemical content of the water has also been determined, concerning the water as a chemical solution which takes part in the processes within the marl complex. The water infiltrating the marls and sampled in the earth-works displays neutral character (pH equal to 7.2±7.5 decreasing after long-lasting rainfalls). Much attention is paid to large general hardness. The general hardness of water changes from 10 mval (260n) to 16.4 mval (460n) whereas that of carbonates varies from 4.2 mval (120n) to 6 mval (160n). High contents of chlorides reaching 67 mg/dm³ of sulphates up to 285 mg/dm³ give an evidence for high salinity of the waters under examination. The content of sulphates decreases, however, even to 33 mg/dm³ in periods of long-lasting precipitation. The content of CaO oscillates between 70 mg/dm³ and 160 mg/dm³, exceeding in general the value of 150 mg/dm³. The results given above were compared with the general composition of the water sampled in the 32 m deep well. This water displays pH about 7.0 hardness from 260n to 300n, carbonate hardness equal to 160n, content of chlorites up to 64 mg/dm³, of sulphates from 248 to 285 mg/dm³ and of calcium from 176 mg/dm³ to 186 mg/dm³. The waters from the Cenoman and Trias levels are characterized by low content of chlorides from 5±20 mg/dm³ and sulphates 17.5±125 mg/dm³, the general hardness varies from 2.7±76 mg/dm³. To sum up the results obtained, it can be stated that waters infiltrating the marls are strongly mineralized whereas their mineral content changes due to precipitation [7]. The specific chemism of the waters infiltrating the marls influences the processes occurring in the rock complex. The water represents a dissolving medium. Flowing through the cracks and fissures of the complex it may be after saturation also a source of coagulation and sedimentation of the transported compounds there, as well as it may generate or take part in process of recrystallization within the cracks.

4 SOME PHYSICOCHEMICAL ASPECTS

The marl eluvium is usually concerned as a granulated, non-cohesive medium. That is why it is treated as debris.

However, in the Opole region it has been observed, that in some years this medium imitates a cohesive soil, the fact which can be concerned as a change of the rocks character from the stage of the very good permeability to the very bad one. This fact is accompanied with an increased level of the ground water which is a direct cause of a partial submergence of building objects and the very high water level in earth-wor, level.

The facts, observed in nature, suggest the following questions: - what is the cause of natural cohesion in noncohesive medium? - What types of processes are responsible for such an insignificant water flow occurring in the waste? Could these processes be identified with those occurring in the cracks and fissures? As far as the first problem is concerned, the occurrence of solidification of the disintegrated marl has been observed for many years in the laboratory for the artificially prepared marl samples saturated and dried during the experiment.

4.1 Chemical Composition

The chemical composition of the marls is defined for the samples taken from the shearing zones of soil by the soil-wedge extrusion method and also from the boreholes located in different regions. The samples were taken every 1 m depth of test bore-holes for pressimmetrical test. The samples after required preparation were investigated in X-ray fluorescence analyser EN 1250 Philips. The results are presented in Table No.1.
Table 1. Some results of X-ray fluorescence tests

<table>
<thead>
<tr>
<th>Bore-hole</th>
<th>d (m)</th>
<th>CaO</th>
<th>Al₂O₃</th>
<th>SiO₂</th>
<th>MgO</th>
<th>Fe₂O₃</th>
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<tr>
<td>The soil-wedge-extraction method</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Shoring zone</td>
<td>No.1</td>
<td>47.64</td>
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<td>4.80</td>
<td>0.33</td>
<td>0.88</td>
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<tr>
<td>No.2</td>
<td>48.65</td>
<td>1.03</td>
<td>4.20</td>
<td>0.31</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>No.3</td>
<td>47.80</td>
<td>1.80</td>
<td>5.10</td>
<td>0.33</td>
<td>0.75</td>
<td></td>
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<tr>
<td>The proximometrical tests</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>1</td>
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<td>1.51</td>
<td>7.00</td>
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<td>0.85</td>
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<td></td>
<td>2</td>
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<td>1.05</td>
<td>5.40</td>
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<td>0.65</td>
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<tr>
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<td>1.18</td>
<td>8.20</td>
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<td>0.87</td>
</tr>
</tbody>
</table>

The obtained results point out small differences in per centage of particular components, appointed on samples coming from different places. For checking purposes the obtained results were analysed comparatively with the research ones of raw material prepared for technology cement plant needs. Then the obtained results were compared with the research results of cores of test bore-holes which define particular deposit of cement plant quarry. The results of investigations, which had been previously carried out for the engineering-geological documentation of building industry, were taken into account too. The investigated analyses showed comparatively small difference in the composition within the same depositing stratum in different places in the town and only several per cent (proportional) differences in profile in the range of the investigated depth.

4.2 Mineralogical Composition

Mineralogical composition was defined by means of an X-ray diffractometer DRON2. Samples subjected to these investigations, were taken from testing ground on which investigations "in situ" were carried out they were of determined chemical composition. The first diffraction analyses showed the limitations of this method in the studies of mineralogical composition of marl. It turned out that calcite brings about the absence of distinct and characteristic reflections of clay minerals from the illite group since the lengths of calcite peaks approximated to the lengths of clay mineral peaks. The calcite was removed by means of 0.2 per cent disodium versenate which allowed to take calcite away quantitatively without any structural changes in clay minerals. The reflections of illite were identified on all the samples being deprived of calcite, Fig.3. This identification was also proved by potassium content amounting to 6 per cent, which was determined by means of a fluorescent analysis. As a result of these studies it has been found that Opole marls consist, first and foremost, of calcite as well as illite and quartz. Cristobalite and other minerals were also registered. Those results were compared with mineralogical and chemical composition of Keuper Marl [1],[2] and partly other marls [13]. The presence of illite explains specific behaviour of marls under the influence of humidity, when the eluvium becomes similar to cohesive soil and swelling plays an important role. Mineralogical and chemical composition of marl eluvium, from different places in the city was tested with the X-ray diffractometer. X-ray fluorescence and the scanning electron microscope. The results of these tests were compared with results of analysis of materials used by cement industry; mineralogical and chemical compositions of these two groups of materials proved to be similar. This statement is very important for another paper by the author, because it allows to treat the investigations "in situ" together, since they refer to the sample deposit.
4.3 Plasticity and Liquidity Limits of the Marl Eluvium

The marl eluvium under conditions "in situ" is a disintegrated medium mainly consisting of rock chips and filling, however it has been noticed, due to the long-lasting rainfalls preceded by dry periods, that the medium becomes similar to cohesive soil. The knowledge of these features enables to forecast the behaviour of the medium under extreme conditions. The determination of the changes of consistency was very important, however, the trial of the definition for disintegrated rock seemed to be absurd. Generally it was admitted to be impossible, therefore the suitable methodology was worked out. Many numerical values of plasticity and liquidity limits were established by making several combinations on samples of different disintegration and dampness. In the method worked out by the author the collected material was subjected to preliminary drying as long as it could be sieved by a sieve with the diameter of meshes < 0.75 mm. Such prepared marl, as mentioned above, was subjected to dampness and mining as long as it became mineable and was similar to cohesive soil. A part of the prepared material was dried and the hardening process was observed. Hardening led to petrification and the tested samples became similar to solid marl. The remaining part of the material was subjected to standard tests of the yield point and the liquid limit. The following results were obtained: the plasticity index was from 22.7 to 24.4 per cent and the liquid limit was from 32.9 to 43.3 per cent.

4.4 Swelling

The phenomenon of swelling under conditions "in situ" was observed and it was the result of the presence of clay minerals of the illite group, found in mineralogical tests, which showed the ability to swelling under certain conditions. To accept such a statement as the right one it should be proved that the phenomenon of swelling occurs in the marl eluvium. However, from the data collected from the source materials, being the result of the investigations carried out for the needs of geological and engineering documentation, and from the laboratory tests made by other researchers it appeared that in Opole marl eluvium the swelling had not been registered at all. The author of this paper also tested swelling in Opole marl eluvium; the tests were carried out according to the accepted standards and methods given in literature and swelling was not observed. Thus, the problem became complicated because it is not possible to explain some processes occurring in the deposit (changes in fissility being a characteristic property of the considered medium, reconstruction of structural bonds, intensive weathering, reconstruction of cohesion) if swelling is not present.

The author decided to check out experimentally the phenomenon on the samples taken directly from marl. A new method for swelling the author discussed in the paper [7], [8]. Such procedure was applied for the Opole marl samples, taken from different places. The swelling process was registered and the maximum swelling observed was 10 per cent. The swelling action is the subject matter of another paper in which the author deals with the research of soil swelling and also the problems connected with the foundation on clay soil with the so-called hidden parameter of swelling. These researches can be used for piling construction by a design engineer on the areas which are characterized by deep deposition of underground water.

4.5 SEM Microstructural Analysis

Photographs showing the microstructural surface were taken with a scanning electron microscope (SEM). They were taken in order to investigate three-dimensional packing of calcite carbonate and clay minerals as well as to explain changes of marl properties, influenced by variable humidification and drying. Material under investigation originated from the testing ground on which examinations "in situ" were made, and from the samples which used for laboratory tests. The microstructure image from solid chip marl and filling fissures and binder from prepared weathering were subjected to the examinations of their microstructure with the use of a scanning electron microscope. The microstructural image of soil marl chips of prepared eluvium are shown in photographs, Figs. 4, 5, 6. After the analysis of photographs taken from various samples it has been found that marl microstructure creates a spatial configuration, rich in calcite from which the sediment is mainly composed and it is of an organic origin. The empty spaces between the visible calcite forms can be packed by air or calcium carbonate in a fine-crystalline amorphous form. One should emphasize that the occurring calcite crystals are characterized by rich forms and originate from fractional precipitation or mechanical grinding of calcareous chips. A part of sediment is built of skeletons of calcareous invertebrates, mainly decomposed, mixed to a various degree with a fine-grained material, which contains many plankton algae.

Figure 4. Microstructural surface of solid chip from marl eluvium observed in SEM - 3000x
fractional precipitation of calcium carbonate from the solution together with illite minerals which occur here considering the specific conditions as flatten illite, visible in many photographs. In order to compare, some additional examinations were made on samples taken from solid marl from several places and from recrystallization material which is the binder of rock spalls, Figs.7, 8.

Figure 5. Microstructural surface of solid chip from marl eluvium observed in SEM - 3000x

Figure 6. Visible well observed coccoliths enlargement - 6000 times

from Coccoliths group. There are also rohdolites, compact silica needles, siliceous sponge needles, etc. Beside chips of organisms, terragenic mineral there occurs binder of rock background or packing mass. This rock background or packing mass present a meshy mosaic of calcite grains which originated by chemical recrystallization of precipitated calcareous alime. The fine-grained mass is partly mixed with colloidal clay substance. Usually the terragenic material is the fine grain in carbonate rocks. Therefore, it can be recognized in the carbonate mass only by great magnification. Calcite elimination is very instrumental by means of disodium versenate. The marl presents such type of rock in which its organic skeletons are not the main rock elements but some attention should be paid to the fact that during the formation of the deposit those organisms could indirectly cooperate in the formation of rock causing to

Figures 7, 8. Microstructural surface of binder of marl eluvium observed in SEM - 3000x

Microstructural images obtained were compared pointing at the existence of fragments of fossil in the compact material in contrast to binder. The compact samples contain a well preserved microfauna which proves that the rock has not been recrystallized. The samples of the second type containing no microfauna are characterized by a great number of empty spaces. It does not exclude the occurrence of amorphous, fine dispersed calcium carbonate and clay minerals. It was proved by microstructural photographs taken from marl eluvium which was granulated in the laboratory, saturated with water and dried and after its solidification. The samples were taken from the compact
material and the binder alining at the SEM analysis, Figs.5,10.

Figure 9. Microstructural surface of binder taken from laboratory samples of granulated marl eluvium after recrystallization of calcite observed SEM - 4200x

It should be stressed that there occurs a distinct similarity of microstructures between the sample taken from greater solid parts of marl eluvium and one which is a binding agent and filling subjected to repeated moisture, and recrystallization. The microstructural similarity of samples taken from solid parts and binder is probably the results of phenomena caused by the changes of humidity and recrystallization and it also proves the correctness of conclusions which state that these processes occur under deposit conditions and the binder petrifaction is a part of their action.

5 THE DESCRIPTION OF PHYSICOCHEMICAL PHENOMENA OCCURRING IN MARL ELUVIUM

Marl eluvium consists of (rock pieces) debris and filling which is the most weathered marl. This marl is found in the form of fine fraction, filling partially or completely gaps and free spaces. In respect of mineralogy the marl and its eluvium consist of calcite and clay minerals from the illite group and in their chemical composition CaCO₃, CaO, Al₂O₃, SiO₂, MgO, Fe₂O₃ prevail and cations K⁺, Na⁺, Mg²⁺, SO₄ also occur. Under conditions of the deposit the processes, described in previous chapters of the paper, in principle, proceed simultaneously but the participation of these processes, considering variable outer conditions may be accepted as random and dependent on many agents. The scale of recognition of the processes permits the author to describe the model of physicochemical phenomena occurring in the marl eluvium with regard to their interaction and the results of the strength and deformation ability of weathering. The description of the conditions of these phenomena, with the extreme values of the geotechnical quality of marl eluvium, is very important because it allows to obtain the correct interpretation of the results obtained in any examinations and it leads to the evaluation of geotechnical parameters.

5.1 The Description of Phenomena Occurring Under Atmospheric Conditions Causing the Processes of Maximum Weathering Extinction

When there is a lack precipitations and temperature outside the deposit is high, the liquid discharges from inside to outside. It is caused by heat transport to the eluvium and non-uniform distribution of humidity, which continually flows from more wet zones to those where humidity is lower. The conditions humidity are the result of not only the external effects but also the weathering ability to the humidity fixation during binder hardening. The structural constraints which came into being through the absorption of water by eluvium, as a result of the formation of gel, are weak and can be spoiled easily by evaporation. The kinetics of water drying begins, first of all, from high pores and capillaries by the upset of physicochemical constraints and by expelling free water (dependant on the gravitation action). Then water evaporates from micropores and microcapillaries, the state of capillary pressure appears, which creates deformation through shrinkage. After evaporation of the capillary water, the structurally bound water, packed in colloidal bonds, begins to be expelled. The humidity shrinkage comes into being, that is to say, together with the change of humidity, the
changes of structure and volume are joined. In a consequence of drying and recrystallization processes, which will be discussed later, threshold humidity must occur. Such humidity causes a tendency to formation of scratches and cracks in the binder. The scratches are magnified by the occurrence of stresses or by the concentration of local stresses which transmit the formation of cracks on the external strata of pieces developing the microstructural situation in the inflow heat zone during drying and recrystallization which can be perceived as the formation of calcite which can be seen in the photographs of scanning electron microscopy. The cracking formation must be stopped with strong local deformation of the vicinity of these limits. The intercrystalline fracture overcomes small cavities (empty spaces bigger or smaller which may also be seen in the microstructural photographs), picked by perite or non-crystallized calcite coming from chemical reactions or by illite molecules and in these areas between granules and packing there are plastic displacement proceeding to scratches and cracks. Besides, as a consequence of heat growing in the coarse-grained area of the heat interaction zone, it appears that the grain boundaries between monolithic and chip parts. Crack growing is also possible as result of the formation of calcite nuclei of recrystallization from the bigger saturation of calcium carbonate. Each change, causing decrease of concentration of CO₂ in water, may be the cause of calcium carbonate precipitation. Calcium carbonate can be precipitated as the result of the evaporation of water containing acid calcium carbonate. This precipitated calcium carbonate can form nuclei of recrystallization in scratches, holes and empty spaces among chips binder. The speed of creating nuclei of recrystallization and their growing may be accelerated by the segregation of cations of other minerals or the separation surface of original particles and on free surface of holes. The total deformations achieved as a result of the connected deformation mechanism are sufficient for the internal stress reduction.

In such a situation a further heat inflow can be observed. It causes water evaporation from capillaries where gravitational water inflows. This leads to cracks and shrinkage. Thus, heat reaches larger and larger surface of the particles and, as a consequence, evaporation of electrostatically bound water can be observed. This water occurs as dipoles in colloidal micelles; it causes rapid increases of moisture shrinkage and determines formation of new cracks on larger areas-surface filled with smaller and smaller fractions. Finally, even big chips crumble. At the same time further moisture inflow may occur through the evaporation from the underlay strata as a result of the decrease in underground water and the transition of the intensity area of moisture contraction to the lower strata. The evaporated water may, sometimes enriched qualitatively by many precipitations, may be absorbed in the newly created cracks and empty spaces. The humidity causes the processes of calcite dissolution, and partial swelling joining with chips and unsaturated eluvium. It which allows the calcite recrystallization processes to recreate which join together loosely composed streak chips or punctually fill fissures of the strengthen eluvium. During moisture decrease and drying intensive cracking occurs, and under extreme drying state crumbling can be observed. In the filling between fissures, as well as in fissures, partially packed, in which a part of evaporated water from the lower strata condenses, new connections form from recrystallization of calcite. It integrates chip walls on which this process is more intensive in consideration of less moisture sorption on the chip surfaces. The process of drying, apparently destructive, leads to indissoluble chip-wall connections where in consideration of small precipitation the processes of recrystallization occur. The above-mentioned reactions occur continually consolidating the weathering and bring about relatively high values of the inner friction angle and little deformability which is documented by the measurements "in situ", presented in the authors another paper. The geotechnical parameters of eluvium, occurring under the humus stratum, achieve the maximum values during long-lasting droughts and when the underground water level is the lowest. Then the marl eluvium achieves the greatest thickness and stores the maximum quantity of heat coming from the atmosphere as a result of ventilation. It is the typical stone debris and its geotechnical characteristics are more favourable than the values achieved by the eluvium deposited in the zone of capillary ascents, e.g., 3-7 m below the ground.

5.2 Description of Phenomena Occurring Under Atmospheric Conditions Causing Maximum Moisture of Eluvium

If long-lasting rainfalls occur after a long-lasting drought, then water, dissolving CO₂ as well as CaO (calcium oxide) cations and anions, becomes a chemical solution, being a carrier of processes discussed in the previous chapters. Calcium oxide dissolving in water shows in a theoretical way solubility product:

$$\text{CaCO}_3 = [\text{Ca}^{2+}] \times [\text{CO}_3^{2-}]$$

If water contains the dissolved CO₂, H₂CO₃ carboxic acid dissociates according to the equation:

$$[\text{H}^+] \times [\text{HCO}_3^-] = \text{L}_1$$

$$[\text{H}_2\text{CO}_3]$$

H⁺ ions join with CO₃²⁻ ions from calcium carbonate to HCO₃⁻ ions, which are a little dissociated according to the equation:

$$[\text{H}^+] \times [\text{CO}_3^{2-}] = \text{L}_2$$

$$[\text{HCO}_3^-]$$

L₁, L₂ denote some constant values here. Joining H⁺ ions with CO₃²⁻ ions causes the loss of balance. Calcium carbonate is dissolved as long as concentration of Ca²⁺ ions increases. The state of system returns to the primary values of solubility product and calcium carbonate precipitates again when H⁺ ions are carried off from the solution.
Ca(HCO₃)₂ = CaCO₃↓ + CO₂↑ + H₂O

The most important factor for calcium carbonate solubility is the concentration of CO₂ which is dissolved in the form of gas in water. As a result of hydration process CaO contained in marl in excess of water passes into Ca(OH)₂

CaO + H₂O = Ca(OH)₂

As a result of the changes of water conditions and evaporation of gravitational water as well as separation of crystal structure, the following reaction occurs:

Ca(OH)₂ + 2H₂O + CO₂ = CaCO₃ + (n+1)H₂O

It creates a difficult microstructural situation. In the first phase of moistness, all the mentioned-above reactions occur, but what is more important, only part of them also occurs in the reverse direction as a result of changes of the concentration of dissolved CO₂ and precipitation reactions. Water, apart from dissolving reactions, in addition, cools granules of tiny fractures forming thermal stresses in crystals and capillarity cracks from high as a result of other hydrosphilic (containing calcite and illite minerals) water extrudes air and surrounds granules separating one from another. Illite minerals cause processes creating colloidal bonds, depending on an amount of the accumulating heat. Water penetrates deeply into capillary cracks and makes their expansion which helps the hydrodynamics and hydration reactions. As a result of this, the reaction between colloidal water with the surface of colloidal micelles becomes the greatest which results from electrostatic forming leading to bigger and bigger packing of water dipoles. This swelling of surface state of colloidal micelles creates mutual approach of colloidal micelles which brings about the bigger packing and the same phenomena occur on the surfaces of more or less weathered chips. This process of colloidal water absorption also promotes the increase in the concentration of Ca²⁺ ions, the state of the system and sometimes return to the primary solubility values after partial calcite precipitation, which at that moment starts to influence, with a restraining effect, the growth of colloid. Water progressively enters the interior of weathered granules and chips or less weathered thicker parts of surfaces and crack surfaces. If bigger water absorption to the interior through bigger surface of colloidal particles arranging greater quantity of water dipoles which causes compacting of particles and uncreating micelles approach each other and as a result of coagulation the micelles connect into bigger aggregates and the formation of the stage of microstructures begins. When such formation is finished, swelling begins to end. Moisture close to the plasticity limit corresponds with such a state, and, since the minimum amount of gravitational water remains, the maximum concentration of Ca²⁺ as well as calcite recrystallization can be observed. It leads to the end of swelling. For the maximum swelling the maximum possible amount of water dipoles is settled and it is indicated by the composition of colloidal micelles. The expanded eluvium resembles cohesive soil, the amount of gravitational water, in which sulbling processes may occur, is minimal. The amounts of calcium and potassium and different cations at the maximum degree are equalized by the electrostatic influence of micelles, the state of the system returns to the primary values of the solubility product. A part of acid carbonate becomes to precipitate in CaCO₃ and the relative state of equilibrium is achieved. Formation of coagulation structure is terminated and the process of intensive recrystallization starts. It is accompanied by the high level of underground water which creates the biggest difference of water level between the oceanic level with respect to the tensile level of Cenomanian water as well as Triassic water. It creates the conditions for the maximum filtration through numerous faults occurring here. After the end of long-term precipitation, water evaporates upwards and its filtering off occurs downwards which under these conditions leads to a very quick decrease in the underground water level.

6 RECAPITULATION

The examination, made by means of a scanning electron microscope, gave the image of geometry of space configuration at the microstructural level. The similarity of the microstructure at the microstructural level was taken from solid parts, binder filling and from recrystallization material, being the packing of rock chips in moulded samples, is the result of the phenomenon-packing with variations in humidity, drying and recrystallization. This proves the correctness of conclusions worked out in the analysis of the processes occurring in a sedimentation basin. The processes, occurring in the deposit are comparable with those occurring during the formation of marl sediment. Obviously under those conditions the sea-water concentration was higher than now as a result of the processes of weathering and the development of various sea organisms. The dynamics of processes depended on the intensity of biological life and the amount of detrital material which flowed into the sedimentation basin as a result of the processes of weathering and erosion from subsidiary river drainage basins. It influenced the participation of chemical calcite in packing between skeletons of organisms and the amount of illite minerals and others. The investigations, carried out on the marl eluvium, allow to find out that under present temperature and moisture deposit conditions, the processes occurring in the deposit are renewed at the macro-and microstructural levels. The participation of these processes in random and their intensity influences the level of moisture and quantity of accumulated heat. The intensive swelling, occurring while the critical humidity (when the weathering becomes similar to the cohesive soil), is the provision against the loss of strength, which is observed on the surface and in excavations (of the construction sites) as a result of soaking and distributing as well as excessive moistening at the contact of debris pieces. During his investigations the author has found that the favourable state of weathering under conditions of deposit is connected by the decrease in the values of geotechnical characteristics which result from the
investigations "in situ". The loss of strength has been registered only during laboratory tests on samples of big moisture-content (19 per cent) in which the processes occurring under conditions of deposit were not simulated [9].

The values described above correspond to the conditions of the model.

Under natural conditions the natural water content of 24 per cent has been registered in which the marl eluvium has kept its strength. The long-lasting research of the author has resulted in an explanation only.

1. Eluvium are the safe basement for the building objects founded in new areas of Opole city. The condition of the slopes of already exploited quarries in the Opole city confines also oriented before. The walls there, namely, vertical and they display the absence of massive movements, i.e., no failures or slips even in the neighbourhood of a busy highway.

The creep analysis answered the question on the behaviour of the slopes of the quarries as well as the building objects founded on the marl eluvium in case of absence of the processes described.

7 CONCLUSIONS

1. It has been determined that the constituent of the Opole marls is calcium carbonate forming a structure filled with illite minerals, relatively resistant to external factors. In consideration of their small per cent share, illite minerals have poor ability to swelling which is accentuated under wet or dry conditions.

2. Eluvium result from the changing weather conditions taking place periodically. The effects of these changes, even in the state of maximum moisture content, are not significant. When conditions of a deposit make the eluvium alike cohesive soil.

3. The effects of these changes are positive since they change, under the conditions of an area, the boundary values which permit to make use of the eluvium as a building foundation. The result of the carried-out pressiometric probing is to determine the migrant eluvium in layer of the Opole marls. The thickness of this layer is limited exclusively to a narrow band connected with the level of underground water and its strength constitutes the numerical minimum of deformability.

4. It has been determined that the assumed effect of the phenomena occurring in the smallest fractions of eluvium is swelling, which under conditions of water saturations, brings about filling the fissures and voids between weathering chips. In addition, swelling protects against unfavourable influence of the excessive increase of humidity. Under the natural conditions of a deposit, the process of an increase in humidity ends on the values of the plasticity limit.

5. The marls eluvium is extremely sensitive to water saturation. The paper takes up a problem from fissure physics and it may be particularly useful to know the geotechnical properties of the material filling fissures.

6. The results of researches, included in the paper, may be also used to forecast the changes of the geotechnical properties of eluvium as the outcome of the processes of weathering forced by lowering the level of underground water.

7. The conditions of formation of marls and limestones were, in principle, congenial in all the regions. The dynamics of the processes occurring during the formation of deposit depended on intensity of biological life and the amount of the detritus material which flowed up to the sedimentation basin as a result of the processes of weathering and erosion from river draining basins.

The analyses of literature and the authors own investigations allow to find that the geotechnical properties of the Opole marls eluvium are comparable with the properties of marls from other regions.

REFERENCES


Variability of compressibility and consolidation parameters evaluated in one-dimensional consolidation test

Variabilité des paramètres de compressibilité et de consolidation évalués dans un essai de consolidation unidimensionnel

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ABSTRACT: Studies of one-dimensional consolidation are carried out under incremental (IL test in oedometer) and continuous (CL test in consolidometer) loadings. Analysis of experimental errors as well as varying parameters of compressibility and consolidation indicate that CL tests have wider application. They provide description of the varying modulus of compressibility in function of wide loading range what makes evaluation of previous geological loading as well as presentation of a soil deformability model possible. The coefficient of consolidation \( c_v \) from CL tests is expressed by a filtration factor of the process. Greater and more uniform values of \( c_v \) are obtained from IL procedures. Interpretation of \( c_v \) from the IL tests should be done in function of the consolidation degree \( U \). It enables to evaluate a filtration-rheological character of consolidation in analysed soils.

RÉSUMÉ: Les essais de la consolidation unidimensionnelle sont réalisés par la charge, appliquée en étapes (essais IL dans l'oedomètre) ou d'une manière continue (essais CL dans le consolidomètre). L'analyse des erreurs expérimentales et la variabilité des paramètres de la compressibilité et de la consolidation montrent que les essais du type CL doivent être appliqués plus souvent. Dans les essais CL nous obtenons la caractéristique de la variabilité du module de la compressibilité dans la fonction des grandes contraintes. Tout cela permet d'apprécier l'influence des charges géologiques sur la compressibilité et formuler le modèle de la déformation du sol. Le coefficient de consolidation \( c_v \) obtenu dans les essais CL caractérise le facteur de filtration de la consolidation. Dans la méthode CL on obtient les valeurs du coefficient de consolidation moins différenciées et plus hautes que dans les essais IL. \( c_v \) des essais IL doit être faite dans la fonction du degré de la consolidation \( U \). Cette méthode permet d'apprécier le caractère de filtration et de fluage dans la consolidation du sol.

I INTRODUCTION

Geotechnical practice proves that parameters of soil compressibility and consolidation from laboratory tests are considerably varying. This fact is due to:
1. physico-structural properties of soils,
2. stress state, and
3. applied methods of laboratory procedures.

If influence of methodical factors is not defined, their determination of varying deformability, dependent on soil structure and stress state, seems impossible. In studies of compressibility we are many a time limited to uniaxial deformations when a sample is put in an aside non-deformable ring. These conditions approximate
the ones at depths to several hundred metres in horizontally-stratified soils. In such environment large-scale changes of stress can occur, caused by lowering of water level or by outer loading in a large area (e.g. under a storage place of production waste or outer heap of caprocks in open mine at considerable distance from a pit). In this case a central part of the loaded area is assumed to present only a vertical component of deformation - as soil in the vicinity is also subjected to the same stress and therefore, no deformation aside can occur. In such circumstances, a test in an aside non-deformable ring is the best approximation of natural conditions, although only some elements of a physical model are recorded in a soil massif. Inadequate model laboratory conditions are also a reason for noted inconformabilities in subsidence, recorded in the field and calculated on the basis of laboratory tests.

2 STUDIES OF ONE-DIMENSIONAL CONSOLIDATION

Studies of parameters of one-dimensional consolidation of soils were done with two loading methods:

IL - incremental load test when loading at particular level is constant and continued as long as full (or almost full) stability of deformations occurs; these are conventional oedometric tests;

CL - continuous loading consolidation tests when a soil sample is put into a consolidometer and measurements of changes of load, strain and pore water pressure are possible (Wissa et al., 1971).

In the IL test a progressing deformation can be observed. Its rate depends only on filtration and rheologic properties of soils. It enables to delimit parameters of consolidation, directly in connection with a classical solution of Terzaghi. In CL tests a deformation rate depends not only on physico-mechanical properties of soils but also on applied rate of incremental loading. Parameters of consolidation are delimited on the basis of analysis of increasing stress, deformation of soil and changing distribution of a pore pressure. In CL tests too low values (if compared to IL tests) of soil deformations are received. Although the CL tests seem to be less perfect than model conditions of the IL tests, values of compressibility and consolidation parameters from the former tests are frequently more reliable. Such opinion is due to:

- results of analysis of experimental errors,
- wide range of loadings to be applied in a consolidometer,
- physical characteristics of consolidation process.

3 ANALYSIS OF METHODS AND EXPERIMENTAL ERRORS

In the IL tests the measurements of soil deformations at incremental loading are varied and difficult to univocal determination. The table 1 presents examples of determinations of these errors, received in a procedure of repeated independent determination of the own deformations of an oedometer. Observed variability of these values is mostly due to incremental loading. Thus, it is not possible to point out univocally a proper value of experimental correction.

Table 1. Own deformations of an oedometer (after Barański, 1992)

<table>
<thead>
<tr>
<th>LOADING RANGE [kPa]</th>
<th>DEFORMATIONS [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>minimum</td>
</tr>
<tr>
<td>0 – 25</td>
<td>0,026</td>
</tr>
<tr>
<td>25 – 50</td>
<td>0,043</td>
</tr>
<tr>
<td>50 – 100</td>
<td>0,064</td>
</tr>
<tr>
<td>100 – 200</td>
<td>0,090</td>
</tr>
<tr>
<td>200 – 400</td>
<td>0,125</td>
</tr>
</tbody>
</table>

Influence of such indetermination on received values of soil compressibility is varying. The figure 1 presents examples of calculations, in which a determination error was calculated for three different moduli of soil compressibility.

To calculate the error, differences between maximum and minimum corrections presented in the table 1 were used. As clearly visible, only at lower values of the modulus of compressibility and at higher loading, the modulus error does not exceed several dozen %. Results of the above-mentioned analysis suggest that the IL test is reliable for very compressive soils only. Corrections for deformations Δh connected with incremental loading can be also defined by analysis of a curve. The way in which the cor-
Fig. 1. Errors of estimation of the oedometric modulus \( E_{ocd} \) corresponding to varying own deformation of the oedometer.

Fig. 2. Estimation of the correction \( \Delta h_o \) from analysis of a consolidation curve.

Fig. 3. Analysis of estimated errors of the compressibility modulus from a consolidometer test.

A) variability of the compressibility modulus with estimation of error range \( \Delta M_k \).
B) errors from consolidometer test.

or a dozen %. The figure 3 presents an example of the analysis of these errors. It has been calculated by the differential, method on the basis of indications independent on deformation gauges and known accuracy of measured force, exerted on a compressed soil sample.

In the CL tests a soil, subjected to continuous loading, indicates a lower deformation than a soil deformed under a constant loading IL in a longer time. Resulting differences in deformations are not any higher than diversity of evaluation of a real soil deformation in the IL tests. This problem is illustrated by examples of compressibility curves from IL and CL tests of ice-
dam lacustrine soils (Fig. 4).

![Graph showing consolidation stress vs. strain](image)

Fig. 4. Compressibility of soil evaluated by different methods.

K-consolidometer test (CL),
EI-traditional oedometer test (IL) soil strain without corrections,
EIΙ-traditional oedometer test (IL) soil strain with corrections from analysis of a consolidation curve (Fig. 2.),

4 COMPRESSIBILITY OF SOILS DETERMINED IN CL TESTS

Presented advantages of the CL tests were received when types of compressibility of soils of varying origin were determined. Studies of normally consolidated tills of the Vistulian (Weichselian) Glaciation (Fig. 5) indicated the U-shaped curve of compressibility (Stamatopoulos 1973). Compressibility at low stress (ε < 100 kPa) is small and corresponds to a repeated part of a stress path equal to preconsolidation. Examination of mineral composition of a clay fraction (kaolinite, illite) indicates that such effect cannot be connected with existence of expansion forces.

After maximum values of preconsolidation stress are overpassed, the value of a modulus of compressibility reaches its minimum (~100 kPa) and then starts to increase in a stress function. Varying moduli connected with these effects are small and could be observed at study procedure burden with a small error. Testing of significance of these effects is confirmed by performed error analysis (cf. Fig. 5).

![Graph showing modular of compressibility vs. consolidation stress](image)

Fig. 5. U-shaped compressibility curve for normally consolidated till of the Vistulian Glaciation.

In consolidometer studies there is in practice a possibility to receive considerably larger stresses than in oedometric tests. It is particularly applicable when deformability of a soil massif at larger depths is determined. Compressibility studies should be carried out in such a case with reference to a loading path. Prognoses of massif subsidence due to draining of mines, are the examples of practical application of moduli, determined in this way. Such procedure was applied to soils around the brown-coal openmine Belchatow in central Poland (Fig. 6a).

Consolidometer tests of ice-dam soils suggest influence of origin on their compressibility. The youngest ice-dam sediments of the Warta (Warthe) Glaciation in the studied area indicate a considerably larger and more diversified deformability than consolidated older sediments of the Odra (Drenthe) Glaciation. Curves of varying compressibility moduli $M_k$ in stress function prove also a drop of $M_k$ in three (for total four) cases, when consolidation stresses are larger than 1.2 MPa. Therefore, a considerable sensibility of deformation of these soils occurs when stresses to a soil in a geological past, were overpassed (Fig. 6b).
in which $M_k$ - is a modulus of compressibility from CL tests in a consolidometer, $\sigma$ - is a consolidation stress, and A or B - are the coefficients that approximate a relation of $M_k$ and $\sigma$; for normally consolidated soils the coefficient $B=1$, and for preconsolidated soils $B<1$.

It results from a theoretical analysis of connection between a logarithmic model of soil deformability

$$\varepsilon = C_e \ln \sigma$$  \hspace{1cm} (2)

and the relation (1) (Dobak 1986).

Examples of characteristics of compressibility moduli concordant with the above model are received for ice-dam sediments of the South Polish Glaciation and the Tertiary cohesive sediments over a brown coal series at Belchatow (Fig. 7).

Fig. 6. Results of a compressibility test for ice-dam lacustrine soils.
A) compressibility curves,
B) variability of the modulus $M_k$.

Most consolidometer investigations indicate the compressibility characteristics which can be described with a formula

$$M_k = A \cdot \sigma^B$$  \hspace{1cm} (1)

Presented examples indicate that a wide range of consolidation stresses from consolidometer tests enables a reliable description of compressibility. The curves indicate in different ways an influence of geological history of loadings on changing compressibility parameters.
5 DETERMINATION OF PARAMETERS OF ONE-DIMENSIONAL CONSOLIDATION

Determination methods of the consolidation coefficient \( c_v \) in IL and CL tests are different. In the IL tests an analysis of the consolidation curve in function of time presents data to calculations of the coefficient \( c_v \). The calculations are based on different methods and refer to various degrees of consolidation. In agreement with a solution of Terzaghi, the coefficient \( c_v \) should be constant at various degrees of consolidation \( U \). In fact, considerably varying values of \( c_v \) are observed, dependent on this fragment of the consolidation curve which is discussed in a given method. Such procedure creates significant difficulties in determination of reliable data to engineering calculations (Parkin 1978, Mroczkowski 1982, Duncan 1993). It seems therefore correct, to analyse varying values of \( c_v \) in the function \( U \) (Dobak 1986). Considerably varying values of \( c_v \) prove that consolidation in the analysed soil significantly departs the model conditions of consolidation, dependent on a pore water filtration - as described by Terzaghi in his solution. The figure 8 presents examples of changeability curves of \( c_v \) in function of the consolidation degree \( U \).

For each test the calculations were done, accepting extreme variants of errors formed at incremental loading. The line Z corresponds to calculations without any corrections. The line G means calculation with maximum error, equal to quick deformation noted during the first two seconds of a test. Independently on applied interpretation methods of experimental data, there is changeability of \( c_v \) in the function \( U \). It indicates influence of soil skeleton creep on rate and values of soil deformations. The values \( c_v \) from modified methods (Glazer 1976) of Taylor present well a middle part of the process only.

Possible dissipation of pore pressure is a decisive factor of consolidation. Evaluation of this factor is possible by the CL tests, in which a coefficient of consolidation \( c_{vf} \) is determined on the basis of pore pressure distribution during the procedure. Reliable values of \( c_{vf} \) are obtained for a quasi-stabilization phase state (Fig. 9.). These values change insignificantly in this state and their drop is due to lower filtration capacity of a soil, resulting from a lower index of porosity at increasing consolidation pressures. Comparison of values of the consolidation coefficient by different methods is presented for tills from the Plock Region (Table 2; M. Barański, 1992).

![Consolidation Degree](image)

**Fig. 8. Variability of the coefficient of consolidation in function of a consolidation degree (oedometer test).**

The presented results of the IL tests indicate very varied values of the coefficient of consolidation, decidedly lower than in the CL tests. In the IL tests it is due to the following facts:

1. interpretation of data is more difficult as corrections to errors in incremental loading of soils cannot be easily determined,

2. values of \( c_v \) are not constant because a significant role is played by creep of soil skeleton, particularly at larger consolidation \( U \) (the so-called: secondary consolidation).

In the CL tests:

1. mean error of \( c_{vf} \) determination is equal to about 10% (Dobak 1984)

2. the parameter \( c_{vf} \) presents mainly a filtration coefficient of consolidation.
It should be underlined that in a prognostic calculation of subsidence, the coefficient of consolidation which describes a filtration coefficient of the process, is more reliable. In classical calculations, the application of the coefficient $c_v$ (charged with effects of rheological deformations) results in a surrealistic longer time of consolidation.

6 CONCLUSIONS

1. In two methods of one-dimensional consolidation, the CL tests are charged with a lower methodical error, have wider range of applied loading and can be carried out in a shorter time.

2. Difference in values of the compressibility modulus from CL and IL tests does not exceed a range of errors and interpretation uncertainties in the IL procedure.

3. Values of the coefficient of consolidation from the CL tests present a filtration factor of the process and are reliable for prognosis of filtration consolidation. When planning the tests, one should remember that saturation of pores of a studied soil $S$, with water should be close to 1, and velocity of increasing loading dependent proportionally on expected real conditions in the field.

4. The IL tests are reliable for soils with a considerable compressibility ($F_{oed} < 10$ MPa). Comparison of results of IL and CL tests can form the basis to evaluate a participation of filtration and a creep of soil skeleton during deformation.

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Predicting and regulating the variation in subgrade density and moisture content
Prédire et régler la variation de densité et teneur en eau d’une plate-forme

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ABSTRACT: Properties of heave soils in the subgrade vary in the process of the road operation in the regions of seasonal frost penetration. The design permits predicting values of soil density and moisture content in autumn, winter, spring, and summer at various recurrence probability. An experience has shown that the variation in soil density and moisture content may be confined by means of facilities for regulating the subgrade water-and-temperature regime.


One of the main problems of road construction in the regions of seasonal frost-penetration is the use of heaving soils in road subgrades.

Because of a probability of heave soil freezing under the road pavement it is necessary to know how properties, density, and moisture content of such soils will change when the construction is completed. Only knowing the above, one can solve the question whether the heave soils should be removed wholly from the zone of freezing or be partially left in place under the structure and whether local heave soils may be used to construct an embankment.

To solve the above problem, observations on the water-and-temperature regime of road subgrades have been carried out. These observations were performed at three special stations and 30 posts in various natural-climatic zones of soil seasonal-frost penetration.

The station included three run-off areas 40 m long situated parallel to each other. At the first area consideration was given to a total run-off from a half-width of the carriageway and its adjacent shoulder, at the second area - to that from the pavement, and at the third area - to that from the shoulder. Water from these areas was collected with troughs and entered measuring pavilions equipped with measuring tanks and water level recorders. Opposite the run-off areas, meteorological grounds were furnished with standard instruments for recording air temperature, pressure and humidity, wind direction and velocity, precipitation quantity and intensity. In the embankment and road pavement courses, temperature gauges were installed. The soil density and moisture content were measured by radiometric instruments to a depth at least 1.5m, and moisture meters were also used. At the carriageway surface, bench marks were placed and their elevations were deter-
mined by means of high-precision levels. Everyday observations were carried out at the stations for 2 to 8 years.

As compared to the above, at the posts the measurements were conducted according to a shortened program.

The water-and-temperature regime of the subgrade, apart from direct observations on roads, was also studied at a ring test stand in Soyuzdornii. The stand includes a concrete channel that consists of three sections, each being connected with its own well regulating the ground water level. The section is about 14m long, 1.5m wide, and 2.0m deep. The channel was filled with soils of various compositions. The soil was overlaid with a road pavement, over which a load trolley was rotated. At the circular stand, measurements were carried out of the soil temperature, density, and moisture content as well as leveling of the road pavement surface was performed. Experiments were also conducted on subgrade models in a freezing chamber, including 1.2x2.7x1.4m plants with soil, and in a climatic chamber where models of smaller dimensions were applied.

Observations have shown that in the course of road operation, the variation in density and moisture content of the subgrade heave soils occurs. This process of variation consists of four main periods: autumn swelling, winter frost heaving, soil settlement during thawing in spring, and drying shrinkage in summer.

Each of these periods is characterized by its own value of the soil compaction coefficient that means a ratio of a given density of dry soil $\rho_d$ to its maximum density $\rho_{d(max)}$ according to the standard compaction method of Soyuzdornii.

Values of the above coefficients may be expressed as follows:

$$K_{comp(a)} = K_{comp(s)} / (1 + e_{sw})$$

$$K_{comp(w)} = K_{comp(a)} / (1 + e_h)$$

$$K_{comp(spr)} = K_{comp(w)} / (1 - e_f)$$

$$K_{comp(s)} = K_{comp(spr)} / (1 - e_{shr})$$

where $K_{comp(a)}, K_{comp(w)}, K_{comp(spr)}, K_{comp(s)} =$ soil compaction coefficient during autumn, winter, spring, and summer, respectively; $e_{sw}, e_h, e_f, e_{shr} =$ relative value of soil swelling, heaving, settlement, and shrinkage, respectively.

Round-the-year variations in density and moisture content repeat over the whole service life of a structure. In summer, when $K_{comp(s)} = K_{comp(p)}$, there occurs a restoration of the initial soil density $K_{comp(o)}$ achieved at the subgrade compaction. Then $K_{comp(s)} < K_{comp(o)}$, the initial soil density does not restore. The loss of soil compaction proceeds so long as there exists a relation $h_{sw} + h_h > (h_f + h_{shr})$, where $h_{sw} =$ soil swelling in autumn, $h_h =$ soil heave in winter, $h_f =$ soil settlement on thawing in spring, $h_{shr} =$ soil shrinkage on drying in summer (cm).

As studies shown, at some minimum value of the soil compaction coefficient after the settlement in spring $h_f > h_h$. In this case the loss of soil compaction ceases and a so-called "everyday" density takes place. And the higher are the initial soil density $K_{comp(o)}$ and the load (p) from the weight of the road pavement and overlying soil courses, the higher is the value of "everyday density".

Values of $e_{sw}, e_h, e_f, e_{shr}$ depend on the rate of absorbing the precipitation by the subgrade and water evaporation from the soil, ground and surface water levels on sections having no surface run-off, volume and rate of the capil-
lary water movement in the thawed soils, temperature field in the subgrade as well as volume and rate of the film water migration in the frozen soils.

Investigations have shown that a process of road subgrade wetting with atmospheric precipitation consists of four periods. During the first period, no absorption occurs, whole precipitation is spent on wetting the vegetation, ground surface, carriageway and shoulder pavement. During the second period of wetting, whole water appearing on the carriageway and shoulder is absorbed by the pavement or soil. During the third period of wetting, water appears on the surface in the form of either jets or a continuous course, and this results in a fact that the runoff occurs simultaneously with the absorption. After rain the fourth period takes place, during which the surface becomes free of water appeared at the third period. This water is spent on the absorption, runoff, and evaporation.

All pavements, asphalt and concrete ones included, are permeable. An amount of the water inflow into the soil underlying the road pavement depends on the pavement condition, cross section (single slope or double slope), carriageway width, precipitation intensity and duration, rain number, value of air humidity deficit, and a season of the year (spring, summer, autumn).

An amount of the precipitation absorbed by the shoulder soil is dependent upon the soil permeability (moisture content before precipitation, moisture content at the liquid limit, capillary moisture capacity, and coefficient of filtration), cross section, width of treated and untreated shoulder portions, surface longitudinal gradient and cross fall, vegetation character (without vegetative cover, with vegetative cover of medium thickness, thick vegetative cover), type of shoulder strengthening (crushed stone of various size distributions, sand-gravel, asphalt and cement concretes) as well as upon meteorological factors (the same as shown above) and water runoff from the carriageway to the shoulder.

As is found from the investigations, a process of the water evaporation from the soil is divided into three periods. Period 1 takes place at the presence of free water in the subgrade upper course = 5 cm. In this case the evaporation intensity is governed by a value of the elasticity of saturated vapour at the boundary with the surface of evaporation, absolute air humidity, air humidity deficit, and wind velocity. As for the hard shoulders, the intensity of evaporation is also dependent on a material used for their strengthening and its density.

Period 1 proceeds until the intensity of evaporation exceeds the inflow of capillary water to the subgrade surface. Otherwise, the soil in a layer as dries to the moisture content corresponding to the bound water. When there is no free water in a layer as, period 2 takes place, which is governed not only by the meteorological factors but also by the rate of inflow of film water, occurring in a layer as, to the subgrade surface.

When the soil moisture content is lower than a maximum hygroscopicity, period 3 appears, and the intensity of evaporation is governed by the meteorological factors, density and depth of a dry soil layer, through which vapour diffuses. The depth of this layer of moisture content lower than a maximum hygroscopicity increases until a moment the intensity of evaporation becomes equal to the water inflow to the lower boundary.

Studies have shown that over a moisture content interval from optimum \( w_{opt} \) to capillary moisture capacity \( w_{c.m.c.} \), a pore system of the soil may be presented by a capillary system that consists of some groups each of which incorporates capillaries of equal cross section. In order to make constructing a moisture curve convenient it is assumed that the soil pore structure consists of four capillary groups.

Each capillary group is charac-
terized by its own values of \( q_c \) and \( \kappa_w \), where \( q_c \) = driving force of a meniscous, Pa, and \( \kappa_w \) = coefficient of water infiltration, m/s.

For main soil types, experimental values of the above indexes of capillary properties are found depending on the plasticity index and soil compaction coefficient.

Depending on a type of capillary water (intrinsic-capillary or capillary-perched), the soil is simulated with a system of isolated or communicating capillaries. In the last case water moves under the action of forces \( \nabla = q_c - q_{\text{min}} \).

Due to these forces, capillaries of smaller size suck in water from those of larger size.

The velocity of capillary water depends on a direction of water movement (upwards, downwards or at angle), a distance from the source of wetting, and capillary properties of individual soil layers along the way of water movement. The movement of capillary-perched water in the subgrade upper layers also depends on the evaporation intensity.

Water migration in the soil of temperature below that of the ice-formation occurs due to a partial freezing-out of the film water on the soil mineral skeleton, release of a portion of the soil particle surface energy, and drawing there an additional quantity of water by the absorption forces. This mechanism of water migration is complemented with the action of crystallization forces. When during the soil saturation there takes place a head, free water may also inflow into the zone of freezing.

The flow rate of film water (\( q_{f(t)} \), m\(^3\)/s) that inflow from the thawed soil into the frozen course is

\[
q_{f(t)} = 0.92 \kappa_h \cdot \gamma_{\text{umfr}(t)} \cdot f(p) \cdot w_s,
\]

where \( \kappa_h \) = coefficient of soil heave, m/s; \( \gamma_{\text{umfr}(t)} \) = gradient of unfrozen film water; \( f(p) \) = function of the load effect on soil heave; \( w_s \) = area of soil cross section, m\(^2\) (\( w_s = 1m^2 \)).

The coefficient of soil heave means the intensity of saturated soil heave at \( \gamma_{\text{umfr}(t)} = 1 \) and \( f(p) = 1 \) under conditions when there is no shrinkage in the subgrade thawed portion. A value of \( \kappa_h \) is determined from soil test data. Values of \( \kappa_h \) depending on the coefficient of soil compaction have been obtained for main soil types.

\[
q_{f(t)} = f\left(w_{\text{opt}}, w_{\text{umfr}(t)}, w_{\text{umfr}(t)}; z_s, z_a, z_b, p_a, p_w\right)
\]

where \( w_{\text{umfr}(t)} \) = unfrozen water content at a depth \( z_a \) and \( z_b \), respectively, at a given moment; \( z_s \) = depth of freezing at the same time moment; \( p_w \) = water density.

The frozen soil moisture content \( (w_f) \) and heave \( (h) \) are dependent upon the initial moisture content of thawed soil \( (w_0) \), rate of its freezing \( (v_f) \), flow rates of the film and capillary waters \( (Q_{f(t)}, Q_{w}) \) that can inflow to the boundary of subgrade freezing.

When \( Q_{f(t)} < Q_{f(t)} \), the soil shrinkage occurs.

\[
h = h_0 - h_{\text{shr}(f)} - h_{\text{shr}(th)}
\]

where \( h_0 \) = rate of soil heave due to the migration ice-accumulation, cm; \( h_{\text{shr}(f)} \) = soil shrinkage within the subgrade frozen layer, cm; \( h_{\text{shr}(th)} \) = soil shrinkage within the subgrade thawed layer in winter, cm.

Investigations have shown that after the subgrade thawing in spring, the soil settlement de-

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pends on the soil density before freezing, rate of heave, and soil loading.

Values of linear shrinkage coefficients have been obtained for main soil types. This allows to determine the soil shrinkage when moisture content values after the settlement and for the summer period are known.

All the foregoing is presented in the form of design relationships that make it possible to predict the variation in heave soil density and moisture content in the course of the road operation. When computing, the following is successively determined: water inflow into the subgrade soils from the precipitation on the road surface in autumn; water inflow from the ground and surface waters on sections having no surface runoff; epures of the soil moisture content and density in autumn before the subgrade freezing; subgrade temperature field in winter; water inflow into the subgrade soils from the precipitation on the road surface in spring; water inflow into the subgrade soils from the ground and surface waters in spring; epures of the soil moisture content and density after the shrinkage in summer. For regions where winter thaws are often observed, water inflow into the soil in winter is determined. The variation in heave soil density and moisture content during the road operation may be confined by means of regulating the subgrade water- and-temperature regime. For this purposes, in the road structure it is expedient to install frost-protective and heat-insulating courses, drainage, waterproof, and capillary-intercepting interlayers, etc. As a capillary-interceptor a two-layered and even one-layered non-woven synthetic material "Bidim" may be used. An investigation, carried out when 10 years passed after laying two courses of "Bidim" of U-34 type above the ground water level, has shown that the clay soil moisture content under these two courses is equal to the capillary moisture capacity and above them - to the optimum moisture content.

Computer application package is prepared, which allows to predict

REFERENCES

Laboratory methods for evaluation of weak soil consolidation in subgrades and embankments of highways

Méthodes de laboratoire pour l'évaluation de la consolidation des sols mou en couches et remblais de routes

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ABSTRACT: New regularities of consolidation of weak soils in embankment foundations and clayey soils with the higher than optimum moisture content in the embankment body are briefly described. The necessity and ways for the improvement of methods for predicting soil settlements in time in soil compacting under the loading generated by the engineering structure weight and soil weight are shown. For the perfection of determining consolidation characteristics of soils the conditions of soil performance in the structure are proposed to be simulated in experiment more thoroughly.

RESUME: On a exposé brièvement les nouvelles conformités à la loi de la consolidation des sols moux dans une fondation d'un remblai routier et des sols argileux à la teneur en eau supérieure à celle optimale à l'intérieur du remblai. On a montré la nécessité et les moyens de modification des méthodes de prévision du tassement dans le temps des sols cités au cours de leur compactage sous la charge due au poids d'un ouvrage et celle due au poids propre. Pour éléver la précision de détermination des caractéristiques de consolidation on propose à l'expérience de moduler plus complètement les conditions du comportement dans une structure.

Idealization of actual soil properties, put into the current methods of predicting soil settlements, affects the accuracy of engineering structure calculations, including road embankments. Differences between idealized soil properties and actual ones become more evident as geotechnical problems become complicated, e.g. the use of weak soils (turf, silts, sapropels, etc) for embankment foundations and that of clayey soils with the higher than optimum moisture content for embankment bodies. All that requires a revision of traditional ideas and an introduction of new laws for deformations, which would reflect actual properties of soils used. It is apparently important to use as a base the results of investigations of physico-mechanical processes occurring in the soil under its static compaction in the engineering structure with taking account of soil performance conditions.

It is this approach to studying the regularities of compaction of weak soils in the embankment foundations (under the load of embankment weight) and clayey soils at \( w > w_{opt} \) in the embankment body (under the load of overlaid soil) that made it possible to determine such character of soil consolidation which differs from the traditional idea about it. It should be noted that if it is a question of foundations, we investigate the behaviour of natural soils and in the case of embankments, the behaviour of disturbed soil (technogenic) during its excavation and filling is investigated.

Thus, it was established that, in the general case (at the defi-
nite moisture content and working conditions of the soil) the consolidation process of the above soils should be considered as the process involving four stages after a symbolically instantaneous settlement: consolidation before filtration, primary filtration consolidation, secondary filtration consolidation and creep consolidation. Each consolidation stage is reflected by a specific section of the experimental curve \( A = f(t) \) according to the geometric character, where \( A \) = relative settlement, \( t \) = time of settlement achieving.

It is possible to show the sections on the relative situation of consolidation curves \( A = f(t) \) of two similar samples, only with different ways of the wringed pore water filtration. The first and last straight sections corresponding to the consolidation before filtration and creep consolidation respectively coincide actually on the curves of two samples. At the second and third stages, i.e. the second and the third curved sections of the consolidation curves of two samples diverge, which testify of the influence of the pore water filtration way on the settlement rate. At the transition of the primary consolidation into the second filtration one a curve fracture and a maximum of the excessive pore water pressure are observed.

On the basis of analyzing the results of compression and consolidation tests of soils of several types with different initial states and structures, and the change of the components (water, air) in compacting under different loadings the factors and processes affecting more significantly the settlement rate were determined.

The value and the rate of settlement at the first stage of natural soil consolidation are conditioned by squeezing out the water (and the air at the incomplete water saturation) from the zones adjacent to the draining surface, by the structure disturbance at the moment of load application, and by the elasticities of the pore liquid and soil skeleton and by the air compression at the incomplete water saturation.

The value and the rate of settle-
coefficient of the first section of
the curve \( \alpha = f(\log t) \) to the
time axis at the stage of consol-
dation before filtration; \( c^w_u \)
the consolidation coefficient, at
the stage of primary filtration
consolidation; \( c^w_u \), at the stage
of secondary filtration consolidation;
\( m^w \), similar to \( m^l \), at the
stage of creep consolidation.

Consolidation coefficients \( c^w_u \)
and \( c^u_u \) are proposed to be calcu-
lated on the known formulas of the
filtration consolidation theory,
but separately on the second and
the third curved sections of the
curve \( \alpha = f(\log t) \). In addition,
it is recommended to take account of
a decrease in the filtration way
during the process of settlement
at the above said stages. The use
of two consolidation coefficients,
\( c^w_u \) and \( c^u_u \), instead of one, \( c_u \),
and in consideration of the vari-
ability of the filtration way allow
to perfect the accuracy of fore-
cast.

Taiter proposed, for the first
time, to average the filtration
way (initial and final) in the
process of compaction. But up to
date nobody has actually supported
that proposal. At the same time
the calculations showed that in a
decrease of the filtration ways
of heavily compressible soils, the
settlement of their layer (ex.,
5m thick) could be reduced almost
5 times in its duration.

Depending on the initial state
of the soil, its structure, defor-
mation properties and its perme-
ability and working conditions,
the soil consolidation can involve
less amount of stages at their va-
rious combinations.

The following combinations of
soil consolidation stages are pos-
sible: 1) consolidation before fil-
tration, primary and secondary
filtration consolidations, creep
consolidation; 2) consolidation be-
fore filtration, secondary filtration
consolidation, creep consolid-
ation; 3) primary filtration con-
solidation, secondary filtration
consolidation, creep consolid-
ation; 4) primary filtration consoli-
dation, creep consolidation; 5) con-
solidation before filtration (pseu-
docreep) and creep consolidation.

For mineral cohesive natural soils
combinations 1, 2, 3, 4 may occur,
for organic natural soils: 3, 4,
for mineral cohesive soils of dis-

turbed structure: 1, 2, 5.

The author derived a differen-
tial equation for the consolidation
of clayey soils with the higher
than optimum moisture content, the
equation describing 4 stages of
soil compaction in the embankment
(under the load of the above-laid
soil weight).

The author's equation is as fol-
"ows:
\[ \frac{\partial H}{\partial t} + \frac{1 + \varepsilon}{\partial \lambda} \frac{\partial H}{\partial \lambda} - \frac{\partial}{\partial \varepsilon} \left( \frac{H}{\varepsilon} \right) \frac{\partial H}{\partial \varepsilon} = 0 \]  

(4)

The first term of the left part
of equation 1 shows the change ra-
te of the excessive pore water
pressure, the second term shows
the change rate of the gaseous
component (air) pressure. In deriving
the equation the relationship of
pressure distribution (P) between
soil components (water, air, sol-
ids) according to Bishop was as-
sumed. The expression of the right
part of equation 1 shows the cor-
responding changes of the porosity
coefficient (\( \varepsilon \)) and coefficient
of consolidation (\( a \)) of the soil due
to a free water permeability (\( k \))
and viscous properties of bound
water (\( H_0 \)), and, also, changes of
the volume compressibility of gas-
eous component at \( t = t_1 \) and at
depth \( z \).

Equation (1) was solved at
\( 0 < t < t_1 \) and at \( z < t < t_1 \).

In this connection, the boundary
conditions are as follows as: at
\( t = 0 \)
\( u_a = P \); \( H = 0 \) at \( t = t_1 \),
\( H = H_0 (z, t) \); \( u_a = 0 \) at \( t > t_1 \).

At \( u_a \to 0 \) equation (1) trans-
forms to V. N. Florin's equation of
consolidation of three-phase nat-
ural soil (2). In deriving equation
(2) the pressure distribution be-
tween soil components (water, sol-
ids) was assumed according to
Terzaghi.

\[ \frac{\partial H}{\partial t} + \frac{1 + \varepsilon}{\partial \lambda} \frac{\partial H}{\partial \lambda} - \frac{\partial}{\partial \varepsilon} \left( \frac{H}{\varepsilon} \right) \frac{\partial H}{\partial \varepsilon} = K_\lambda \frac{\partial^2 H}{\partial \lambda^2} \]  

(2)
where \( k_e \) = filtration coefficient. Boundary conditions of equation (2) are as follows: at \( t = 0 \) \( H = H(2) \) at \( t = 0 \) \( z = 0 \) \( \frac{\partial H}{\partial t} = 0 \) \( z = h \) \( H = p/H \).

At \( \omega \to 0 \) equation (2) transforms to Taitor's or Tertzagi's-Cherkos-
vyan's equations.

Since the solution of equation (2) entails some difficulties, the author proposes the system of equations (3) for practical calculations of the completion time for the prescribed relative settlement (a) i.e. for the prediction of the instantaneous settlement of the above soils in the structure. Each of the equations shows a corresponding stage of consolidation described above:

\[
\begin{align*}
\lambda(t) = & \lambda^I = \lambda_{Inst} + \int_{0}^{t} f_1(t_1) f_2(t_1) dt_1 \\
& \lambda^{II} = \lambda + \int_{0}^{t} f_1(t_1) f_2(t_1) dt_1 \\
& \lambda^{III} = \lambda + \int_{0}^{t} f_1(t_1) f_2(t_1) dt_1 \\
& t_{Inst} < t_1 < t^I \\
& t^I < t_1 < t^{II} \\
& t^{II} < t_1 < t^{III} \\
\end{align*}
\]

where \( \lambda_1 \) and \( t_1 \) = relative settlement and its completion time respectively (any moment at a given consolidation stage); \( \lambda^I - \lambda^{III} \) and \( t^I - t^{III} \) = final relative settlement at a given consolidation stage and its completion time respectively; \( \lambda_{Inst} \) and \( t_{Inst} \) = conventionally instantaneous settlement and conventionally instantaneous time of its taking place respectively. Indices I, II, III, IV refer to the stages of consolidation.

Functions \( f_1 \) and \( f_2 \) of the 2nd equation of the above system of equations (3) signify primary and secondary filtration consolidations respectively. To solve the functions \( f_1 \) and \( f_2 \) the equations and formulas of the filtration consolidation theory can be used. But in this case, as it was noted, consolidation coefficients should be determined separately at the second \( (\lambda^{II}) \) and third \( (\lambda^{III}) \) stages of consolidation.

In order to determine compression and consolidation characteristics of the above soils more perfectly, the simulation of soil performance conditions in the embankment body and its foundation is required to be carried out. For this aim it is necessary to set a regime of load application corresponding to that of filling an embankment; to create actual conditions of draining (one-side or two-side) with taking into consideration a weak layer location in relation to the permeable and water-proof layers in the engineering structure, i.e. to provide an open or closed system of tests; to reproduce the pressure gradient of squeezing water from the sample, corresponding to the actual pressure gradient of squeezing water from the weak soil layer.

Existing apparatus provide the first two conditions of simulation (regime of loading and draining conditions). To provide the third condition of simulation, it is necessary either to change the apparatus construction (to lengthen a filtration way of water, for, in the experiment, we create the pressure gradient of squeezing water essentially higher than the actual one) or to develop special methods for processing the results of consolidation tests carried out by means of standard apparatus.

The author developed a graphic method for drawing the consolidation curve by means of standard compression apparatus. The essence of the method is to reconstruct the trial consolidation curves \( u = f(t) \) for several compacting loadings \( (p_L = const) \) into the curves \( u = f(p) \) for definite time moments \( (t_L = const) \). By their interpolating the values of \( u_n \) are determined for the value of load \( (p) \) at which the actual gradient of water squeezing out is provided. Then for this value of \( p \) the curve \( u = f(t) \) is plotted, i.e. the desired curve of consolidation at a real gradient of water squeezing out.
Consolidation curves of soils plotted in experimental modelling the real, abovementioned soil performance conditions will reflect, to a higher degree, the actual properties of soils. This, with taking account of the author's revision of soil compaction regularities will make it possible to improve a reliability of the design of road embankments, using weak soils at \( w > w_{op} \) in the embankment body.

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Shear and resistivity behaviour of sandstones
Résistance et résistivité des grès

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ABSTRACT: The article presents the experimental results of studies conducted to understand shear and electrical resistivity behaviour of sandstones under high pressure triaxial tests. The influence of saturation on tangent modulus of elasticity, Poisson’s ratio, cohesion and angle of internal friction of these rocks has been evaluated. The combined effects of confining pressure, axial stress and degree of saturation on electrical resistivity of these sandstones have been investigated. The findings may be conceived as preliminary approach towards prediction of rock behaviour at different depths through electrical resistivity soundings in the field.

RESUMÉ: Dans cet article, les résultats des études expérimentales sont présentés pour mieux comprendre cisaillement et le comportement résistivité électrique des grès soumis à les tests hautes pression triaxiales. L’influence de la saturation sur le module tangent d’elasticité, le ratio de Poisson, la cohésion et l’angle de friction interne de ces grès a été estimé. Les effets conjugués celles de la pression restreinte, de la tension axiale et le degré de la saturation sur résistivité électrique de ces grès ont été étudiés. Ces études peuvent être considérées comme une approche préliminaire pour prédire le comportement de roches à profondeurs divers par les sondages de résistivités électrique dans le champs.

1 INTRODUCTION

The electrical resistivity technique is one of the geophysical methods which now-a-days is frequently used by Geotechnical engineers for subsurface exploration of large areas. The technique is not only a non-destructive but possess manifold advantages such as its suitability for variety of subsurface conditions, rapidity, simplicity of interpretation etc. over the conventional methods. To exploit its potential further, the technique is being researched world over to derive information about in-situ rock strengths and other parameters at different depths required for planning and design of civil engineering projects.

In view of this technique being rated as indirect method of subsurface exploration, the interpretation of field electrical resistivity soundings data is generally aimed at extracting the qualitative information about rock formations at different depths. However, certain studies (Donaldson and Keller, 1974; Kate and Rao, 1989) do indicate that electrical resistivity of rock may serve as an index of rock strengths. Which means a possibility to predict the rock behaviour at different depths under certain conditions through interpretation of resistivity soundings data alone.

Thus, it becomes imperative to understand the relationship between electrical resistivity of rocks and their strengths. The triaxial shear tests which provide strength behaviour of rock under simulated overburden stress existing at different depths in the field can be considered most suitable for such studies. In the light of this, present investigation has been
planned to understand electrical resistivity and shear strength behaviour of sandstones through high pressure triaxial tests conducted in the laboratory.

2 STRENGTH AND RESISTIVITY OF ROCKS

The variation between uniaxial compressive strength of rocks and their electrical resistivity reported by Donaldson and Keller (1974) is a curvilinear band on log-log plot showing increase in resistivity with strength. Similar variation between Young’s modulus of elasticity with resistivity has been noticed by them. A curvilinear increase in uniaxial compressive strength, shear strength, tensile strength and Young’s modulus of elasticity with increase in resistivity of dry sandstones has been observed by Kate and Rao (1989).

The effect of confining pressure on electrical resistivity of water saturated crystalline rocks reported by Brace et al. (1965) indicated initial small increase in resistivity at low pressure followed by large decrease in resistivity at high pressures. Kate and Rao (1989) noticed bilinear decrease in resistivity under increasing high confining pressures for sandstones in both dry and saturated conditions, which was mainly attributed to change in porosity taking place under pressures.

The effect of uniaxial pressure on electrical resistivity of rocks observed by Parkhomenko and Bondarenko (1960) indicated a decrease in resistivity at low pressures followed by resistivity increase at very high pressures. Brace and Orange (1968) conducted studies on electrical resistivity of different rocks during fracturing under the application of axial stress. They noticed a slight rise and then fall in resistivity with axial stress up to about half the fracture stress, beyond which it rapidly decreased. This rapid decrease in resistivity was attributed to rapid increase in volumetric strain of rock as a result of dilatancy.

The water contents in rock influences its resistivity considerably (Scott et al. 1967). The studies by Alvarez (1973) on the effect of atmospheric moisture on resistivity showed that, the rocks of resistive type are affected whereas, conducting type do not. A linear decrease in resistivity with increase in degree of saturation on log-log plot has been reported by Kate and Rao (1989) for water saturated sandstones.

3 TRIAXIAL TESTS

The rocks, their specimen preparation, experimental set-up used and the procedure adopted to determine their shear and electrical resistivity behaviour under different confining pressures (σ3), axial stresses (σ1) and degrees of saturation (Ss) through high pressure triaxial shear tests etc. are briefly described below.

3.1 Rocks and specimen preparation

The rocks studied herein are three sandstones namely Jhingurda sandstone (JS), Karnali sandstone (KA) and Kota sandstone (KS). Typical Scanning Electron photomicrographs and X-ray diffractograms of these sandstones are shown in Figs. 1 and 2 respectively. The textural and mineralogical compositions alongwith ranges of certain physical properties of these sandstones are illustrated in Table 1.

The cores of 38 mm diameter drilled out from rock blocks were cut to 76 mm length and were given a smooth finish conforming to tolerances as per ISRM (Brown, 1981). All the rock specimens thus prepared were dried up initially in an oven at 105°C, cooled down and kept for saturation in separate desiccators, each maintaining different relative humidity (depending upon degree of saturation required) till the moisture equilibrium was reached.
3.2 Set-up and test procedure

High pressure triaxial equipment consists of mild steel triaxial cell designed to withstand internal hydraulic pressure of 20 MPa, hydraulic dead weight system for applying and maintaining high confining pressures and loading frame of 100 tonne capacity, which facilitates application of axial compressive load at constant rate of loading.

In order to measure axial and diametral strains, four strain gauges with appropriate orientations were fixed on to rock specimen at its mid height along its circumference and were soldered with eight thin wires. For the determination of electrical resistivity of rock specimen, both its flat ends were held firmly between two copper discs of same diameter. These copper discs brazed to thin wire leads were covered with mica discs for insulation purpose. This arrangement adopted for measurement of electrical resistivity is similar to that used by Kate and Rao (1989). The specimen assembly enclosed in rubber membrane was placed within the triaxial cell and outer ends of all the ten wires were brought out through grooves made inside the cell body caps.

The outer ends of eight wires of the strain gauges were connected to digital strain indicator (12 channel) whereas, outer ends of two wires brazed with copper discs were connected to the terminals of digital multimeter for resistance measurement. After the placement of rock specimen, end plottens, sealing of rubber membrane and sleeve etc., the cell was placed on the platform of loading frame. Suitable connections of hydraulic line with
Table 1. Composition and properties of rocks

<table>
<thead>
<tr>
<th>Item</th>
<th>Jhingurda sandstone (JS)</th>
<th>Karnali sandstone (KA)</th>
<th>Kota sandstone (KS)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Texture and mineralogy</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grains</td>
<td>Well cemented (argillaceous), coarse, flaky</td>
<td>Cemented (silicious), fine, sub-angular</td>
<td>Cemented (ferruginous), fine to medium, sub-rounded</td>
</tr>
<tr>
<td>Voids</td>
<td>Interconnected large voids</td>
<td>Fine voids</td>
<td>Interconnected medium voids</td>
</tr>
<tr>
<td>Minerals</td>
<td>Kaolinite, Quartz, Feldspar</td>
<td>Quartz, mica, Kaolinite</td>
<td>Predominantly Quartz</td>
</tr>
<tr>
<td>Isotropy</td>
<td>Almost isotropic</td>
<td>Anisotropic</td>
<td>Anisotropic, thin bedding planes</td>
</tr>
</tbody>
</table>

**Physical Properties**

<table>
<thead>
<tr>
<th>Density (Kg/m³)</th>
<th>Dry</th>
<th>Saturated</th>
<th>Specific gravity</th>
<th>Porosity (%)</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1700-1900</td>
<td>2400-2550</td>
<td>2.60</td>
<td>21.5-22.9</td>
<td>13.3-14.1</td>
</tr>
<tr>
<td></td>
<td>2000-2255</td>
<td>2450-2600</td>
<td>2.59</td>
<td>1.9-2.4</td>
<td>1.6-2.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.6-7.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.1-3.6</td>
</tr>
</tbody>
</table>

Cell were made and confining pressure of known magnitude was applied through dead weight system. The axial stresses were then applied and corresponding strains and resistances were simultaneously observed till the failure of rock specimen.

The electrical resistivity (ρ) is determined from known values of resistance (R), cross-section area (A) and length (L) of rock specimen using the following equation.

\[
\rho = \frac{R.A}{L}
\]  

(1)

The values of strains, both axial and diametral could be read directly on the digital strain indicator.

3.3 Parameters studied

Triaxial shear tests have been conducted on intact specimens of JS, KA and KS rocks during which, their electrical resistances and strains were simultaneously recorded under following conditions.

(i) Confining pressures varied from 1 to 10 MPa in steps of 1 MPa (upto 7 MPa). Under each confining pressure, at least four samples of each rock type were tested.

(ii) Axial stress increased gradually from zero till ultimate failure of rock specimen.

(iii) Rock samples tested for their degrees of saturation of 0, 10, 30, 60 and 100 %.
In order to maintain uniformity under different confining pressures, the same rock specimen with any particular degree of saturation was tested for its electrical resistances by increasing $\sigma_3$ in steps of 1 MPa without the application of $\sigma_1$.

4 RESULTS

4.1 Stress-strain behaviour

Typical variations of axial strains ($\varepsilon_a$) and diametral strains ($\varepsilon_d$) with deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) are illustrated in Fig. 3 for Kota sandstone under different confining pressures at 100% degree of saturation. It can be seen from the figure that, both the strains increase curvilinearly with increase in deviator stress. At any particular deviator stress, the curves exhibit decrease in both the strains with increasing confining pressure. The above stress-strain relationships indicate brittle failure of rock specimens. The curves also demonstrate that, with increasing confining pressures, the failure stress ($\sigma_f$) and corresponding strains (both $\varepsilon_a$ and $\varepsilon_d$) also increase. Similar trends have been noticed for other sandstones also under all the confining pressures and degrees of saturation studied here.

The values of tangent modulus of elasticity ($E_t$) and Poisson's ratio $\nu$ under various testing conditions have been evaluated from stress-strain curves, at 50% of ultimate stress and corresponding strains. In order to demonstrate the influence of degree of saturation on $E_t$, $\nu$ and $\sigma_f$ of sandstones, typical variations between them corresponding to $\sigma_3$ of 4 MPa are illustrated in Fig. 4. The figure indicates that all the three $E_t$, $\nu$ and $\sigma_f$ decreases curvilinearly with increase in degree of saturation for these sandstones. Similar trends have been noticed at other confining pressures also.
4.2 Strength Parameters

The strength parameters cohesion ($c$) and angle of internal friction ($\phi$) have been extracted from modified failure envelopes i.e. plots of $(\sigma_1 - \sigma_3)/2$ versus $(\sigma_1 + \sigma_3)/2$ using the equations given below.

\[
\sin \phi = \tan \alpha \quad \text{and} \quad 2(a)
\]
\[
c = \frac{d}{\cos \phi} \quad \text{and} \quad 2(b)
\]

Wherein, $\alpha$ is the slope angle of modified failure envelope and $d$ is its intercept on ordinate as illustrated in Fig. 5, which corresponds to sandstones at 10\% degree of saturation. The variations between $c, \phi$ and degree of saturation illustrated in Fig. 6 clearly show curvilinear decrease both in $c$ and $\phi$ with increasing saturation for these sandstones.

4.3 Electrical resistivity behaviour

The influence of $\sigma_3$ on the resistivity of dry and fully saturated sandstones without any axial stress is shown in Fig. 7, which indicate bilinear decrease in $\rho$ with increase in $\sigma_3$. The electrical resistivity values under zero confining pressure and axial stress ($\rho_0$ values) are $9.7 \times 10^5, 8.9 \times 10^5$ and $4.4 \times 10^5$ ohm cm for dry JS, KA and KS rocks respectively.

The typical variations of $\rho$ with $\sigma_1$ under different confining
Fig. 7 The influence of $\sigma_3$ on resistivity of sandstones

pressures are presented in Fig. 8(a) for completely dry KS and in Fig. 8(b), (c) and (d) for KS, JS and KA rocks at 100% degree of saturation respectively. The curves in this figure indicate exponential decrease in resistivity with increase in axial stress. Initially the decrease is rapid whereas at axial stress corresponding approximately to failure stress, curves become nearly asymptotic. Such trends have been noticed for these sandstones under all the confining pressures applied in the present study. The initial rapid decrease in resistivity may be due to reduction in void spaces and closing of microcracks on the application of axial stress. The curves also show that, under any particular axial stress the resistivity decreases with increase in confining pressure. Similar trends have been observed for these sandstones at other degrees of saturation also. A comparison of such curves for different degrees of saturation as expected, showed a decrease in resistivity with increase in saturation under any particular value of $\sigma_1$ and $\sigma_3$.

The values of electrical resistivity at failure ($\rho_f$) under axial stress were recorded for different degrees of rock saturation and confining pressures. Typical variation between $\rho_f$ and $S_r$ on semilog plot illustrated in Fig. 9 for Kota sandstone under different $\sigma_3$ shows a curvilinear decrease in $\rho_f$ with increase in $S_r$. The curves in Fig. 9 also show decrease in $\rho_f$ with increasing $\sigma_3$. Similar trends have been observed for JS and KA rocks also.

4.4 Application

As a preliminary approach towards practical utility of (typical) curves presented in Fig. 8 and 9 may be visualised to understand problem cases at the site. At the problematic location with similar rocks, a comparison of electrical resistivity subsurface soundings data before and after the construction of structure may provide clues about rock failure along with its probable depth.

5 CONCLUSIONS

The high pressure triaxial shear tests conducted on sandstones to understand their strength, deformation and electrical resistivity behaviour leads to conclusions summarised below:

(i) The saturation of sandstones influences their strength and deformatonal behaviour. There is a noticeable decrease in the magnitudes of $c$, $\phi$, $E_i$, and $\nu$ of these rocks with increase in saturation.
(ii) The electrical resistivity of sandstones decreases considerably on increasing the saturation. There is a remarkable decrease in the magnitudes of resistivity at failure of these rocks with increase in saturation. These effects have been noticed for all the confining pressures and axial stresses applied.

(iii) The application of confining pressure and/or axial stress decreases electrical resistivity. The resistivity decreases exponentially with increasing axial stress.
REFERENCES


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NOTATIONS

A area of cross section
C cohesion
d intercept of modified failure envelope
\( E_t \) tangent modulus of elasticity
JS Jhingurda sandstone
KA Karnali sandstone
KS Kota sandstone
L Length
R Resistance
\( S_r \) Degree of saturation
\( \alpha \) angle of modified failure envelope
\( \varepsilon_a \) axial strain
\( \varepsilon_d \) diametral strain
\( \mu \) micron
\( \nu \) Poisson's ratio
\( \rho \) electrical resistivity
\( \rho_0 \) electrical resistivity of dry rock under zero external stress
\( \rho_f \) electrical resistivity at failure
\( \sigma_1 \) major principal stress, axial stress
\( \sigma_3 \) minor principal stress, confining pressure
\( \sigma_f \) failure stress
\( \phi \) angle of internal friction
Behaviour of cohesive soils subjected to dynamic loadings
Comportement des sols cohésifs soumis à des chargements dynamiques

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ABSTRACT: The paper presents the results of laboratory tests concerning the behaviour of cohesive soils subjected to dynamic loadings. The tests have been carried out in triaxial chamber with the application of pneumatic-hydraulic loadings enabling cyclic tests in the scheme: compression-extension. As a result of the tests the some dependences have been established between: the number of cycles needed to reach the failure of the soil and the frequency at the constant amplitude of loadings; and between the number of cycles and the amplitude of loadings at the constant frequency.

RÉSUMÉ: Dans l'article on présente les résultats des investigations en laboratoire concernant de comportement des sols cohérents pendant les essais dynamiques. Les investigations ont été exécutés avec l'appareil triaxial et application de chargement pneumatique-hydraulique, permettant l'étude cyclique dans le schéma: compression-extension. Les résultats ont permis d'établir la dépendance entre le nombre de cycles nécessaire pour rupture de sol et la fréquence quand l'amplitude de chargement est constante ainsi que entre le nombre de cycles et l'amplitude de chargement quand la fréquence est constante.

1 INTRODUCTION

Dynamic loadings (vibrations) occur commonly. In the soil media their range exceeds considerably the area of their induction. The propagation of vibrations through the soil is very complex. The dislocations of the soil particles in the result of dynamic loadings cause the creation of additional loadings. Thus the soil must carry some value of loading induced by vibrations in addition to static loadings. The destructiveness of these loadings is mainly the result of repeatability of dislocations despite even little deformations. Dependence: loading-deformability varies essentially compared to the relations obtained in static tests.

Hitherto existing investigations (1-12) indicate that dynamic loadings induce the increase of water pressure in pores causing decrease of effective shear strength. This leads to decrease of ultimate bearing capacity. For the soils which make foundation for building objects it is necessary to consider the influence of dynamic loadings on strength and deformability characteristics of the soils.

In the paper, the results of dynamic tests will be presented carried out on cohesive soil of lacustrine type (varved clays). These types of soils occur in the foundation of the Warsaw tube. All the tests have been carried out on a few dozens of samples with the application of prototypical apparatus.

2 THE TEST SOIL

The varved clays chosen for the dynamic experiments were deposited in the Warsaw marginal lake in the time of Middle Polish Glaciation.
The structure of soils is laminar: dark lamina interlayered with light ones. The dark lamina - clay type reach 4-6 cm in thickness and light lamina - about 7 cm. The presented results concern only type which is characterized by the following parameters:
- clay fraction content (<0.002 mm) 24 %
- silt fraction content (0.002-0.05 mm) 64 %
- specific density 2.67 Mg/m³
- natural volume density 1.94 Mg/m³
- porosity 43 %
- liquidity limit 31.2 %
- plastic limit 23.3 %
- plasticity index 7.9 %
- natural water content 27 %
- colloidal activity 0.33
- ratio of saturation 0.96

The test soil is low consolidated. Considering the mineral composition, among main minerals quartz and illite are distinguishable, among auxiliary: the minerals from the group of smectite, kaolin, chlorite and carbonates. The weight percentage (%) of the mineral composition is following: SiO₂=55-65, Al₂O₃=8-12, Fe₂O₃=4-8, CaO=5-10, MgO=6-9, Na₂O=1, K₂O=3.

The quantitative microstructural tests (SEM Hitachi S-800, programme Stiman ver.2.05 - Sokolov 1990) of the analyzed soil enable to determine its structure as matrix-skeletal, and coarsely-dispersed type C-II-B. The orientation of structural elements is poor.

In triaxial tests in the triaxial shear apparatus the following results have been obtained for the natural state of the test soil:
- the angle of internal friction $\phi=18^\circ$, $\phi'=21^\circ$
- cohesion $C_s=27$ kPa, $C'_s=24$ kPa.

In the same tests but for disturbed state of soil as well as water content and density equal to natural ones the results have been as follows:
- the angle of internal friction $\phi=3.5^\circ$, $\phi'=11^\circ$
- cohesion $C_s=28$ kPa, $C'_s=24$ kPa.

### 3 THE APPARATUS FOR DYNAMIC TESTS

Modelling of dynamic loadings in laboratory conditions takes place in dependence on the construction and capabilities of the applied apparatus, mostly by the sequence of repetitive loadings. The apparatus is based on a typical triaxial chamber for static tests, but it was rebuilt and equipped with the feeding set supplying pressured air (Fig. 1). The rebuilt chamber enables independent changes of lateral pressure and vertical loading exerted on the sample. The required values of the pressure can be obtained by regulating the feeding pump and output pipe of the pressured air. In the described apparatus the regulation of pressure is carried out by connecting the chamber with the air-compressor keeping constant pressure on required level. The valve linking the chamber with the compressor consists of the casing and the air pipes leading to the chamber and the compressor. Inside the casing there is a rotating ring with holes which close and open the inlet of compressed air. Changing the frequency of rotations of the ring one may regulate the number of cycles per second to which the sample is subjected (Fig. 2). The concept ensures that in the first half of the cycle the sample is compressed and in the second one extended which makes the scheme: "compression-extension".

The above construction enables to make tests in the frequencies reaching up to 50 Hz, the pressure in the chamber can be changed up to 70 kPa and the total vertical loading can reach 70 N.

In the course of testing the following parameters are measured:

![Figure 1. The photograph of the triaxial apparatus.](image-url)
the vertical deformation of the sample, the pressure in the chamber, the vertical loading and the pressure of water in the pores. The signals from the measuring sensors are amplified then they are transformed into signals of direct current with voltage reaching up to 10 V proportionally to the measured values. Then through the analogue-digital transformer PCL-812 card the signals are directed to the computer memory (Fig. 3). The measurement system is managed by the computer program ZPG-JS, which enables to make all measurements, record them on the hard disk and to make graphical presentation on the screen of the monitor. In addition the program performs Fourier's transforms of the measured parameters.

4 THE RESULTS OF THE INVESTIGATIONS

Two cycles of tests have been carried out in the apparatus presented above:
- the influence of frequency of dynamic loadings on shear strength with the constant amplitude of loadings,
- the influence of the amplitude of the loadings on shear strength with the constant value of frequency $f_i=1$ Hz, $f_2=2$ Hz.

All tests have been made on the samples with disturbed structure formed of the tested soil keeping: water content in the range 26-29 %, volume density 1.94-2.0 Mg/m³. Totally over 70 tests have been made. The tests have been continued until the failure of the sample. In most cases the shape of the samples after the test was similar to "barrel-like". During the test the samples mostly extended their length. The increase of water pressure in pores occurred stepwise or rapidly until it reached the value of pressure in the chamber.

In the first cycle of tests the constant pressure $q=50$ kPa was applied whereas the frequency was changing. All the samples were destroyed by liquefaction, $u/q = \text{about 1}$ ($u$ - water pressure in pores). It has been observed that with the increasing frequency of vibration ($f$), the number of cycles ($n$) necessary to diminish the effective strength of soil to zero, decreases. Only for the range of frequencies up to 5 Hz the functional relation has been determined, the formula being $n = e^{-1.15f+0.314}$ - Fig. 4.

In the second cycle of tests the amplitude of loadings was changed within the range 10-70 kPa, the
frequency being constant. It has been also observed that with the increase of the amplitude of the loading \( q \) the number of the cycles necessary to destroy the sample \( n \) decreases. The determined functional relations \( n = f(q) \) are as follows: 
\[ f_1 = 1 \text{ Hz}; \quad n_1 = e^{0.0673q+1.237}; \quad (f_2 = 2 \text{ Hz}; \quad n_2 = e^{0.0554q+1.536}) \]  - Fig. 5.

Having dynamic tests finished, the microstructure of the soil was determined on the basis of qualitative measurements. The structure of the soil subjected to cyclic loadings in relation to untested soil may be described as "disrupted". The silt particles are surrounded by clay ones; there is clear increase of the anisotropy coefficient in relation to the increase of frequency of vibrations.

![Figure 4](image1.png)

![Figure 5](image2.png)

**Figure 4.** The results of tests of influence of frequency of dynamic loadings on shear strength (silty varved clay, disturbed structure, the applied loading \( q = \text{constant} = 50 \text{ kPa} \)).

**Figure 5.** The results of test of influence of amplitude of dynamic loadings on shear strength (silty varved clay, disturbed structure and frequency: a - \( f = \text{constant} = 1 \text{ Hz} \), b - \( f = \text{constant} = 2 \text{ Hz} \)).

5 CONCLUSIONS

In the conditions without the outlet of water, the shear strength diminishes with the increase of the number of cycles of dynamic loadings. The number of cycles loading to the failure of the sample depends on the level of cyclic shearing loadings. The functional relation has been determined between the number of cycles \( n \) and the amplitude of dynamic loadings \( q \): 
\[ n = f(q; \text{ kPa}) \] for frequencies 1 Hz and 2 Hz. Another functional relation has also been determined between the number of cycles and frequency of dynamic loadings: \( n = f(f; \text{ Hz}) \) with the constant value of the loading - \( q \) (this dependence exists for the frequency not exceeding 5 Hz).

The first quantitative structural tests (SEM) of the investigated soil subjected to dynamic loadings indicate clear changes in disposition of particles, their orientation, changes in porosity. The investigations should be continued as they create opportunity for the closer scientific approach to the process of the failure of the soil subjected to dynamic loadings.
REFERENCES


Variability of the deformability parameters of the boulder clay of the last Scandinavian glaciation in Poland, in the Plock region

Variabilité des paramètres de déformabilité de l'argile à blocs de la dernière glaciation Scandinave en Pologne, dans la région de Plock

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ABSTRACT: The paper presents engineering - geological characteristics of clay of the last Scandinavian glaciation in Poland in the Plock region. The characteristics covers the determining physical properties, grain composition, mineral and chemical composition as well as quantitative analysis of microstructures SEM of tested soil. Particular attention has been paid to presenting changeability of parameters of the one-dimensional deformability of boulder clays i.e. of modules of compressibility and coefficients of consolidation. These parameters have been calculated on the basis of site investigation and laboratory tests, applying various methods of interpretation. The conclusions present the influence of microstructures of boulder clay, research methods and ways of interpretation on changeability of deformability parameters of the tested soil.

RESUME: Dans l'article on a présenté la caractéristique des argiles glaciaires comme la base de construction. Ces argiles de la région de Plock ce sont les dépôts glaciaires de la dernière glaciation sur le territoire de la Pologne. La caractéristique contient les propriétés physiques du sol, la granularité, la composition minérale et chimique, puis l'analyse quantitative des microstructures SEM. L'article présente particulièrement la variabilité des paramètres de la consolidation unidimensionnelle: le module de compressibilité et le coefficient de consolidation. Ces paramètres ont été calculés par les différentes méthodes de l'interprétation pour les donnés des essais in-situ et de laboratoire. Les conclusions présentent l'influence de la microstructure des argiles glaciaires, de la méthode des essais, du mode de l'interprétation sur la variabilité des paramètres de la déformation du sol.

1 INTRODUCTION

The last Scandinavian glaciation in Poland is called Vistula glaciation (in Western Europe decribed as Weischel).

The maximum extent of the Vistula glaciation in Central Poland was marked in the region of Plock, situated north-west of Warsaw.

Postglacial deposits of this glaciation are represented by the boulder clay. These soils, about 4 m in thickness are the basement soil for the living buildings of the new-erected district of Plock.

The proper statistical calculations of the earthen foundation require good knowledge of soil deformability parameters.

The aim of the article is to present changeability of deformability parameters of boulder clays obtained on the basis of pressuremeter, oedometer and consolidometer tests and to show the influence of structural factors, ways of testing and methods in interpretation of measured data on this changeability.

2 CHARACTERISTICS OF THE TESTED BOULDER CLAY

Laboratory tests have been carried out on samples of the boulder clay taken from the depth 1.5 m below ground level that is from the bottom of the excavation.
Grain composition: the tested soil is sandy boulder clay with the content of: sandy fraction - 52 \% 
silt fraction - 29 \% 
clay fraction - 19 \%

Physical properties of the boulder clay:
Physical parameters of the boulder clay are following: specific density \( \rho = 2.66 \text{ Mg/cm}^3 \), volume density \( \rho = 2.11 \text{ Mg/cm}^3 \), porosity \( n = 31.4 \% \), porosity index \( e = 0.458 \), natural water content \( w_n = 15.1 \% \), plastic limit \( w_p = 12.6 \% \), liquid limit \( w_l = 22.5 \% \), plasticity index \( I_p = 1.66 \% \), liquidity index \( I_l = 0.17 \),

degree of saturation \( S = 0.81 \)

Mineral composition: quartz - 71 \%, clay minerals - 20 \% (beidellite > kaolinite > illite), calcite - 9 \%.

Chemical composition:
\( \text{SiO}_2 - 74.8 \% \), \( \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 - 9.8 \% \),
\( \text{FeO} - 0.6 \% \), \( \text{MgO} - 2.8 \% \), \( \text{CaO} - 4.5 \% \),
\( \text{Na}_2\text{O} + \text{K}_2\text{O} - 2.9 \% \).

**SEM ANALYSIS**

The quantitative and qualitative microstructural SEM Analysis (Grabowska-Olszewska 1984, Osipov 1989, Czajka 1992) of boulder clays was made with the application of the system SEM Hitachi - S - 800.

Also the STIMAN ( ver. 2.05 ) program was applied to processing of SEM data ( Sokolov 1990 ). The main qualitative type of microstructure of boulder clays from Plock region is determined as skeletal-matrix ( figure 1 ). Matrix microstructure is distinguished by the presence of a continuous unoriented clay mass ( matrix ), which contains irregularly arranged inclusions of silt and sand grains.

**Skeleton microstructures** consists of a loosely uniform porous skeleton, composed mostly of primary silt grains ( Grabowska-Olszewska 1984 ). The results of analysis have enabled to determine of quantitative microstructural parameters: the number of pores ( in the tested sample ) \( 240 \times 10^3 \) porosily 38.04 \%,

total pore area of pores \( 28.83 \times 10^3 \mu \text{m}^2 \),
total pore perimeter of pores \( 290.6 \times 10^3 \),
average diameter \( 0.157 \mu \text{m} \),
average area \( 0.120 \mu \text{m}^2 \),
average perimeter \( 1.209 \mu \text{m} \),
filtration coefficient \( 0.003875 \text{ mD} \).

Having determined of form index ( \( K_f \) ) defined as the ratio between two margin measurements of pores it was possible to determine the morphological types of pores ( figure 2 ). The form index value change from 0.133 to 0.910 ( with the average 0.536 ).
The anisometric pores ( \( K_f < 0.66 \) ) prevail than isometric ( \( K_f > 0.66 \) ). The application of method which enables to assess the level of intensiveness of the image allows to assess the level of orientation of microstructural elements.

**Fig.1 Example of SEM photo of boulder clays; magnification x 250; (skeleton-matrix microstructure).**

**Fig.2 Example of distribution according to form index of pores of boulder clays from Plock region.**
The figure 3 show the orientation diagram of microstructural elements. The value of anisotropy coefficient \( K_a \) \[ 1 - (S_{1} - S_{2})/(S_{1} - S_{3}) \times 100 \% \] (1) (where \( S_{max} \) the radial area of a signal) is 16.4 %, which confirms medium anisotropy of structural elements (Sokolov 1990). In addition, main direction in pores orientation has ascertained about 150 degree.

3 SITE INVESTIGATION

In order to determine deformability of boulder clays of Vistula glaciation the pressuremeter test have been made with the use of pressuremeter type E, probe GC (Baguelin F., Jezquel J. F., Shields D. H. 1978).

Pressuremeter tests have been made in 5 holes at depths: 1.5, 2.5 m below ground level. The value of pressuremeter modules \( E_p \), limit pressures \( P_l \), creep pressures \( P_c \) have been presented in the table 1. Direct comparison of pressuremeter modules with oedometric ones \( M_s \) is not possible because stress path in the soil in pressuremeter test differs from the stress path in oedometric tests and this has remarkable influence on the soils behaviour (Bukowski M., Dudyca D. 1977).

There is theoretical basis for recalculating pressuremeter modulus to oedometric one (Bukowski M., Dudyca D. 1977). The formula of recalculating \( E_p \) modulus on \( M_s \) modulus is the following:

\[ M_s = k_s E_p \] (2),

where \( k_s \) - correction coefficient

\[ k_s = 1/(1 + 2\nu) \] (3),

where \( \nu \) - Poisson coefficient dependent on the plasticity index.

The value of original compressibility modules determined on the basis of pressuremeter tests are shown in the table 2.

Analysing the results of compressibility modules for the levels of testing 1.5 and 2.5 m one states that they are often higher for the first depth and lower for the second one. This is caused by lower water content in boulder clay at the level 1.5 m. The value of confarmability coefficient \( Z \) calculated by the formula \( Z = M_s/(k_s E_p) \) (4), are shown in the table 3. The \( Z \) coefficient depending on the loading, varies in the range 0.6 - 0.81; that is compressibility modules obtained from pressuremeter in traditional way.

4 LABORATORY TESTS

Deformability of boulder clay from Plock region has been determined in a laboratory with the use of oedometer and consolidometer within the range of loadings 0 - 400 kPa. This two types of tests differ concerning the way of the loading to the sample of soil. In oedometer test the loading is exerted rapidly and increased in steps.

In consolidometer test the sample is subjected to continuous and increasing loading. In both types of tests are obtained the value of parameters describing one dimensional deformability of the tested soil, that is compressibility.
Table 2 Compressibility modules calculated on the basis pressuremeter modulus.

<table>
<thead>
<tr>
<th>Level of Test [m]</th>
<th>Range of stress [kPa]</th>
<th>Compressibility modulus M [kPa]</th>
<th>Average modulus M [kPa]</th>
<th>σ [kPa]</th>
<th>ν [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>1.5</td>
<td>0-25</td>
<td>16264</td>
<td>32742</td>
<td>16050</td>
<td>32956</td>
</tr>
<tr>
<td></td>
<td>25-50</td>
<td>28890</td>
<td>32742</td>
<td>16650</td>
<td>16478</td>
</tr>
<tr>
<td></td>
<td>50-100</td>
<td>38520</td>
<td>32742</td>
<td>32314</td>
<td>33170</td>
</tr>
<tr>
<td></td>
<td>100-200</td>
<td>32956</td>
<td>32956</td>
<td>21828</td>
<td>33384</td>
</tr>
<tr>
<td></td>
<td>200-400</td>
<td>16692</td>
<td>19046</td>
<td>22642</td>
<td>22470</td>
</tr>
<tr>
<td>2.5</td>
<td>0-25</td>
<td>16478</td>
<td>16906</td>
<td>16264</td>
<td>16906</td>
</tr>
<tr>
<td></td>
<td>25-50</td>
<td>16906</td>
<td>17120</td>
<td>20330</td>
<td>8560</td>
</tr>
<tr>
<td></td>
<td>50-100</td>
<td>16478</td>
<td>34240</td>
<td>32528</td>
<td>11556</td>
</tr>
<tr>
<td></td>
<td>100-200</td>
<td>16692</td>
<td>17120</td>
<td>32528</td>
<td>13910</td>
</tr>
<tr>
<td></td>
<td>200-400</td>
<td>10700</td>
<td>9416</td>
<td>14766</td>
<td>10272</td>
</tr>
</tbody>
</table>

modules and parameters of consolidation serving to determining deformation of soil in time - coefficients of consolidation.

Natural water content in the samples assigned to laboratory tests was increased to about 20% - table 4.

4.1 Oedometer test

One should remember, that different oedometers on each level of loading reveal different self-deformations, this why it is necessary make rectification of all instruments used for the tests. Probably, the individual characteristics change in time.

Undoubtedly, these are instrumental limits in oedometer tests.

Compressibility modules calculated without and with consideration of correction caused by self-deformations in oedometers ( max, min average corrections ) are presented in the table 5 and figure 4.

Compressibility modules calculated without considering the corrections are the least credible. Their values are influenced not only by natural changeability of behaviour of the tested soil but also, to remarkable extent, self-deformations in oedometers.

Having obtained measured data from oedometer tests, additionally the values of coefficient of consolidation c, have been determined by means of:

\[ r = \frac{\Delta H_{100}}{\Delta h} \] (5)

Table 3 Oedometer compressibility modulus.

<table>
<thead>
<tr>
<th>Number of Test</th>
<th>Compressibility modulus in the range of stress [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-25</td>
</tr>
<tr>
<td></td>
<td>E_s</td>
</tr>
<tr>
<td></td>
<td>11800</td>
</tr>
<tr>
<td></td>
<td>16200</td>
</tr>
<tr>
<td></td>
<td>1950</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
</tr>
</tbody>
</table>

- method \( \log t \) (Casagrande's method)
- method \( t^{1/2} \) (Taylor's method)

The authors of this article don’t intend to describe particular methods of interpretation enabling to assess values of coefficient of consolidation. But it should be mentioned, that not always was it possible to obtain proper diagrams, allowing to calculate the values of coefficient of consolidation. This testifies that consolidation of boulder clay proceed in different way than it is described Terzaghi's one-dimensional consolidation theory - this also confirmed by analysis of lithologically and genetically different soils (Dobak 1986).
The determined values of coefficient of consolidation according to two different methods are shown in the table 6. In the table 6, the value of parameters was shown, which is called consolidation index (Minh 1976):
Fig. 4 Plots of changeability of the oedometric compressibility modulus.

Table 4 Physical properties of the samples boulder clays.

<table>
<thead>
<tr>
<th>Number of Test</th>
<th>Water content w [%]</th>
<th>Density of soil ρ [Mg/m³]</th>
<th>Degree of saturation S [%]</th>
<th>Liquidity index l, [l]</th>
</tr>
</thead>
<tbody>
<tr>
<td>E₁</td>
<td>20.31</td>
<td>2.03</td>
<td>0.91</td>
<td>0.80</td>
</tr>
<tr>
<td>E₂</td>
<td>20.73</td>
<td>2.02</td>
<td>0.92</td>
<td>0.82</td>
</tr>
<tr>
<td>K-80</td>
<td>19.74</td>
<td>2.10</td>
<td>0.94</td>
<td>0.65</td>
</tr>
<tr>
<td>K-60</td>
<td>19.92</td>
<td>2.04</td>
<td>0.94</td>
<td>0.82</td>
</tr>
<tr>
<td>K-40</td>
<td>20.85</td>
<td>2.03</td>
<td>0.98</td>
<td>0.84</td>
</tr>
<tr>
<td>K-24</td>
<td>20.56</td>
<td>2.06</td>
<td>0.94</td>
<td>0.72</td>
</tr>
<tr>
<td>K-16</td>
<td>20.12</td>
<td>2.10</td>
<td>0.94</td>
<td>0.66</td>
</tr>
<tr>
<td>K-66</td>
<td>20.02</td>
<td>2.03</td>
<td>0.92</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Table 5 Oedometer compressibility modulus.

<table>
<thead>
<tr>
<th>Number of Test</th>
<th>Range of stress [kPa]</th>
<th>Compressibility modulus calculated M₀:</th>
<th>without correction X₀</th>
<th>with correction Xₗ₀</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kPa]</td>
<td>[kPa]</td>
<td>[kPa]</td>
<td>[kPa]</td>
</tr>
<tr>
<td>E-3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-25</td>
<td>1866</td>
<td>2041</td>
<td>2146</td>
<td>2347</td>
</tr>
<tr>
<td>25-50</td>
<td>6851</td>
<td>8981</td>
<td>9151</td>
<td>10325</td>
</tr>
<tr>
<td>50-100</td>
<td>9103</td>
<td>11455</td>
<td>11596</td>
<td>11730</td>
</tr>
<tr>
<td>100-200</td>
<td>11996</td>
<td>14319</td>
<td>15098</td>
<td>15237</td>
</tr>
<tr>
<td>200-400</td>
<td>6255</td>
<td>6660</td>
<td>6666</td>
<td>6675</td>
</tr>
<tr>
<td>E-6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-25</td>
<td>2358</td>
<td>2874</td>
<td>3145</td>
<td>3521</td>
</tr>
<tr>
<td>25-50</td>
<td>3695</td>
<td>4236</td>
<td>4351</td>
<td>4640</td>
</tr>
<tr>
<td>50-100</td>
<td>6690</td>
<td>7639</td>
<td>8220</td>
<td>8613</td>
</tr>
<tr>
<td>100-200</td>
<td>8996</td>
<td>10145</td>
<td>10265</td>
<td>10335</td>
</tr>
<tr>
<td>200-400</td>
<td>4590</td>
<td>4759</td>
<td>4794</td>
<td>4980</td>
</tr>
</tbody>
</table>

secondary which is a result of creep of the soil skeleton. Analyzing the values of the index r one can say that the whole process of consolidation is governed both by filtration factor, and by reological one ( Minh 1976, Dobak 1986).

Generally it is contended that values of coefficient of consolidation c, determined by Taylor’s method bigger that c, values in Casagrande’s method.

4.2 Consolidometer test

In the course of test the loading put to soil increases continuously and simultaneously deformation of the tested soil proceeds. Physical-mechanical features and speed of the testing of soil have influence on the values of deformation. Stabilization of deformations during consolidometer test is impossible because of continous increase of loadings, the registered values of deformations are smaller than in oedometer tests. Consolidometer test has been made on 6 sample of boulder clay with various speeds of testing: 8, 6, 4, 2.4, 1.6, 0.8 mm/h. The obtained values are present in the table 7 and at figure 5 and 6.

Generally, the values of modules obtained in tests with speed 8, 6, 4 mm/h are higher than the remaining ones. It is caused by the fact the increase in deformations is reversally proportional to the speed of loading, whereas the increase values of porous pressure is directly proportional to the speed of increase loadings.
Table 7 Compressibility modulus determined on the basis of consolidometer tests.

<table>
<thead>
<tr>
<th>Number of Test</th>
<th>Speed of Test [mm/h]</th>
<th>Compressibility modulus in the range of stress [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0-25 25-50 50-100 100-200 200-400</td>
</tr>
<tr>
<td>K-80</td>
<td>8</td>
<td>1100 9459 8697 7276 11758</td>
</tr>
<tr>
<td>K-60</td>
<td>6</td>
<td>1530 7452 5971 6939 10751</td>
</tr>
<tr>
<td>K-40</td>
<td>4</td>
<td>1480 6743 5266 6988 11895</td>
</tr>
<tr>
<td>K-24</td>
<td>2.4</td>
<td>900 4546 3953 6770 9487</td>
</tr>
<tr>
<td>K-16</td>
<td>1.6</td>
<td>551 5507 4601 6528 11240</td>
</tr>
<tr>
<td>K-08</td>
<td>0.8</td>
<td>894 3111 4852 8058 13371</td>
</tr>
<tr>
<td>average modulus N [kPa]</td>
<td></td>
<td>1076 5970 5269 7168 11417</td>
</tr>
<tr>
<td>σ [kPa]</td>
<td></td>
<td>344 1798 978 541 1181</td>
</tr>
<tr>
<td>ν [%]</td>
<td></td>
<td>32 30 19 8 10</td>
</tr>
</tbody>
</table>

Observe the diagrams at figures 5 and one may easily notice, that with decreasing testing speed the diversification of max, mean and min compressibility modules decreases. Using the measured data from consolidometer tests, the values of coefficient of consolidation have been calculated. The values of coefficient of consolidation c<sub>ν</sub> from consolidometer tests at various testing speeds have been shown in the table 8.

Table 8 Coefficients of consolidation determined on the basis of consolidometer tests.

<table>
<thead>
<tr>
<th>Number of Test</th>
<th>Speed of Test [mm/h]</th>
<th>Range of Stress [kPa]</th>
<th>coefficient of consolidation c&lt;sub&gt;ν&lt;/sub&gt; [m²/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-25</td>
<td>25-50</td>
<td>8.2 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-80</td>
<td>8</td>
<td>50-100</td>
<td>4.4 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>2.8 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>2.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-60</td>
<td>6</td>
<td>25-50</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50-100</td>
<td>1.2 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>2.9 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>1.6 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-40</td>
<td>4</td>
<td>25-50</td>
<td>9.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50-100</td>
<td>3.1 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>1.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>1.8 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-24</td>
<td>2.4</td>
<td>25-50</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50-100</td>
<td>1.6 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>9.8 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>1.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-16</td>
<td>1.6</td>
<td>25-50</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50-100</td>
<td>1.2 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>1.0 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>1.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td>K-08</td>
<td>0.8</td>
<td>25-50</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50-100</td>
<td>7.5 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-200</td>
<td>8.6 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200-400</td>
<td>1.1 x 10&lt;sup&gt;-5&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

The influence of speed of testing on the values of the modules was visible only of remarkable difference of testing speeds. Generally are may testify the influence of testing speed on the value of compressibility modulus. The influence of structural and textural differences between particular samples may be visible at moderate speeds of consolidometer testing.

It's easily notifiable that the values of coefficient of consolidation determined in consolidometer tests are bigger than values of this parameter determined in oedometer tests. Generally one may conclude decreasing tendency coefficient of consolidation with the decreasing
testing speed, which might have been expected. Coefficients of consolidation obtained in K-08 test (testing speed 0.8 mm/h) are the closest to the values of this parameter determined in oedometer tests by Taylor’s method. Possessing the values of the coefficient of consolidation the authors assessed coefficient of permeability \( k \), difficult to laboratory calculation. Its value ranged within \( 6.3 \times 10^{-10} - 2.1 \times 10^{-9} \) m/s.

5 CONCLUSIONS AND DISCUSSION

The boulder clay which occur in the region of Plock are youngest deposits of the last Scandinavian glaciation in central Poland. Proper geotechnical calculations of earthen foundation require good knowledge of deformability parameters, that is: compressibility modules and coefficients of consolidation.

For practical and scientific the site investigation and laboratory tests been made in order to reveal changeability of the mentioned parameters.

This changeability is influenced by: soil structure, kind of test, and way of interpreting the measured data.

On the basis of results of quantitative analysis of microstructures of the boulder clay, the clear anisotropy has been determined. The boulder clay from Plock has skeleton-matrix structure.

Oedometric modules calculated from measured data of pressuremeter tests revealed sufficient confirmability with correction coefficient is 0.60 - 0.81.

Laboratory tests have been made on boulder clay samples with water content about 20 %. The tests purpose was to determine the values of compressibility modules and coefficients of consolidation. The tests have been carried out in oedometers and consolidometers within the range of loadings 0 - 400 kPa.

One should say, that compressibility modules calculated within considerably self-deformations of oedometers are the least credible. Analyzing diagrams of change of mean compressibility modules obtained in oedometers are the least credible. Analyzing diagrams of change of mean compressibility modules obtained in oedometer and consolidometer tests it may concluded that the character of curves differs.

On the basis of data measured in oedometer tests the coefficient of consolidation have been calculated using methods of Casagande and Taylor. It should be emphasized that not always has it been possible to obtain proper diagrams useful to the calculation of coefficient of consolidation, which testifies, that consolidation proceeded in different way compared to the one-dimensional theory of consolidation by Terzaghi.

The values of coefficient consolidation determined by Taylor’s method are bigger than values of the same parameters calculated by Casagande’s method. Analysing the values of consolidation index \( r \) are concludes the filtration factor as well as reological one of soil skeleton have influence on the process of consolidation. In addition, on the basis of data from consolidometer tests the values of coefficient of consolidation have been calculated. The values of this parameter are bigger than in oedometer tests. With the decrease of testing speed the coefficient of consolidation decrease and approach to the values of coefficient consolidation determined in oedometer test according to Taylor’s method . Consolidometer tests allow to determine parameters of deformability in any range of loading. The large advantage of consolidometer tests is that they enable to calculated compressibility of soil much faster than in oedometer test in which the errors connected with the long duration of the test are inavitable.

REFERENCES


Czajka, R. 1992. Quantitative characteristic of the pore space of the
The effect of soft coatings on shear strength of rock joints
L'effet d'un film d'argile sur la résistance au cisaillement des joints

S. Manolopoulos
Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

ABSTRACT: The results of a series of direct shear tests on rock joints with a thin clayey coating are reported. Planar saw-cut and rough undulating surfaces, replicas of natural rock joints were used. A thin layer of kaolin having a water content of 45% was applied and the samples were tested under three different normal stresses. The peak shear strength of kaolin coated joints is lower than that of the same joints without coating (clean), from 35% to 45% for planar saw-cut surfaces and from 39% to 62% for rough joints, depending on the surface roughness and the normal stress. On the other hand, the dilation increases slightly, which shows that the main action of the kaolin coating is "lubrication", i.e. the reduction in frictional resistance between the two rock walls.

RESUMÉ: Présentation des résultats d'une série d'essais de cisaillement direct sur des éprouvettes de roche revêtue d'un film d'argile. L'utilisation d'un découpage en surfaces planes ou onduelées a été fait pour reproduire les joints naturels dans la roche. Une mince couche de kaolin, à teneur en eau de 45%, a été appliquée sur ces surfaces. La valeur maximale de la résistance au cisaillement, des joints revêtus de kaolin, est inférieure à celle des joints propres. En fonction de la rugosité et de la contrainte normal appliquée, cette variation peut aller de 35% à 45% pour les surfaces planes et de 39% à 62% pour les surfaces rugueuses. Par ailleurs, la dilatation s'accroît légèrement. Ce fait démontre que l'action principal du kaolin est la "lubrification", c'est-à-dire la diminution de la résistance au frottement qui se développe entre deux surfaces rocheuses.

1. INTRODUCTION

Rock joint coatings occur as a result of chemical alteration or disintegration of the rock walls, or transportation of clayey materials from the surface by groundwater. Coatings such as chlorite, graphite, talc, serpentine, clayey minerals etc. are often, and since minerals usually have lower friction angle than rock materials, they may cause a considerable reduction in shear strength of rock joints, especially when wet.

Horn & Deere (1962) and Kenney (1967) report friction angles of several minerals which may be as low as 5°. Chapell (1975) found very low (friction coefficient 0.29) shear strength of graphite coated joints in shale whereas Gyenge & Herget (1977) report friction angles of 15-25° for molybdenum-coated rock joints, 13° for graphite-coated and 24° for joints coated with chlorite.

Jaeger (1971) reports similar results for graphite-coated shear surfaces. Williams & Pells (1981) suggest a reduction of 75% for the side resistance of rough rock sockets covered with a thin cake of bentonite.

From the above it is evident that when a rock joint is covered with a thin layer of a low friction mineral, a considerable reduction in strength may result. In this paper the results of a series of direct shear tests on modelled rock joints covered with a layer of wet kaolin are presented.

2. EXPERIMENTAL PROGRAM

In order to investigate the shear behaviour of coated rock joints, a research programme was carried out, consisted of a series of direct shear tests on modelled
rock joints covered with a thin layer of a wet clayey material, namely kaolin. A special cement mortar was used to model rock joints, since plaster based model materials are unstable in the presence of water. The main properties of the model material are given in Table 1.

Table 1. Typical properties of model material

<table>
<thead>
<tr>
<th>Property</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compressive strength</td>
<td>MPa</td>
<td>6.0</td>
</tr>
<tr>
<td>Tensile strength (Brazilian)</td>
<td>MPa</td>
<td>0.9</td>
</tr>
<tr>
<td>Point load strength</td>
<td>MPa</td>
<td>0.9</td>
</tr>
<tr>
<td>Density</td>
<td>g/cm³</td>
<td>2.2</td>
</tr>
<tr>
<td>Porosity</td>
<td>%</td>
<td>30</td>
</tr>
<tr>
<td>Friction angle of saw-cut surfaces</td>
<td>°</td>
<td>33</td>
</tr>
</tbody>
</table>

Planar saw-cut surfaces and replicas of two different rough undulating sandstone joints (designated RJ-1 and RJ-2), were used in this programme. Contours and typical profiles are given in Figure 1.

Dry kaolin powder was mixed with water to form a creamy substance with a measured water content w=45%±2% and a thin layer was spread on the two opposite rock walls of each surface. The two parts of the joint were pressed together, and tested in direct shear. The resulted thickness of the kaolin layer before testing was approximately 0.5mm. The mineralogical composition of dry kaolin is as follows: kaolinite 82.8%, mica 12.1%, quartz 2.1%, feldspar 1.5%, other clay minerals 1.5%, whereas the Atterberg limits are: liquid limit 52%, plastic limit 30% and plasticity index 22%. The shear strength parameters were determined in a conventional Wykeham Farrance direct shear box and found as follows: cohesion 7 kPa, peak friction angle 12° and residual friction angle 11° (Figure 2).

The samples were 250mm long and 120mm wide and tested under three normal stress levels, namely 50, 100 and 150 kPa, at a shear rate 0.4 mm/min in a specially designed shear box. Details of the experimental procedure are described elsewhere (Manolopoulos, 1991).

The main characteristics of the two rough joints, based on measurements of 13 longitudinal profiles 1cm apart, are given in Table 2, whereas the peak shear strength envelopes are given in Figure 2.

Table 2. Characteristics of rock joints

<table>
<thead>
<tr>
<th>Property</th>
<th>Units</th>
<th>RJ-1</th>
<th>RJ-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean roughness amplitude</td>
<td>mm</td>
<td>10.4</td>
<td>6.0</td>
</tr>
<tr>
<td>Highest-lowest point distance</td>
<td>mm</td>
<td>16.0</td>
<td>9.8</td>
</tr>
<tr>
<td>Maximum asperity angle</td>
<td>°</td>
<td>18.0</td>
<td>11.2</td>
</tr>
</tbody>
</table>

Figure 1. Typical profiles of joints

Figure 2. Peak shear strength envelopes of rock joints and kaolin.
3. RESULTS

3.1 Flat surfaces

The first series of tests involve tests on planar saw-cut surfaces. The resulted peak shear strength is given by the equation

\[ \tau = 5 + \sigma \tan 10^\circ \]  

(1)

The friction angle is much lower than the friction angle of uncoated surfaces (33°) but higher than the friction angle of the kaolin itself. Table 3 gives the peak shear strength of planar saw-cut surfaces coated with kaolin as compared with the peak shear strength of the same joints without any coating. The coating causes a reduction in shear strength from 35% to 45%, the higher decrease corresponding to higher normal stresses.

Table 3. Peak shear strength of planar saw-cut surfaces (kPa)

<table>
<thead>
<tr>
<th>Normal stress (kPa)</th>
<th>Coated joint</th>
<th>Clean joint</th>
<th>Reduction %</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>21.2</td>
<td>32.5</td>
<td>34.8</td>
</tr>
<tr>
<td>100</td>
<td>37.5</td>
<td>64.9</td>
<td>42.2</td>
</tr>
<tr>
<td>150</td>
<td>53.7</td>
<td>97.4</td>
<td>44.9</td>
</tr>
</tbody>
</table>

3.2. Rough surfaces

In Figure 3 typical shear stress - shear displacement and normal displacement - shear displacement diagrams for joint RJ-1, clean and coated for normal stress 100 kPa are shown.

In the case of rough joint the shear stress - shear displacement diagram for coated joints are quite similar with that of the same joint uncoated (clean), the only difference being the shear stress magnitude, which is much lower in the case of coated joint. The normal displacement - shear displacement diagrams are almost identical in the two cases. The reduction in shear strength is similar to that caused when a lubrication substance is inserted between two metal surfaces. Figures 4 and 5 give the shear strength envelopes for the two joints both coated and clean. The shear strength of kaolin and saw-cut coated surfaces are also given for comparison.

The reduction in peak shear strength caused by the coating varies from 54% to 62% for joint RJ-1 and from 39% to 53% for joint RJ-2 as shown in Table 4. The reduction in strength is considerable in both cases and greater than that observed in planar coated joints. The larger reduction corresponds to the rougher joint (RJ-1).
Table 4. Reduction (%) in peak shear strength of kaolin coated rough joints.

<table>
<thead>
<tr>
<th>Normal stress (kPa)</th>
<th>RJ-1</th>
<th>RJ-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>61.8</td>
<td>53.3</td>
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<tr>
<td>100</td>
<td>60.5</td>
<td>50.5</td>
</tr>
<tr>
<td>150</td>
<td>54.2</td>
<td>38.6</td>
</tr>
</tbody>
</table>

\[ \mu = \mu_{\text{min}} + (\mu_{\text{max}} - \mu_{\text{min}})^n \]  \hspace{1cm} (2)

with

\[ n = (1 - \frac{1}{c})^{m} \]  \hspace{1cm} (3)

and \( \mu_{\text{max}} \) and \( \mu_{\text{min}} \) stress ratios (expressed as percentages) corresponding to the peak shear strength of the coated joint, the potential maximum shear strength (= the peak shear strength of the same joint without coating, i.e. clean) and the potential minimum shear strength (usually a value between the peak shear strength of the coating itself and of the rock wall-coating interface).

\( f \) is the thickness of the coating,

\( a \) the mean roughness amplitude of the joint,

\( c \) the critical \( f/a \) ratio at which the shear strength approaches the minimum shear strength of the system (\( \tau = \tau_{\text{min}} \)) and

\( m \) an experimentally derived constant.

Figure 5 shows the application of this model with \( c=1 \) and \( m=2 \) for joint RJ-1 and \( c=1 \) and \( m=4 \) for joint RJ-2.

Brumund & Leonards (1973) give a variation of coefficient of friction from 0.60 for interfaces between sand and cement mortar to 0.32 for the same interfaces coated with graphite, i.e. a reduction in shear strength of 47%, which is in good agreement with the results of this study. Williams & Pells (1981) suggest a reduction of 75% in side resistance of rock sockets drilled with the use of bentonite as rock wall stabilisation slurry. This underlines the importance of such coatings in several engineering problems.

A series of tests were carried out with increasing thickness of coating. In this case, after the initial considerable drop for thin coatings, the peak shear strength decreased at a much lower rate with the thicknesses of coating, and reached a constant value - lower than the shear strength of kaolin - when the thickness was approximately equal to the mean roughness amplitude of the joint (Figure 6).

Papailiakos et al. (1993, 1994) suggested a general empirical model for the estimation of peak shear strength of filled rock joints. This model has the following expression

\[ n = (1 - \frac{1}{c})^{m} \]

Figure 6. Variation in peak shear strength with the thickness of kaolin layer.

3.3 Dilation

Representative normal displacement-shear displacement diagrams have already given in Figure 3. Coated joint shows the same or higher dilation than the uncoated (clean) joint. Thus the geometrical component of shear strength either remains unchanged or increases slightly in the case of coated
joint, emphasizing the lubricating role of kaolin. Figure 7 shows the dilation angle corresponding to the peak shear strength of the two rough joints, coated and uncoated, for the three normal stresses used.

![Graph showing dilation angle vs normal stress](image)

Figure 7. Dilation angle at peak shear strength of rough coated and clean joints.

4. CONCLUSIONS

A thin coating of a soft clayey material on the surface of rock joint causes a considerable reduction in shear strength. This reduction is of the order 35-45% for planar saw-cut surfaces and 30-62% for rough undulating surfaces. The rougher the joint and the lower the normal stress the greater the reduction in shear strength. The geometrical component of shear strength is at least equal to that of the same joint uncoated, suggesting that "lubrication" is the main action of coating.

5. REFERENCES


Caractéristiques géotechniques du Miocène de Lisbonne

Engineering geological properties of the Miocene formations of Lisbon

Isabel Moitinho de Almeida
Department of Geology University of Lisbon, Portugal

RESUMÉ: Le Miocène de Lisbonne est constitué par une épaisse série sédimentaire à large variation latérale et verticale de faciès, correspondant à plusieurs cycles. Les sols, fortement surconsolidés sont, à la surface, décomprimés et moins résistants.

On constate, malgré les différences de composition, l'uniformité des caractéristiques géotechniques des sols génétiquement liés.

La texture et la composition minéralogique imposent aux sols cohérents des limites d'Atterberg correspondant à une plasticité moyenne, presque toujours au-dessus de la ligne "A" dans le diagramme de plasticité c avec des corrélations assez élevées.

Les teneurs d'eau "in situ" sont généralement, même pour les sols décomprimés, légèrement inférieures à la limite de plasticité et supérieures à la limite de retrait, et l'indice de liquidité proche de zéro, correspondant à des sols très résistants au cisaillement.

Dans la détermination des caractéristiques de résistance "in situ", les sols mioçènes surconsolidés, avec des argiles dures fissurées et des sables très compacts, contrastent avec les sols superficiels décomprimés.

ABSTRACT: Lisbon Miocene is represented by a thick sedimentary series, with a wide horizontal and vertical variability, corresponding to several cycles.

Miocene soils, with stiff fissured clays and compacted sands show, at ground surface, degradation of geotechnical properties due to decompression by removal of overburden and erosional processes.

In spite of composition variability, the uniformity of geotechnical properties shows their genetic relations.

Texture and mineralogical composition of those clays impose values of Atterberg limits corresponding to inorganic clays of low to high plasticity, lying above the A-line on the plasticity chart.

Moisture content, even in surface soils, is usually near plasticity and shrinkage limits. Liquidity index, closed to zero corresponds to soils with high shear strength.

1. INTRODUCTION

La plupart du sous-sol de Lisbonne est constitué par des terrains miocénes (Fig. 1), correspondant à une épaisse série sédimentaire à large variation latérale et verticale de faciès.

La genèse de cette séquence est liée à l'affondrement du bassin du Tage, dont la zone vestibulaire a subi une intense subsidence tectonique, qui a été compensée par une sédimentation active.

Par son importance stratigraphique, due à sa position géographique entre la Méditerranée et l'Atlantique, et à l'alternance d'influence marine et continentale, le Miocène de Lisbonne a été l'objet de nombreuses études.

L'intense développement urbain depuis la fin du XIXème siècle a conduit à l'exécution de nombreux travaux de prospection géologique-géotechnique, dans les terrains du Miocène de Lisbonne, publiés sous la forme de rapports privés.

Paradoxalement, si l'intérêt pour la stratigraphie du Miocène de Lisbonne se maintient, les travaux de synthèse sur les caractéristiques géotechniques sont encore rares.


2. GÉOLOGIE

Du point de vue tectonique, la série miocéne fut affectée surtout par la phase bétique de l'orogénie alpine. A Lisbonne ont peut définir une structure principale en monoclinal, inclinant doucement vers E et ESE. Les principaux accidents sont liés à la base de la série, sans signes d'activité, ou au parcours du Tage avec des failles parallèles à la vallée principale et à la gorge actuelle, dans ce cas avec une bonne probabilité d'activité néotectonique.
Dans la longue bibliographie de la géologie de Lisbonne, on doit nommer les travaux de Cotter (1896, 1904, 1956) qui a fini du XIXème siècle a étudié "in situ" presque toute la série, et a défini 13 unités lithostratigraphiques correspondant, malgré la variation latérale de faciès, à de grands complexes lithologiques.

La chronostratigraphie de Cotter a été revue, parmi autres, par Antunes (1971, 1979), mais ce sont les divisions distinguées auparavant qui sont considérées dans les cartes géologiques (SGP, 1940, 1944, 1950; Almeida, 1986).

Du point de vue géotechnique la carte de 1986, à l'échelle 1:10.000, est la plus intéressante parce que, malgré l'absence de nouvelles données de surface, elle s'appuie sur les résultats de nombreux forages.

2.1 Composition

À Lisbonne la série miocène, avec presque 300 m d'épaisseur, est constituée par de rares niveaux rocheux et par des sols représentés par des sables limo-argileux, des argiles limoneuses et des marnes qui, ayant subi un long processus de diagenèse, ont acquis élevés degrés de surconsolidation.

Les phénomènes de pétification liés à la diagenèse et à la pedogenèse, associés à l'évolution de ces sols, ont conduit à la formation de quelques niveaux plutôt rocheux.

Au long de la série ont peut vérifier la variation globale de la composition lithologique (Fig. 2) associée à l'alternance de cycles sédimentaires.

La fraction congloméristique, avec quelques galets de quartz, est presque nulle et est limitée à la base de la série.

La fraction sablonneuse est constituée par des grains de quartz et des fragments bioclastiques.

La composition de la fraction argileuse est dominée (Almeida, 1991a) par des smectites et illites, traduisant les conditions climatiques et de drainage et l'apport des sources d'alimentation. Les minéraux des argiles sont généralement bien cristallisés. Dans les unités plus argileuses il y a parfois des niveaux très riches en matière organique, souvent associée à sulfures (pyrite) et des sulfates (gypse).

La fraction carbonatée est dominante dans plusieurs niveaux, indiquant la sédimentation d'une vase carbonatée.

3. CARACTÉRISTIQUES GÉOTECHNIQUES

La plupart des roches de la série miocène sont tendres.

Les sols non cohérents sont, pour la plupart, des sables siltieux (SM), des sables argileux (SC), des sables mal gradés (SP), et des mélanges de ces groupes avec SP-SM, SP-SC (sables mal gradés, limoneux et limo-argileux).

La plupart des sols cohérents sont des argiles de plasticité moyenne, presque toujours au-dessus de la ligne "A" dans le diagramme de plasticité, compris entre des argiles peu plastiques (CL) et des argiles très plastiques (CH). Il y aussi des sols à comportement limoneux (ML et MH) et des sols organiques très plastiques (OH).

3.1. Limites d'Atterberg

La teneur en eau "in situ" et les limites d'Atterberg varient au long de toute la série dans un intervalle assez étroit (Table 1). Les valeurs de moyenne et écarts types dans les différents unités sont, dans l'ensemble, de 40 ± 10 pour la limite de liquidité; 21 ± 4 pour la limite de plasticité et de 14 ± 3 pour le limite de retrait.

Les paramètres statistiques permettent d'analyser l'uniformité des caractéristiques des sols d'une formation. Rhébault (1988) suggère des intervalles de valeurs du coefficient de variation (CV) pour les unités homogènes avec 0,09 - 0,28 pour la limite de liquidité (wL) 0,06 - 0,16 pour la limite de plasticité (wP) et 0,18 - 0,40 pour l'indice de plasticité (IP = wL/wP).

L'unité MIVa (Table 1) est apparemment moins homogène mais ce qui marque la différence est la présence, dans quelques-uns de ces sols, de matière organique et de fraction fine plus limoneuse.

Les coefficients de corrélation entre la limite de liquidité et l'indice de plasticité sont presque toujours supérieurs à 0,9 (Table 2).

Considérant la ligne-A de Casagrande:

\[ I_p = 0.73 \times (w_L - 20) \]  \hspace{1cm} (1.1)

En constate que les sols de la série miocène avec:

\[ M_{IVa} \quad I_p = 0.76 \times (w_L - 15) \]  \hspace{1cm} (1.2)
\[ M_{IVb} \quad I_p = 0.89 \times (w_L - 19) \]  \hspace{1cm} (1.3)
\[ M_{VA} \quad I_p = 0.91 \times (w_L - 18) \]  \hspace{1cm} (1.4)
\[ M_{IVa} \quad I_p = 0.71 \times (w_L - 15) \]  \hspace{1cm} (1.5)
\[ M_{IVa} \quad I_p = 0.66 \times (w_L - 14) \]  \hspace{1cm} (1.6)
\[ M_{IV} \quad I_p = 0.83 \times (w_L - 16) \]  \hspace{1cm} (1.7)
\[ M_I \quad I_p = 0.72 \times (w_L - 13) \]  \hspace{1cm} (1.8)
Fig. 2 - Le Miocène de Lisbonne: A - Lithostratigraphie (Adapté de Antunes, 1979): 1 - clastes; 2 - sables grossiers et moyens; 3 - sables fins; 4 - argiles siliceuses; 5 - siltes; 6 - calcaires; 7 - lignite; 8 - glauconite; 9 - coquilles; 10 - coraux; 10 - passages latéraux); B - Distribution des types lithologiques (I - roches; II - argiles; III - sables)
Table 1 - Paramètres statistiques des limites d’Atterberg, teneur en eau et indice de liquidité

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Maximum</th>
<th>Moyenne</th>
<th>Écart type</th>
<th>Coef. Var.</th>
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</tr>
<tr>
<td>(w)</td>
<td>11.8</td>
<td>36.6</td>
<td>19.8</td>
<td>4.5</td>
<td>0.23</td>
<td>71</td>
</tr>
<tr>
<td>(I_L)</td>
<td>-0.70</td>
<td>0.68</td>
<td>-0.03</td>
<td>0.28</td>
<td>8.43</td>
<td>27</td>
</tr>
</tbody>
</table>

se distribuent dans une bande allongée parallèlement à la ligne-A.

On a soumis un groupe de 94 échantillons, à des corrélations croisées sur l'ensemble de paramètres mesurés, où la limite de retrait \((w_S)\) et l'indice de retrait \((I_S = w_L - w_S)\) sont inclus (Table 2).

On constate que l’indice de plasticité et l’indice de retrait ont non seulement une forte correlation avec la limite de liquidité, mais aussi entre eux.

L’importance de ces corrélations permet (Almeida, 1991b) l’utilisation des valeurs de \(w_L\) et \(I_P\) dans l’évaluation de la retrait:

\[
I_R = 0.72 + 1.35 I_P \\
I_R = -16.0 + 1.03 w_L
\]  

(2.1)  
(2.2)

3.2. Indices Physiques

Dans la série miocène les poids spécifiques et l’indice de vides se rangent dans des intervalles assez étroits (Table 3).

La variation de ces paramètres est limitée à la zone décomprimée où l’on trouve les minimums du poids spécifique humide \((\gamma)\) et à sec \((\gamma_d)\) et les maximums de l’indice de vides \((\varepsilon)\). Malgré l’uniformité de valeurs ont remarqué dans les moyennes une tendance générale du sommet vers la base de la série, traduisant le taux de surconsolidation.

Les valeurs du poids spécifique des grains \((\gamma_g)\) mesurés varient entre 27.6 kN/m\(^3\) et 24.8 kN/m\(^3\), dans un échantillon riche de matière organique.

La moyenne de 26.6 ± 0.65 kN/m\(^3\), franchement supérieure au poids spécifique du quartz (26 kN/m\(^3\)) est proche du poids spécifique des smectites (26,9 kN/m\(^3\)).

Les teneurs en eau "in situ" \((w)\) sont généralement, même pour les sols décomprimés, légèrement inférieures à la limite de plasticité et supérieures à la limite de retrait (Table 1).

L’indice de liquidité \((I_L = (w_L - w)/I_P)\) est ainsi toujours proche de zéro. On peu remarquer, comme pour l’indice de vides et les poids spécifiques, la variation de ces paramètres au long de la série traduisant les différences dans l’évolution subi.
Table 2. Coefficients de correlation entre les limites d'Atterberg.

<table>
<thead>
<tr>
<th></th>
<th>Limite de</th>
<th>Limite de</th>
<th>Limite de</th>
<th>Indice de</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Liquidité</td>
<td>Plasticité</td>
<td>Retrait</td>
<td>Plasticité</td>
</tr>
<tr>
<td></td>
<td>( w_L )</td>
<td>( w_p )</td>
<td>( w_S )</td>
<td>( I_p )</td>
</tr>
<tr>
<td>S. mioc. (*)</td>
<td>0.730</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-0.088</td>
<td>0.240</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.910</td>
<td>0.382</td>
<td>-0.264</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.954</td>
<td>0.606</td>
<td>-0.381</td>
<td>0.924</td>
</tr>
<tr>
<td>( M_{IVa} )**</td>
<td>0.914</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.266</td>
<td>0.182</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.868</td>
<td>0.593</td>
<td>0.0307</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.899</td>
<td>0.850</td>
<td>-0.183</td>
<td>0.747</td>
</tr>
<tr>
<td>( M_I )**</td>
<td>0.794</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-0.141</td>
<td>0.196</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.950</td>
<td>0.563</td>
<td>-0.293</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.985</td>
<td>0.729</td>
<td>-0.308</td>
<td>0.963</td>
</tr>
</tbody>
</table>

* Ensemble de la série miocène (n = 94)
** Unité \( M_{IVa} \) (n = 21)
*** Unité \( M_I \) (n = 58)

Table 3 - Paramètres statistiques des indices physiques.

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Maximum</th>
<th>Moyenne</th>
<th>Écart type</th>
<th>Coef. Var.</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{VII} )</td>
<td>e</td>
<td>0.56</td>
<td>0.72</td>
<td>0.65</td>
<td>0.05</td>
<td>0.07</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m²)</td>
<td>17.6</td>
<td>20.1</td>
<td>19.3</td>
<td>0.7</td>
<td>10</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m²)</td>
<td>14.5</td>
<td>16.1</td>
<td>15.4</td>
<td>0.6</td>
<td>10</td>
</tr>
<tr>
<td>( M_{Vla} )</td>
<td>e</td>
<td>0.47</td>
<td>0.76</td>
<td>0.63</td>
<td>0.08</td>
<td>0.13</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m²)</td>
<td>14.6</td>
<td>21.5</td>
<td>20.0</td>
<td>1.6</td>
<td>0.08</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m²)</td>
<td>12.1</td>
<td>18.0</td>
<td>16.3</td>
<td>1.3</td>
<td>0.08</td>
</tr>
<tr>
<td>( M_{Ve} )</td>
<td>e</td>
<td>0.53</td>
<td>0.81</td>
<td>0.60</td>
<td>0.06</td>
<td>0.11</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m²)</td>
<td>18.6</td>
<td>21.6</td>
<td>20.4</td>
<td>0.5</td>
<td>0.03</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m²)</td>
<td>15.6</td>
<td>18.4</td>
<td>17.0</td>
<td>0.7</td>
<td>0.04</td>
</tr>
<tr>
<td>( M_{IVa} )</td>
<td>e</td>
<td>0.54</td>
<td>0.63</td>
<td>0.59</td>
<td>0.04</td>
<td>0.06</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m²)</td>
<td>19.4</td>
<td>21.5</td>
<td>20.3</td>
<td>0.6</td>
<td>0.03</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m²)</td>
<td>15.9</td>
<td>18.1</td>
<td>16.6</td>
<td>0.6</td>
<td>0.04</td>
</tr>
<tr>
<td>( \gamma_s )</td>
<td>(kN/m²)</td>
<td>26.0</td>
<td>27.4</td>
<td>26.7</td>
<td>0.4</td>
<td>0.01</td>
</tr>
<tr>
<td>( M_{II} )</td>
<td>e</td>
<td>0.27</td>
<td>0.66</td>
<td>0.49</td>
<td>0.07</td>
<td>0.14</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m²)</td>
<td>19.2</td>
<td>24.3</td>
<td>20.8</td>
<td>0.9</td>
<td>0.04</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m²)</td>
<td>15.7</td>
<td>21.4</td>
<td>17.6</td>
<td>1.0</td>
<td>0.06</td>
</tr>
<tr>
<td>( \gamma_s )</td>
<td>(kN/m²)</td>
<td>26.0</td>
<td>27.4</td>
<td>26.7</td>
<td>0.4</td>
<td>0.01</td>
</tr>
<tr>
<td>( M_I )</td>
<td>e</td>
<td>0.33</td>
<td>1.17</td>
<td>0.53</td>
<td>0.14</td>
<td>0.26</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>(kN/m³)</td>
<td>18.4</td>
<td>22.9</td>
<td>20.6</td>
<td>0.8</td>
<td>0.04</td>
</tr>
<tr>
<td>( \gamma_d )</td>
<td>(kN/m³)</td>
<td>14.0</td>
<td>20.5</td>
<td>17.2</td>
<td>1.2</td>
<td>0.07</td>
</tr>
<tr>
<td>( \gamma_s )</td>
<td>(kN/m³)</td>
<td>24.8</td>
<td>27.6</td>
<td>26.7</td>
<td>0.7</td>
<td>0.03</td>
</tr>
</tbody>
</table>

3.3. Activité

Les valeurs de l'activité colloïdale (Skempton, 1953), évaluées par le rapport de l'indice de plasticité à la teneur en "argile": \( A_c = I_p / \% < 2 \mu m \), permettent de classer les sols miocènes (Table 4) en sols actifs (\( A_c > 1.25 \)) et sols inactifs (\( A_c < 0.75 \)).

Les sols inactifs sont plus abondants, soit dans l'ensemble des sols miocènes, soit dans les divisions individualisées (Almeida, 1992). La plupart des valeurs d'activité supérieure à 1.25 correspondent à des sols limoneux dont la teneur en particules argileuses est très basse, quoiqu'il y ait des agrégats argileux dans la fraction siltreuse.

Puisque l'activité dépend non seulement de la texture mais aussi de la composition minéralogique et de l'histoire géologique, on peut évaluer dans la série miocène l'activité de chacun des minéraux présents (Almeida, 1992).

La régression obtenue avec 60 échantillons:
Table 4 - Paramètres statistiques de l'activité.

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Maximum</th>
<th>Moyenne</th>
<th>Écart type</th>
<th>Coef. Var.</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>M\textsubscript{Va}</td>
<td>0.25</td>
<td>9.20</td>
<td>0.44</td>
<td>1.67</td>
<td>1.85</td>
<td>8</td>
</tr>
<tr>
<td>M\textsubscript{Vb}</td>
<td>0.35</td>
<td>1.05</td>
<td>0.64</td>
<td>0.25</td>
<td>0.39</td>
<td>7</td>
</tr>
<tr>
<td>M\textsubscript{Va}</td>
<td>0.29</td>
<td>17.40</td>
<td>2.01</td>
<td>4.04</td>
<td>2.02</td>
<td>18</td>
</tr>
<tr>
<td>M\textsubscript{I}</td>
<td>0.36</td>
<td>2.43</td>
<td>0.83</td>
<td>0.49</td>
<td>0.59</td>
<td>18</td>
</tr>
<tr>
<td>M\textsubscript{I}</td>
<td>0.17</td>
<td>2.50</td>
<td>0.79</td>
<td>0.40</td>
<td>0.50</td>
<td>71</td>
</tr>
<tr>
<td>S. Miocène</td>
<td>0.17</td>
<td>17.40</td>
<td>1.01</td>
<td>1.78</td>
<td>1.76</td>
<td>122</td>
</tr>
</tbody>
</table>

Ip = 6.93 + 0.65 Sm + 0.37 I - 0.02 K + 0.49Calc (3)

(R = 0.75) permet la comparaison avec les valeurs trouvées par divers auteurs.

De même, 0.49 est trop élevé pour l'activité de la calcite (Calc), qui devrait être proche de 0.18 (Salas e Alpañas, 1975).

Par contre, 0.65 est trop bas pour les smectites (Sm), bien que dans l'intervalle de 0.32 à 3.09, prévu pour les montmorillonites riches en Ca et Mg (Salas e Alpañas, 1975).

Les données disponibles suggèrent que, puisque l'abondance de calcite est associée à ces smectites, une partie de l'activité de celle-ci est masquée par la calcite. Il faut aussi considérer que les smectites sont bien cristallisées.

Quant aux ilites (I), elles aussi bien cristallisées, la valeur de 0.37 correspond aux données habituelles sur la bibliographie (Salas e Alpañas, 1975; Attewell e Farmer, 1976).

3.4. Résistance

L'analyse globale des valeurs des essais de laboratoire (résistance au cisaillement, compression simple et triaxiale et consolidation) montre que l'échantillonnage affecte souvent les caractéristiques de ces sols. Ce fait, associé à l'hétérogénéité de composition et situation rend difficile la quantification de ces paramètres.

À Lisbonne la prospection dans les terrains miocènes est habituellement fait par des forages à percussion à la caveure avec des essais de pénétration dynamique (SPT).

La compilation (Almeida, 1991a) de plus de 10000 résultats d'essais SPT montre que dans les argiles dures fissurées et les sables très compacts, le nombre de coups (N) est en général très supérieur à 60. Dans les sols superficiels décomprisés le nombre de coups diminue vers des valeurs qui peuvent correspondre à des sols de consistance moyenne ou moyennement compacts.

4. CONCLUSIONS

La large variation latérale et verticale de fécies de la série miocène est exprimée dans l'hétérogénéité lithologique. Malgré les différences on constate l'uniformité des caractéristique géotechniques des sols génétiquement liés.

Les valeurs des limites d'Atterberg varient dans des intervalles assez étroits et ont des coefficients de correlations souvent supérieurs à 0.9.

Les indices physiques permettent d'évaluer les différences de composition et d'évolution.

Les valeurs les plus dispersées correspondent à la dégradation de caractéristiques à la surface. Le contraste dans les valeurs des essais de pénétration (SPT) montre que cet essai peut définir l'épaisseur de la couche décomprimée.

REFERENCES


Lisbonne.

Influence sur la stabilité des ouvrages et l’environnement des échanges cationiques sur les argiles
Influence of the cationic exchanges in clays on the stability of structures and the environment

J.Baudrago
Université Paul Sabatier, CRMTG, UA CNRS, Toulouse, France

RESUME: La circulation de solutions de natures différentes entraîne sur les argiles des roches des échanges cationiques que nous avons étudiés sur le grès argileux de Berea par percolations successives de solutions de CaCl₂, MgCl₂, KCl et NaCl de force ionique I=0.01. D’une façon générale, tout changement de solution s’accompagne d’une brutte variation de la perméabilité. Celle-ci augmente lors des échanges Ca²⁺/Mg²⁺ et Na⁺/Ca²⁺, elle diminue pour les échanges Mg²⁺/K⁺ et K⁺/Na⁺ et ces variations sont réversibles. Pour les échanges de cations de même valence (Ca²⁺/Mg²⁺, K⁺/Na⁺ ou inverses), les isothermes sont du type de Langmuir et les énergies d’échange sont faibles et ont la même valeur (0,70 kJ/mole). Pour les échanges de cations de valences différentes, les isothermes sont plus complexes et les énergies beaucoup plus élevées (2.12 kJ/mole, pour Mg²⁺/K⁺, 3.41 kJ/mole pour Na⁺/Ca²⁺). Ces phénomènes sont importants dans tous les grands travaux (exploitation, stockage, environnement,...) où la perméabilité est un critère de choix.

ABSTRACT: The circulation of different solutions in clayey sandstone entails cationic exchange on the clays of rocks. These phenomena have been studied on the Berea sandstone by percolations of the successive solutions of CaCl₂, MgCl₂, KCl and NaCl with ionic strength I= 0.01. Generally speaking any change of solution is accompanied by a sharp variation of the permeability. Permeability increases during the exchanges Ca²⁺/Mg²⁺ and Na⁺/Ca²⁺, it decreases for Mg²⁺/K⁺ and K⁺/Na⁺. These variations are reversible. For exchanges of cations of equivalent valence (Ca²⁺/Mg²⁺, K⁺/Na⁺ or reverses), the isotherms are of the Langmuir type and the exchange energies are weak and have the same value (0.70 kJ/mole). For exchanges of cations of different valences, the isotherms are more complex and energies a lot higher (2.12 kJ/mole, for Mg²⁺/K⁺, 3.41 kJ/mole for Na⁺/Ca²⁺). These phenomena are important in all great works (exploitation, stockage, environment,...) where permeability is a criterion of choice.

1 INTRODUCTION
On sait que la variation de composition des solutions au contact des roches n’est pas sans influence sur leur perméabilité. La substitution d’une solution par une autre de nature différente va entraîner des phénomènes d’échanges sur les argiles au contact des fluides. Ces échanges dépendent de la nature des argiles et des solutions, de la température et de l’ordre de passage des cations. La littérature abonde d’études relatives à ces phénomènes, mais l’influence des échanges sur la perméabilité des roches a été peu étudiée.
Nous avons donc examiné, sur un grès bien connu, le grès de Berea, les cinétiques et les énergies d’échanges sur les argiles qu’il contient et l’incidence de ces échanges sur sa perméabilité.

2 DESCRIPTION DU GRES
Le grès de Berea est une roche à grain fin (0,15 à 0,35 mm), bien consolidée, de couleur gris clair. Il est essentiellement constitué de quartz monocrassill, plus rarement polycristallin, anguleux ou subanguleux, de feldspaths potassiques (microcline et orthose), de plagioclases sodiques en faible proportion. Quartz et feldspaths sont souvent entourés d’une aureole de silice secondaire. Accessoirement, on trouve de la calcite, des painelles de micas, des cristaux de rutile, de zinc et de pyrite. La fraction argileuse est constituée de kaolinite (6%), dillite (1%) et de chlorite (1%). La CEC de la fraction argileuse est de 12 meq/100 g d’argile; celle de la roche totale de 0,93 meq/100 g de roche. Dans ce grès nous avons prélevé une éprouvette de 40 mm de diamètre et 40 mm de hauteur dont la perméabilité à l’air est de 600 mdy, la porosité totale, de 19,80% et la porosité communicante, de 19,69%.

3 DESCRIPTION DE L’APPAREILAGE
L’appareil de percolation (Baudrago, 1978) est constitué (Fig.1):
- d’un générateur de pression qui maintient la solution

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de percolation à une pression constante tout le long des essais, choisie de façon à obtenir un débit de filtrat voisin de 10 cm³ par heure;
- d'un porte-échantillon, cellule triaxiale étanche avec chambre de frettage permettant le maintien à une valeur constante de la pression de confinement. Le porte-échantillon est placé dans une enceinte thermostatée dont la température est commandée par un programmateur;
- des dispositifs de mesure et d'enregistrement de la perméabilité, du volume et de la température de l'échantillon, du débit, de la conductivité et de la température du filtrat;
- d'un plateau collecteur automatique pouvant recueillir 16 fractions de volume prédéterminé.
Les filtrats sont recueillis en fractions de volumes suffisamment petits dans lesquelles sont dosés les cations contaminants et compensateurs.

Figure 1. Schéma de l'appareil de percolation utilisé.

L'éprouvette grès est soumise à 20 °C et sans jamais interrompre le débit, à des percolations successives d'eau déminéralisée et de solutions de CaCl₂, MgCl₂, KCl et NaCl de force ionique égale à 0,01 dont l'ordre de passage est le suivant:

\[
eau / CaCl₂ / MgCl₂ / KCl / NaCl / CaCl₂ / eau \\
CaCl₂ / NaCl / KCl / MgCl₂ / CaCl₂ / eau
\]

La pression de percolation est de 0,08 bar et la pression de frettage de 2 bars. Les essais sont prolongés jusqu'à ce que les échanges, contrôlés par la conductivité du filtrat, soient terminés. Ils ont ainsi une durée variant de 25 à 50 heures en fonction du temps nécessaire à l'adsorption/déssorption des cations.

4 RESULTATS

4.1 Perméabilité

Pour tous les essais réalisés, après les échanges cationiques, la perméabilité décroît régulièrement en fonction du temps (figure 2). Comme ils correspondent à des temps expérimentaux différents, nous avons calculé les variations de la perméabilité en fonction du temps \( (\Delta k)_{t} \) en les ramenant à un volume unité de filtrat. Ce volume est pris égal au volume poreux \( V_0 \) de l'éprouvette \( (V_0 = 10,72 \text{ cm}^3) \). Ces variations \( \Delta \) sont définies par:

\[
(\Delta k)_{t} = \left( \frac{k_t - k_i}{k_i} \right) \frac{1}{nV_0}
\]

\( k_i \), est, pour une solution donnée, la perméabilité mesurée dès que le régime de variation devient stationnaire, \( k_t \), la perméabilité à la fin du passage de cette même solution.

On a aussi calculé les variations de la perméabilité au cours de la substitution des cations \( (\Delta k_{s12}) \):

\[
\Delta k_{s12} = \frac{k_{s1} - k_{s2}}{k_{s1}} 100
\]

\( k_{s1} \) est la perméabilité à la fin du passage de la solution 1; \( k_{s2} \), la perméabilité au début du passage de la solution 2, après stabilisation du régime d'écoulement. Ainsi, \( \Lambda \) positif correspond à une
Figure 2. Variations en fonction du temps (exprimé en volumes de pores) de la perméabilité au cours des divers essais réalisés avec des solutions de nature différentes.
augmentation de la perméabilité, Δ négatif, a une diminution (tableau 1).

<table>
<thead>
<tr>
<th>Sens Direct</th>
<th>Sens Inverse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sol. CaCl₂ MgCl₂ KCl NaCl CaCl₂ Eau</td>
<td>CaCl₂ NaCl KCl MgCl₂ CaCl₂ Eau</td>
</tr>
<tr>
<td>(Δk)_0 (‰/V₀)</td>
<td>-0,24 -0,31 -0,24 -0,73 -0,46 -1,41</td>
</tr>
<tr>
<td>Δk₁₂ (%)</td>
<td>+57,16 -9,47 +2,21 +3,41 -6,27 -56,4</td>
</tr>
</tbody>
</table>

4.1.1 Perméabilités aux sels

Les perméabilités à l'eau et aux sels décroissent toutes en fonction du temps, de façon plus nette pour l'eau que pour les solutions salines. Dans le cycle direct, les amplitudes de diminution se classent ainsi par valeurs croissantes: Eau > Na > Ca > Mg > K. Dans le cycle inverse, les perméabilités décroissent aussi en fonction du temps, mais avec des amplitudes différentes qui se classent ainsi: Eau > Ca > Mg > K > Na. L'eau se distingue par des amplitudes de variations importantes (-1,41 et -7,20 ‰/V₀). Pour les sels, elles sont plus faibles et relativement voisines entre elles. Compte tenu de la différence des débits, les écarts observés entre ces valeurs ne sont guère significatifs.

4.1.2 Variations de la perméabilité aux interfaces cation/cation

Les interfaces eau/Ca⁺⁺, Ca⁺⁺/Mg⁺⁺ et Na⁺/Ca⁺⁺, dans le sens direct, et Na⁺/K⁺, K⁺/Mg⁺⁺, dans le sens inverse, se traduisent par une augmentation de la perméabilité. Par contre, les interfaces Mg⁺⁺/K⁺, K⁺/Na⁺, Ca⁺⁺/eau, dans le sens direct et Ca⁺⁺/Na⁺, Mg⁺⁺/Ca⁺⁺, Ca⁺⁺/eau, dans le cycle inverse, se traduisent par une diminution de la perméabilité.

On peut noter que les sens de variation de la perméabilité aux interfaces sel 1/sel 2 et sel 2/sel 1 sont reversibles dans le cycle réalisé, mais les amplitudes de variations sont différentes:

- Ca⁺⁺/Mg⁺⁺: +5,63 %  Mg⁺⁺/Ca⁺⁺: -6,27 %
- Mg⁺⁺/K⁺: -6,91 %  K⁺/Mg⁺⁺: +3,41 %
- K⁺/Na⁺: -5,39 %  Na⁺/K⁺: +2,21 %
- Na⁺/Ca⁺⁺: +4,65 %  Ca⁺⁺/Na⁺: -9,47 %

Seul le sens de variation de la perméabilité est important car l'amplitude dépend un peu du débit du filtrat. Comme celui-ci est variable d'un essai à l'autre, on ne peut accorder à ces différences d'amplitude une trop grande signification.

L'influence du temps est nettement soulignée par la boucle d'hystérésis (figure 2) décrite par la perméabilité au cours du cycle complet d'échange.

4.1.3 Corrections de la perméabilité de la différence de taille des cations

Comme les cations ont des dimensions différentes, les substitutions de solutions de natures différentes entraînent des variations de la dimension des canaux de circulation du fluide. On peut se demander alors si elles ne sont pas responsables des variations obtenues.

Nous avons donc calculé théoriquement l'influence de ces dimensions sur la perméabilité du milieu poreux. Pour cela, nous avons choisi le modèle de pore le plus simple, pore cylindrique. A partir de l'équation de Kozeni: 

\[ k = \frac{\phi^2}{8} \]  
(k: perméabilité; \( \phi \): porosité), nous avons calculé la variation relative de la perméabilité résultant d'une variation \( \Delta \alpha \) de dimension par:

\[ \frac{\Delta k}{k} = -\alpha \left( 3 \frac{3}{\phi} - 1 \right) \]  
(Aoubouaza, 1992).

Le grès de Berea contient essentiellement de la kaolinite. La variation de la dimension des canaux ne peut provenir que de la variation de l'épaisseur de la couche adsorbée du fait de la différence de taille des cations et de leur environnement ionique.

En solution, les cations sont hydratés. Or les données bibliographiques sur les rayons hydratés sont assez divergentes (Pytkovich, 1979; Helgeson et al., 1981; Koster Van Groos et Guggenheim, 1987; Dandurand et Schott, 1992). Les rayons de Debye-Hückel et de Helgeson et al (1981) étant difficilement compatibles avec nos conditions expérimentales, nous avons adopté les rayons calculés par Dandurand et Schott (1992). D’après ces auteurs, le rayon hydraté, \( r_h \), est égal au rayon cristallographique augmenté de l'épaisseur des couches d'eau adsorbées: \( r_h = r_e + n r_{2D} \). sachant que \( r_{2D} = 1,55 \) Å, les valeurs de rayons relevées et calculées sont portées dans le tableau 2.
Tableau 2. Valeurs (en Å) des différents rayons des cations étudiés (r_H2O = 1,55 Å).

<table>
<thead>
<tr>
<th>Cations</th>
<th>rayon cristallographique (r)</th>
<th>nombre de couches d'eau (n)</th>
<th>rayon hydraté (nH)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H⁺</td>
<td>0,35</td>
<td>2</td>
<td>3,45</td>
</tr>
<tr>
<td>Na⁺</td>
<td>0,97</td>
<td>1</td>
<td>2,52</td>
</tr>
<tr>
<td>K⁺</td>
<td>1,33</td>
<td>1</td>
<td>2,88</td>
</tr>
<tr>
<td>Ca⁺⁺</td>
<td>0,99</td>
<td>2</td>
<td>4,09</td>
</tr>
<tr>
<td>Mg⁺⁺</td>
<td>0,66</td>
<td>2</td>
<td>3,76</td>
</tr>
</tbody>
</table>

4.3 Influence de la vitesse de circulation des solutions sur les échanges cationiques

Nous avons vérifié l'influence de la vitesse de circulation des solutions sur les cinétiques d'échanges cationiques en réalisant le cycle d'échange suivant : CaCl₂ / MgCl₂ / KCl / NaCl / CaCl₂ à deux débits initiaux très différents : rapide (Q = 150 cm³/h) et lent (Q = 10 cm³/h).

On voit (figure 3) que les vitesses d'échanges sont les mêmes pour les deux débits utilisés et cela pour tous les cations examinés.

Tableau 3. Calculs des variations relatives de la perméabilité résultant des différences de taille des cations.

<table>
<thead>
<tr>
<th>Échange</th>
<th>H⁺/Ca⁺⁺</th>
<th>Ca⁺⁺/Mg⁺⁺</th>
<th>Mg⁺⁺/K⁺⁺</th>
<th>K⁺/Na⁺</th>
<th>Na⁺/Ca⁺⁺</th>
<th>Ca⁺⁺/H⁺</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δk/k (%)</td>
<td>-0,02</td>
<td>+0,02</td>
<td>+0,06</td>
<td>+0,02</td>
<td>-0,12</td>
<td>+0,02</td>
</tr>
</tbody>
</table>

Avec ces valeurs de Δk/k nous avons alors, à partir du modèle de pore élaboré, calculé les variations de la perméabilité résultant de ces différences de taille (tableau 3). On voit que, dans tous les cas, ces variations sont extrêmement faibles et n'ont aucune incidence sur les résultats expérimentaux analysés plus haut.

4.2 Concentrations en cations

Les variations des concentrations en cations adsorbés et désorbés sont symétriques pour les cations de mêmes valences, elles présentent une dissymétrie importante pour les substitutions de cations de valences différentes. Les cations de même valence se remplacent cation à cation : 1 Mg⁺⁺ remplace 1 Ca⁺⁺ et 1 Na⁺, 1 K⁺ alors que pour les substitutions des cations de valences différentes, les remplacements se font ainsi : 1 K⁺ remplace 0,5 Mg⁺⁺ et 0,5 Ca⁺⁺ remplace 1 Na⁺. C'est vrai aussi pour les échanges inverses.

En outre, pour les cations de mêmes valences, les vitesses d'adsorption et de désorption sont égales et atteignent un maximum au début des essais (figure 4). Le maximum est plus intense et l'échange est plus rapide pour les cations monovalents que pour les cations bivalents. Dans le cas des cations de valences différentes, l'échange est plus lent et la vitesse d'adsorption ou de désorption du cation monovalent est toujours plus grande que la vitesse de désorption ou d'adsorption du cation bivalent.

On retrouve, avec nos résultats expérimentaux, des faits connus sur les phénomènes d'échange.

L'exploitation théorique de nos données expérimentales n'est possible que si les équations thermodynamiques des échanges cationiques peuvent être appliquées. Cela suppose que les conditions d'équilibre soient réalisées. Or, en percolation nous sommes en système dynamique hors équilibre et on peut se demander si les résultats trouvés ne sont pas dépendant de la vitesse de circulation des solutions.

Ainsi donc, la vitesse de circulation des solutions est sans effet sur les vitesses de réaction et, dans ce cas, on peut considérer que, vis-à-vis des échanges, les conditions d'équilibre sont remplies. Les lois de la thermodynamique des échanges peuvent alors être appliquées.

5 APPLICATION DE LA THERMODYNAMIQUE DES ÉCHANGES CATIONIQUES

Les équations d'échanges reposent le plus souvent sur l'application de la loi d'action de masse à l'équilibre d'échange. Mais l'objection principale à cette application, est que le système d'échange est hétérogène et la masse des particules ne prend aucune part à l'équilibre superficiel des ions.

Afin de lever cette objection et de prendre en compte les phénomènes électriques aux interfaces, après un tour d'horizon des différentes théories et équations...
applicables, Mabaret du Basty (1963) a eu l'idée d'associer la théorie de l'adsorption monocouche au concept de double couche de Gouy-Chapman. L'avantage de ce raisonnement est d'introduire un terme énergétique qui tient compte de l'affinité de l'ion pour l'argile et d'utiliser la notion de double couche qui caractérise la structure de ces minéraux.

5.1 Rappel de la théorie de Mabaret du Basty (1963)

Il est généralement admis (Baudracco, 1990) que les particules argileuses au contact d'une solution sont entourées d'une double couche ionique fixée sur leur surface. Il faut alors admettre que l'échange des cations se fait à travers cette double couche et que les conditions d'équilibre thermodynamiques dépendent de celles des deux couches. Cette prise en compte a été réalisée par Mabaret du Basty (1963).

Les calculs rappelés en annexe 1 montrent que la loi limite d'échange est égale à:

\[ \frac{C_1}{A} \cdot \frac{C_1}{V} = \frac{k_2}{k_1} e^{\frac{w_2 - w_1}{RT}} \frac{C_2}{C_1} \]

où:
- \( A \) est le nombre total de moles de cation participant à l'équilibre,
- \( V \) est le volume de la phase liquide,
- \( C_1 \) est la concentration molaire moyenne du cation compensateur dans la double couche,
- \( C_2 \) celle du cation contaminant au même lieu,
- \( k_1 \), la constante de désorption du cation compensateur,
- \( k_2 \), la constante d'adsorption du cation contaminant,
- \( w_2 - w_1 \) représente l'énergie d'échange.

Cette équation est celle d'une hyperbole qui admet comme asymptote la droite d'ordonnée \( \frac{A}{V} \), concentration maximum des cations compensateur en solution, c'est-à-dire, la capacité d'échange totale du système considéré (figure 4).

La forme de l'équation de la loi limite suggère l'usage des coordonnées \( \frac{C_1}{A} \cdot \frac{C_1}{V} \) et \( \frac{C_2}{C_1} \), c'est-à-dire

le choix des variables qui expriment les rapports des quantités ioniques dans la double couche et en solution.

La loi limite se transforme alors en une droite passant par l'origine des coordonnées et admettant la pente:

\[ p = \frac{k_2}{k_1} e^{\frac{w_2 - w_1}{RT}} \]

Cette droite est par définition une asymptote de l'isotherme d'échange réelle.

Figure 4. Représentation schématique de l'hyperbole représentant la loi limite d'échange.

Ainsi, à condition de pouvoir admettre que la concentration en électrolyte contaminant est assez forte pour que les hypothèses développées en annexe 1 soient applicables, l'isotherme expérimentale permet une estimation directe de l'énergie d'échange \( W_2 - W_1 \).

On a vu que:

\[ k = \frac{\lambda}{\lambda'} \sqrt{\frac{RT}{2\pi m}} \]

où: \( \lambda \) est la constante de vitesse d'adsorption; \( \lambda' \), la constante de vitesse de désorption; \( R \), la constante des gaz parfaits; \( T \), la température; \( m \), la masse du cation échangé.

Le rapport des constantes \( \frac{k_2}{k_1} \) est alors égal à

\[ \frac{k_2}{k_1} = \frac{\lambda_2}{\lambda_1} \left( \frac{\lambda_1}{\lambda'} \sqrt{\frac{RT}{2\pi m_2}} \right) \]

\[ k_1 = \frac{\lambda_1}{\lambda'} \sqrt{\frac{RT}{2\pi m_1}} \]

Si l'on peut admettre que le rapport des constantes des vitesses d'adsorption et de désorption des cations compensateur et contaminant est égal à 1, alors:

\[ k_2 = \frac{\lambda_2}{\lambda_1} \left( \frac{\lambda_1}{\lambda'} \sqrt{\frac{RT}{2\pi m_2}} \right) \]

et:

\[ p = \sqrt{\frac{m_1}{m_2}} e^{\frac{w_2 - w_1}{RT}} \]

Par la mesure de la pente de la droite, on peut donc estimer l'énergie d'échange: \( W_2 - W_1 \).

5.2 Application de la loi limite aux résultats expérimentaux

A partir de la loi limite précédemment définie, nous
avons, pour chacun des échanges étudiés, tracé les isothermes théoriques et expérimentales d'échange (figure 5).

On constate que pour trois d'entre eux, Ca++/Mg++, K+/Na+ et Na+/Ca++, les isothermes théoriques et expérimentales sont voisines, elles sont assez différentes par contre pour l'échange Mg++/K+. Il est à noter que du Basty obtient, dans des conditions d'équilibre habituellement rencontrées dans les échanges, exactement les mêmes résultats.

Ce comportement particulier pourrait provenir de la différence des vitesses d'adsorption et de désorption des deux cations. En effet, dans nos essais, la vitesse d'adsorption du magnésium est environ deux fois plus lente que la vitesse de désorption du potassium. Ceci a pour effet de laisser la surface solide nue pendant un temps donné, entraînant des variations locales du potentiel de surface et de double couche. Dans ce cas, les conditions d'équilibre ne sont plus respectées et, de plus, on s'écarte des conditions normales d'échange qui supposent qu'un cation compensateur est automatiquement remplacé par un cation contaminant.

Pour les échanges inverses, la correspondance entre isothermes théoriques et expérimentales est nettement moins bonne, sans doute à cause du cycle expérimental adopté qui voit, chaque cycle inverse précédé de plusieurs essais après le cycle direct.

Conformément à la théorie, les pentes des transformées linéaires des isothermes d'échange des cations de même valence sont voisines de 1. Elles sont voisines de 1,5 dans les échanges mono/divalent et de 0,65 dans le sens di/ménonovalent (tableau 4).


<table>
<thead>
<tr>
<th>Echange</th>
<th>Pentes</th>
<th>Énergies (KJ/mole)</th>
<th>Echange</th>
<th>Pentes</th>
<th>Énergies (KJ/mole)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca++/Mg++</td>
<td>0,98</td>
<td>-0,66</td>
<td>Mg++/Ca++</td>
<td>1,02</td>
<td>+0,66</td>
</tr>
<tr>
<td>Mg++/K+</td>
<td>0,66</td>
<td>-2,12</td>
<td>K+/Mg++</td>
<td>1,52</td>
<td>+2,13</td>
</tr>
<tr>
<td>K+/Na+</td>
<td>0,98</td>
<td>-0,70</td>
<td>Na+/K+</td>
<td>1,10</td>
<td>+0,88</td>
</tr>
<tr>
<td>Na+/Ca++</td>
<td>1,53</td>
<td>+3,41</td>
<td>Ca++/Na+</td>
<td>0,65</td>
<td>-3,42</td>
</tr>
</tbody>
</table>

Les énergies d'échange sont du même ordre de grandeur pour les échanges de cations de même valences (0,66 KJ/mole pour l'échange Ca++/Mg++ et Mg++/Ca++, 0,70, pour K+/Na+ et 0,88, pour Na+/K+). Elles sont de 2,12 KJ/mole pour les échanges Mg++/K+ ou K+/Mg++, 3,41 KJ/mole pour Na+/Ca++ ou Ca++/Na+.

Dans le cas de l'échange d'une montmorillonite-Ca/NH₄⁺, du Basty (1963), obtenait une énergie de 2,93 KJ/mole dans le sens Ca++/NH₄⁺, de 2,72 KJ/mole, dans l'échange inverse. Ces valeurs sont un peu différentes de celles qui nous avons obtenus dans ce même sens, mais il est vrai, avec un couple différent et, surtout, dans des conditions expérimentales très différentes. Toutefois, elles sont suffisamment voisines pour considérer que l'approche de du Basty (1963), peut très bien être appliquée à nos conditions dynamiques d'essais.

Si l'on rapproche maintenant, les énergies d'échange et les variations de la perméabilité (tableau 5), on se rend compte de l'analogie de comportement et du lien qu'il y a entre ces deux paramètres.

Tableau 5. Valeurs des énergies d'échange et des variations de la perméabilité obtenues au cours des échanges examinés.

<table>
<thead>
<tr>
<th>Echange</th>
<th>ΔΔk_{s12} (%)</th>
<th>Energies (KJ/mole)</th>
<th>Echange</th>
<th>ΔΔk_{s21} (%)</th>
<th>Energies (KJ/mole)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca++/Mg++</td>
<td>+5,63</td>
<td>+0,66</td>
<td>Mg++/Ca++</td>
<td>-6,27</td>
<td>-0,66</td>
</tr>
<tr>
<td>Mg++/K+</td>
<td>-6,91</td>
<td>-2,12</td>
<td>K+/Mg++</td>
<td>+3,41</td>
<td>+2,13</td>
</tr>
<tr>
<td>K+/Na+</td>
<td>-5,39</td>
<td>-0,70</td>
<td>Na+/K+</td>
<td>+2,21</td>
<td>+0,88</td>
</tr>
<tr>
<td>Na+/Ca++</td>
<td>+4,65</td>
<td>+3,41</td>
<td>Ca++/Na+</td>
<td>-9,47</td>
<td>-3,42</td>
</tr>
</tbody>
</table>

On voit, en effet, que les diminutions de la perméabilité correspondent à une énergie d'échange négative (c'est le cas des échanges Mg+++/K+).
Figure 5. Isothermes hystéro-élastiques et expérimentales traitées pour chacun des échanges étudiés.
K⁺/Na⁺, Mg⁺⁺/Ca⁺⁺ et Ca⁺⁺/Na⁺⁺), les augmentations, à une énergie positive (c'est le cas de Ca⁺⁺/Mg⁺⁺, Na⁺/Ca⁺⁺, K⁺/Mg⁺⁺ et Na⁺/K⁺).

Si la taille des cations ne semble pas avoir d'influence sur la perméabilité, celle-ci par contre, paraît sensible à l'affinité des cations pour l'argile. Ainsi, la libération d'énergie lors de l'échange cation/cation entraîne une diminution de la perméabilité, l'adsorption d'énergie, une augmentation de la perméabilité. On peut penser que cette énergie libérée ou adsorbée est dans la solution circulante, ce qui augmente ou réduit l'énergie interne du fluide et donc, les forces de viscosité et par là, le débit fluidé.

6. CONCLUSION

Nous avons examiné le comportement d'un échantillon du grès de Berea soumis à percolation successives de solutions de CaCl₂, MgCl₂, KCl et NaCl à l=0.01.

Les résultats montrent une étroite relation entre les variations de la perméabilité et les phénomènes d'échange cationiques au niveau des argiles. Les vitesses d'échange cationiques sont égales pour les cations de mêmes valences, alors que pour les cations de valences différentes, les vitesses d'adsorption ou de désorption du cation monovalent sont toujours plus grandes que celles des cations bivalents. Les isothermes expérimentales et les isothermes calculées à partir du modèle thermodynamique de du Basty (1963) sont très voisines pour les échanges entre cations de mêmes valences, elles divergent par contre dans le cas des échanges entre cations de valences différentes. Les énergies d'échange calculées sont en bon accord avec celles qui sont obtenues à l'équilibre. Ces phénomènes ont des conséquences pratiques importantes pour toutes les roches pouvant être au contact de solutions différentes. En effet, un même milieu peut être perméables à certaines d'entre elles et étanches à d'autres. C'est une donnée fondamentale dont il faut tenir compte pour toute implantation d'ouvrage et dans les problèmes d'environnement.

REFERENCES


ANNEXE 1: APPLICATION DE LA THEORIE DE LA DOUBLE COUCHE AUX ECHANGES CATIONIQUES SUR LES ARGILES (du Basty, 1963)

La double couche suppose l'existence d'une couche adsorbée à la surface des argiles et d'une couche diffuse, dite de Gouy, dans laquelle la distribution des ions n'est pas homogène. Pour concilier échange et structure des particules d'argile dans une solution, Mabaret. du Basty (1963) a considéré que l'adsorption sur la surface de l'argile obéissait à la loi de Langmuir et que la distribution des cations dans la couche diffuse était régie par l'équation de Maxwell-Boltzmann.

1 Equilibres ioniques superficiels

1.1 Equilibre monoionique

Pour un équilibre monoionique, la combinaison de la loi d'adsorption de Langmuir et de la loi de distribution des vitesses de Maxwell, permet d'aboutir à l'équation d'équilibre superficiel monoionique:

\[ Q_0 e^{-\frac{\theta_0}{T}} = k (1 - \theta) C_d \]  \hspace{2cm} (A.1)

\( \theta_0 \) es le degré de recouvrement de la surface solide, \( Q \), l'énergie moléaire d'adsorption, \( C_d \), la concentration ionique molérale au voisinage de la surface, \( k \), un paramètre lié à l'échange.

\( k \) est en effet égal à:

\[
 k = \frac{\lambda R}{2N m^2} \sqrt{\frac{T}{2\pi m}}
\]

où:

\( \lambda \) est la constante de vitesse d'adsorption; \( \lambda' \), la constante de vitesse de désorption; \( R \), la constante des gaz parfaits; \( T \), la température; \( m \), la masse du cation.
échangé.

1.2 Équilibre ionique binaire

Quand l’argile est mise au contact d’un électrolyte, la phase superficielle contient à la fois le cation compensateur qui appartient à l’argile et le cation contaminant apporté par l’électrolyte. On a alors un équilibre superficiel binaire dont chaque constituant peut être caractérisé par l’équation de l’équilibre monoionique à condition de considérer que le comportement de chaque espèce ionique est identique à celui qu’elle aurait si elle était seule.

La fraction non recouverte étant dans les deux cas égales à $1 - (\theta_1 + \theta_2)$, les relations d’équilibre s’écrivent:

$$\theta_1 e^{-\frac{Q_1}{RT}} = k_1 \left[1 - (\theta_1 + \theta_2)\right] C_{d1} \quad (A.2)$$

et

$$\theta_2 e^{-\frac{Q_2}{RT}} = k_2 \left[1 - (\theta_1 + \theta_2)\right] C_{d2} \quad (A.3)$$

1.3 Energetique de l’échange d’ions

On démontre que la quantité d’énergie mise en jeu au cours de l’adsorption d’un cation est égale à:

$$Q = W - F z \Psi_d$$

$W$, est l’énergie spécifique qui serait mise en jeu si les effets de répulsion électrique étaient nuls; c’est donc l’énergie spécifique molaire libérée par l’adsorption des cations quand la couche diffuse n’existe pas, c’est-à-dire dans le cas limite de l’argile saturée en électrolyte.

$F$, est la constante de Faraday,

$z$, la valence du cation,

$\Psi_d$, le potentiel au point considéré.

Dans un équilibre d’adsorption binaire, la différence des énergies $W_2$ et $W_1$ qui correspondent aux constituants ioniques de type 2 et de type 1, est égale à l’énergie libérée à la limite, par la substitution d’un cation d’un type donné par un autre cation d’un autre type sur le même site d’adsorption superficiel. Elle est donc égale à l’énergie d’échange des cations des deux types.

2 Loi limite de l’échange d’ions

Dans une solution monoionique la loi de distribution de Boltzmann peut s’écrire:

$$C = C_0 e^{-\frac{F_{\text{exp}}}{RT}} \quad (A.4)$$

Si l’on affecte l’indice 2 au cation contaminant, on peut alors écrire:

$$C_{d1} = C_1 e^{-\frac{F_{\text{exp}}}{RT}} \quad (A.5)$$

et:

$$C_{d2} = C_2 e^{-\frac{F_{\text{exp}}}{RT}} \quad (A.6)$$

En portant ces deux équations dans les expressions correspondantes (A.2) et (A.3), on a:

$$\theta_1 e^{-\frac{W_1}{RT}} = k_1 \left[1 - (\theta_1 + \theta_2)\right] C_1 \quad (A.7)$$

$$\theta_2 e^{-\frac{W_2}{RT}} = k_2 \left[1 - (\theta_1 + \theta_2)\right] C_2 \quad (A.8)$$

Ainsi, l’équilibre ionique superficial ne dépend pas de l’état de la couche diffuse mais simplement des concentrations ioniques molaires de chaque cation au cœur de l’électrolyte.

En divisant les deux équations précédentes membre à membre, on a:

$$\frac{\theta_2}{\theta_1} = \frac{k_2 e^{-\frac{W_2-W_1}{RT}} C_2}{k_1 C_1} \quad (A.9)$$

Quand la concentration en électrolytes augmente, l’extension de la couche diffuse diminue rapidement et, à la limite, le rapport des quantités d’équivalents de chaque cation retenus dans la double couche devient égal à: $\frac{\theta_2}{\theta_1}$.

Sachant que chaque équivalent de cation contaminant retenu dans la double couche libère un équivalent de cation compensateur, le rapport $\frac{\theta_2}{\theta_1}$ peut s’exprimer, à la limite, en fonction de la concentration $C_1$ du cation compensateur au sein de l’électrolyte:

$$\frac{\theta_2}{\theta_1} \rightarrow \frac{C_1}{A - C_1} \quad (A.9)$$

Il en résulte que la loi limite de l’échange d’ions à laquelle on aboutit lorsque la concentration en électrolytes devient importante, peut s’écrire:

$$\frac{C_1}{A - C_1} = \frac{k_2 e^{-\frac{W_2-W_1}{RT}} C_2}{k_1 \frac{V}{C_1}} \quad (A.10)$$

$A$, est le nombre total de moles de cation participant à l’équilibre,

$V$, est le volume de la phase liquide,

les autres paramètres sont déjà définis.
Effect of joint planes on physico-mechanical behaviour of rocks – A model approach

Modélisation de l’effet des discontinuités sur le comportement mécanique des roches

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ABSTRACT: The paper discusses the effect of joint planes on physico-mechanical behaviour in weak materials of silica sand, Plaster of Paris, mica powder, borax mixture under controlled loading conditions. The results obtained from these models suggest that the strength reduces as the number of jointing increases. The stress-strain behaviour of model indicates that the intact model have higher stress than the jointed model. These properties will be supplemented by the weakening coefficient to get the mass property of rocks, which can be used in the field and may significantly reduce time and costs of experiments.

RÉSUMÉ: La communication traite l'effet des surfaces de joints sur le comportement physico-mécanique de matériaux tendres (mélange de sable siliceux, plâtre de Paris, mica en poudre et borax) soumis à des conditions de charge contrôlées. Les résultats montrent que la résistance réduit avec l'augmentation de joints. Le comportement contrainte-déformation du modèle indique que le modèle intacte présente une résistance plus élevée que le modèle avec des joints. En introduisant un coefficient d'affaiblissement pour décrire les propriétés du massif, l'auteur arrive à la conclusion que les résultats correspondent au comportement du massif et que, donc, des épaisses de temps et de budget sont possibles.

1 INTRODUCTION

The deformational properties of jointed rockmass are significantly different from that of intact rock. The mechanical properties of jointed planes are essential for assessing the stability or instability of the rockmass. The objective of the model study therefore, should be the effect of planar discontinuities on the strength and deformability of a rockmass.

An in-depth knowledge of the physico-mechanical properties of rockmass is primary requisite for planning various mining activity right from geological exploration to mineral beneficiation. It has great significance in the design of rock structures both underground and surface, drilling, selection of explosives, blasting, etc. (Lajai, 1967; Lena et al., 1978; Singh and Singh, 1975).

Structural discontinuity present within the rockmass make it highly anisotropic. Their presence affects the strength and deformability characteristics of the rockmass to a great extent.

There are various method and approaches for obtaining a jointed sample from field such as drilling through jointed strata or producing artificial joints in the laboratory (Goodman, 1976). Modelling of jointed rockmass is a quite common practice in engineering geology. It is mostly based on strength properties, angle of friction, model materials and cohesion.

2 MODELLING OF JOINTED ROCKMASS

The modelling involved development of artificial material, the response of which could be identical to proto system upto a point of failure (Singh, 1986). Several researchers have described different methods for the simulation of jointed rockmass in a model.

Punagalli (1963, 1967) incorporated the major discontinuities in the model itself whereas minor discontinuities were simulated by reducing the strength of the solid model material. However, it is not appropriate to simulate joint like this because they do not occur in the same fashion as the prototype.

A number of researchers simulated joints by pilling up blocks within the frame
(Lajtai, 1967; John, 1969; Brown and Trollope, 1970; Ergun, 1970). This technique is not realistic because spacing between blocks are relatively more than the prototype. Also, the joint distribution is absolutely uniform in the model, contrary to the field distribution of joints. For these reasons, joints in a rockmass are usually described as an assemblage rather than individual (Derschonitz and Einstein, 1988). The peak and residual strength of the piled blocks are much smaller as compared to prototype. Glue was inserted between two layers to avoid joints gaps but it is a crude method of simulation of joints. It is also expensive and not systematic method for simulation of jointed rockmass in the model. Joint configuration was only schematic but not realistic as it occurs in nature (John and Rantenstrach, 1979). Barton (1974) developed a technique to simulate jointed rock by generating tension crack indirectly with the help of a guillotine machine in two dimensional models.


3 STATEMENT OF PROBLEM

The present study deals with the effect of joint orientation on the rockmass properties. An attempt was also made to study the influence of composition of material of the model as well. To know the effect of physico-mechanical properties from intact to jointed rockmass, different strength properties were determined eg. compressive strength, tensile strength, shear strength, density, angle of friction. The relationship in between these parameters was also analysed. It was also decided to know the stress-strain behaviour of jointed and intact models.

4 MATERIAL AND METHODS

The model materials can be composed of actual rock or a combination of rock with some cementing material. It may also be equivalent model material which normally contain silica sand, mica powder, plaster of Paris, borax powder in different proportion. In the present study, silica sand, IP grade plaster of Paris, mica flakes and borax were used as equivalent material.

Different types of small scale models were prepared for different tests. The model making procedure was same as described by Singh and Singh (1992). Table 1 gives an idea about the types of specimen prepared for different tests.

Different types of material proportions were casted in different models for different type of testing. The specimens were left for 15 days to let them dry. Then, the samples were tested on universal testing machine as per specification of International Society of Rock Mechanics (Brown, 1982). The properties determined were density, uniaxial compressive strength, tensile strength, shear strength and angle of friction. The strength properties of different compositions are given in Table 2.

Load and deformation curves were obtained from servo-controlled stiff testing machine for unjointed and jointed models. The specimen were subjected to a uniform strain rate of 2500 unit strain/second. This reduced the shattering of the specimen and enabled to record the dropping part of the stress-strain curve. By the help of load and deformation curves, stress-strain curves were computed for intact and jointed model. The amount of joints were 60° and 75°.

<table>
<thead>
<tr>
<th>Test</th>
<th>Type of specimen</th>
<th>Intact or jointed</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive</td>
<td>Cube</td>
<td>Intact</td>
<td>12 cm cube</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jointed (J1)</td>
<td>12 cm cube with 6 layers of 2 cm each</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jointed (J2)</td>
<td>12 cm cube with 6 layers of 2 cm each, notch time was 40 min having 60 inclined joints (Joint spacing 2.5 cm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jointed (J3)</td>
<td>75 inclined joints (Joint spacing 2.5 cm)</td>
</tr>
<tr>
<td>Tensile</td>
<td>Thin disc</td>
<td>Unjointed</td>
<td>5 cm dia, 2.5 cm thick</td>
</tr>
<tr>
<td>Shear &amp; angle</td>
<td>Cylindrical</td>
<td>Unjointed</td>
<td>5 cm dia, 10 cm length</td>
</tr>
</tbody>
</table>

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### Table 2. Physico-mechanical properties of equivalent materials (EM)

<table>
<thead>
<tr>
<th>Material group</th>
<th>Composition %</th>
<th>Volume or Mechanical properties, MPa</th>
<th>Density</th>
<th>Angle of friction (%)</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td>PP SS Mica</td>
<td>Compressive strength</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>IA</td>
<td>15 10 75</td>
<td>J</td>
<td>0.032</td>
<td>0.008</td>
</tr>
<tr>
<td>IB</td>
<td>15 20 65</td>
<td>J</td>
<td>0.072</td>
<td>0.014</td>
</tr>
<tr>
<td>IC</td>
<td>15 30 55</td>
<td>J</td>
<td>0.028</td>
<td>0.005</td>
</tr>
<tr>
<td>ID</td>
<td>15 40 45</td>
<td>J</td>
<td>0.104</td>
<td>0.019</td>
</tr>
<tr>
<td>IE</td>
<td>15 50 35</td>
<td>J</td>
<td>0.046</td>
<td>0.005</td>
</tr>
<tr>
<td>IF</td>
<td>15 60 25</td>
<td>J</td>
<td>0.121</td>
<td>0.019</td>
</tr>
<tr>
<td>IG</td>
<td>15 70 15</td>
<td>J</td>
<td>0.147</td>
<td>0.022</td>
</tr>
</tbody>
</table>

W = Unjointed  J = Jointed

### 5 Results and Discussion

Results obtained from the tests are given in Table 2. It was observed that in intact samples, the maximum failure strength was found to be 0.284 MPa whereas minimum failure strength was calculated 0.032 MPa for the composition II, and IA (Table 2). The failure strengths were further reduced due to presence of joint planes. It indicates that the presence of joint planes and percentage of mica decreases the strength of the model. As the joint inclination with horizontal was increased the strengths were further reduced which leads to the fact that the strength of jointed models highly depends upon the orientation of joints also.

#### 5.1 Compressive strength vs density

A graph between compressive strength and density is plotted, which is not showing a definite trend, but it seems that as the density increases, the compressive strength increases. The increase in density indicates compaction of the materials. It is also evident that the compactness is not only the governing factor of the strength but also the nature of individual grain and cementing material which is also affecting the strength directly (Fig. 1).

#### 5.2 Compressive strength vs tensile strength and shear strength

It is estimated that rocks are very weak in tension. Therefore, tensile strength of the rockmass is always less than shear or compressive strength. There is no definite correlation between them as shown in

![Fig. 1. Plot between density and compressive strength.](image-url)
Figures 2 & 3. Same is true for shear strength vs compressive strength. Fig. 2 shows that the compressive strength increases as the tensile strength increases in a linear manner.

Fig. 2. Plot between compressive strength and tensile strength.

Fig. 3. Plot between compressive strength and shear strength.

Table 3. Mean stress-strain values of EM samples

<table>
<thead>
<tr>
<th>Strain %</th>
<th>I_A</th>
<th>I_B</th>
<th>I_C</th>
<th>I_D</th>
<th>I_E</th>
<th>I_G</th>
<th>IIA</th>
<th>IIB</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unjointed Samples:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>0.83</td>
<td>8.58</td>
<td>14.30</td>
<td>54.37</td>
<td>17.17</td>
<td>57.23</td>
<td>57.23</td>
<td>60.09</td>
<td>143.09</td>
</tr>
<tr>
<td>1.66</td>
<td>11.44</td>
<td>25.75</td>
<td>72.57</td>
<td>35.77</td>
<td>135.95</td>
<td>157.39</td>
<td>171.70</td>
<td>314.79</td>
</tr>
<tr>
<td>2.50</td>
<td>14.30</td>
<td>42.92</td>
<td>78.69</td>
<td>72.97</td>
<td>200.33</td>
<td>186.01</td>
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<tr>
<td>3.33</td>
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<td>100.16</td>
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<td>8.33</td>
<td>40.06</td>
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<td>143.09</td>
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<tr>
<td><strong>Jointed Samples:</strong></td>
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<td>17.11</td>
<td>30.47</td>
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<td>8.33</td>
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<td>31.27</td>
<td>49.21</td>
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<td>9.16</td>
<td>18.21</td>
<td>32.77</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tbody>
</table>

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Fig. 4. Stress-strain curve of equivalent material at 75° joint set

5.3 Stress-strain relationship

The modulus of elasticity was determined with the help of material testing machine. Different stress-strain values were obtained by the deformation and load curves at different position of deformation. Curves were drawn from the computed values of stress-strain (Table 3).

From the curves, it is clear that confining stress at which transition took place is largest for intact model and smaller for 75° joints (Fig. 4). In addition, there is systematic increase of this transition stress (Fig. 5).

The initial flat portion curves in jointed model is probably the result of the closing of micro-cracks, closing of joints and setting due to loading apparatus.
6 CONCLUSIONS

Importance of modelling seems to be obvious for planning and design for mining and civil engineering construction. It may significantly reduce time and cost of experiments.

As the joint orientation from the horizontal is increased, the failure load and strength decreases in the same model. In the range of the normal stress, the intact sample fail by fracturing, generally at 45°, while jointed model failed by sliding along the favourably inclined pre-existing joint at lower stresses. Relationships between tensile, shear and compressive strength are different for each model type and no uniformity could be found in any one. On density - compressive strength correlation, it can be seen that compressive strength increases with increasing density.

Thus, the mechanical properties tests are valuable in understanding and in predicting the response of jointed rockmass in field.
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Goodman, R.E. 1976. Method of geological engineering in discontinuous rock, west St Paul, MN.


Mineralogy, crystallinity and activity of Lisbon’s basaltic soils
Minéralogie, cristallinité et activité des sols basaltiques de Lisbonne

Luís Melo Duarte, Isabel Moitinho de Almeida & Silvério Prates
Department of Geology University of Lisbon, Portugal

ABSTRACT: Neocretaceous basaltic volcanism in Lisbon-Mafra region, produced thick lava flows interlayered with pyroclastic beds. The weathering of the Lisbon Volcanic Complex (LVC) originates typical alteration soils. In this article the textural and mineralogical characteristics of these cohesive soils are analysed in relation with geotechnical parameters.

RESUMÉ: L’activité volcanique neocréatrice a produit dans la région de Lisbonne-Mafra des épaisseurs coulées basaltiques, intercalées des niveaux pyroclastiques. Les phénomènes de météorization des roches du Complex de Volcanique de Lisbonne (LVC) ont originé des sols d’altération très typiques. Dans ce travail on analyse la texture et les caractéristiques minéralogiques de ces sols et les relations avec le comportement géotechnique.

1. INTRODUCTION

1.1 Objectives

The present study is mainly focused on the properties of the cohesive alteration soils from the Lisbon’s Volcanic Complex, particularly in relation with textural parameters, clay mineralogy and crystallinity and their influence on Atterberg limits and colloidal activity.

1.2 Geology

During the Neocretaceous the Lisbon-Mafra region was the centre of important subaerial basaltic volcanism, which originated the geologic formations nowadays known as the Lisbon’s Volcanic Complex (LVC). These events, related with the opening of the North Atlantic, had their beginning (Ribeiro et al., 1979) some 70 million years ago, as shown by several isotopic determinations and geologic relations.

At the present time, the LVC outcrops discontinuously through an area of about 200 km², reaching the region of Torres Vedras (Fig. 1). The majority of the eruptive centres seems to be located in the Cheléiros-Mafra region, where the remains of several important chimneys were found (Serralheiro, 1978).

From a petrographic point of view (Perez, 1985 and Alves et al., 1980) the LVC presents some diversity, from basalts, largely predominant, to rhyolites. In addition to lava flows, pyroclastic beds and chimneys, several sills and dikes also occur.

The geologic formations which constitute the LVC correspond, in general, to thick lava flows interlayered with pyroclastic beds. The evident predominance of lava flows shows that the volcanism in this area was mainly effusive. The presence of paleosols, alluvium beds and remains of old lagoons amongst the volcanic rocks and the existence of weathered zones in the older lava flows are good indicators of intermittence in the volcanic activity and of the early beginning of weathering and pedogenesis.

Attending to the region climate, the present alteration degree exhibited by these rocks, particularly in the more superficial zones, may be considered, in general, high. Cohesive soils, the object of the present study, are often associated to the altered pyroclastic layers. The weathering of LVC produced agricultural soils of excellent characteristics that progressively have been occupied for the urban development that is taking place in the surroundings of Lisbon.

2. METHODOLOGY

The sample preparation for the routine identification tests followed the Portuguese specification LNEC E 195-1966.
Fig. 1 - Lisbon's alpine volcanism: LVC - Lisbon Volcanic Complex (basalts); SSC - Sintra Subvolcanic Complex (granites)
Fig. 2 - Texture of Lisbon's basaltic soils.

2.1 Grain-size distribution analysis

In order to evaluate the soils texture, samples were in first place sieved through a sieve # 200 (ASTM).

The silt and clay fraction was then analysed by a laser particle sizer (Fritsch Particle Sizer - Analysette22).

2.2 Determination of Atterberg limits

The proceedings followed for the determination of the Atterberg limits were in accordance with the Portuguese standard NP-143 (1969).

2.3 Clay mineralogy

The clay fraction (< 2µm) mineralogical composition was determined by X-ray diffraction, using the Kα radiation of Cu, with a Philips PW1011 equipment.

Oriented aggregates were prepared and the diffractograms of three types of sample preparations (normal, glycolated with ethylene glycol vapour and heated to 550°C) analysed.

Semi-quantitative evaluation of clay mineralogy was made by measuring the areas of the (001) peaks in the glycolated aggregates diffractograms and correcting the obtained values with the specific reflectance of the mineral.

As the predominant minerals are smectites their crystallinity was estimated following the method proposed by Biscaye (in Thorez 1976), using the v/p index.

In order to identify the specific type of smectites present, X-ray diffraction of random powder preparations and the Green-Kelly's Li-test (in Thorez, 1975) was utilised.

3. RESULT ANALYSIS

The tested samples correspond chiefly to soils resulting from the weathering of pyroclastic layers.

3.1 Texture

From a textural point of view the studied samples reveal significant similarity between themselves. Clay fraction does not exceed 21% while the silt fraction often surpasses 75%. Only in few samples the coarse fraction exceeds 30%.

In fact, as shown in the triangular textural diagram presented in Fig. 2, these soils can be classified, in general, as clayey silts.

3.2 Atterberg limits

Obtained values for the Atterberg limits (Table 1) usually agree with the soils textural composition. In the Plasticity Chart (Fig. 3) the large majority of the soils is located in the domains of the high to low plasticity silts (MH-ML).

| Table 1. Lisbon's basaltic soils: Atterberg limits statistics |
|-----------------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Liquid Limit (wL) | Plastic Limit (wp) | Shrink. Limit (ws) | Plasticity Index (Ip) | Shrink. Index (If) |
| Mean | 52.6 | 39.3 | 20.0 | 13.3 | 32.6 |
| St. d. | 14.3 | 11.0 | 3.1 | 5.6 | 14.5 |
| Min. | 29.0 | 19.0 | 15.0 | 7.0 | 12.0 |
| Max. | 92.0 | 66.0 | 28.0 | 26.0 | 73.0 |
| Cv | 0.27 | 0.28 | 0.15 | 0.43 | 0.45 |

*29 tested samples.

Correlation analysis (Fig. 4 and table 2) show the relationships between liquid limit (wL), plastic limit (wp), shrinkage limit (ws), plasticity index (Ip=wL-wp) and shrinkage index (If=ws-wL).

| Table 2. Correlation matrix |
|--------------------------------|-----------------|-----------------|-----------------|-----------------|
| wL | - | wp | ws | Ip |
| wp | 0.930 | - | - | - |
| ws | 0.037 | 0.285 | - | - |
| Ip | 0.715 | 0.407 | -0.450 | - |
| If | 0.978 | 0.857 | -0.174 | 0.799 |

*29 tested samples.

Accordingly, the higher correlations found suggest the following equations:
Fig. 3 - Lisbon's basaltic soils. Plasticity Chart.

Fig. 4 - Lisbon's basaltic soils. Correlations between plasticity parameters.
\[ w_p = 1.81 + 0.71 w_L \]  \hspace{1cm} (1)
\[ I_p = -1.81 + 0.29 w_L \Rightarrow I_p = 0.29 (w_L - 6.3) \]  \hspace{1cm} (2)
\[ I_R = -19.6 + 0.99 w_L \]  \hspace{1cm} (3)
\[ I_R = -12.0 + 1.13 w_p \]  \hspace{1cm} (4)
\[ I_R = 5.77 + 2.01 I_p \]  \hspace{1cm} (5)

3.3 Colloidal activity (Ac)

Though showing some variability, the determined values for the activity parameter \( Ac \) (Ip / %<2μm) allowed us to classify these soils, using the criteria of Skempton (1953), as "active" (Ac > 1.25) to "normal" (0.75 < Ac < 1.25).

In Table 3 some statistical data referring colloidal activity are presented.

Table 3. Activity and clay crystallinity of Lisbon's basaltic soils.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Cryst. (y/p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>1.44</td>
</tr>
<tr>
<td>St. Deviation</td>
<td>1.03</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.47</td>
</tr>
<tr>
<td>Maximum</td>
<td>4.43</td>
</tr>
<tr>
<td>CV</td>
<td>0.72</td>
</tr>
</tbody>
</table>

* 29 tested samples.

3.4 Clay fraction mineralogy and crystallinity

As referred above, the mineralogical composition of the clay fraction present in these soils consists, almost exclusively, in smectites. However, in a few samples, some vestigial palygorskite was found. The smectites revealed to be beidellite, in general well crystallised. Prudêncio et al. (1990) refer also the presence of halloysite, however the soils tested by these authors resulted exclusively from the weathering of lava flows.

In Table 3 are presented some statistical data referring the crystallinity of the clay minerals found in the analysed samples.

4. DISCUSSION

The tested soils revealed to be clayey silts, presenting a plastic behaviour that may be considered typical of this type of materials. Attending to mineralogical composition of the clay fraction (smectites) an higher plasticity would, eventually, be expected. However we may not forget that the clay fraction only in few samples exceeded 15%, never surpassing 21%. Therefore, the obtained values can be considered as normal.

Concerning specifically the Atterberg limits, the variability found in these parameters should not be considered very significant. In the plasticity chart (Fig. 2) it is clearly seen that the large majority of the samples fall below the A-line, defining a relatively restricted domain. This fact is generally in accordance with the results obtained in previous works (Nascimento, 1954; Almeida, 1991 and Silva and Sobral, 1985), where samples of this type of soils were also tested.

The charts presented in Fig. 4 show good correlation between some of these parameters.

5. CONCLUSIONS

In this paper preliminary data obtained by the authors during the present study of cohesive soils of the LVC are presented. However, as a preliminary work, only few data are now available. The study in progress contemplates the analysis of a wider group of samples and the research of other parameters (geocchemistry, specific surface, expansibility, etc.).

As main conclusions we may consider:

1. The LVC's alteration soils are mainly silts including some clay and, not so often, a coarser fraction composed essentially by medium to fine sand and is also present.
2. In what concerns clay mineralogy, beidellite, in general well crystallised, is undoubtedly the predominant species.
3. In the tested samples the existence of a direct relation between crystallinity / Atterberg limits or crystallinity / Ip is not evident. In this case, plastic behaviour should be essentially associated with textural characteristics, as seen in the plasticity chart.
4. The obtained values for the Atterberg limits and $I_p$, even though not very homogeneous, do not present a real significant dispersion. The range of values, specially those showing lower $I_p$, agree, in general, with the soils texture.

5. Colloidal activity shows some variability and does not exhibit an obvious relation with clay crystallinity or plastic behaviour. A reason for this value dispersion could be the presence, in some of the tested soils, of clay clusters that would remain in the silt fraction.

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Lab shear tests in rock slope stability assessment
Essais de cisaillement au laboratoire pour l’analyse de la stabilité des pentes rocheuses

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ABSTRACT: Rock slopes' stability assessment often requires the use of simple methods. However, even the most simple graphical methods need the basic information about shear strength along the discontinuity planes. The simple lab method for determining this parameter is described in the first part of this paper. Using the shear strength parameter in the design of safety slopes during the road construction in Central Slovakia is shown in the application part of the contribution.

RÉSUMÉ: En évaluant la stabilité des versants rocheux surtout il faut souvent appliquer les méthodes de solution rapides et simples. Même les méthodes graphiques les plus simples exigent les informations de base sur la force de cisaillement des discontinuités. Dans la première partie de cet article, on présente les méthodes simples de laboratoire concernant la force de cisaillement. Les résultats acquis sont appliqués à la projection des versants fiables pour la construction des routes dans la région de la Slovaquie centrale.

1 INTRODUCTION

The construction of highways and railroads results in problems connected with stability of rock cuts and half cuts, especially in the mountainous areas. Solution to these problems is based upon the analysis of rock mass structure and its relation to the proposed orientation of the cut. Various graphical methods are used in this analysis. Advantages of all of them is the rapidity and simplicity. The design of technical remedial measures is based either on the results of the graphical analyses, or - in the more complicated cases - on the results of various stability computing methods.

One of the oldest and the simplest graphical methods is the stability analysis according to Markland (1972). This method locates all the discontinuities which decrease the rock slope stability. Moreover, very simple graphical result of this analysis leads to the prediction the type of slope failure - plane, wedge or toppling failure (Figure 1).

Although many new graphical stability methods were introduced or improved recently (Hock & Bray 1977, John & Deutsch 1974, Patton & Deere 1971 and many other authors), each needs the same range of basic input data - information about discontinuity orientation in the rock mass, the orientation of designed rock cut and values of strength on discontinuity planes.

Information about rock cut orientation follows from the highway or railroad design and orientation of principal rock mass discontinuities are obtained as a result of engineering-geological investigation of the given area, or, at least, during excavation works. The most problematic is the question of shear strength along the discontinuity planes. Usually the values are taken from geological literature (Barton & Choubey 1977, Hock & Bray 1977), or qualified estimations are used. Complicated and expensive in situ shear tests are used rarely - only in very important cases.

Since there was available a lot of drill cores from extensive engineering - geological investigation in Slovenské Rudohorie mountains
prepared bearing surface and discontinuity plane (Dp) was minimum 10 mm. Before testing the angle $\beta$ (dip of discontinuity plane) was measured. Measurements of discontinuity surface roughness were done using induction receiver. Because of the small discontinuity plane, the measurements (Figure 2B) were completed with detailed description and visual evaluation of discontinuity surface roughness.

Prepared sample of drill core with discontinuity was inserted into the loading jack. The friction between the sample and the loading jack was eliminated by the cylinder-bearing (Cb). The vertical strength ($F$) increased till the moment, when the permanent motion (shear) of the upper part of sample along the discontinuity surface occurred. The vertical loading strength ($F$) was divided into normal (N) and tangential (T) component - Figure 2C. Different proportions of normal and tangential strain depend on the dip angle ($\beta$) of the discontinuity plane. The different values of $\beta$ in each sample enabled the determination of the various proportions of N and T and thus the shear strength parameters for given rock mass. The resolution of the strength $F$ and the calculation

(investigations were carried out in connection with PSPP Ipe I and storage reservoir Málinec project preparation), we used this material and derived simple method for shear strength on discontinuity planes determination. The results of lab tests were used as the input data for rock slopes assessment during construction the new road.

2 DETERMINING THE SHEAR STRENGTH PARAMETRES ON DRILL CORES

The proposed lab method for determining the shear strength parameters on discontinuities planes is very simple and economical. It uses the drill cores whose compactness was broken by different inclination of the discontinuity planes.

The samples (drill cores with discontinuity) were prepared for testing according to Figure 2A. Bearing surface (Bs) was perpendicular to core axis and was prepared by cutting and whetting. The distance $h_1$ between artificial

Fig. 1 Principles of Markland's graphical method of rock slope stability evaluation. a - great circle presenting the slope face, Na - the pole of the slope face; $S_1$, $S_2$ - the poles of discontinuities, which may cause: $S_1$ - plane failure of the slope, $S_2$ - wedge failure, $S_3$ - toppling failure of the slope

Fig. 2 The scheme of lab shear test on discontinuity plane. A - the scheme of drill core sample, B - results of roughness measurement on the discontinuity surface, C - resolution of the loading strength $F$ to normal (N) and tangential (T) component. a - axis of the borehole, Bs - bearing surface of the sample, Dp - discontinuity plane, Cb - cylinder-bearing
The collection of 39 samples of crystalline rocks was examined according the above-mentioned laboratory method. The results of the tests are shown on Figure 3B. The line "a" is derived for the whole collection of samples, the lines "b" and "c" are typical for samples with average roughness on discontinuity planes less or more than 1.5 mm (smooth / rough planes). The difference of the friction angle values is not significant - from $35^\circ$ to $37^\circ$ (Hyáneková et al. 1991).

3 STABILITY ANALYSE OF ROAD ROCK CUTS

As mentioned above, the experimental rock material (drill cores) have been obtained from the engineering-geological investigation for two large hydrotechnical projects - storage reservoir Málinc, which is under the construction, and pumped-storage power plant (PSPP) Ipéf, which is in stage of preliminary investigation. The project of PSPP Ipéf is situated about 5 km NE from the storage reservoir Málinc. Both structures are located in the same geological environment - in the crystalline rock mass, which consists of granodiorites, granites, crystalline schists, gneisses and migmatites. Based on the rock environment similarities, we used the results of lab tests to solve the stability problems in the construction of the new road along the water reservoir Málinc - Figure 4.

3.1 Geographical and geological characteristics of the locality

Storage reservoir Málinc is situated in the valley of Ipéf river. The dam construction was finished in 1993 and the filling of the reservoir is currently underway. The construction of such a large storage (about 26.7 mil. m$^3$) resulted in supplemental construction, such relocating a road along the eastern bank of water reservoir. The length of the new road is 4.51 km.

As mentioned before, the line of road is situated in crystalline rock mass. From engineering-geological point of view the lithological heterogeneity is not as significant as the tectonic deterioration of the rock mass. The
opened mostly in the weathered and tectonically deteriorated crystalline rocks. Although optimum technical stabilizing methods were observed during the excavation, several problems were encountered, as well as the questions of the long-time stability of rock slopes. The stable conditions of road cuts in each section of the line were evaluated on the basis of geotechnical properties of the rock mass. Stability and the design of the optimum stabilizing methods had to be determined promptly during the excavation. Therefore, simple Markland’s graphical method was used.

The line of road was divided into 25 sections depending on geological conditions and type of road line (cut or embankment). Engineering-geological evaluation of each section was done regarding the physical and geotechnical state of the rock mass (including the results of lab shear tests on drill cores). Examples of engineering-geological evaluation and design of stabilizing measures in chosen sections of the road are described in the following chapter.

3.3 Engineering-geological evaluation and the design of technical remedial measures of the chosen sections

Section No 8 (1.71 - 2.06 km) - the cut of 12 m height and 65° dip was designed in weathered hybrid granodiorites with significant crush zone consisting predominantly of gneisses and mylonites. The most important joint set (S1 - Figure 5) is perpendicular to the road line direction and its influence on the stability conditions of the rock slope is not substantial. On the basis of lab tests, the characteristic angle of friction of 37° on discontinuity planes has been chosen - Figure 5. Because the results of graphical analysis show high degree of rock slope stability, very simple technical remedial measures were designed. The dip of the upper part of the slope was decreased and the lower part of the slope was protected against the fall of weathered blocks and lumps of rocks by the steel net. The general ratio of the slope is 2:1 now, and the toe of slope is covered by fallen weathered material - Figure 5.

3.2 Characteristics of the line of road

The line of the road leads into half cuts and cuts and rarely on the embankments. The cuts are

regional crush zone, which follows to the River Ipeľ valley, exerts the influence upon the physical state of the rock mass. This fact must be considered in rock cuts design.
Section No 13 (2.43 - 2.60 km) - the cut of 7 m height and 80° dip (after excavation) is situated in the hybrid granodiorites, which are intensively weathered especially in the upper part of the rock slope. Stability conditions of the slope are complicated mainly due to the joint set S1 (Figure 6) of the dip direction 220 - 230° and the dip 70 - 88°, which creates the condition for plane failure. The value of friction angle is 35°, characteristic for this type of rock, and it is too low for long-term stability. Therefore, despite of small height of slope, the stabilization measures were more exacting - they consisted of a retaining wall of 3 m height, which was constructed from the gabion baskets, filled by the rock debris - Figure 6. The upper part of the slope (in intensively weathered rocks and deluvial loams) was modified to the 1:1 ratio (or less), the berm was constructed, and the surface of slope was stabilized by hyroseed.

Section No 17 (3.2 - 3.28 km) - the cut was designed in one of the steepest slopes in Ipel River valley (the natural dip of slope was from 25 to 35°). The height of cut's right slope was about 16 m. In this part of the rock mass hybrid granodiorites predominate along with minor gneiss. Dominate joint sets may cause various types of the slope failure. The joint set S1 may cause plane failure, the sets S1, S2, S3 wedge failure and set S4 toppling failure of the slope. Characteristic angle of friction on discontinuity planes was considered as 37°. Heterogeneity of the rock mass structure and steepness of the natural slope lead to a remedial measures design - the slope was divided into three parts by berms, constructed in two levels. On the upper the gabion basket retaining wall was constructed. Slopes in the central and lower part were modified to the 2:1 and 3:1 ratio and reinforced by hyroseed - Figure 7.

Fig. 5 Section No 8 - the graphical analyse and final design of the slope. 1 - debris, 2 - weathered rocks, 3 - migmatites, 4 - significant joints, 5 - steel net, a, b - various degrees of the discontinuity poles concentration (b - max. concentration of poles)

Fig. 6 Section No 13 - the graphical analyse and final design of the slope. 1 - backfill, 2 - deluvial loams, 3 - weathered rocks, 4 - granodiorites, 5 - significant joints, 6 - gabion baskets, 7 - shape of the primary excavation
REFERENCES


4 CONCLUSIONS

The paper shows a modified method of solving the rock slope stability, based on the simple graphic method. The input data were obtained from the engineering-geological investigation and simple laboratory tests. The method of determination the shear strength parameters on the discontinuity planes, using drill cores, is described in more details. Stability analysis, based on these input data, suggests the most appropriate stabilizing measures for the rock slopes. Illustrated examples of simple practical solutions of these problems are used from the selected cuts in the new road construction in Central Slovakia crystalline rock mass.
The experience of using a compression-tension method for engineering geological investigations

L’expérience d’utiliser une méthode de compression-traction pour des investigations de géologie de l’ingénieur

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ABSTRACT: The compression-tension method has high accuracy of strength parameters of clayey soils with consideration of curvilinearity of Mohr’s circles envelope at engineering geological investigations and makes it possible to refuse the use of the plain shear and triaxial compression methods at short-term tests. The method is used in engineering geological tests to obtain standard strength parameters (cohesion and angle of inner friction) of clayey soils, to estimate stress state of clayey soils on slip slopes, clayey soils resistance to fracture and in other cases where tensile strength presence is possible in soils.

RÉSUMÉ: L’extension-compression est une méthode très précise pour la détermination des paramètres géotechniques du sol argileux à l’usage des investigations ingénieur-géologiques. Le recensement d’une enveloppe du cercle de Mohr donne la possibilité de l’essai vie sans déplacement de plan et sans compression triaxiale. La méthode a été employée pour l’identification d’une cohésion, d’angle de frottement interne et de résistance par traction du sol argileux, et pour l’évaluation d’état de tension dans une pente ébouleuse. Elle est applicable à d’autres situations avec des contraintes de traction dans le sol.

The compression-tension method has been developed in Russia and used for a long time to obtain accurate parameters of strength (cohesion and angle of inner friction) for clayey soils in any state.

![Diagram](image.png)

Fig.1 Mohr’s circles group and strength parameters, obtained by the use of the compression-tension method.

The method allows in an experimental way to get the intersection point of the Mohr’s circles envelope and axis of tangent stresses $\tau_n$ (Fig.1) in $\tau_n - \sigma_n$ coordinates and to construct the part of the envelope $A_1-B$ with the use of the experimental data but not by extrapolation and to obtain the clayey soils strength parameters taking into account the envelope curvilinearity of Mohr’s circles at the axis of tangent stresses.

Using conventional methods of tests (triaxial compression) the part of the A-B envelope is defined by extrapolation, which depending on the degree of the envelope curvilinearity, may bring to the different value of disagreement between strength parameters $C_0$ and $\varphi_0$ and their real values $C'_0$ and $\varphi'_0$.

In the use of plane shear the envelope curvilinearity is neglected because the process of its flattening into straight line takes place. The point of its intersection with the axis $\tau_n$ defines some average meanings of $C_0$ and $\varphi_0$.

The compression-tension method, used in Russia, doesn’t have these drawbacks.
Fig. 2 The scheme of stress distribution in the soil sample of special form during the test in the device of simultaneous compression-tension.

To construct Mohr's circle I (Fig.1) which cuts the axis of tangent stresses, the sample in the form of a special coil (Fig.2) is used in the tests. Under hydrostatic pressure P on the soil sample through the rubber cover within the limits of ABCD contour the following stresses appear in the central working part of the sample: \( \sigma_1 = \sigma_2 \) perpendicular to the sample axis, \( -\sigma_3 \) - along the sample axis.

The value \( -\sigma_3 \) is determined by the depth of the lateral surface recess and calculated by the formula

\[
-\sigma_3 = P \left( 1 - \frac{S_T}{S_0} \right)
\]

(1)

\( P \) - hydrostatic pressure, kg/sq.cm,
\( S_T \) - square of the face part of the sample, sq.cm,
\( S_0 \) - sample square in the mean cross-section, sq.cm.

The stress state is obtained in the soil sample:

\[
\begin{align*}
P &= \sigma_1 = \sigma_2 \\
\sigma_3 &< 0
\end{align*}
\]

(2)

According to the values \( \sigma_1 = \sigma_2 \) and \( -\sigma_3 \) in coordinates \( \tau_n - \sigma_n \) one can construct Mohr's circle (Fig.1).

To construct the part of the envelope A_1-B it is necessary to make a test of one more cylinder sample and destroy it under pressure in the same chamber.

The sample acquires the following stress state:

\[
\begin{align*}
P &= \sigma_1 = \sigma_2 \\
\sigma_3 &= 0
\end{align*}
\]

(3)

The test made according to this scheme is called "side reduction". Using Mohr's circle 2 we build the part of the envelope A_1-B and get accurate values of strength parameters \( C_0 \) and \( \phi_0 \).

The method permits to make simple and reliable tests of clayey soils on simple tension by limiting side surface of the sample in the form of a coil by rigid split yoke. In this case the hydrostatic pressure created on the ABCD contour influences only the parts of the sample AB and CD making a tensile force along its axis. The stress state in the sample can be represented by the following stresses:

\[
\begin{align*}
\sigma_1 &= \sigma_2 = 0 \\
\sigma_3 &< 0
\end{align*}
\]

Practically, using 3 main schemes of sample loading it is possible in the experimental way to construct the whole part of the envelope DB (Fig.1) from a circle of simple tension \( \sigma_1 \) to the circle of "side reduction" \( \sigma_0 \). This enables to model the stressed state of soils with any combination of tensile and compressive stresses which are as a rule present in the soils of slip slopes, edge parts of structures, in the places of adjoin of earth dams to the edges, etc.

To use the method there were made a few modifications of the simultaneous compression-tension (SCT) method device for different hydrostatic pressures from 6 to 20 kg/sq.cm which enables to test both cryptofluid clayey soils and semi-rock material, including those ones with the content of fragments to 20% of the volume.

The devices of simultaneous compression-tension, made in Russia are distinguished by their original construction, simple use, reliability, high productivity, accuracy, little size, it doesn't need bulky loading devices. There is no analogy to it in other countries. The peculiarities of structure and the accuracy of the strength parameters \( C_0 \) and \( \phi_0 \) by the compression-tension method permitted to refuse the use of the plane shear and triaxial compression methods to obtain the strength parameters of clayey soils in short-time tests during engineering geological investigations. Besides it, these devices make it possible, if necessary to determine the parameters of long-term soil strength.

The sphere of application of the tension-compression method and the main results of the investigations on strength characteristics of clayey soils are as follows:

1. The comparison of the strength parameters (cohesion and angle of interval friction) obtained
by the compression-tension method for clayey soils with the parameters of strength, obtained by the plane shear and triaxial compression methods showed that the value of divergence between them depends on the value of curvilinearity of Mohr's circle envelope, that in its turn is bound up with the character of structural ties in the soil. The more elastic ties are in the soil, the more is the divergence and inaccuracy of strength parameters of the methods mentioned in comparison with the parameters obtained by the compression-tension method.

For the soil with plastic ties the cohesion value, obtained by the plane shear and triaxial compression is higher than the cohesion value, obtained by compression-tension in the limits 0-8%; the value of the angle of interval friction is less in the limits 0-15%, that is located in the limits of accuracy of determining the characteristics. We may say, that the strength parameters, obtained by different methods for plastic soils are identical.

For the soils with elastic-plastic ties the cohesion value, obtained by the plane shear and triaxial compression is 5-22% higher and the angle of the internal friction is 20-55% less.

For the soils with elastic ties the cohesion value obtained by the plane shear and triaxial compression is 22% higher and the angle of internal friction decreases more than 55% in comparison with the results obtained by compression-tension. These investigations show that for the parameters of strength, obtained by the plane shear and triaxial compression methods for the soils with elastic and elastic-plastic ties, the stage of design envisages excess safety factor which leads to the design of more expensive foundations of buildings and structures and to the rise in the cost of construction on the whole.

2. The determination of clayey soils resistance to fracture showed that it can be more, equal or less than the value of soil cohesion and depends on its plastic ties. For the soils with plastic and elastic-plastic ties the resistance to fracture is equal or more than the value of cohesion of soil. For the soils with elastic ties the less is the resistance to fracture, the more are the elastic ties of the soil. On the whole the resistance to fracture is 1.2-1.8 times less than the cohesion value.

The results obtained, permit to evaluate the degree of soil resistance to the occurrence of separation cracks in the places of the tensile stress (slip slopes, dam edges, etc.).

3. The maximum values of tensile stresses and deformations of the clayey soils in the upper zone of the earth dam at the Nureck hydroelectric power station in the places of contact with the edges were defined. The formation of separation cracks is possible at the dam as a result of its settlement in the edge and river-bed parts. The possibility of the cracks formation was estimated by the use of the compression-tension method on the samples of silty sands with the size of particles to 2 mm.

It was stated that the maximum tensile deformation and rupture stress which the soil can stand conforms to the optimum moisture-density of silty sand put to the core of the dam, as it was recommended at the construction. The calculation of the dam's core state under stress and deformation showed that possible limiting stresses and deformations are below admissible meanings, obtained by the compression-tension method at the soil samples.

4. The value of the resistance to fracture, obtained by compression-tension was compared with the value obtained by the conventional methods which used rigid grip of the sample. It was found out that the resistance to fracture, obtained by compression-tension is 1.5-2 times higher by the absolute value, because the soil sample doesn't possess the violation of the stressed state connected with the tensile effort transmission.

5. At the investigation of clayey soil stressed state at the slip slope the compression-tension method was used to define the critical tensile effort in soils according to the depth. This permits to locate already existing zone of slip and the zones which are only at the stage of formation which are potentially dangerous. The distribution of stress in the soils which form the upper and middle parts of the slope is characterized by the presence of positive stresses i.e. natural pressure and tensile stresses, occurring to a definite depth and provoking the appearance of the separation and sliding surfaces and depending on the angle of inclination of rocks on the slip slope.

It was established that the soils located in the zone of slip slope possess isotropic tensile characteristics at which the value of natural pressure is equal both to the value of limit tensile stresses and the value of soil cohesion.

This fact makes it possible to obtain critical tensile stress by the compression-tension method for these depths at making tests of soils samples of special form (a coil) from various depths of the slope. Comparing the critical tensile strength value with the value of natural pressure at this depth it is possible to locate both the existing sliding zones and the zones under formation. The use of the compression-tension method permitted to find out and locate in the Caucasus slip slopes previously unknown sliding zones which are at the stage of formation at the elevation of soil moisture due to
precipitation, changes of the slope stress state due to the extra loads in the upper part or cut of the slope, the zones may become active and bring to catastrophe in case of the slope movement. In this case the compression-tension method helps to forecast hazardous geological processes. The example based on the use of the compression-tension method at the engineering geological investigations open many other possible fields of its use.

REFERENCES


Caractérisation minéralogie et géotechnique des ‘Terres Noires’ du Sud-Est de la France, en vue d’applications routières

Mineralogical and geotechnical characterization of the ‘Black Lands’ of the South-East of France, having in view road applications

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ABSTRACT : The "Terres Noires" formation is a series of marls, which age ranges from middle to upper Jurassic. They are widespread over considerable areas in the southeastern part of France and thus they are frequently involved in engineering works, particularly for motorways. The authors describe the special weathering processes and the main geotechnical characteristics in view of using these marls as embankments.

RÉSUMÉ : Les Terres Noires correspondent à une formation marneuse d’âge Jurassique moyen à supérieur qui occupent de très importantes surfaces dans le Sud-Est de la France. Elles se trouvent ainsi impliquées dans de nombreux travaux de génie civil et particulièrement la construction des autoroutes. Les auteurs décrivent les processus d’altération et les caractéristiques géotechniques en vue de leur emploi en remblai routier. Des recommandations sont formulées pour cet usage.

1 INTRODUCTION

La zone externe des Alpes occidentales françaises est constituée de terrains sédimentaires formés d’une alternance de séquences de marnes et de calcaires dont les âges s’échelonnent du Lias au Crétacé. La séquence marneuse la plus ancienne est celle des "Terres Noires" (Bathonien à Oxfordien) dont l'épaisseur, quoique très variable, peut-être considérable et dépasser les 2000 m. Pour cette raison, et compte-tenu du plissement, cette formation affleure sur des surfaces considérables dans tout le secteur délimité à l'Ouest par la vallée du Rhône, au Nord par la latitude de Grenoble, à l'Est par le front des nappes de la zone interne alpine et au Sud par les chaînons provençaux.

Cette extension considérable fait que la plupart des travaux de génie civil réalisés dans cette zone rencontrent les formations de Terres Noires. Ceci est particulièrement vrai pour les tracés autoroutiers lesquels posent le problème du remplacement des déblais et cela de la stabilité des talus. La liaison autoroutière en projet, entre Grenoble et Sisteron en offre un bon exemple. Les "Terres Noires" y représentent la majorité des affleurements rocheux rencontrés selon les divers tracés envisagés. L’altérabilité des divers faciès de marnes a donc été particulièrement étudiée dans le cadre du projet.

2 APERÇU STRATIGRAPHIQUE

Les travaux de P.A. ARTRU (1972) ont montré que la formation des "Terres Noires" comprend généralement deux membres, essentiellement marneux et très épais, séparés par un niveau repère médian.

Le membre inférieur appartient au Jurassique moyen (Bajocien supérieur à Bathonien). Il est constitué de marnes noires à débit en fines plaquettes, très caractéristique.

Le niveau médian, plus carbonaté, est un calcaire argileux, parfois dolomitique. Sa patine rousse et sa résistance plus forte aux agents atmosphériques (il forme assez facilement des ressauts) en font un bon niveau repère. Son âge correspond sensiblement à la limite Bajocien-Bathonien.

Le membre supérieur est à nouveau constitué de marnes en plaquettes, mais celles-ci renferment des nodules plus carbonatés de teinte grise ou beige. Son âge s'étend du Bathonien à l'Oxfordien moyen.

3 CARACTÉRISATION DES DIVERS FACIÈS

Compte-tenu de la grande extension géographique de la formation il n'était pas certain a priori que ses caractéristiques géotechniques soient suffisamment constantes pour que l'on puisse raisonner globalement quant à leur utilisation routière. Trois sites géographiquement très distincts ont donc été étudiés à
des fins de comparaison, dans le Trièves, les environs de Digne et de Barcelonnette.

![Figure 1. Évolution des courbes granulométriques entre le tamisage par voie sèche et par voie humide](image)

3.1 Constitution minéralogique

Les divers faciès de "Terres Noires" dérivent du mélangage, en proportions variables, de deux phases principales :
- une phase carbonatée représentée principalement par de la calcite (et parfois un peu de dolomie) ;
- une phase détritique sableuse (quartz dominant et un peu de feldspaths) et argileuse (argiles et interstratifiés ainsi que phyllosilicates variés - micas, chlorites).

La phase carbonatée (roche saine) représente de 70 à 80% du volume total. Les faciès correspondants sont donc des marnes calcaires, voire des calcaires argileux, renfermant une fraction siltée non négligeable. Au sein de la phase argileuse l'ilite est largement prédominante ainsi que la chlorite. La kaolinite est absente dans les échantillons provenant de la région de Barcelonnette mais présente dans les autres (en faibles quantités). Il en va de même pour les interstratifiés (chlorite-smectite ou illite-smectite le plus abondant).

Les résultats des essais au bleu de méthylène sont en bon accord avec les données de la minéralogie. Les valeurs de bleu (V.B.) sont en général faibles comme le montre le tableau suivant ce qui confirme la nature siltée de la fraction détritique fine :

<table>
<thead>
<tr>
<th>Localité</th>
<th>V.B. roche saine</th>
<th>V.B. roche altérée</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trièves</td>
<td>0,7 à 1,7</td>
<td>1,3 à 2,5</td>
</tr>
<tr>
<td>Drax</td>
<td>1 à 3,2</td>
<td></td>
</tr>
<tr>
<td>Barcelonnette</td>
<td>0,5 à 0,6</td>
<td>0,5 à 1,1</td>
</tr>
</tbody>
</table>

On remarque en particulier les valeurs très faibles des échantillons du secteur de Barcelonnette où, justement, les interstratifiés à smectite font défaut.

3.2 Le processus d'altération des "Terres Noires"

Le processus d'altération met en jeu, classiquement, des phénomènes physiques et chimiques. L'action des premiers est la plus évidente. Les surfaces de marnes exposées aux agents atmosphériques subissent une desquamation et un écaillement poussés qui produisent un matériau meuble, constitué de fines plaquettes, dont la taille varie, selon le degré d'évolution, de quelques millimètres à quelques centimètres. Ce matériau, doté d'un indice de vides élevé, est très perméable. L'action des précipitations sera différente selon que du ruissellement pourra prendre naissance ou non. Dans le premier cas, on assistera à l'ablation très rapide d'une partie de la couverture meuble et à la dégradation de ces plaquettes en produits siltieux. L'évolution des courbes granulométriques entre un tamisage par voie sèche et par voie humide met bien en évidence cette dégradation.

La charge solide entrainée par les filets liquides et, par conséquence, par tous les ruissellements, sera très importante ce qui pose des problèmes pour tous les ouvrages hydrauliques ainsi que pour la conservation des sols.

Dans le second cas une infiltration épidermique se produira au sein du manteau altéré superficiel, rapidement limitée vers la profondeur par la présence de la marge compacte imperméable. Une action chimique pourra alors prendre naissance dont l'effet principal est une décarbonatation des produits meubles. Les eaux, chargées de carbonates, percoleront alors vers le bas des pentes et leur évaporation conduira généralement au dépôt du carbonate, lequel pourra conférer une certaine cohésion entre les plaquettes. Ceci permet la tenue du manteau meuble sur des pentes atteignant 35 à 40°.
3.2.1 Essais pénétrométriques

L'appréciation de l'épaisseur de la tranche altérée et la détermination de ses caractéristiques de résistance sont des éléments déterminants. Elles ont été réalisées par l'utilisation d'un pénétromètre léger, portable, bien adapté, conçu et réalisé à l'IRIGM. Les graphiques obtenus sont très révélateurs du degré d'altération, de la qualité de la couche altérée superficielle ainsi que de son épaississeur.

<table>
<thead>
<tr>
<th>Rd (kg/cm²)</th>
<th>0</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>160</th>
<th>180</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enfoncement (cm)</td>
<td>-10</td>
<td>-20</td>
<td>-30</td>
<td>-40</td>
<td>-50</td>
<td>-60</td>
<td>-70</td>
<td>-80</td>
<td>-90</td>
<td>-100</td>
<td>-110</td>
</tr>
</tbody>
</table>

Figure 2. Exemples de graphiques pénétrométriques dans la couche superficielle

3.2.2 Calcimétrie

La mise en évidence d'une variation significative des teneurs en carbonates entre la roche saine et la roche altérée est difficile puisqu'une partie de la calcite retirée aux plaquettes est souvent redéposée sous forme d'un enduit superficiel. Par ailleurs on a considéré comme roche saine la roche massive (refus au pénétromètre) mais rien ne prouve qu'il n'y a pas déjà eu transformation par rapport au faciès originel. Toutefois, la comparaison avec des analyses pratiquées sur des carottes de sondage montre que ce critère est acceptable. Les calcimétries réalisées sur 122 échantillons indiquent les tendances suivantes (teneurs moyennes en CO3Ca):

<table>
<thead>
<tr>
<th>Roche saine</th>
<th>Roche altérée</th>
</tr>
</thead>
<tbody>
<tr>
<td>% CO3Ca</td>
<td>% CO3Ca</td>
</tr>
<tr>
<td>Thiès</td>
<td></td>
</tr>
<tr>
<td>Membre supérieur</td>
<td>45</td>
</tr>
<tr>
<td>Membre inférieur</td>
<td>33</td>
</tr>
<tr>
<td>Drinx</td>
<td></td>
</tr>
<tr>
<td>Membre supérieur</td>
<td>45</td>
</tr>
<tr>
<td>Membre inférieur</td>
<td>38</td>
</tr>
</tbody>
</table>

Les résultats de Barcelonnette sont trop peu nombreux pour être significatifs mais on notera que les valeurs ci-dessus montrent bien la tendance à la décarbonatation entre les faciès sains et altérés, notamment pour le membre supérieur.

3.2.3 Analyse minéralogique

La comparaison des résultats d'analyses par diffraction X de la marnes saines et de ses produits d'altération révèle les faits suivants:
- les variations dans la teneur en carbonates ne peuvent être appréciées valablement ;
- les paragenèses argileuses sont inchangées.

3.2.4 Observations au microscope électronique à balayage

L'observation au MEB à des grossissements variant de 500 à 7500 permet de mieux cerner la progression de la décarbonatation lors de l'altération. Les principaux faits observés peuvent être résumés comme suit :
- la texture de la marnes est étroitement calcite argile et grains de quartz. Trois types de texture sont reconnaissables : compacte (beaucoup de calcite), alvéolaire (calcite partiellement dissoute et paillettes d'argile, floconneuse - argile dominante) ;
- la dissolution de la calcite est très perceptible et la disparition des grains rhomboédriques laisse fréquemment des vides de forme géométrique reconnaissables. La porosité augmente nettement lorsque l'on passe des faciès sains aux faciès altérés ;
- la texture la plus fréquemment représentée est la texture alvolaire caractérisée par l'abondance de micropores au voisinage desquels des cristaux de calcite restants sont corrodiés. Il est difficile d'établir si la différence entre texture compacte et texture alvolaire est entièrement due à de l'altération.

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3.2.5 Le mécanisme de l'altération

Il est désormais possible de résumer la succession des phénomènes conduisant à l'altération du faciès "Terres Noires". La décomposition naturelle liée à l'érosion facilite la pénétration de l'eau dans une couche superficielle de quelques décimètres. L'humidité provoque le gonflement des interstratifiés à smectites ou de la pyrite (par oxydation ethydratation ou formation d'acide sulfurique puis de gypse - secteur de Barcelonnette). La desquamation du matériau s'amorce en retrouvant les zones de faiblesse du sédiment (lamines). La percolation de l'eau dans le matériau meuble en voie d'élaboration contribue à la décarbonatation et à l'accroissement de la porosité (une diminution de 20 à 30 % de la teneur en carbonate conduit à un quadruplement de la porosité). Le renouvellement périodique des circulations superficielles conduit peu à peu à une diminution de la granulométrie du matériau altéré (la taille des plaquettes décroit progressivement) et l'évolution s'accélère par libération des silts lesquels ne représentent probablement rien d'autre qu'une partie de la phase détritique sédimentaire initiale.

4 APTITUDE DES "TERRES NOIRES" À LA CONFECTION DE REMBLAIS

Les indications précédentes montrent bien que les marnes siliceuses étudiées sont en fait très sensibles à l'action de l'eau laquelle se trouve à l'origine de la détérioration des caractéristiques géotechniques du matériau. La question se pose de savoir si une telle évolution est possible en remblai ce qui aurait pour conséquence inévitable des tassements inacceptables notamment pour une chaussée autoroutière. Divers essais complémentaires ont donc été réalisés pour préciser le comportement du matériau et aboutir à des recommandations pour la mise en place en vue de limiter leur détérioration avec le temps. Étant donné que les conditions de laboratoire étaient, selon toute probabilité, plus sévères que celles prévalant dans un remblai, un contrôle a été réalisé par carottage dans des remblais existants constitués à partir des marnes du membre supérieur (niveau seul sera concerné par les travaux autoroutiers en projet).

4.1 Les essais relatifs à l'évolutivité des "Terres Noires"

Il s'est agi principalement d'essais d'altérabilité (mesure de la réduction du D10 d'un granulat 10/20 après 4 cycles d'humidification séchage - indice DG) et d'essais de fragmentation selon la méthode Schaeffner (évolution du D10 d'un granulat 10/20 après pilonnage à la dame Proctor - indice FR) ou par fragmentation dynamique - indice FD. Les résultats ont été les suivants :

<table>
<thead>
<tr>
<th>Localité</th>
<th>DG</th>
<th>FR</th>
<th>FD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Déblai du Fau</td>
<td>1,07</td>
<td>3,6</td>
<td>27,7</td>
</tr>
<tr>
<td>Remblai des Blancs</td>
<td>8,50</td>
<td>5,5</td>
<td>30,4</td>
</tr>
<tr>
<td>Remblai Col du Fau</td>
<td>31,40</td>
<td>3,6</td>
<td>31,8</td>
</tr>
</tbody>
</table>

On constate que, dans l'ensemble, les matériaux empruntés au membre supérieur sont peu fragmentables mais que leur altérabilité peut varier dans de fortes proportions. Le déblai du Fau est en fait entaillé dans une marnes particulièrement riche en carbonate (de 55 à 79%).

4.2 Essais Proctor et CBR

Des essais Proctor, CBR, oedométrique et de cisaillement ont été réalisés sur des échantillons carottés en sondage au Col du Fau (marnes du membre supérieur). La roche a été broyée et passée aux tamis de 20 et de 2mm. Le tamisat à 20 mm a été utilisé pour les essais Proctor et CBR, celui à 2 mm pour les essais oedométrique et de cisaillement.

L'énergie de compactage joue un rôle particulièrement important dans le cas des matériaux évolutifs. Une énergie trop faible laisse subsister des vides d'où une possibilité de poursuite de l'altération et de tassement ultérieur. Une énergie plus importante accroît la pression de gonflement et son amplitude. Il a donc été procédé à ces essais Proctor normal et modifié. Pour estimer l'influence de l'altération il a été en outre réalisé un essai Proctor modifié sur le matériau après un cycle d'humidification-séchage. Les résultats sont donnés sur la figure 3.

![Figure 3. Essai Proctor sur un échantillon normal (1) et après humidification séchage (2)](image-url)
L'emploi d'une énergie élevée sur le matériau initial montre que la fraction fine voit son rôle minoré. L'essai après altération montre au contraire que la production supplémentaire de fines conduit à une courbe plus pointue d'où une moindre latitude quant à la teneur en eau en vue d'atteindre l'optimum de compactage.

L'essai CBR a été réalisé tout d'abord à la teneur en eau optimum. Proctor modifié soit 6,6% puis après imbibition, soit un indice de saturation compris entre 87 et 100% (figure 4).

### 4.3 Essais oedométriques

Les échantillons d'essai ont été réalisés comme indiqué précédemment. Les masses volumiques sèches et les teneurs en eau sont celles de l'optimum Proctor modifié. Le compactage de l'échantillon a été réalisé en quatre couches et ses dimensions avant essai étaient de 70 mm de diamètre et 24 mm de hauteur. Les courbes obtenues pour diverses valeurs de contrainte normale (de 25 à 400 KPa) montrent un tassement immédiat et une absence de tassement secordaire due au fait que l'échantillon n'était pas saturé.

Les essais de gonflement ont été réalisés après 24 heures de chargement et deux heures d'imbibition. Sous 400 KPa on constate un tassement supplémentaire d'environ 1% qui correspondrait à l'affaissement structural de J. CAMAPUM DE CARVALHO - (1985) Le regroupement des courbes de tassement et de gonflement permet de déterminer la pression de gonflement laquelle est de l'ordre de 220 kPa.

### 4.4 Essais de cisaillement

Des essais de type consolidé-drainé ont été réalisés sur des échantillons préparés comme indiqué plus haut, à l'aide d'une boîte de Casagrande. Après un compactage à l'Optimum Proctor modifié on a obtenu c= 12 kPa et \(\phi'= 56^\circ\) ce qui est considérable. Après imbibition ces valeurs tombent respectivement à 6 kPa et 38° ce qui apparaît encore trop élevé. Il est probable que la surconsolidation entraînée par le compactage intense (plus intense que celui des remblais étudiés) est à l'origine de cette résistance au cisaillement élevée.

### 5 CONCLUSIONS

Les essais réalisés ont mis en évidence le processus d'altération des Terres noires, essentiellement par décarbonatation mais sans modification de la phase argileuse. La présence d'ahrone et d'argile les modalités de l'écoulement superficiel très spécifique de ce matériau. Sur le plan pratique il est particulièrement intéressant de noter que les membres inférieurs et supérieurs de la formation ne se distinguent pas de ce point de vue en dépit d'une teneur en carbonate légèrement plus forte dans le membre supérieur. Les essais mécaniques ont révélé que le matériau était peu fragmentable, mais sensible à l'eau avec évolution de la granulométrie vers les fines et chute de l'indice CBR (éffondrement structural). Par contre le compactage accroît sensiblement la résistance au cisaillement.

L'observation de carottes réalisées dans des remblais anciens montre que l'altération est moins importante que ce que l'on pouvait penser (il subsistait...
beaucoup de blocs résiduels emballés dans une gangue argileuse). Il apparaît donc que l'on puisse réaliser des remblais relativement élevés à l'aide des Terres-Noires à condition de prendre quelques précautions :
- lors de la mise en place il faut homogénéiser au maximum le matériau et accélérer la production de fines par arrosage-séchage (ce qui sera toutefois plus contraignant au niveau des teneurs en eau de compactage);
- le compactage devra être réalisé par levées minces (de l'ordre de 30 cm) à l'énergie de l'OPM pour obtenir un matériau bien serré;
- protéger le corps du remblai des infiltrations d'eau;
- veiller à poser le remblai sur un terrain stable. Eviter en particulier de le fonder sur une couche de terrains superficiels de mauvaises caractéristiques comme cela fut le cas au remblai des Blancs sur la RN 85. Les instabilités observées avaient été attribuées à tort aux matériaux du remblai eux-mêmes.

REFERENCES


Gonflement et indices d’activité des sols cohérents de la dépression de Grenade et ses alentours (Espagne)
Swelling behaviour of coherent soils from the Granada basin (Spain)

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ABSTRACT: The swelling behavior on fine soils depend of a large number of factors. Being significant the clay fraction activity and the clay-water relationship. A correlation analysis of factors on one hundred twenty samples from the Grenade basin (Spain) leads to a better understanding of the relative influence of the properties determining the swelling of the studied soils.

RÉSUMÉ: Le comportement gonflant des sols fins dépend de nombreux facteurs extrinsèques et d’autres intrinsèques, parmi ces derniers l’activité de la fraction argileuse et le rapport argile-eau jouent un rôle important. Les corrélations établies sur cent vingt échantillons provenant de la dépression de Grenade et ses alentours ont permis de mieux apprécier quelques facteurs déterminant le gonflement de ces sols.

1 INTRODUCTION


L’échantillonnage a été réalisé sur une zone qui se situe, géographiquement, dans l’Andalousie orientale et elle intègre les alentours de la dépression intramontagneuse de Grenade (fig. 1).

Du point de vue géologique, il s’agit de formations d’âge allant du Crétacé au Néogène, dont les lithologies varient de carboneaux aux marno-calcaires et marnes en passant par des sables, des silt et des conglomérats, ces quatre derniers constituent les matériaux de comblement de la dépression.

Dans ce qui suit, nous ne nous intéresserons qu’aux faciès argileux et marneux.

2 CLASSIFICATION DES SOLS ÉTUDIÉS SUIVANT LE SYSTÈME UNIFIÉ (U.S.C.S.)

Cette étude porte sur cent vingt sols provenant d’une part de la série marno-argileuse d’âge Néogène et d’autre part de la série marneuse Crétacé.

Le classement des sols étudiés a été fait selon les critères de la classification du S.U.C.S., suivant la norme ASTM D 421/85.

La représentation sur l’abaque de plasticité de Casagrande des données de consistance des échantillons (fig. 2) indique qu’il s’agit pour les sols d’âge Néogène, d’argiles et limons très plastiques en nombre de neuf, de limons peu plastiques (49), d’argiles peu plastiques (29), d’argiles sableuses (6), de
limons sableux (3) et d'argiles limoneneuses (4). Tous les sols d'âge Crétacé sont des argiles très plastiques.

3 LES ESSAIS D'IDENTIFICATION

3.1 La sédimentométrie

Parallèlement à la granulométrie des gros- siers, nous avons réalisé l'essai de Sédimentation, suivant la norme ASTM D 422/63, à fin de déterminer les teneurs en argiles (fraction < 2µm) et en limons (fraction < 50µm) des différents sols (fig. 3).

Fig. 2 Diagramme de plasticité de Casagrande.

3.2 Les limites d'Atterberg

L'évaluation des paramètres de consistance des sols a été réalisée selon la norme ASTM D 4318/84.

Les sols étudiés possèdent des limites de liquidité variables entre 18,7% et 96,5% et des indices de plasticité compris entre 0,6% et 60% (fig. 2).

D'après la classification proposée par Skempton (1963), la fraction argileuse de ces sols, est inactive dans le 45-52% des cas, normalement active pour le 27% et active pour le 28% restant.

De même, nous avons déterminé les limites et les indices de retrait (l'indice de retrait étant la différence entre la limite de liquidité et la limite de retrait). La fourchette de variation de la limite de retrait est de 7,75 - 45,84%, les valeurs élevées sont obtenues pour les sols peu plastiques. Les indices de retrait se répartissent selon l'intervalle 0 - 69,3%.

3.3 L'essai au bleu de méthylène

La surface spécifique des argiles est un bon critère pour comprendre leur comportement puisqu'elle varie de 5-20 m²/g pour une kaolinite, 40-60 m²/g pour une illite et 800 m²/g pour une montmorillonite (Grém, 1962).

Le bleu de méthylène en mesurant la surface totale, externe et interne, (Tran, 1980) des argiles est un bon indicateur de la quantité d'eau qui peut être fixée par celles-ci, quand elles sont à l'état naturel.

L'essai utilisé ici, est empreint de la norme NF P18-592 (1980), mise au point par le LCPC.

Connaissant la quantité de bleu introduite (en g) dans une suspension aqueuse, nous avons calculé:
- la valeur de bleu qui correspond à la quantité de bleu en (g) adsorbée par 100g de sol sec.
- la surface d'échange globale développée par la fraction argileuse, exprimée par la formule suivante (Lautrin, 1989):

\[ S_a = 20,93 \cdot V_{cc} \cdot \frac{1}{P_s} \]

où \( S_a \): surface active (m²/g); \( V_{cc} \): volume de solution utilisé (cc); 20,93: surface correspondant à 1cc de bleu de méthylène; \( P_s \): poids sec du matériau.

- l'activité colloidale ou "nocivité" (Lautrin, 1989) de la fraction argileuse définie par:

\[ A_{cb} = \frac{100 \cdot \text{VB}}{C_2} \]

où VB: valeur du bleu du sol (g/100g); \( C_2 \):
Le diagnostic montre que la valeur du bleu des sols varie entre 0,2 (g/100g) pour les sols peu plastiques à 7,2 (g/100g) pour les sols très plastiques.

Le diagramme d’activité du bleu pour des argiles à saturation calcique (Lautrin 1989), montre que les indices d’activité du bleu (A66) des échantillons traités s’étendent entre 1 et 13, d’où une gamme très large de variabilité de l’activité des sols étudiés (Fig. 10).

3.4 La diffraction aux rayons X

Les résultats des analyses de diffracation aux rayons X sont représentés dans le tableau 1. Ces analyses ont été réalisées par la méthode des "pâtes orientées" pour les éléments inférieurs à 2 μm recueillis par centrifugation et décantation (Voinovich, 1971).

3.5 Les essais oedométriques

Ces essais ont été réalisés sur des échantillons remaniés, compactés sous des conditions hygroscopiques et de densités sèches contrôlées.

Les indices de gonflement des argiles très plastiques du Crétacé sont six fois supérieurs à ceux du Néogène.

L’évaluation du taux de gonflement suivant le "Technical Committee on Expansive Soils" montre que les sols présentent un potentiel de gonflement faible, significatif et même critique (Alimi 1991).

4 RÉSULTATS ET DISCUSSIONS

Les diagrammes de la figure 3 montrent une bonne corrélation entre les paramètres de liquidité et de plasticité et l’indice de retrait d’une part et entre ceux-ci et la valeur du bleu du sol d’autre part. L’indice de plasticité donne une mesure de l’étendue du domaine de plasticité du sol, il est d’autant plus élevé que la fraction argileuse du sol est importante. Or, une forte teneur en argile favorise la capacité d’adsorption d’eau du sol et l’indice de retrait est généralement plus élevé dans ce cas (exemple des fentes de retrait dans les formations argileuses). Ainsi, lorsque les indices de retrait et de plasticité augmentent, la quantité de bleu adsorbée croît également et la limite de retrait décroît. La teneur en sables est inversement proportionnelle à l’indice de retrait (r= -0,514), autrement dit les fractions limoneuses et sableuses défavorisent la plasticité et le retrait des sols étudiés. Dans le cas des limites de liquidité et de plasticité, on constate que, pour des teneurs en particules inférieures à 2 μm variable de 3 à 25%, nous obtenons sensiblement les mêmes valeurs. D’où, la fraction argileuse n’a d’influence significative sur le comportement plastique des sols qu’à partir d’un certain seuil (Fig. 5).
Pour les sols Néogènes, peu plastiques, à fraction argileuse polyminérale (Tableau 1) et à forte teneur en sables et limons, il paraît que le seuil qui marque le passage vers un comportement de type colloïdal, c'est-à-dire, le seuil au-delà duquel la phase argileuse impose son comportement se situe vers 20 à 25% d'argile (Fig. 4).

Pour des teneurs en argiles inférieures à ce seuil, comme l'explique Lautrin, "toujours de même si les valeurs des limites correspondaient à une eau interstitielle masquant l'activité colloïdale spécifique des minéraux argileux".

D'autre part, d'après la définition physico-chimique des paramètres d'Atterberg proposée par Grim (1962), et sachant que la capacité d'adsorption d'eau par une particule d'argile augmente en fonction de la charge partielle dans le sens Kaolinite < illite < montmorillonite, on pourrait penser que les limites seraient indicatrices de la composition minéralogique des sols.

La figure 6 relative à de nombreux échantillons analysés montre que les valeurs des paramètres d'Atterberg sont d'autant plus élevées que les sols contiennent de forts pourcentages d'argiles et que ces argiles sont actives. C'est ainsi qu'un sol qui contient 51,3% de smectite a une limite de liquidité de 60% et une limite de plasticité de 36,6% alors qu'un sol qui a 60,5% d'illite a une limite de liquidité de 38% et une limite de plasticité de 23,85% (échantillons 2 et 19 du tableau 1). Pour de plus faibles pourcentages en argiles, les corrélations deviennent plus aléatoires et le comportement plastique des échantillons est indépendant du taux d'argile. Aussi, il est aisé de penser, que des échantillons qui ont des granularités et des limites comparables (échantillons 5 et 19 du tableau 1) pourraient être considérés comme des sols identiques. En fait ce n'est pas exact puisque la valeur de bleu permet de calculer une activité colloïdale adsorbante et une surface réactive différente.

La figure 7 montre que la valeur du bleu augmente avec le pourcentage d'argile. Pour divers sols possédant la même teneur en argiles cette relation varie dans une fourchette d'autant plus étendue que ce pourcentage est élevé. Ceci illustre que la nature de la fraction argileuse joue un rôle important. En plus, la figure 7 nous indique une différenciation des échantillons en groupes de sols dont chacun montre une tendance de variabilité de la valeur du bleu en fonction du pourcentage d'argile. En prenant le cas des sols possédant une teneur en argile variable entre 15 et 28%, on remarque que l'activité croît en fonction du pourcentage des smectites, autrement dit, de la réactivité des minéraux argileux. (Fig. 8)

Malgré la corrélation significative entre les deux indices d'activité (A0 et A0) (Fig. 9), des sols à pourcentages de smectites élevés (adsorption de bleu élevée) seraient classés selon la classification de Skempton (1963), comme argiles normales. Donc, cette activité n'est pas indicatrice de la minéralogie.

Ceci peut être expliqué par le fait que ces faibles valeurs d'activité obtenues, se-
<table>
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Tableau 1 Caractéristiques géotechniques et minéralogiques des sols.

Fig. 7 Variation de la valeur de bleu en fonction de la teneur en argile.

Les résultats des essais oedométriques montrent que le gonflement des sols est d'autant plus élevé que la plasticité et le retrait sont importants. La valeur de bleu augmente parallèlement au gonflement, ce qui verifie le rôle de la surface réactive hautement adsorbante des argiles dans le comportement gonflant des sols.

D'où, le gonflement des sols étudiés est en fonction de leurs plasticités, leurs capacités d'adsorption d'eau et leurs pouvoirs de retrait (Fig. 11).

Dans le tableau (2) on présente les résultats obtenus d'après les méthodes de classifications de Williams (1957), Ranganathan.
Les diagrammes de la figure (12) mettent en évidence l'influence de la minéralogie de la fraction argileuse sur le taux de gonflement. Ainsi, un sol à faible valeur de bleu et à faible activité, correspond à un sol dont la fraction argileuse est chimiquement stable.

Les corrélations entre ce taux de gonflement et les pourcentages d'illite et de kaolinite sont négatives.

Fig. 8 Influence de la minéralogie des argiles sur l'activité de bleu de méthylène.

Les résultats obtenus à partir de la réalisation des différents essais mettent en évidence la relation entre le gonflement des sols étudiés, leur plasticité, leur pouvoir de retrait et leur capacité d'adsorption d'eau. Or, nous avons vérifié que les facteurs intrinsèques qui conditionnent le comportement gonflant de ces sols sont déterminés par leurs degrés d'argilosite, et par l'activité de leur fraction argileuse. L'activité ou la nocivité des sols étant proportionnelle au pourcentage des smectites, nous avons constaté que le gonflement potentiel des sols étudiés dépend de ce pourcentage.

De même, signalons que la plupart des classifications proposées pour la caractérisation du gonflement des sols mènent à des attributions souvent contradictoires.

D'après nos résultats, la norme (NF) Afnor 18-952 (1989) appliquée complète les essais classiques (Tran 1989, Lautrin 1989), donnant ainsi d'une manière simple et indirec-

Fig. 9 Corrélation entre l'indice d'activité de Skempton et l'activité de bleu.

6 CONCLUSIONS

Les résultats obtenus à partir de la réalisation des différents essais mettent en évidence la relation entre le gonflement des sols étudiés, leur plasticité, leur pouvoir de retrait et leur capacité d'adsorption d'eau. Or, nous avons vérifié que les facteurs intrinsèques qui conditionnent le comportement gonflant de ces sols sont déterminés par leurs degrés d'argilosite, et par l'activité de leur fraction argileuse. L'activité ou la nocivité des sols étant proportionnelle au pourcentage des smectites, nous avons constaté que le gonflement potentiel des sols étudiés dépend de ce pourcentage.

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avec * est le potentiel de gonflement
Tableau 2 Évaluation du potentiel de gonflement d'après différents auteurs.

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<td>Sol</td>
<td>IP(I)</td>
<td>C₂(I)</td>
<td>*</td>
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Fig. 10 Diagramme d'activité de bleu. Intervalles d'activité: 0-1: Sols non argileux. 1-3: Sols peu argileux. 3-5: Sols inactifs. 5-8: Sols normaux. 8-13: Sols actifs. > 13: Sols très actifs. >18: Sols nocifs.

te, une approximation à la minéralogie des argiles qui les composent et par-là à leur nocivité.

Références bibliographiques
Altmeyer, W.T. 1955. Discussion of engenee

Fig. 11 Corrélations entre les paramètres d'Atterberg, la valeur de bleu et taux de gonflement.
Fig. 12 Influence de la minéralogie des argiles sur le taux de gonflement.


Remerciement:
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Laboratory testing aspects of the Athenian Schist
Aspects des essais en laboratoire du Schist Athenien

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University of Patras, Greece

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ABSTRACT: A large number of intact rock specimens of the Athenian schist were tested in the laboratory and the results were statistically analysed and evaluated. The specimens were classified in seven groups according to their lithological type and grade of weathering. The main results refer to the uniaxial compressive strength and point load index, while other physical and mechanical characteristics, such as dry density, sonic velocities, tensile strength, shear strength parameters and deformability are examined.

RESUME: Un grand nombre des samples intacts, du schist dit "athénien", testé en laboratoire, est analysé statistiquement. La roche est classée en sept groupes suivant son type lithologique et son grade d'alteration. Les résultats traités concernent, principalement, la résistance à compression simple et le "point load index" en même temps que d'autres caractères physiques et mécaniques comme la densité sèche, la vitesse des ondes soniques, la résistance à la traction, les paramètres de cisaillement et de deformation.

1 INTRODUCTION

The bedrock of the city of Athens is the geological formation known as "Athenian Schist". This include schists of low-grade metamorphism, metasedimentary types of shales (sericitic, chloritic etc), marls, breccia, siltstones and sandstones, mainly in the upper horizons, and lenses of brecciated or crystalline limestones. Locally, small bodies of mostly decomposed metagneous rocks of diabase-splite type or serpentinitized peridotites are encountered. The formation represents, according to Marinos et al (1971), a flyschoid phase of delta-type deposits, upper Cretaceous in age.

The intense tectonism which is reflected in the macro and micro-structures of the formation, the different lithological types and the various degrees of weathering and alteration, have resulted a high heterogeneity of the material. Generally, the main behaviour of the formation ranges from that of soft to hard rock, while some members are characterized as transitional material, lying in the geotechnical area between soil and rock. Sometimes the "soil-like" members exhibit geological structures, faults, joints, defects etc. and they behave as heavily over-consolidated clay soils, in the soil mechanics concept (Anagnostopoulos, 1981). Generally, Athenian Schist is a difficult material to study, because it is easily disturbed during drilling, sampling and specimen preparation, while regular shape specimens very often disintegrate and fall along planes of weakness.

The prediction of the mechanical behaviour of the Athenian schist, is mainly based upon practical experience gained from shallow excavations for building foundations, excavation for subways and also from the early stages of Athens Metro construction.

Although, the published information concerning the physical and mechanical characteristics of the Athenian schist is rather limited (Myrianthi & Leach, 1978; Christoulas & Tsiambaos, 1983; Papadopoulos and Marinos, 1992; Kolaiti et al, 1993), the geological and geotechnical problems of this formation were earlier discussed in a one-day scientific meeting organised by the Technical Chamber of Greece (1981), while Sabatakakis (1991) in the engineering geological study of the Athens basin refers to the engineering properties of Athenian schist. In the present paper, emphasis is given to the physical and mechanical characteristics of the intact rock specimens of the Athenian schist, after their
classification in seven groups according to their lithology and grade of weathering.

2 SAMPLING AND LABORATORY TESTING

An extensive sampling was carried out from about 100 exploratory boreholes using double tube barrel, providing rock cores of 70-90 mm in diameter and in which a detailed logging was made regarding the lithology, core recovery, R.Q.D., grade of weathering etc. The samples were classified in seven groups as follows:

Group A: Completely decomposed shale, schist, peridotite, diabase; marl: stiff soil like material.

Group B: Highly decomposed shale, schist, peridotite, diabase; highly disintegrated marlstone, siltstone, sandstone: soft rock like material.

Group C: Moderately decomposed shale, schist, peridotite, diabase; slightly to moderately disintegrated marlstone, siltstone, sandstone: soft to weak rock.

Group D: Slightly disintegrated-highly discorded shale, schist, peridotite, diabase; moderately discoloured sandstone: weak rock.

Group E: Moderately discoloured shale, schist, peridotite, diabase: medium strong rock.

Group F: Slightly discoloured schist, peridotite, diabase; slightly discoloured to fresh sandstone: medium strong to strong rock.

Group G: Fresh schist, peridotite, diabase, graywacke: strong rock.

It is noted that limestone and breccia samples have been excluded from the above classification.

From these samples a great number of specimens were prepared and tested in the laboratory. The procedure for specimen preparation and the execution of characterization tests of the intact rock material, as well as the determination of the index and strength parameters, were in accordance with ISRM (1981, 1985) suggested methods. More specifically, the indices determined by tests carried out in the laboratory and performed in dry conditions for a better comparison of the results, were as follows: dry density (d), slake durability (Id), sound velocities (Vp, Vs), point loading Is(50), as well as tensile (Brazilian) and uniaxial compressive strength.

Furthermore, a number of about 400 specimens were tested under uniaxial compression with measurement of the axial and lateral strain for the determination of static Young’s modulus (E) and Poisson’s ratio (v), while the shear strength parameters (c, φ) of about 20 specimens were determined under triaxial compression.

3 TEST RESULTS AND DISCUSSION

The dry bulk density of specimens tested, ranges from 21,3 KN/m³ to 27,3 KN/m³ and, as shown in figure 1, the greater percentage of specimens has values from about 23 KN/m³ to 26 KN/m³, with a mean value of 24,6 KN/m³. Slake durability testing showed that all specimens tested, regardless their grade of weathering, have values of slake durability index (measured after the second cycle of slaking) greater than 96 percent, while only specimens of completely decomposed schist (group A) exhibited lower values.

The compressional wave velocity, Vp, for specimens of groups 4 to 7 ranges from 2396 m/sec to 5873 m/sec and the shear wave velocity, Vs, from 1581 to 3866 m/sec. The greater percentage of the specimens tested, show values of Vp and Vs, 3000 - 5000 m/sec and 2000 - 3500 m/sec respectively, as shown in Figure 1. Correlation of the Vp, Vs values for the specimens tested, is expressed by the linear equation: Vp = 1.51 Vs, with a correlation coefficient r = 0.70 (figure 2). Specimens of different groups of the Athenian schist were tested under point loading and uniaxial compression. Point loading results of group A specimens were not considered because these specimens exhibited very low values of point load index, Is(50).

Values of Is(50) for all specimens tested range from 0.4 to 8.0 MPa. As shown in figure 3, specimens of group B show values of Is(50) from 0.4 MPa to 1.2 MPa, group C: 0.6 - 2.2 MPa, group D: 1.0 - 3.6 MPa, group E: 1.5 - 4.5 MPa, group F: 1.8 - 6.0 MPa and group G: 2.7 - 8.0 MPa.

The scatter of Is(50) values of specimens with a lower grade of weathering is mainly due to the high anisotropy of specimens. Previous work (Papadopoulos and Marinos, 1992) also showed a dramatic decrease of point load strength of Athenian schist specimens with the increase of weathering grade. This decrease was more pronounced at a direction of loading normal to schistosity planes. Values of anisotropy index ranged from just below 1 (very weathered specimens) to 3.1 (slightly weathered specimens). However, specimens of weathering grade I (fresh, strong schist) although possessing the highest point load strength, presented
Fig. 1. Frequency histograms for dry density and sonic velocities
Fig. 2. Compressional versus shear wave velocities

Fig. 3. Point load index for different material groups

Fig. 4. Uniaxial compressive strength for different material groups
Fig. 5. Frequency histogram for Young Modulus

Fig. 6. Frequency histogram for Poisson Ratio

Fig. 7. Frequency histogram for Dynamic Young Modulus

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Fig. 8. Correlation of Uniaxial Compressive and Tensile Strength

Fig. 9. Shear strength parameters of Group C and F material group
again an anisotropy index lower than the slightly weathered specimens, probably due to the strong bonds developed between the schistosity planes of fresh rocks. Similar results were obtained from point loading tests on other schist type rocks in Greece (Marinos et al. 1986).

Values of uniaxial compressive strength, $\sigma_c$, for all the specimens tested range between 1.3 MPa and 90.0 MPa. As shown in figure 4, group A specimens show values of $\sigma_c$ from 1.3 MPa to 10.0 MPa, group B: 3.0 - 17.0 MPa, group C: 6.0 - 25.0 MPa, group D: 10.0 - 35.0 MPa, group E: 15.0 - 50.0 MPa, group F: 20.0 - 70.0 MPa and group G: 30.0 - 90.0 MPa.

Referring to deformability parameters, values of Young's modulus, $E$, for specimens classified in groups $E$, $F$ and $G$ vary from 3.0 GPa to 42 GPa (figure 5), while Poisson ratio presents values from less than 0.10 to 0.37 (figure 6).

Based on the sonic velocities values, the dynamic Young's modulus, $E_d$, was also computed with values of $E_d$ ranging from 11 GPa to 70 GPa (figure 7). In comparison with the values of static Young's modulus it seems that dynamic values are about three times the static ones.

Brazilian test results showed values of indirect tensile strength, $\sigma_t$, ranging from 0.3 MPa to 6.4 MPa.

Furthermore, correlation of uniaxial compressive and tensile strength for the specimens tested is illustrated in figure 8. As shown in this figure there is a scatter of data (shaded area) and the limit lines are expressed by the equations $\sigma_c = 4.0 \sigma_t$ and $\sigma_c = 13.0 \sigma_t$.

Finally, triaxial compression tests carried out on specimens of groups C and G showed that mean values of shear strength parameters (cohesion $c$ and friction angle $\varphi$) are as follows (figure 9):

Group 3 specimens: $c = 2.40$ MPa, $\varphi = 27^\circ$
Group 7 specimens: $c = 14.43$ MPa, $\varphi = 45^\circ$

4 CONCLUSIONS

4.1 Athenian schist is an heterogeneous formation including variable rocks of different composition, grade of weathering and anisotropy.

4.2 Laboratory testing results of the proposed seven different group members of this formation showed a great scatter, and the determined correlations should be used only for soil and rock like material characterization and classification.

4.3 Design parameters for Athenian schist must be based only upon detailed site and laboratory investigation results.

REFERENCES


A contribution to the geotechnical and mineralogical characterization of the clayey and shaly complexes involved in mass movements affecting historical towns in the Tosco-romagnolo Apennines

Une contribution pour la caractérisation des formations argileuses associées à des mouvements de masse affectant des villes historiques de l'Appénine

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ABSTRACT: The paper offers a preliminary contribution to the geotechnical and mineralogical characterization of the clayey and shaly complexes involved in mass movements affecting some towns in the Northern Apennines, in Tuscany. The aim is to define the behaviour of these materials with respect to the slope instability. Various analyses were carried out in order to determine the relationship between both the strength parameters and swelling characteristics, and the type of the clay minerals and the carbonate content.

RESUME: Cette note représente une contribution préliminaire pour la caractérisation géotechnique et minéralogique des schistes argileux impliqués dans des mouvements de pente concernant des villes de l'Apennin Septentrional, en Toscane. Le but c'est d'établir la conduite de ces matériaux en considération de l'instabilité des versantes. Les différentes analyses ont été conduites pour déterminer les liaisons entre les paramètres de résistance, les caractéristiques de regonflement, le type de minéral argileux et le contenu de carbonate de calcium.

1 INTRODUCTION

The main geotechnical characteristics of the materials of the clayey and shaly complexes in the Northern Apennines were analysed in the context of the research currently underway on the instability problems of the terrains in large areas of Tuscany. These problems often affect historical towns and important infrastructures.

The slopes composed of the structurally and lithologically complex clayey formations are in fact affected by numerous landslide phenomena which are often great in size and of a complex typology.

They are mainly flow phenomena, at times these are mud flows having also a translational component, and sometimes multiple slides which are rototranslational when they propagate backward. Moreover, many of these phenomena can be related to huge ancient landslides which have in part been reactivated with the same sliding characteristics or with a different typology and movement mechanism.

In other cases they are triggered off by or connected to other phenomena which have a slower evolution but are of even greater dimension, such as deep seated gravitational phenomena, like the lateral spreads which occur in the case of rigid plates overlying the clayey units under examination. Creep type processes in the weathered reworked sheets, swelling and softening phenomena in certain portions of the terrains and bulging and stress relaxation type phenomena related to morphological and geological phenomena are all connected to these movement typologies.

The aim of the research, to which this study represents a preliminary contribution, is to analyse the behaviour of the clayey and shaly complexes which make up the slopes affected by instability phenomena, and to try and evaluate the influence of the mechanical and chemical-
physical weathering processes. Indeed, in many situations, both in the landslide bodies which correspond to ancient movements and in weathered belts which are at times deep, it is quite complicated to determine the shear strength parameters with a sufficient level of reliability. Among other things there is, in fact, very little data available on this subject in literature and no systematic studies have been carried out on these terrains (Bertocci et al., 1991a; Bertocci et al., 1991b; Esu, 1977). Therefore, at this stage of the research, we feel that it is more useful to try and make an initial classification of these complex behaviours by carrying out a series of standard analyses. Although they are only preliminary they may help provide a guideline for future tests, including strength ones, on the materials in the various slope conditions.

We therefore carried out mineralogical and X-ray diffractometric analysis, both on the *tou venant* and on the clayey fraction. We also carried out traditional classification analysis, such as the one proposed by Casagrande and activity maps were drawn up in relation to the clayey fraction, following methods proposed by various authors (Seed et al., 1962, Van der Merwe, 1964).

Tests were then carried out on the calcium carbonate content.

The swelling characteristics were also analysed.

Finally, oedometric tests were carried out on some samples in order to determine the compressibility of the materials in relation to overconsolidation, in an attempt to show, in particular, the degree of swelling in terms of pressure.

2 CASED EXAMINED

Instability phenomena in Tuscany are mainly associated with the pelitic sediments outcropping in the mountain range of the Apennines, in particular with the argillaceous melanges (Cretaceous-Eocene, belonging to the Allochthonous terrains of the Ligurian Units). These formations are made up of clay shales with chaotic structure due to intense tectonization (tectonites or tectonic melanges) or to submarine gravitational processes (olistostromes, sedimentary melanges, sedimentary breccias).

They are commonly formed by the assemblage of rock blocks (sandstones, ophiolites, calcarenites, limestones) embedded in a matrix of sheared clay shales. These materials are well known in the Apennines under the informal names of "Chaotic Complex" or "Argille Scaglioce" (Scaly clay shale). The geomechanical behaviour of these formations is rather complex and critically dependent on the relative proportions between strong rock and weak matrix. However, in the most typical outcrops in the Apennine mountain range, the clay shale component is dominant, hence they can be analysed by a classical soil mechanics approach.

Therefore four main formations outcrop in the areas analysed and the clayey complexes can be subdivided into these 4 groups. These units differ not only from the point of view of the sedimentation period but above all because of their history and tectonic position within the Allochthonous Units.

The cases examined (Fig. 1) refer to:

The slope of San Marcello Pistoiese, an ancient town in the Apennine relieves where the terrains of the Heterogeneous Complex outcrop. Here we can observe flow phenomena with a maximum length of 800 metres and a depth of up to 15 metres. There is a thick belt of weathered and fractured material due to tectonic gravitational phenomena and to more surface slope deformations.

The area of Impruneta, an important historical town in the Chianti area, south of Florence. A few years ago, in an urban expansion area here, an 18 metres deep ancient translational slide-flow type landslide, on the terrains of the Sillano Formation, was cleared.
The slopes of M. Simone, where the reliefs are also affected by lateral spreads, are composed of the terrains of the Val Marecchia Shale Complex and of the Olistostrome.

The numerous landslide phenomena are mainly of the flow type, at times debris flows and earth-flow and mud-flow type phenomena, with active badlands processes and bulging and creep systems and areas.

3 SAMPLING

The sampling included the terrains of various complex shaly units directly involved in landslide phenomena of various types.

In particular, the majority of the samples came from the Sillano Formation. The latter is affected by landslide phenomena having very varied movement mechanisms and geometry related to different mechanical behaviours, even in adjacent areas. This will also be clearly shown by the very widespread distribution of some geotechnical data obtained from the analyses.

Some data come from literature concerning research currently underway on landslide susceptibility. For this reason every sample may not have undergone all the tests foreseen in the research programme.

X Ray diffractometric analyses, in particular, were only carried out on some samples and they represent one single landslide phenomenon.

The samples were taken from the surface or near the surface and they refer both to detachment or in situ outcropping areas and also to the landslide body in various reworking, and therefore alteration, stages.

4 MINERALOGY

The mineralogical characterisation of a terrain is important to evaluate probable ranges in the geotechnical properties, their variability and sensitivity to variation in the surrounding conditions (pore pressure, stress history, chemism of the pore fluids, etc.). The
Fig. 2 Three component clay mineral variations in the formations under study. 1: Heterogeneous Complex; 2: Val Marecchia Shales; 3: Olistostrome; 4: Sillano Formation (La Verna); 5: Sillano Formation (Impruneta); 6: Sillano Formation (S.Ellero-Carbonile).

Mineralogical composition of the clayey fraction, in turn, is influenced by many factors, among which the grain size, that controls in particular the kind of predominating clayey minerals.

As regards the clay minerals content, the samples analysed highlight certain high concentration areas in the diagram of fig. 2.

Among these the area of the samples belonging to the Val Marecchia stands out for its high content of expandable clay minerals.

The Sillano Formation presents the highest degree of variability in its mineralogical composition. The samples, which were taken in three areas of Tuscany, present an expandable mineral content which varies from 20/25 to 70% of the clayey fraction and an illite content generally less than 40%.

The expandable mineral content of the samples coming from the Heterogeneous Complex of S.Marcello is very low.

Samples relating to the same place do not show substantial differences in their mineralogical composition, depending on the sampling sites (in landslides or not).

5 CLASSIFICATION OF THE MATERIALS

Many of the factors influencing the swelling mechanisms also affect, or are affected by physical soil properties such as plasticity.
The expansion processes can be divided into mainly mechanical or physical-chemical ones. Soils may have the same swelling potential, according to their classification, but exhibit very different amounts of swelling (Seed et al., 1962), because of mechanical swelling (elastic and time-depended stress unloading following tectonic uplift and erosion).

This study only analysed the physical-chemical phenomena, using qualitative assessments of the swelling potential to do this. Various diagrams and graphs are used. These are not standardised but are the ones generally used in scientific literature. The aim was to try and compare the results with the ones obtained from the oedometric tests carried out on undisturbed and reconstituted material.

The Atterberg limits were determined on material passing at 40 ASTM sieve. This fact may lead to the underevaluation of the swelling potential and the activity index. In actual fact, Taylor & Smith (1984) stated that the potential is evaluated better if the Atterberg limits are determined on material passing the n. 200 ASTM sieve. However, the data would not have been comparable with the ones obtained from existing literature.

From the point of view of plasticity the materials can be classified as follows (Fig. 3).

Sillano formation can be classified as CL, with LL values ranging from 25% to 50%, most of the samples fall in the fields CL and CI; the values increases regularly in a position close to curve A.

The samples of the Heterogeneous Complex in the San Marcello area can be classified as CI, with IP values ranging from 20% to 30%.

We have more dispersed data for the Olistostrome of Sasso di Simone, i.e. between CI and CH, and for the Val Marecchia shale samples we have values of extreme plasticity, with LL values of over 100% and PI values greater than 60% (CE).

Therefore we have four clear groups of samples and they can easily be identified from the point of view of plasticity.

Lower friction angles and greater cohesion generally correspond to greater plasticity and greater activity (Mitchell, 1976).

When we classify the material from the point of view of potential expansiveness (Van der Meere, 1964)(fig.4), based on the degree of activity of the material, we can clearly identify 4 descriptive fields for the behaviour of the material. The samples of the Sillano formation...
Fig. 4 Determination of potential expansiveness of soils (symbols as in Fig. 2).

can be defined as having low, or at the most, medium swelling characteristics; the activity remains below 0.5 in most cases.

San Marcello Heterogeneous Complex presents a high potential of expansiveness, with an activity index close to 1, in spite of its low content of expandable clay minerals.

As far as the Olistostrome of M.Simone-Simoncello is concerned, the samples are characterised by a low swelling potential and an activity index lower than 0.5.

Finally, the Val Marcechia samples differ in this case too. They are defined as having very high expandability and an activity index of between 1 and 2.

In this case too it may result that the values are underevaluated with respect to the ones which can be determined using values obtained on material sieved at 200.

In fig. 5 the materials are classified according to the Seed et al. (1962) diagram, which defines the swelling potential on the basis of the relationship between Ac and CF. Once again here we have considerable dispersion of the data relating to the Sillano Formation which is classified as having a low-medium potential, while the Heterogeneous Complex is low and the Olistostrome is medium-high. The Val Marcechia shales differ once again, with a very high swelling potential.

As far as the FSI value is concerned, it remains below 100%, and below 50% in most cases. Only the Val Marcechia materials exhibit FSI values over 150%, up to 450%.

Holtz and Gibbs (1956) stated that soils having FSI as low as 100% may exhibit considerable expansion in the field when settled under light loading. Although soils with free swell values below 50% are not considered to exhibit appreciable volume change, Dawson (1953) reported that several Texas clays with free swell values in the range of 50% have caused considerable damage through expansion (Nelson & Miller, 1992)

6 CALCIUM CARBONATE CONTENT

As well as being controlled by the mineralogy, the degree and speed of expansion in undisturbed samples are also controlled by diagenetic bonds and by cementation. In this
Fig. 5 Evaluation of swelling potential according to Seed et al., 1962 (symbols as in Fig. 2).

sense we tried to observe if there was a link, and if so, what link there was between the parameters which describe the behaviour of the disaggregated materials (plasticity, activity, FSI) and the calcium carbonate content.

Calcium carbonate is present in these formations as precipitated and recrystallized, in the micro fissures that separate scales, during diagenesis and tectonic stressing; it can also be precipitated by fluids circulating in the most surficial weathered belts.

The influence of the calcium carbonate on the mechanical behaviour of the clayey rocks is not well understood, depending on the different scale of the mass.

We have considerable dispersion of the data in all the graphs correlated to the calcium carbonate content (Fig. 6 and 7).

The variations in volume are linked not only to the mineralogy but also to the water content, the nature and concentration of the interstitial solutions and the structure and degree of cementation. In order to have some idea of the type of structure present in the shales under examination, analyses are carried out using a scanning electron microscope (S.E.M.)

Tests are underway on all the samples, in the body and outside the landslide, as a preliminary result. Fig. 8 shows the micro texture of a sample coming from a landslide body which started in the San Marcello Heterogeneous Complex, which is composed mainly of illite and also of kaolinite and chlorite. Here we can observe subparallel orientations of the single domains, in a very compact structure. In the San Marcello samples all the structures are greatly disturbed, even the samples taken from outside the landslide area.

Oedometric tests on a wide range of pressures were carried out on samples of the Sillano Formation and the Val Marecchia shales, using a special cell with a diameter of 27.5 mm. As a term of reference, tests were carried out on the same samples, reconstituted with a water content close to 1.2-1.3 times the liquid limit, in order to obtain the intrinsic compressibility curves (Budland, 1990). The intrinsic curves of the Sillano samples are all very similar, given their similar compression characteristics, and they show that the materials are fairly compressible and prone to swelling after remoulding. The properties of the undisturbed materials show distinctly lower compressibility index values Cc with respect to the intrinsic ones. This is due to the effects of the structure which was produced by the complex stress history and the incipient lithification (SEM and calcium carbonate...
Fig. 6: PI versus CaCO₃ content (symbols as in Fig. 2).

Fig. 7: FSI versus CaCO₃ content (symbols as in Fig. 2).

Fig. 8: Scanning Electron Microscope of Heterogeneous Complex (landslide body).

Fig. 9: Compressibility test data. Sillano Formation, La Verna: reconstituted (1) and undisturbed (2) samples. Val Marecchia Shales: undisturbed (3) samples.
content analyses seem to confirm this interpretation).

The results of the oedometer tests carried out on the Val Marecchia shale samples show higher values, both for the compressibility and the swelling indexes, with respect to the ones determined for the Sillano Formation. The data on the undisturbed samples have not allowed us to make a reliable estimate of the swelling and overconsolidation pressures as the samples were taken near the surface. The values lie in the range of 0.6-0.8 MPa for the swelling pressure and $\sigma'=1$ MPa for the effective overconsolidation pressure.

7 CONCLUSIONS

The forecast of the mechanical behaviour of the clayey materials in relation to expansion and variations in the shear strength, without geotechnical data, or to complete the existing data, is provided by the analysis of some parameters, which are quick and easy to determine, such as the mineralogical composition, in particular that of the clay fraction, and the determination of the plasticity characteristics and the grain sizes of the materials.

In the case of highly overconsolidated clays and shales, the qualitative assessment of the mechanical behaviour, supplied by classifications based on the determination of CF and PI values, is often insufficient. This is because it is determined on properties of the disaggregated terrain and fails to take the effects linked to the structure of the material and its stress history into account (Taylor & Smith, 1984).

In particular, the degree and speed of the variation in volume, resulting from variations in the water content and variations in the in situ stress, are controlled by diagenetic bondings, cementation and fabric orientation, as well as by the mineralogy.

However, the preliminary oedometric data obtained for some terrains (La Verna, Simoncello) seem to agree with the evaluations obtained from the classifications used.

It was not possible to assess the influence of the calcium carbonate content on the swelling parameters as the variables which control the phenomenon are many and are not easy to isolate. In particular, it is difficult to separate the influence of the simultaneous presence of many clay minerals characterised by different values of activity, which determines sensible variation in the complicative activity of the material. This leads generally to a reduction in the activity indexes.

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Experimental research on the causes of the fissures in the fissured clay 

L'étude expérimentale sur la cause des fissures dans l'argile fissurée

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ABSTRACT, Through simulated tests on the formation-transformation of the deposited clay and the analyses of the microstructure of the fissure clay and the non-fissure clay, the following paper discusses the surroundings and conditions in which the fissures in the fissure clay came into being. It puts forward the idea that after the deposition of clay, the clay body had not yet turned into stable textural system and the firm jointing power had not formed, the covering layer became eroded, and vertical rebounding occurred because of unloading, thus the fissures were created.

RESUME, Dans cet article, l'auteur a discuté des environnements et des conditions qui ont causé la formation des fissures dans l'argile fissurée au travers de la simulation de la formation et la transformation de l'argile sédimentaire et l'analyse de la microstructure de l'argile fissurée et de l'argile sédimentaire. On a donné une idée que, après la sédimentation de l'argile, le corps de l'argile n'a pas encore produit d'un système de texture stable et l'adhésion solide n'a pas été formée, la couverture avait été érodée, les fissures ont donc été formées à cause de la décharge produisant le rebondissement vertical.

1. Introduction

Fissure clay is widely found all over the world. For example, the London Clay of Britain, Hannover Clay of Germany, Bangkok Clay of Taiwan, South Africa Fissure Clay and Chengdu Clay in China are all typical fissure clays. Because the existence of the fissures in the fissure clay spoil the texture of the clay and changed the nature of the clay body, the peak strength of the clay became 50-30% lower or even lower (1). In the areas in which fissure clay is distributed, a lot of clay slopes came into being, the ground slides due to the instability, and the accidents of the building industry have attracted extensive attention. Since K.V. Terzaghi first studied fissure clay in 1936, the scientists of many countries have conducted a lot of systematic research work in this field. They have now acquired a rather extensive knowledge of the fissures in fissure clay, both theoretically and practically. However, about the causes of the fissures of the fissure clay, they have put forward different hypotheses and cannot agree (3). These hypotheses are as follows:

1. Due to the synoptosis and compaction and the chemical changes during the deposition,

2. Because of the natural physical-chemical changes caused by the forces of groundwater, weathering and ion exchange after the deposition and compaction of the clay,

3. Owing to the stress of structures and earthquakes during the flood or shearing of the rock stratum,

4. By the non-tectonic movements such as wriggling, unloading rebounding or the relief of stress during erosion,

5. Coming from the underneath rock.

The above hypotheses can be classified into two main groups, i.e., formed by the tectonic forces and non-tectonic forces such as physical and gravitational forces, into which intensive research has been made. Their distribution, state and characteristics are strictly controlled by applied forces, and each has its own regularities, quite different from the unidirectional net fissures which exist widely in the fissure clay. The weathered clay fissures can only be found in the body of top clay, while the "dehydrated shrunk fissures" of needs a further intensive research. This article will try to make a further research into
the causes of fissures in the fissure clay created by deposition. The fissure clay to be discussed in this article is defined as, the unidirectional not fissures with dip angles between 10 to 60 which exist widely in the clay. The length of the fissures varies from microns to several meters. The surfaces of the fissures are smooth with wave-like undulation, along with the clay is subject to splitting under external forces.

2. The Experiment of the Causes of the Formation of Fissures

Most of the fissure clays over the world were formed by underwater deposition. Each of them had its own geological development of deposited formation and transformation. However, there was one thing in common, i.e., the clay particles of deposited materials came from suspended substances which distributed in the running water. In this experiment, the samples were made from suspended clay liquid.

2.1 The conditions of the experiment and the making of the samples

The pure red-yellow clay was taken from the Brick Factory in the East Suburbs of Chengdu, Sichuan provience. The clay is made up mainly of illite and kaolinite with a little montmorillonite. The physical nature and chemical composition for the clay used are shown as follows:

<table>
<thead>
<tr>
<th>Content</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Content of granulated clay (2x10^-10 m)</td>
<td>56.52</td>
</tr>
<tr>
<td>Specific gravity T</td>
<td>2.74</td>
</tr>
<tr>
<td>Moisture content W (%)</td>
<td>21.40</td>
</tr>
<tr>
<td>Natural bulk density ρ (kg/m³)</td>
<td>1900.00</td>
</tr>
<tr>
<td>Dry unit weight rd (kg/m³)</td>
<td>1700.00</td>
</tr>
<tr>
<td>Content of chemical composition</td>
<td></td>
</tr>
<tr>
<td>SiO₂</td>
<td>45.20</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>26.30</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>9.56</td>
</tr>
<tr>
<td>MgO</td>
<td>2.37</td>
</tr>
</tbody>
</table>

Colour: Red-yellow

The sample clay was put into a container fitted with distilled water and stirred with a machine for a long time. Thus the granulated clay was fully scattered in the water which became clay-suspended liquid. The clay suspended liquid was then poured into a glass container which was 4x10^-10 m in diameter and 5x10^-10 m in height. While the depth of the liquid is about 2x10^-10 m. After it was put still and the clay deposited, a five stainless steel circular cutters, 4x10^-10 m in diameter and 4x10^-10 m in height, were put in with blades upward and a piece of permeable stone under each cutter. The five cutters were even on the same surface with certain distance between the cutters and between glass container. Every day some more clay suspended liquid was slowly added to the container and the clay deposited into the circle cutters as well as toward the bottom of the container. When the height of the deposited clay became 4x10^-10 m, three sheets of filter paper and a iron plate with small holes in, both of which were of the same diameter as the container, were put into the container. A clump of iron was put on the plate, which exerted stress of 5kPa. Water was forced out through the filter paper and the holes in the iron plate. When this was done and the state of the clay became stable, the cutters with deposited clay was carefully taken out and put aside as samples for further compression experiment.

A similar experiment was being made with only one difference, i.e., whenever the depth of deposited clay became 1x10^-10 m, several brown paper cups with an epoxy resin coating were put even at the same spot. The cups were 2x10^-10 m in diameter and 2x10^-10 m in height with the mouths upward. The clay deposited into the cups. When this deposited clay became 4x10^-10 m deep, no more clay-suspended liquid was poured in. After the clay had been put still for two months, the cups with clay in were carefully taken out. The clay was dried in originat shape with the acetone-isopentyl alcohol method and put aside for electronic scanning. It is worth white to mention that there were unidirectional fissures formed during the deposition, which were visible to the naked eye. W.A. White has named such fissures as synerisis fissures of clay, and regarded them as the fissures in the fissure clay(C).

2.2 Compaction Experiment

In order to stimulate the geological development environment of the deposition formation-transformation under different geological conditions and the loading process, the experiment was done in two different ways.

2.2.1 Compaction-undloading experiment

The experiment was made on the compression instrument according to the saturated samples of the Experiment Regulations(C). The largest loading weight were respectively 200, 400, 600, 800, 2000, 3000kPa. When the samples gained their largest loading weight, they were given a long static period (about 15 days). The samples were compacted and fired, and the transformation tended to be stable. Then the samples were being unloaded gradually. A certain amount of the loading weight was kept...
removed by 1/2, the loading weights were kept unchange for a long time so that they were able to rebound fully till the change of shape became stable. Then the water in the water groove of the instrument was drained off and the samples were slowly made to dry under the 1/2 largest loading weight. The dried samples were taken out as samples for electronic scanning.

2.2.2 Compaction-non-unloading Experiment

The compression experiment was conducted in the same way as described above. However, when all the samples gained their largest loading weight and were stabilized for a long time, they were compacted and firm, the transformation tended to stable. Then the water in the water groove of the instrument was drained off and the samples were gradually dried under the effect of the largest loading weight. The dried samples were put aside as samples for electronic scanning.

2.3 Microstructural Features

2.3.1 Deposited Clay Samples

When the deposited samples in the paper cups were dried, they were observed vertically and horizontally with an electronic scanning microscope. It was seen that the structure of the deposited clay was porous and there were roundish cavities 5x10−5x10 m in diameter scattered in the clay body. See Photo.1. No "synereisis fissures" can be that were visible to the naked eye during the early period of the deposition, Deposited clay is a very high porosity clay body with tunneling structure. From Photo.2 it can be seen that the porous honeycomb microstructure made up of flocculent of micro-assembly granular have been formed from the clay mineral surfaces touched with each other. Such porosities are mainly made up of honeycomb porosities, which are 5x10−15x10−m in diameter.

2.3.2 Compaction-unloading Samples

In Photo 3 and Photo 4 is a sample with a 600KPa loading weight which has been compacted and unloading to 300KPa. Under the influence of loading weight and later loading weight, the flocculent has turned into the microstructure in the micro-assembly clay granular mainly consist of little. Because of unloading, fissures have been formed in the clay body and appears to be wave like in shape. Directional arrangement can be seen along the two sides of the fissures. The dip angles of the fissures were chiefly from 20° to 50°. Observed from different positions, the directions of fissures appear to be unidirectional.

In Photo, 5 and 6 are samples with a loading weight of 1000KPa turning to 500KPa after being unloaded. In Photo, 7 and 8 are samples with a loading weight of 3000KPa after being compacted and firm. With the increase of the loading weight, the degree of compaction of the clay become higher and unidirectional fissures come into being, with dip angles mainly from 10° to 50°, while the undulated surfaces of the fissures gradually become even and straight. The micro-assembly clay granulars become more directional and the fissures cross through the clay body.

2.3.3 Compaction-Non-unloaded Samples

Photo, 9 are non-unloaded samples with loading weight of 600KPa. The flocculated assemblage clay granulars indicate the transformation of honeycomb microstructure into matrix microstructure. The structure of the clay body is loose and even. There are axile like crevices with similar radius or very narrow crevice among assemblage clay granulars, two to ten micron in diameter. The structure is continuous without fissures.

Photo, 10 are samples of non-unloaded samples, whose loading weight is 1000KPa. Flocculated assemblage clay granulars appear to be matrix microstructure. The joints of the granulars are rather close. Crevices become smaller and axile like with similar radius or very narrow, among the assemblage clay granulars 0.5-6 micron in diameter. The structure of the clay body is even and continuous without crevices.

2.4 Discussion

Why at the early stage of the clay deposition there were dehydrated fissures visible to the naked eye, while in microstructure of the clay in the original state there was no fissures? This is because during the deposition, the clay body was compacted and the fissures were closed. A flocculated structure was created among the assemblage clay granulars and thus the fissures disappeared.

Under the influence of the covering loading weight, clay would be gradually dehydrated, shrink and compact. The loose honeycomb microstructure formed during the deposition would gradually turn into rather compacted matrix microstructure. If the covering loading weight remained unchange, the clay body would continue to dehydrate and solatify. The colloids of the
clay would gradually turn into a rather compacted matrix microstructure. If the covering looding weight remained unchange, the clay body would continue to dehydrate and solidify. The colloids would become less active while the contacting effect in the microstructure of the clay and the physical-chemical effect become stronger. The strength of the structural system increases.

The clay gradually grows rocky due to "ageing" (in this experiment dehydration-drying method was adopted). No fissure system would occur in the clay body and the clay will not become fissure clay. After dehyration, shrinkage and compaction, if unloading occurred before the clay turned into rock through "ageing", rebounding would happen in the clay and the volume would expand. This was due to the spherical stress like changes in the clay body during the deposition and compaction. Because of unloading, the horizontal stress becomes larger than the vertical stress and thus deviator stress come into being. Under the effect of certain deviator stress, the former vertical compression of the clay body would gradually turn into vertical expansion. Thus fissures formed in the clay body, such is the cause of the fissures in the fissure clay.

3. The Comparison of the Microstructural Features of Clays

In order to compare the microstructure of the fissure clay and the non-fissure clay, we have analyzed the microstructures of Chengdu clay and Shi Geda clay, both of which have similar composition and deposition, but have experienced different geological development environment.

3.1 The Microstructural Feature of the Chengdu Clay, A Typical Fissure Clay,

Chengdu clay is a typical fissure clay(6), which is distributed over the three-stage land of Chengdu Plain. Formed in Middle Pleistocene Epoch, it is the clay layer of running water deposition. At the very beginning, the thickness of the clay was 25 meters. Owing to the denudation, today the thickness parts are 12 meters. The microstructure of Chengdu clay is mainly matrix microstructure(7). See Photo. 11. Not fissures are easily seen in the clay body. Seen Photo. 12. The fissures have spoiled the original structure of the clay body. The fine arrangement of the assemblage clay granular can be seen along the surface of the fissures. See Photo. 13. Chengdu clay is of the same microstructural features and micro-fissure state as those of the compression-unloading clay described above.

3.2 The Microstructural Features of the Non-fissure Clay Shi Geda Clay

Shi Geda stratum is widely distributed in the south-west Sichuan, which was a take deposited stratum of the early Pleistocene Epoch, with a thickness of over 200 meters. The middle and lower parts chiefly consist of clay layers and dusty sand clay layer, both of which are mixed together with one taylor above another. The upper part is made up mainly of fine sand layers which sandwich the clay layers. This kind of stratum is well-known for its tendency to slide. The Shi Geda clay stratum which is distributed in large areas arranges most in horizontal. The stratum is stable, the structure intact and the body compact, without the development of net fissures. Therefore it is not counted as fissure clay.

The basic microstructural unit of Shi Geda clay is the micro-assemblage granular made up mainly of clay minerals, including illites, kaolinite and a small amount of montmorillonite (8). Between the assemblage granular is flocculated structure of transition mud and some facies which is formed into conglutination like contact of the surface to side and surface to surface. See Photo. 14. Along the surfaces of the microstructural units, fine direction arrangement can be seen. See Photo. 15. Most the structural cavities, 10 to 15 microns in length, are narrow crack in shape and are among the granulars. The clay body is compact without net fissures. See Photo. 16. During its deposition, the Shi Geda clay experienced compact dehyration due to the covering load. After the deposition, as the second Glaciation Epoch had began, the clay was thus covered by red grit layer of the upoch (9). Therefore the Shi Geda clay was subjected to a rather heaving loading so that the assemblage clay granulars turn into a concervation-same facies mould like and stream like microstructure. Because quite a few Ca and Fe particles have changed into bonds and iron-static electricity jointing. The anti-shear strength of Shi Geda clay may be as high as $\gamma = 35^\circ - 45^\circ$, $C = 150-220kPa$. The single exist anti-pressure strength may be as high as $C = 1000-1500kPa$. After the erosion of the upper layer and unloading, not fissures would not appear in the clay body with such stable structure and strong jointing effect.
4. Discussion of Developmental Surroundings and Conditions of Fissure Clay

Through simulated experiment of the formation-transformation developmental geological environment and conditions in which the fissure clay came into being, it is clear that there were specific environment and conditions for the formation of the clay.

4.1 Depositional and Flocculated Effect

During the deposition, assemblage clay granulites formed from the assembled clay minerals turned into various microstructures owing to compaction under the inter-effects of long distance motecenes, magnet and earth's gravity. Physical-chemical compaction and syneresis are important for this change. In this stage, the structural system of clay gradually became compact, and structural strength grew greater, but net fissures would come into being in the clay structure. Therefore, such clay was not fissure clay.

If deposit clay continued to be subjected to a rather heavy loading and dehydration for some time, the area of contact surface would become larger and larger, the flocculated contact would gradually turn into transition mold or some facies would contact, and bond particle-static electric effect and physical-chemical cementing would become the dominated jointing force. The structural strength would be even greater. But this kind of clay still was not fissure clay.

4.2 Unloading and Rebound

When the conservation contacted microstructure formed from deposited clay substances had not been further dehydrated and turned into some facies contact or conservation contact like structure, i.e., had not formed into bond, particle-static electric or physical-chemical cementing, the covering stratum would erode and the clay body would be largely unloading. At this time, the horizontal stress was much stronger than vertical stress. With the development of unloading, when the difference between horizontal stress and vertical stress increased to a certain value, the clay would expand and rebound in the vertical direction, and the structure of the clay body would be spoiled so as to form a unidirectional net microfissure system with small to medium dip angle. The clay would now turn into fissure clay. 400 meters of covering layer of the marine deposited London Fissure clay was eroded due to the rising after deposition. It was typical that 1000 meters of covering layer of Upper Lias Fissure clay was eroded due to the rising after deposition. A.W. Skempton discovered that the horizontal stress is much greater than the horizontal stress in the London Clay. (10) Skempton has measured that in the London Clay, the pressure coefficient Ko at rest earth (Ko-horizontal stress/vertical stress) varies with the depth, from 1.65 of the depth of 30 meters to 2.5 of 7 meters. (See Fig. 1)

![Fig. 1](image)

Fig. 1 In London Clay, the clay pressure coefficient Ko at rest earth varies with the depth under underground. (According to Skempton, 1961)

4.3 The Late Evolution Fissures

The micro-fissures in fissure clay was subjected to the coefficient of natural forces, such as weathering, the geological structure and sliding gravity, especially the movement of ground water and weathering. The microfissures would be continued, developed and transformed. Fissures would be opened, developed and linked up by various influences. The arrangement of assemblage clay granulites along the fissures would be more directional form into smooth fissure surfaces and the composition of minerals may change, thus the fissures in the fissure clay came into being, which we can see macrocosmically today.

5. Summary

1. The "syneresis contracted fissures" in the early stage of deposition of the deposited clay would disappear as the deposition continued and no such fissures as are seen in fissure clay would come into being.

2. A series of physical and chemical
changes would happen in the clay under the effect of covering loading. If this loading was not removed the clay would turn to compact, but no fissure system would be created in the clay body. No fissure clay would be formed.

3. If the covering stratum was eroded and unloading occurred in the clay body of the compact deposited clay before the firm structural jointing formed, the horizontal stress would gradually become larger than the vertical stress. When the horizontal diviator stress came to a certain value, i.e., greater than structural strength in the clay body, rebounding would occur in the vertical direction in the compact clay body, the structure of the clay body would be spoiled and unidirectional mild-medium dip angle would be formed in the micro-net fissure system. The clay would become fissure clay. 1:2 of the covering soil was removed in this experiment. It has been also discovered that with the increase of the covering loading the compaction of the clay would increase, unidirectional fissures would develop, the wavy surfaces of the fissures would tend to be even and micro assemblage clay granular along the fissure surfaces would become more directional. The dip angles of the fissures were usually $10^\circ-50^\circ$.

4. Though the analysis of the geological surrounding changes of the typicl fissure clay and the non-fissure clay, and the analysis of the microstructural features of the clay, it has been further proved that the fissures in the fissure clay were formed by the unloading-rebounding of the clay body before stable structural system and firm jointing were formed in the clay.

This research was made when the writer was studying at Chengdu Institute of Geology under the guidance of Prof. Kong Defang. Assoc. Prof. Zhang Huiling, Prof. Zhao Zhanan and Mr. Li Rongqing, a PhD student, all gave me a lot of aid. Here the writer expresses his heartly gratitude towards all these people.

Reference


Variation of physical and mechanical behaviors of volcanic rocks in Cappadocia as a function of temperature
Variations des comportements physiques et mécaniques des roches volcaniques en Cappadoce, en fonction de la température

M. Erdogan & E. Yuzer
Technical University of Istanbul, Faculty of Mining, Turkey

ABSTRACT: Cappadocia region is generally covered by various volcano-sedimentary rocks. Stratigraphically five different volcanic rocks are differentiated in the study area. This volcanic units have been named as "Urgüp formation". Some of the rock types, such as tuffs, lahars and ignimbrites, are evaluated as building materials due to their lightness and high quality of insulation properties.

In this article, variation of the physical and mechanical properties of the volcanic rocks as a function of temperature and their behaviors are explained. It was observed during the laboratory experiments that the Kavak tuffs were disintegrated above 800°C and Sarımadetepe ignimbrites around 300°C. The mechanical properties of the Çavuşini tuffs, which have high alkaline oxide ratio, were improved up to 700°C. The rock was deformed by melting above this temperature. The mechanical properties of the Tahar tuffs were twice improved and local deformation were noted above this temperature related to the partial melting. It was also determined that the Karadan ignimbrites could be heated up to 1450°C without any partial melting. This sample was excessively expended after 800°C, because of its secondary active silica content.

RÉSUMÉ: La région de Cappadoce est couvert par les séries volcano-sédimentaires, ou nous avons distingué cinq unités volcaniques émises par différentes époques géologiques, qui composent "la formation Urgüp" et se change du tufs aux ignimbrites dont les propriétés physico-mécaniques sont très favorables à l'utilisation comme la pierre de construction. D'autre part, au niveau de tufs et de tuffites qui ont l'avantage d'une facilité d'extraction et d'excellents propriétés d'isolation thermique, dans les quels on peut rencontrer des églises rupestres, des villes souterraines historiques et des autres cavités étant bénéficiées, actuellement, de dépôts pour conserver des fruits et des légumes.

Dans cet exposé, on a expliqué les résultats de la recherche réalisée, dans le laboratoire, sur les roches volcaniques pour pouvoir constater la variation du comportement physico-mécanique en fonction de la température. On a constaté que, sous l'effet de température, le tufs de Kavak à 800°C et l'ignimbrite de Sarımadetepe à 300°C ont été éclatés et le tuf de Çavuşini a été fondu entièrement. Par ailleurs, les propriétés physico-mécaniques du tuf de Tahar et de l'ignimbrite de Karadan ont résisté à la chaleur jusqu'au 1280°C - 1450°C.

1 INTRODUCTION

The study area is situated in Cappadocia, in the middle of Anatolia as seen in Figure 1. Historical Churches, underground cities and fairy chimneys are found in the region covered with volcanic formations. The rocks having high porosity values are very good isolating materials against temperature and noise and have been used for building materials since the antiquity. They are decorative and easily cut.
Figure 1. The situation of the investigation area.

As a result of field studies, the volcanic stratigraphy is divided into five units. Rock samples were collected from each unit and subjected to laboratory tests. During the heating tests the samples were heated gradually up to 1400°C and it is observed that they behaved differently and the research is directed towards this phenomenon.

The physico-mechanical tests were carried out almost on 1000 samples and the effect of temperature on the mechanical behavior of volcanic rocks have been tried to explain.

2 REGIONAL GEOLOGY

The region emerged after late Eocene and lakes were formed depending on paleotopography. Phyroclastics were deposited in late Miocene, as a result of Island arc volcanism caused by the collision of the African and Anatolian plates. These volcanic rocks are characterized by tuffs at the bottom, lahar in the middle and ignimbrites at the top, and they are called as Ürgüp Formation (Figure 2). As shown in figure 2, there are other sedimentary deposits with abundance of volcanic clasts in this unit indicating interruptions of the volcanic activity. The volcanic activity continued during entire Pliocene and ended in the Quaternary with basalts. This formation includes: a) Kavak unit, tuff and tuffites, b) Sarmsadentepe unit, ignimbrites, c) Çavuşini unit, pumiceous tuff, d) Tahar unit, lahar and Karadağ unit, ignimbrites. This unit shows dastic, andesitic and rhyolitic compositions indicating that they are products of a calc-alkaline volcanism. The mineralogical

<table>
<thead>
<tr>
<th>STRATUM</th>
<th>FORMATION</th>
<th>UNITROCK</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>EOLIANE</td>
<td>NALCAY</td>
<td></td>
<td>Less porosity caused of rock, crevices and solution with the upper parts are made up of basaltic rocks with fresh water minerals and structures with epidote.</td>
</tr>
<tr>
<td>EOLIANE</td>
<td>NALCAY</td>
<td></td>
<td>Less porosity caused of rock, crevices and solution with the upper parts are made up of basaltic rocks with fresh water minerals and structures with epidote.</td>
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<td></td>
<td>Less porosity caused of rock, crevices and solution with the upper parts are made up of basaltic rocks with fresh water minerals and structures with epidote.</td>
</tr>
</tbody>
</table>

Figure 2. The columnar section of the investigation area.

compositions of these units are listed in the columns of figure 2. This research concerns of laboratory analyses of the Kavak, Sarmsadentepe, Çavuşini, Tahar and Karadağ units.

3 PHYSICAL PROPERTIES AND HEAT EFFECT

Hardness, linear expansion, porosity, gas permeability and thermal conductivity tests were conducted on the samples taken from the Kavak, Sarmsadentepe, Çavuşini, Tahar and Karadağ units. The tests mentioned above were realized at different temperatures below the melting point.

The testing temperature values for the Sarmsadentepe ignimbrite increased up to 300°
C and later the sample was fractured. However the testing temperature could increased up to 600° C for Çavuşini tuff and 800° C for Kavak tuff and later the samples started melting. For the Tahar tuff and Karadağ ignimbrite the test temperature was increased up to 1200° C.

As seen in figure 3, the rock hardness decreases with increasing water content. However the hardness increases with testing temperature.

![Figure 3. The variation of hardness of volcanic rocks with testing temperature.](image)

As the hardness values of the Tahar tuff increase suddenly after a temperature of 1000° C, in other rock samples the hardness values show very small increases. Measured values of coefficient of linear expansion of the volcanic rocks subjected to the research, increased at different rates for a temperature up to 600° C. The highest linear expansion was measured at Karadağ Ignimbrite (Figure 4). The porosity values are found to be changed between 18 % and 44 % at room temperature and increased with testing temperature (Figure 5). In the examinations done on the test samples, it was determined that local meltings by the effect of heat, shrinkage developed along the boundary of kaolized feldspar minerals and fractures caused by intensive expansion, were effective in the increase of porosity. Permeability tests carried out on the volcanic rocks with a gas permeameter showed that pores of the Kavak tuff, the Tahar tuff and the Karadağ ignimbrite were not connected to each other (Figure 6). In these rocks gas permeability gradually increased due to local meltings and intensive dilatation under the effect of heat. As seen on the same figure, pores in the Sarımadentepe Ignimbrite and the Çavuşini tuff were connected even at room temperature. It is seen that the gas permeability of the Çavuşini unit which is completely formed by pumiceous tuff increases rapidly after a temperature of 100° C (Figure 6). In addition, results obtained from water permeability tests conducted on these rocks were found to be parallel to the results of gas permeability tests (Figure 7).
those of other volcanic rocks showing the character of being tuff and having higher porosity values. Thermal conductivity value increases with increasing moisture content values (Figure 8). On the other side, thermal values of the test samples remained at the different temperature levels decreases due to removed absorbed and molecular water from the sample (Figure 9).

Figure 6. The variation of gas permeability with temperature in volcanic rocks.

Figure 7. The variation of permeability in volcanic rocks with temperature.

It was determined that thermal conductivity values of ignimbrite rocks were higher than

Figure 8. The variation of thermal conductivity with temperature in volcanic rocks.

Figure 9. The variation of thermal conductivity with temperature in volcanic rocks.
4 THE EFFECT OF TEMPERATURE ON THE MECHANICAL PROPERTIES OF VOLCANIC ROCKS

Uniaxial and triaxial compressive strength tests were carried out in order to see the effect of temperature on the mechanical behavior of volcanic rocks.

Stress-strain curves of the rock samples are given in figure 10. The following conclusions can be drawn from these figures.

Figure 10. The variation of uniaxial compressive strength with temperature in volcanic rocks.

Uniaxial compressive strength values of Kavak and Çavuşini tuffs increase with temperature up to 400° C and decrease thereafter due to the occurrence of the micro fractures with the effect of high temperature values.

The strength of Sarımadantepe ignimbrites increases up to 300° C and thereafter the samples break suddenly.

The uniaxial compressive strength values of Tahar tuffs remain constant up to 800° C and increases rapidly between 800° - 1200° C due to the effect of sinterization and samples start melting after 1280° C.

The strength of Karadağ Ignimbrites increases up to 700° C and decreases thereafter due to the occurrence of micro-fractures.

The mechanical behavior of the rock samples under triaxial compression tests are given in figures 11, 12, 13,14 and 15. The results are in parallel to the tests results given above.

Figure 11. The behavior of Kavak Tuff under different conditions and triaxial stresses.

Figure 12. The behavior of Çavuşini Tuff under different conditions and triaxial stresses.

Figure 13. The behavior of Sarımadantepe ignimbrite under different conditions and triaxial stresses.
For the reason of the vitric texture of Sarımadentepe ignimbrite, it rapidly dilated and disintegrated after 300 °C. This showed that the rock does not resist to the fire.

Tahar tuff resisted to melting up to the temperature of 1280 °C and all the mechanical properties of the rock increased after 1000 °C. It is concluded that the rock has fire resistance property. It is also proved form the additional tests that after the rock is cut properly and sintered, it may be used as a fire brick in the industry not requiring high temperatures.

The Karadağ ignimbrite did not have any tendency to melt up to 1450 °C. However, the rock rapidly dilated after 800 °C and the heat expansion reached to 5 % at 1450 °C. With this extensive expansion, capillary cracks were formed on the specimens. For this reason the mechanical properties of Karadağ ignimbrite decreased after 800°C. It is proved that the extensive expansion of the rock was a result of the active silica minerals in the rock.

REFERENCES


Innocenti & al. 1975 The neogene calc-alkaline volcanism of central Anatolia geochronological data on Kayseri-Nigde area. Geol. mag. no. 112/4, p.33-60, Cambridge.


5 CONCLUSIONS

The rocks tested belonged to calc-alkaline type of magma.

The physical properties of the rocks (hardness, porosity, permeability, thermal conductivity) improved dramatically with the increasing test temperature.

Since Kavak and Çavuşini tuffs contain alkaline oxides, they melted at low temperatures 800-600 °C.
Perspectives using time-analogy methods for determination of long-term strength and strain of frozen soils

Perspectives des méthodes des analogies temporelles pour la détermination de la résistance et de la déformation des sols gelés à court terme

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Moscow State University, Russia

ABSTRACT: On the base of experimental data shown possibility to use methods of time-analogies for prediction of long-term strength and strain of frozen soils.

RÉSUMÉ: On utilise les méthodes des analogies temporaires pour la vérification des déformations à long terme des sols d'accord les résultats des essais à court terme.

OBSERVATIONS

Time-analogy methods have found wide application in prediction of long-term strength and strain of solid materials (J. Ferry, 1952; Urzhumtsev, 1982). The essence of these methods is that they accelerate strain and reduce stress by factors which affect these processes. Such factors are higher temperature, load, load rates designated physical properties (higher moisture, density, etc.)

The impact of these factors on strain and strength is similar to that of time. The aim of time-analogy methods is to define a relationship necessary to attain the same strain or strength at different values of the accelerating factor. This relationship is referred to as the reduction coefficient (a = t/t'). Once defined the reduction coefficient accelerates the study of strain development and reduction in strength, permits the use of accelerated test procedures and modelling of material behaviour under load.

There are different theoretical explanation of reduction coefficient recommended that consolidation rate be used as a function of time and temperature (Ferry, 1952; Ilyushin, Pobedria, 1970) established an inter exchangeable affect of temperature and time on stress-strain relationship by considering the dependence of the viscosity coefficient on temperature; other authors treated as a function of coefficients of thermal expansion and diffusivity (Molchanov, Andrievsky, 1967).

Our research of frozen peat (Roman, 1987) show, that rheological properties displayed by frozen soils permit one to use analogy methods for prediction of long-term strength and strain.


The possibility of obtaining experimental data has been determined from by many kind of tests. The test conducted by author proved the validity of the temperature-, stress- and concentration-time analogies.

TEMPERATURE - TIME ANALOGY

Temperature-time analogy method can be used for frozen soils on base data from uniaxial compression, spherical- and plate-loads tests. The tests should be run under strictly observed isothermal condition. Identical samples should be tested minimum at six different temperatures. Stress specified for each test temperature have a considerable im-
impact. Since temperature should be leveled out, i.e., the load effect on strain should be brought to the same level throughout the test temperature range.

For uniaxial compression or plate-loading tests, this requirement is met by stress being equal fraction of instantaneous strength. In this case, the impact of loads is identical at all temperatures and the strain depends solely on rheological processes affected by temperature-dependent molecules mobility. A series of tests is conducted to select stresses for test temperature:

$$\sigma_i = \left( \frac{\sigma_0}{R_{\theta,0}} \right) R_{\theta,i}$$  \hspace{1cm} (1)

where $\sigma$ is the stress applied to the sample at the base temperature; and are ultimate short-term strength at base and temperature, respectively.

In spherical-load test soil stress varies in the course of the experiment. During tests at varying temperatures identical stressed states will be achieved if strain-time dependence is invariant. In this case, a requirement should be met which specifies relationship between settlement and the spherical load diameter (d) for equivalent cohesion tests: $0.05 < d_S < 0.05 d$; $S_d$ is load settlement for 15min (P'yotov, 1973). Parameter I can be found for plastic frozen soils under spherical-load tests from equation (Roman, 1967):

$$J = 4S_d^{1/2} (\rho - S_d)^{1/2} \frac{1}{S_d (1 - M_S)}$$  \hspace{1cm} (2)

where $P$ is load on the spherical load; $S_d$ is load settlement over time period $t$; $d$ is load diameter; $M_S$ is mass coefficient.

Physically, the application of temperature-time analogy for frozen soil cases can be substantiated by the following reasons: Two major processes which reduce creep with decreasing temperature can be single out: 1) lower unfrozen water content; 2) lower mobility of the restfrozen soil components. The model of a viscoelastic body was used to determine the temperature-time reduction coefficient. It considers a strong temperature impact on the behavior of viscous elements and significantly smaller one on elastic elements. The validity of the model for the frozen-soil cases is supported by the studies of the instantaneous elasticity modulus $E_{\text{inst}}$ for frozen soils and ice (Roman, 1987). It has been established that at temperatures below intensive phase changes temperature has little effect on $E_{\text{inst}}$. For ice $E_{\text{inst}}$ is practically constant beginning from freezing temperature and lower. Hence, reasons for the application of the temperature-time analogy should be generally valid for frozen-soil cases. Yet, the impact of moisture phase changes which significantly affect thermomechanical behavior should be taken into account. Two types such behavior are distinguished.

1. Compliance at temperature variations solely depends on molecule mobility. Viscous and elastic compliance curves for different temperatures are similar. The reduction coefficient is a function of only one argument, i.e., temperature. Plots $J-t$ can be shifted along the time axis. Their mutual similarity will not be disturbed. This permits graphical determination of the general compliance curve. Bodies with such strain refer to the category of thermorheologically simple.

2. The strain is affected by structural transformations which make $J-\Delta t$ relationship more complex. Curves cease to be similar. The reduction coefficient becomes dependent on strain time and temperature. $J-\Delta t$ curves obtained at different temperatures shift both horizontally and vertically. Bodies with such strain refer to the category of thermorheologically complex.

Experimental data is processed in the following sequence: computation of viscoelastic compliances:

$$J = \frac{\dot{\epsilon}}{\sigma}$$  \hspace{1cm} (3)

where $\dot{\epsilon}$ is mean creep strains; $\sigma$ is stress; plotting of $J-\Delta t$ with confidence intervals; selection of the "base" temperature $\theta_b$ equal to any experimental. If the reduction coefficient is found graphically, $\theta_b$ is chosen closer to the lower temperature interval. This is followed by determination of the time shift $\Delta \ln Q$ between each pair of neighboring curves. If test samples are thermorheologically simple bodies
\( \Delta \ln A(t) \) is taken as equal to the mean value of \( \Delta \ln A(t) \) over the test period. Experimental values of \( \Delta \ln A(t) \) are sequentially added together beginning with the zero value, i.e. at \( t = 0 \), which is followed by plotting of \( \ln A(t) - (\theta - \theta_b) \) expressed in the analytical form \( \ln A(\theta) - f(\theta - \theta_b) \). Experimental points on curve \( J - \theta_b t \) are shifted along the log-time axis by a corresponding value of \( \ln A(t) \). The new time for these curves is

\[
\ln (t, A) = \ln (t) + \Delta \ln A(t) \tag{4}
\]

Shifting of experimental curves by the quantity of the new time yields general compliance curve at the "base" temperature for time which exceeds experimental period by several orders of magnitude. The suggested procedure for plotting general curve is applicable to thermoreologically simple bodies when temperature variations affect only horizontal time shift of compliance. In this case, \( J - \theta_b t \) curves are similar.

Determination of the reduction coefficient for thermoreologically complex bodies requires more time. Hence, when the impact of the vertical shift is insignificant it is ignored. If such assumption lead to significant errors the vertical shift is considered based on the reduction coefficient as a function of temperature of time. To this end, Ferri's equation (Ferri, 1953) may be used

\[
\Delta \ln A(t) = \ln A(t) - (\theta - \theta_b)(1 + f(T)) \tag{5}
\]

where \( t_0 \) is any "base" time in the beginning of the test period; \( T \) is time increment relative to the "base" value, \( T = \ln t - \ln t_0 \).

Processing of test data for plastic frozen soils revealed that the behaviour under load corresponds to thermoreologically complex strain. Fig. 2 shows compliance curves for the top complex past subjected to uniaxial compression. Following are mean values of physical properties of test samples: density = 1.08 g/cm³, moisture = 6.47%, particle density = 1.56 g/cm³, degree of decomposition = 17%; freezing point = -0.1°C, Unfrozen water content is given in table 1.

The general curve has been plotted for two cases:
1) test soil is treated as a thermoreologically simple body;
2) test soil is treated as a thermoreologically complex body.

Table 1. Unfrozen water content in peat test samples

<table>
<thead>
<tr>
<th>( \theta ) (°C)</th>
<th>-25</th>
<th>-45</th>
<th>-65</th>
<th>-85</th>
<th>-95</th>
<th>-13</th>
<th>-25</th>
</tr>
</thead>
<tbody>
<tr>
<td>( W_c )</td>
<td>5</td>
<td>38</td>
<td>24</td>
<td>16</td>
<td>14</td>
<td>13</td>
<td>12</td>
</tr>
</tbody>
</table>

![Fig. 1. Compliance curves for top peat. General curve plotted for thermoreologically simple (1) and complex (2) bodies.](image-url)

As can be seen from the diagram, incorporation of the vertical shift affects the position of the general curve. Long-term test data at the "base" temperature coincide with the general curve plotted for thermoreologically complex body. Since test data processing in this case is more complex the author has found the way to reduce the impact of the vertical shift.

Different continuity of frozen soils tested at different temperatures is mainly responsible for the complex thermoreology of the frozen soil. This is due to moisture phase changes. The experiment revealed that the impact of the unfrozen water and gas content on the compliance can be allowed for if stress is calculated for the area occupied by soil and ice particles instead of cross-section.
area of test samples. The former can be found approximately by assuming it to be proportional to relative soil and ice content per unit volume of frozen soil:

$$K = \rho_d \left[ \frac{1}{\rho_d} + \frac{(W - W_u)}{\rho_i} \right]$$  \hspace{1cm} (6)

where \( \rho_d \), \( \rho_i \), and \( \rho_d \) are dry soil, soil particles, and ice densities respectively in g/cm\(^3\), and \( W \) and \( W_u \) are total moisture and unfrozen water content respectively, in unit fractions.

For experimental evidence shown in Fig. 1, values for \( K \) are given in Table 2.

Table 2. \( K \) values for peat test samples

<table>
<thead>
<tr>
<th>( \theta^\circ )</th>
<th>-1</th>
<th>-25</th>
<th>-45</th>
<th>65</th>
<th>85</th>
<th>-13</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K         )</td>
<td>052</td>
<td>063</td>
<td>076</td>
<td>083</td>
<td>085</td>
<td>088</td>
</tr>
</tbody>
</table>

Compliance for the area occupied by soil and ice particles is \( J_k = (\delta/\sigma)K \). Plots \( J_k - \ln t \) and the general curve at the same temperature for the thermomechanically simple bodies are shown in Fig. 2.

The data obtained reveals that incorporation of coefficient \( K \) brings the general curve closer to the experimental evidence, eases its determination and makes it possible to predict long-term strain as for thermomechanically simple bodies.

With the general compliance curve \( J = \ln t \) and the reduction coefficient as a function of temperature \( \Delta \ln t = (\theta - \theta_0) \) available, we can find compliance over any period of time for the entire test temperature range.

The author has also developed procedures for graphic determination of the long-term compliance for all test samples.

The author proceeded from the following assumptions. The similarity of compliance curves \( J_k - \ln t \) makes them smooth and parallel in the log. scale \( \ln J_k - \ln t \) thus making it possible after plotting \( \ln J_k - \ln t \) for the general compliance curve to extrapolate these dependencies to the entire test temperature range.

In addition to long-term strain, temperature-time analogy can be used to predict long-term strength. Fig. 3 illustrates how the long-term equivalent cohesion can be obtained. Top complex peat with undisturbed structure has been tested at the temperature range from -2.5 to 25\(^\circ\)C. Physical properties of samples have been given before (same as for the uniaxial compression test). As one can see equivalent cohesion (calculated with incorporation of parameter \( K \)) curves are similar in the semi-logarithmic scale (Fig. 3). This permits one to obtain reduction coefficient \( \Delta \) for each curve \( C_k - \ln t \) (Fig. 3b) and plot general long-term cohesion curve for all test temperatures by horizontal shifting of test points. The general curve with confidence intervals is shown in Fig. 3b. It describes variations in equivalent cohesion with time for samples tested at -25\(^\circ\)C. The prediction period for the equivalent cohesion exceeds 50 years which is comparable to the service life of engineering structures.

The discussed graphic test data processing permits one to predict strength with general creep curve for a particular temperature, the
lowest in the test range as a rule. To determine long-term strength at other temperatures a three-stage combination of graphic and analytic procedures should be used which involves:

1) plotting of the prediction curve;
2) derivation of equation for it;
3) computation of strength for the time period $t_j$ obtained by the above equation, where $j$ is the period in which strength at a given temperature is to be defined, $A_j$ is the reduction coefficient derived from $A_j$ vs. $t_j$ plot for $i$-th temperature.

The obtained strength will be true for $t_j$ and $t$ conditions.

**STRESS - TIME ANALOGY**

Stress-time analogy procedure consists in plotting a family of compliance curves based on static loads isothermal creep tests. To obtain such family a broad test stress range is required. In this context certain difficulties arise. The application of analogy methods is based on the theory of viscous elasticity suggested by A. Ilyushin and P. Ogibalov (1966), which considers materials with physical non-linearity and exhibiting under load only viscoelastic, recoverable strains. In frozen soils most of the strain does not recover after unloading. Post-load soil strain strain has been exhaustively discussed by S. Vyalov (Vyalov et al., 1981). S. Vyalov demonstrated that conditionally instantaneous strain recovers in whole or in part immediately after unloading. This is true for the elastic part of strain. Damping creep strain also recovers, but with time and partially. This means that it consists of elastic and plastic strains. The total creep strain at any period of time includes recoverable and residual strains, the latter being generally greater. The total creep strain is a sum of instantaneous, damping and steady viscoelastic strains. The indicated differences in frozen soil strain as compared with viscoelastic bodies affect temperature-time reduction coefficient and thermomechanical behaviour.

Stress-time analogy tests revealed that thermomechanical behaviour of plastic-frozen soils has a complex nature. However, the vertical shift of creep curves is insignificant, it affects prediction of the long-term strain less than test data spread. To reduce the effect of non-uniform physical properties when computing compliance curves stress should be found for solid soil components by incorporating $k$-parameter like in the
temperature-time analogy case.

Fig. 4 illustrates stress-time analogy prediction of the long-term strain for top peat tested for uniaxial compression at -4.5°C.

It should be noted that if time-analogy methods are applied total strain should not include instantaneous elastic strain. Our computations ignored this requirement, i.e. compliance was taken as equal to total strain-stress relationship. Such assumption did not disturb the similarity of compliance curves family. It is valid for frozen soils, since instantaneous elastic strain constitutes an insignificant part of the total strain. This is proved by the analysis of the instantaneous elasticity modulus (Roman, 1987). Instantaneous elastic compliance taken as an inverse value of the instantaneous elasticity modulus for frozen peat ranges from 1·10^-6 to 2·10^-6 MPa, while compliance computed from total strain varies within 1·10^-3-8·10^-4 MPa, i.e. is by an order or too of magnitude greater.

![Diagram](image)

Fig. 4. Determination of long-term strain by method of stress-time analogy (uniaxial compression test at θ = -4.5°C)

Log plotting of compliance curve \( \text{C} = 60 + t \) revealed that these dependencies are linear and form a family of parallel straight lines each of which corresponds to its stress value. Identical inclination of all straight lines is indicative of the similarity of compliance curves. Hence, test samples in terms of strain can be referred to as thermomechanically simple bodies with sufficient for practical applications accuracy. Reduction coefficient-vs-stress relationship is also linear. Table 3 shows values for long-term relative settlements.

Table 3. Frozen peat long-term relative settlements (5·10^-5) predicted by stress-time analogy

<table>
<thead>
<tr>
<th>Time, years</th>
<th>Relative stress, MPa</th>
<th>0.2</th>
<th>0.36</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>2.74</td>
<td>2.82</td>
<td>2.74</td>
<td>3.49</td>
<td>4.78</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>3.36</td>
<td>3.53</td>
<td>3.63</td>
<td>4.58</td>
<td>6.06</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>3.46</td>
<td>4.46</td>
<td>3.82</td>
<td>4.69</td>
<td>6.33</td>
<td></td>
</tr>
</tbody>
</table>

The results obtained have been compared with calculations by the creep equation. The power function best suits for the description of the frozen peat strain process. The power function which incorporates creep curves similarity and Vyaliyov's assumption (1978):

\[
\delta = \left( \frac{\sigma}{A} \right)^{\beta} \left( \frac{t}{T^*} \right)^\alpha
\]

where \( A, m, T^*, \beta \) are experimental parameters.

Table 4 shows values for relative strains derived from Eq. (7) and stress-time analogy data.

Table 4. Comparison of relative strain predictions based on stress-time analogy and creep equation (peat, θ = -4.5°C, σ = 0.2 MPa)

<table>
<thead>
<tr>
<th>Loading time, years</th>
<th>Relative strain determination from</th>
<th>experiment</th>
<th>Eq. (7)</th>
<th>general curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>908 h</td>
<td></td>
<td>1.65</td>
<td>1.68</td>
<td>1.74</td>
</tr>
<tr>
<td>25 years</td>
<td></td>
<td>2.31</td>
<td>1.64</td>
<td>2.05</td>
</tr>
<tr>
<td>50 years</td>
<td></td>
<td>2.65</td>
<td>2.60</td>
<td>3.48</td>
</tr>
<tr>
<td>100 years</td>
<td></td>
<td>3.51</td>
<td>3.51</td>
<td>4.48</td>
</tr>
</tbody>
</table>

1012
The result obtained by the two methods are in close agreement. However, the time-analogy procedure is much simpler since it does not require time-consuming determination of the creep equation parameters.

CONCENTRATION - TIME ANALOGY

Frozen soil creep rate increases with salt, peat and ice content. The impact of their content on the strain rate along with load and temperature is identical to that of time. This makes it possible to use salt and ice content and vegetation residue as strain accelerating factor and use concentration-time analogy in long-term strain predictions. The investigations are already under way. Satisfactory applicability of the salt-time analogy has been established (Roman and Kuleshov, 1990). Fig. 5 shows saline loam uniaxial compression test data obtained by A. Alipanov. Given further are mean values for physical properties of the test material: frozen soil density - 2 g/cm³ and soil particle density - 2.71 g/cm³. Samples contaminated with Na₃SO₄ have been tested. The salt content Dₘ₄ was 0.2; 0.4; 0.6; 0.8; 1.0 and 1.5%.

The static constant load during creep test at -3°C was 1.26 MPa. Since different salt content is responsible for different unfrozen water content the investigators used coefficient K to compute stress for the area occupied by solid components. The impact of the salt content on strain is sufficiently regular. Similar influence has been exhibited by vegetation residue and ice inclusions. Together with stress and temperature the content of these components can serve as a creep-rate acceleration factor, permitting the use of the concentration-time analogy for long-term strain cases.

CONCLUSIONS

1. Frozen soils exhibit rheological properties in such a way that the impact of temperature and stress together with salt and ice content and vegetation residue on strain and strength is similar to that of time thus making it possible to use time analogies for long-term strain and strength predictions. Temperature, stress- and concentration- time analogies best suit to accomplish these ends. In terms of behaviour under load frozen soils are thermodynamically complex bodies. However, the computation of stresses for soil solids permit one to allow for the impact of phase changes and structural non-uniformity and predict long-term strain and stress as for thermodynamically simple bodies.

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The experience of using a compression-tension method for engineering geological investigations

Expérience de l’usage de la méthode extension-compression dans les investigations géotechniques

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ABSTRACT: The compression-tension method has high accuracy of strength parameters of clayey soils with consideration of curvilinearity of Mohr’s circles envelope at engineering geological investigations and makes it possible to refuse the use of the plain shear and triaxial compression methods at short-term tests. The method is used in engineering geological tests to obtain standard strength parameters (cohesion and angle of inner friction) of clayey soils, to estimate stress state of clayey soils on slip slopes, clayey soils resistance to fracture and in other cases where tensile strength presence is possible in soils.

RÉSUMÉ: L’extension-compression est une méthode très précise pour la détermination des paramètres géotechniques du sol argileux à l’usage des investigations ingénieur-géologiques. Le recensement d’une enveloppe du cercle de Mohr donne la possibilité de l’essai vite sans déplacement de plan et sans compression triaxiale. La méthode a été employé pour l’identification d’une cohésion, d’angle de frottement interne et de résistance par traction du sol argileux, et pour l’évaluation d’état de tension dans une pente éboulée. Elle est applicable pour autres situations avec des contraintes de traction dans le sol.

The compression-tension method has been developed in Russia and used for a long time to obtain accurate parameters of strength (cohesion and angle of inner friction) for clayey soils in any state.

![Diagram of Mohr's circles group and strength parameters](image)

Fig. 1 Mohr’s circles group and strength parameters, obtained by the use of the compression-tension method.

The method allows in an experimental way to get the intersection point of the Mohr’s circles envelope and axis of tangent stresses $\tau_n$ (Fig.1) in $\tau_n - \sigma_n$ coordinates and to construct the part of the envelope $A_1-B$ with the use of the experimental data but not by extrapolation and to obtain the clayey soils strength parameters taking into account the envelope curvilinearity of Mohr’s circles at the axis of tangent stresses.

Using conventional methods of tests (triaxial compression) the part of the A-B envelope is defined by extrapolation, which depending on the degree of the envelope curvilinearity, may bring to the different value of disagreement between strength parameters $C_0$ and $\varphi_0$ and their real values $C'_0$ and $\varphi'_0$.

In the use of plane shear the envelope curvilinearity is neglected because the process of its flattening into straight line takes place. The point of is intersection with the axis $\tau_n$ defines some average meanings of $C_0$ and $\varphi_0$.

The compression-tension method, used in Russia, doesn’t have these drawbacks.
The test made according to this scheme is called "side reduction". Using Mohr's circle 2 we build the part of the envelope A₁-B and get accurate values of strength parameters C₀ and \( \phi'₀ \).

The method permits to make simple and reliable tests of clayey soils on simple tension by limiting side surface of the sample in the form of a coil by rigid split yoke. In this case the hydrostatic pressure created on the ABCD contour influences only the parts of the sample AB and CD making a tensile force along its axis. The stress state in the sample can be represented by the following stresses:

\[
\begin{cases}
\sigma_1 = \sigma_2 = 0 \\
\sigma_3 < 0
\end{cases}
\]  

Practically, using 3 main schemes of sample loading it is possible in the experimental way to construct the whole part of the envelope DB (Fig.1) from a circle of simple tension \( \sigma_1 \) to the circle of "side reduction" \( \sigma_0 \). This enables to model the stressed state of soils with any combination of tensile and compressive stresses which are as a rule presented in the soils of slip slopes, edge parts of structures, in the places of adjoin of earth dams to the edges, etc.

To use the method there were made a few modifications of the simultaneous compression-tension (SCT) method device for different hydrostatic pressures from 6 to 20 kg/sq.cm which enables to test both crypto-fluid clayey soils and semi-rock material, including those ones with the content of fragments to 20% of the volume.

The devices of simultaneous compression-tension, made in Russia are distinguished by their original construction, simple use, reliability, high productivity, accuracy, little size, it doesn't need bulky loading devices. There is no analogy to it in other countries. The peculiarities of structure and the accuracy of the strength parameters C₀ and \( \phi'₀ \) by the compression-tension method permitted to refuse the use of the plane shear and triaxial compression methods to obtain the strength parameters of clayey soils in short-time tests during engineering geological investigations. Besides it, these devices make it possible, if necessary to determine the parameters of long-term soil strength.

The sphere of application of the tension-compression method and the main results of the investigations on strength characteristics of clayey soils are as follows:

1. The comparison of the strength parameters (cohesion and angle of interval friction) obtained
by the compression-tension method for clayey soils with the parameters of strength, obtained by the plane shear and triaxial compression methods showed that the value of divergence between them depends on the value of curvilinearity of Mohr's circle envelope, that in its turn is bound up with the character of structural ties in the soil. The more elastic ties are in the soil, the more is the divergence and inaccuracy of strength parameters of the methods mentioned in comparison with the parameters obtained by the compression-tension method.

For the soil with plastic ties the cohesion value, obtained by the plane shear and triaxial compression is higher than the cohesion value, obtained by compression-tension in the limits 0-8%: the value of the angle of interval friction is less in the limits 0-15%, that is located in the limits of accuracy of determining the characteristics. We may say, that the strength parameters, obtained by different methods for plastic soils are identical.

For the soils with elastic-plastic ties the cohesion value, obtained by the plane shear and triaxial compression is 5-22% higher and the angle of the internal friction is 20-55% less.

For the soils with elastic ties the cohesion value obtained by the plane shear and triaxial compression is 22% higher and the angle of internal friction decreases more than 55% in comparison with the results obtained by compression-tension. These investigations show that for the parameters of strength, obtained by the plane shear and triaxial compression methods for the soils with elastic and elastic-plastic ties, the stage of design envisages excess safety factor which leads to the design of more expensive foundations of buildings and structures and to the rise in the cost of construction on the whole.

2. The determination of clayey soils resistance to fracture showed that it can be more, equal or less than the value of soil cohesion and depends on its plastic ties. For the soils with plastic and elastic-plastic ties the resistance to fracture is equal or more than the value of cohesion of soil. For the soils with elastic ties the less is the resistance to fracture, the more are the elastic ties of the soil. On the whole the resistance to fracture is 1.2-1.8 times less than the cohesion value.

The results obtained, permit to evaluate the degree of soil resistance to the occurrence of separation cracks in the places of the tensile stress (slip slopes, dam edges, etc.).

3. The maximum values of tensile stresses and deformations of the clayey soils in the upper zone of the earth dam at the Nureck hydroelectric power station in the places of contact with the edges were defined. The formation of separation cracks is possible at the dam as a result of its settlement in the edge and river-bed parts. The possibility of the cracks formation was estimated by the use of the compression-tension method on the samples of silty sands with the size of particles to 2 mm.

It was stated that the maximum tensile deformation and rupture stress which the soil can stand conforms to the optimum moisture-density of silty sand put to the core of the dam, as it was recommended at the construction. The calculation of the dam's core state under stress and deformation showed that possible limiting stresses and deformations are below admissible meanings, obtained by the compression-tension method at the soil samples.

4. The value of the resistance to fracture, obtained by compression-tension was compared with the value obtained by the conventional methods which used rigid grip of the sample. It was found out that the resistance to fracture, obtained by compression-tension is 1.5-2 times higher by the absolute value, because the soil sample doesn't possess the violation of the stressed state connected with the tensile effort transmission.

5. At the investigation of clayey soil stressed state at the slip slope the compression-tension method was used to define the critical tensile effort in soils according to the depth. This permits to locate already existing zone of slip and the zones which are only at the stage of formation which are potentially dangerous. The distribution of stress in the soils which form the upper and middle parts of the slope is characterized by the presence of positive stresses i.e. natural pressure and tensile stresses, occurring to a definite depth and provoking the appearance of the separation and sliding surfaces and depending on the angle of inclination of rocks on the slip slope.

It was established that the soils located in the zone of slip slope possess isotropic tensile characteristics at which the value of natural pressure is equal both to the value of limit tensile stresses and the value of soil cohesion.

This fact makes it possible to obtain critical tensile stress by the compression-tension method for these depths at making tests of soils samples of special form (a coil) from various depths of the slope. Comparing the critical tensile strength value with the value of natural pressure at this depth it is possible to locate both the existing sliding zones and the zones under formation. The use of the compression-tension method permitted to find out and locate in the Caucasus slip slopes previously unknown sliding zones which are at the stage of formation at the elevation of soil moisture due to
precipitation, changes of the slope stress state due to the extra loads in the upper part or cut of the slope, the zones may become active and bring to catastrophe in case of the slope movement. In this case the compression-tension method helps to forecast hazardous geological processes. The example based on the use of the compression-tension method at the engineering geological investigations open many other possible fields of its use.

REFERENCES


Rod extensometers to detect deformation of excavation cut
Extensomètres à barreau de fer pour la détermination de la déformation de la pente d’une excavation

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National Technical University of Athens, Greece

ABSTRACT: The use of rod extensometers to detect the displacement of the excavation cut of the Pournari spillway in Western Greece is examined. The spillway is excavated in a flysch formation. Larger displacements are detected in the siltstone layer at the lower part of the excavation, than at the sandstone layer at the upper part, with a dependence on the size of rock wedge removed in front of the cut. Seasonal variation due to temperature is more pronounced in the sandstone than in the siltstone layer.

RÉSUMÉ: L’application des extensomètres à baton de fer a été étudié pour la détermination de la déformation de la pente de l’excavation de l’évacuateur de crue du barrage de Pournari en Grèce de l’Ouest. L’évacuateur de crue est excavé dans une formation de flysch. Les déformations les plus grandes sont observées dans les couches des schistes argileux dans la partie inférieure de l’excavation, que dans la partie supérieure des couches des grès, avec dépendance de la masse du rocher excavé. Les variations des déformations saisonnières à cause de la température, sont plus importantes dans les couches des grès que dans les couches des schistes argileux.

1 INTRODUCTION

The performance of the excavation cut presented in this paper refers to the spillway of the Pournari hydropower installation in Western Greece. The Pournari earth dam, the spillway on the right abutment and the powerplant on the left are shown from downstream in the picture in Fig. 1.

The spillway was cut into flysch layers of sandstone overlying siltstone. The deformation was monitored by 12 rod extensometers installed on horizontal beams into 15 m deep drill holes in the excavation cut, as the excavation proceeded from top to bottom.

The processing of extensometer readings indicated the deformation of the cut face, and the specific characteristics of deformation in the sandstone and siltstone layers. The deformation of the excavation cut within a period of four years is described and discussed.

2 SPILLWAY CUT

The Pournari hydropower installation is on the Arachthos river in Western Greece, approximately 4.5 km east of the town of Arta. An index map showing the location of the Pournari installation is shown in Fig. 2 (top right). The reservoir behind the earth dam provides water to a 300 MW powerplant and to 20,000 hectares of cultivated land in the plain of Arta.

The Pournari earth dam has a maximum height of 102 m. The riverbed excavation is at elevation 25 m and the crest of the dam at elevation 127 m. The spillway associated with the Pournari earth dam is an 84 m-deep excavation at the right abutment of the dam, with its long excavation axis oriented towards the river valley downstream from the dam. The maximum design flood of 6,100 m³/s is supposed to pass through the 40 m by 10 m cross section of the spillway chute which is concrete lined. The flow is controlled by three tainter gates.

The geology at the dam site consists of Cenozoic sedimentary rocks, mainly a flysch formation composed of alternating strata of sandstone and siltstone. The flysch was formed in the Eocene and Oligocene of the Tertiary time period, and is one of the formations of the Ionian
zone in western Greece. The Ionia zone consists of Pliocene to Eocene rock formations thrust eastwards over older formations of the Pindus zone. The flysch consists of alternating strata of siltstone (70%), sandstone (20%) and conglomerate (10%). At the dam site coarse-grained sandstone is present at higher elevations, while siltstone with alternating thin bedded siltstone and sandstone layers of 5 to 10 centimeters thickness, is present at lower elevations. Upstream from the dam site interbedded siltstones and sandstones are present, while pure siltstone is present downstream from the dam. At the dam site sandstone is present at the riverbed covered by a 10-15 m thick layer of alluvial sandgravel. The spillway is cut into the outcropping sandstone of the right abutment of the dam which overlies a thick bed of siltstone. The sandstone layers are interrupted by thin layers of siltstone. The monocline present at the location of the dam has a trend of N76°E and an inclination of 22°N.

The two distinct layers of sandstone and siltstone exposed by the spillway cut are shown in the plan of Fig. 2, with indication of the boundary line as it
Fig. 2. Index map and plan of the spillway excavation indicating the positions of the extensometers and the sandstone - siltstone boundary.

emerges from the excavation cut. The cut was covered by a 5 cm layer of gunite placed on wire mesh anchored on the rock surface right after the excavation. Weep holes were drilled to relieve water pressure. Twelve rod extensometers were placed to monitor the behaviour of the rock cut.

The excavation has a trend of N45°W and an inclination of 35°NE. Berms of 6m width were constructed at 15m or 10m at elevations of 172m, 157m, 142m, 128m and 118m. The slopes of the cut between the five berms have a 1:1 slope. A cross section of the cut is shown in Fig. 3. The ground surface upstream from the cut parallels the strike and dip of the sandstone beds. Downstream from the cut, the ground surface on the siltstone beds trends N27°E and is inclined 34°SW.

The spillway cut was continuously excavated from the top at elevation 187m to the bottom at el. 102 m, which is the floor of the spillway shute. The cut slope was stabilized by prestressed anchors 15 m long, to which forces were applied at constant time intervals. These forces did not affect the expansion of the spillway cut after the excavation, which was due to the relief of the initial stresses in the rock mass.

3 ROD EXTENSOMETERS

Each rod extensometer consists of a multiple rod reference head with dial depth gauge, a remote reading displacement transducer, the extensometer rods with protective sleeves, a range adjustment unit, the groutable and expanding shell anchors, and a sleeve support clamp plate. The application of the rod extensometers includes monitoring of the relaxation of rock in large excavations, in tunnels and in underground openings.

Multiple rod extensometers monitor displacements at various depths using rods of varying length. Several rods may be installed side by side in sleeves within a single large hole. The complete assembly is grouted in place, thus fixing the ends of the anchors into rock but allowing free movement of each rod within the sleeve. A single reference housing receives all the rods from one installation. A meter or digital remote readout, and a visual alarm system to give warning of excessive movements and to indicate when readings are required is furnished with the rest of
the rod extensometer equipment.

The multiple rod extensometers employ 6 rods, each anchored at one end within the drillhole, passing into a reference tube fixed in the hole collar. Relative movement between the end anchor and the reference tube is measured either with a dial depth gauge or with an electric transducer inserted through the reference tube and registered on to the free end of the rod. A range adjustment screw fitted to the end of the rod extends the reading range beyond that of the dial gauge.

The extensometer rods comprise 9 mm diameter invar rods epoxy painted, externally threaded at both ends and supplied with one rod coupling made of stainless steel. The multiple rod reference head comprises cadmium plated mild steel datum plate, sleeve guide plate, ragbolts, epoxy painted aluminum alloy datum housing and protective cap. The protective sleeves comprise PVC tubes of 21.2 mm outer diameter, 15.5 mm interior diameter, flush coupled for cementing with PVC adhesive to secure the flush couplings.

The readout equipment comprises a dial depth gauge 75 mm diameter with a stainless steel boss to locate in the reference bushes of the respective reference heads. The dial gauge plunger made of stainless steel is fitted with a flat anvil to locate on the hemispherical end of the adjustment units. The recommended operating temperature range is -10°C to 50°C. The travel length is 50 mm and the linearity is better than .50% of the total working range. The temperature coefficient sensitivity is plus or minus 0.025% per degree C. It is supplied in a wooden box complete with calibration tube.

The 12 rod extensometers were installed horizontally in 15m-deep holes, drilled 2-3m above the level of each berm. The extensometers E1 and E2 were placed from el. 157m, extensometers E3, E4, and E5 from el. 142m, extensometers E6, E7, E8, and E9 from el. 128m, and extensometers E10, E11, and E12 from el. 118m. The positions of the extensometers are shown in the excavation plan of Fig. 2. The rod extensometers are anchored in the rock at depths of 1m, 3m, 6m, 9m, 12m, and 15m.

4 DEFORMATION OBSERVED

The extensometer readings for a period of 4 years were processed to produce plots of displacement versus time shown in Fig. 4 and Fig. 5. The displacements of each of the six rods of the extensometers are plotted in the drawings, with the largest displacements corresponding to the 15m-long rods, and the smallest displacements to the 1m-long rods.

The extensometers placed in sandstone are E1, E3, E4, E6, E7 and E10, while those in siltstone are E2, E5, E8, E9, E11 and E12, as shown in Fig. 2 (extensometer E2 is only partially in sandstone). Readings
Fig. 4. Displacement versus time for extensometers E1, E3 and E7 in the sandstone layer during a period of 4 years.

Fig. 5. Displacement versus time for extensometers E2, E5 and E9 in the siltstone layer during a period of 4 years.
Table 1. Information on extensometer elevation, rock wedge characteristics and maximum displacement.

<table>
<thead>
<tr>
<th>extensometer elevation (m)</th>
<th>Small Wedge Height Base (m)</th>
<th>Large Wedge Height Base (m)</th>
<th>wedge ratio SW/LW</th>
<th>maximum displacement (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
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<tr>
<td>1</td>
<td>160</td>
<td>17</td>
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<td>74</td>
</tr>
<tr>
<td>3</td>
<td>144</td>
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<td>88</td>
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<td>10</td>
<td>120</td>
<td>28</td>
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<td>Siltstone</td>
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<td>12</td>
<td>120</td>
<td>28</td>
<td>45</td>
<td>47</td>
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</tbody>
</table>

were taken 3 times a month, every 10 days. Displacements versus time (4-year period) for extensometers E1, E3 and E7 in sandstone are shown in Fig. 4. Similarly, displacements versus time for extensometers E2, E5 and E9 in siltstone are shown in Fig. 5. Both figures indicate the rate of displacements and the effect of yearly temperature variation on the displacements.

The maximum displacements in the 4-year period of the 15m long extensometer rods for all extensometers installed are shown in Table 1, along with tabulation of the elevations of the tip of the extensometers. Also, the characteristic dimensions (height and width) of the rock wedges (in two dimensions) removed in front of each extensometer are shown in Table 1. The rock wedge dimensions are measured from the front of the position of each extensometer at sections normal to the excavation. The characteristic dimensions of the wedges and the ratio of the rock wedges (area of the small wedge ABC above the instrument to the area of the large wedge ADE above the spillway chute, shown in Fig. 3), are presented in Table 1.

The maximum displacements of the 12 extensometers in the 4-year period versus wedge ratio in the sandstone and the siltstone layers are shown in Fig. 6. The displacements (d in mm) of extensometers E1, E3, E4, E6, E7, and E10 positioned in sandstone are found along the line d = 1.96 + 0.034 w, which is a function of wedge ratio (w in percent) and is found at lower displacement levels. The displacements of extensometers E2, E5, E8, E9 and E11 positioned in siltstone are found along the line d = 4.18 + 0.175 w, at higher displacement levels. Extensometer E12 positioned in siltstone indicates very large displacements and is found outside the two distinct groups of the extensometers presented in Fig. 6.

5 DISCUSSION

The performance of the two groups of extensometers indicates that the group with lower displacements is located in the sandstone layer and the group with larger displacements is in the siltstone layer. Both groups indicate a linear behaviour of displacement versus wedge ratio, which means that greater displacements are present at lower elevations than at higher elevations.

The group of extensometers in the siltstone layer indicates displacements 2.7 to 3.8 times larger than the displacements in the sandstone layer for wedge ratios of 10 to 60 percent.
Fig. 6. Displacements versus load ratio of the rock wedges for the 12 extensometers in the sandstone and siltstone layers.

respectively. This is attributed to the larger Modulus of elasticity in the siltstone than in the sandstone. Inferences on the Modulus of elasticity and Poisson's ratio of sandstone and siltstone, as well as on the initial stresses at the positions of the extensometers could be made, if a numerical model were used to simulate the unloading of the rock mass due to excavation.

The reason for considering the rock wedges in front of the position of each extensometer is to simulate the original stress condition and show the relation of displacements observed to the size of rock wedge removed, for two different types of rock with different elastic properties. However, the actual performance of the rock slope indicates a stepwise plastic behaviour, since the rate of displacement is larger within the first 200 days of the excavation, and smaller in the subsequent years, with a trend to become zero in longer periods.

The seasonal variation of the displacements is more pronounced in the sandstone layer than in the siltstone layer. The temperature effect on the displacements in Fig. 4 and Fig. 5, is indicated as a 0.5mm to 0.8mm increase in displacement during the summer months and the same decrease during the winter months. The seasonal variation of the displacements is a sinusoidal variation along the general trend of the variation of the displacements with time, which also indicates a gradual lowering of the displacements.

The deformations described here may be compared with the results of numerical models to identify the in situ strength parameters of sandstone and siltstone, and make assumptions for the initial stress field in the rock slope.

6 CONCLUSIONS

The performance of a spillway excavation at the right abutment of an earth dam, in Western Greece was studied. Extensometers on the 84m deep excavation, which crosses a sandstone layer overlying a siltstone layer, indicated larger displacements at the lower part of the excavation, with a dependence upon the size of rock wedge removed.

The sandstone layer indicated smaller displacements than the siltstone layer. Seasonal variation due to temperature was more pronounced in the sandstone layer than in the siltstone layer.
Continuous evaluation of rock mechanics and geological information at drilling and boring
Évaluation instantanée des informations géologiques et géotechniques pendant l’exécution de forages

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ABSTRACT: The paper presents the technique of evaluation of rock mechanics and geological information based on the monitoring of the drilling or boring. On the basis of the logged data of drilling or boring is possible to evaluate the stress state of the rock mass in surroundings of the working and to capture the change of the mechanical properties of rocks with significant accuracy.

The advantage of the technique is a possibility of continuous and immediate obtaining of information. The continuous evaluation of the monitored data is secured by the computer based system which, besides the data handling, enables optimization of the drilling or boring.

In the paper the examples of the evaluation of the monitored data under in situ conditions are introduced.

In our previous papers (Krepetka, 1993; Krupa, 1993; Sekula, 1992), we have dealt with the possibilities of interpretation of theoretic knowledge of classic theory of elasticity for the area of the rocks disintegration. This theory defines the specific energy of elastic deformation as

\[ w = \frac{1}{2} \frac{\sigma^2}{E}, \]  \hspace{1cm} (1)

respectively

\[ w = \frac{1}{2} \sigma \epsilon, \]  \hspace{1cm} (2)

where \( \sigma \) is stress, \( E \) is modulus of elasticity, and \( \epsilon \) is relative deformation. The equations (1) and (2) were derived for elastic stress field, i.e. for the domain of perfectly reversible deformations. At the rock disintegration, when the plastic deformations is not possible to neglect, the quadratic law of increasing of a specific energy with increasing of stress is changing to the linear law (Košták, 1982)

\[ w = k \sigma_R, \]  \hspace{1cm} (3)

where \( k \) is a constant.

In the technical research the specific energy necessary to disintegrate the rock is possible to express, e.g. in rotary drilling or fullface boring, by means of the equation

\[ \psi = \frac{2\pi n M_c}{3v}, \]  \hspace{1cm} (4)

where \( n \) are revolutions of the disintegration tool, \( M_c \) is the torsional moment of the disintegration tool, \( v \) is the penetration rate of advancing of the tool into the rock, and \( S \) is surface of the disintegrated face of working. The torsional moment and penetration rate of advancing of the tool into the rock are variables size of which is influenced, besides the properties of the disintegrated rock and disintegration tool, by the regime parameters of the disintegration tool. So these variables depends on the thrust and revolutions. If we want to interpret and to investigate the rock properties directly during the disintegration, it is necessary to exclude the influence of a remaining variables.

For example, at the boring by means of the fullface tunnel boring machines this condition complies with the regime of the boring at the constant penetration rate of boring in the interval of boring in which no substantial change of the functional surfaces of the disintegration disc cutters under influence of the wear of the tools was observed. It is possible to illustrate it by means of a model procedure. The disintegration output of the fullface
tunnel boring machine, expressed in the numerator of the equation (4), is the variable that can be monitored during boring. From the disintegration output the torsional moment $M_t$ is calculated. Based on the premise that all disc cutters are acting on the face of the working by means of the same force the average values of the cutting forces acting on the particular discs is possible to calculate. This premise is expressed by the equation

$$F_R = \frac{M_t}{\sum_{i=1}^{N} I_{D0_i}} ,$$  \hspace{1cm} (5)$$

where $I_{D0_i}$ are the perimeters of the trajectories of the particular disc cutters fitted on the cutterhead of the boring machine, $N$ is number of the disc cutters. For the fullface tunnel boring machines with the construction distance $s$ between the trajectories of the particular disc cutters is valid the equation

$$I_R = sN ,$$  \hspace{1cm} (6)$$

where $I_R$ is the perimeter of the cutterhead of the fullface tunnel boring machine, and for the sum of the perimeters of the trajectories of the disc cutters is valid the equation

$$\sum_{i=1}^{N} I_{D0_i} = N \frac{r_R}{2} = \frac{r_R^2}{2} .$$  \hspace{1cm} (7)$$

This equation can be rewritten into the form

$$\sum_{i=1}^{N} I_{D0_i} = \frac{s}{2 \pi} \cdot \frac{S}{2} .$$  \hspace{1cm} (8)$$

where $S$ is the surface of the face of the tunnel. By substituting (8) into the (5), at using the definition equation of the specific energy (4), we obtain the average value of the cutting force of the disc cutter as

$$F_R = \frac{w v g h}{n} = \frac{w v g h}{n} ,$$  \hspace{1cm} (9)$$

where $h$ is the depth of chipping out of the rock under the disc cutter that can be calculated from the immediate penetration rate and revolutions of the cutterhead as

$$h = \frac{v}{n} .$$  \hspace{1cm} (10)$$

In order to disintegrate the rock it is necessary to create, under the disc cutter, a critical stress $\sigma$. This stress is possible to calculate as the ratio of the force acting on the disc cutter and the projection of the contact surface of the disc and rock into the direction of the acting force. The contact surface of the disc cutter with the rock in the direction of the force $F_R$ can be defined as

$$S_{k} = \frac{p^2 \tan \alpha}{2} ,$$  \hspace{1cm} (11)$$

where $p$ is the depth of the penetration of the disc cutter into the rock, and $\alpha$ is the angle of the cutting edge of the disc cutter. The experimental research shows the ratio of the depth of the penetration of the disc cutter into the rock and the depth of chipping out of the rock under the disc cutter is constant that does not depend on the primary fracturing of the rockmass (Krupa, 1993). This ratio defines the cutting constant of the rock $R_D$ in the form

$$R_D = \frac{F_R}{h} .$$  \hspace{1cm} (12)$$

From the above stated knowledge results that in the direction of the force $F_R$, under the disc cutter, the stress $\sigma$ occurs. This stress is possible to express by the equation

$$\sigma = \frac{F_R}{R_D} = \frac{w \cdot s \cdot h}{R_D \cdot h^2 \cdot \tan \frac{\alpha}{2}} .$$  \hspace{1cm} (13)$$

By means of rearrangement of the equations (3) and (13) we obtain for the parameter $K$ the expression

$$K = \frac{R_D \cdot \tan \frac{\alpha}{2}}{s} .$$  \hspace{1cm} (14)$$
which indicates the previously formulated statement that at the constant depth of chipping out of the rock, respectively at the constant penetration rate of the boring, the linear relation is valid between the specific energy defined according (4) and the cutting stress.

Sanio (1983) have defined the pro-rata relation between the cutting force of the disc cutter \( F_N \) and the normal force \( F_N \) acting on the disc cutter as

\[
F_N = \frac{4}{5} \sqrt{E_d F_N}
\]  

(15)

and the lateral force \( F_L \) as

\[
F_L = \frac{1}{2 \tan \frac{\alpha}{2}} F_N
\]  

(16)

From these relations results that also for the normal stress \( \alpha \) defined as the ratio of the normal force \( F_N \) and the projection of the contact surface of the disc cutter and rock into the direction of this force the linear dependence in relation to the specific energy is valid. The same is possible to prove for the lateral stress \( \alpha_L \). Between the stresses the following relations are valid

\[
\sigma_R = \frac{4}{5} \sigma_N
\]  

(17)

\[
\sigma_L = \frac{3}{4} \sigma_N
\]

Theoretically, we can define also the shear stress caused by pushing of the disc cutter into the rock; and we can prove that the linear relation between this stress and the specific energy, at keeping the condition of the constant penetration rate of boring, is valid also for shear stress.

The illustration figure demonstrates the results of the monitoring of the boring of andesite by means of the tunnel boring machine Wirth TB-II-330H at Hodruša locality, Slovakia. In the Figure a) the dependence of the disintegration stress on the overturned value of the immediate penetration rate of the boring is illustrated; and in the Figure b) the dependence of the same stress on the specific energy of disintegration is plotted. Both figures confirm the validity of the equation (13).

- **Figure. The results of the monitoring of boring**

By the monitoring of a selected parameters of the process of boring under in situ conditions is possible to obtain information about relative stresses needed to disintegrate the rock on the face of the tunnel. Under the precisely given conditions of boring these stresses of disintegration enable to determine, in relative proportionate values, the approximate properties of the disintegrated rock and so the continuous monitoring of the changes of the rocks. By means of the
presented mathematic procedure is possible to determine for given type of tunnel boring machine the mutual domain of values of monitored and calculated parameters corresponding to the stable and unstable parts of the tunnel. This method enable to identify the weak zones and faults in the rockmass during boring, and because these weaknesses are in most cases accompanied by the changes of the rock properties in its surroundings, enables to predict the occurrence of weaknesses.

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Using stereological methods to estimate the volumetric proportions of blocks in melanges and similar block-in-matrix rocks (bimrocks)

Méthodes stéréologiques pour l'évaluation des proportions volumétriques des blocs en mélanges et des roches de bloc-en-matrice (bimrocks)

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ABSTRACT: Melanges are part of a larger family of bimrocks (block-in-matrix rocks), composed of hard blocks within weak matrices. Since recent work by other shows that the overall strength of a bimrock is proportional to the volume of blocks, means of estimating the volumetric proportion of blocks in-situ must be developed. A stereological method is described in which the volumetric proportion is estimated by measurements of block intercept lengths in drill core, and the results compared to data for a California construction site.

RÉSUMÉ: Mélanges sont une partie d'une famille plus grande de bimrocks (les roches de bloc-en-matrice), composées de blocs durs dans matrices faibles. Parce que les travaux récents par les autres démontrent que la force d'un bimrock est proportionnel d'après le volume des blocs, c'est nécessaire ainsi développer le moyen de évaluer le proportion volumétrique par blocs in situ. Une méthode stéréologique est décrite par lequel la proportion volumétrique est évaluée par faire les mesures de la longueur dans segments de droite dans le noyau de forêt, et les sont comparés aux résultats d'une location de construction en Californie.

1 INTRODUCTION

Melanges (from the French, mélange, or "mixture") are rock masses composed of competent rock blocks of varying size, embedded within a weaker argillaceous matrix. The chaotic fabric of melange is characteristic of rocks formed within the accretion prisms of subduction zones. Indeed, the presence of melanges within the sedimentary stack is considered to be necessary evidence for a subduction history. However, despite general accordance on these principal facts, many geologists disagree about the details of melange formation (Raymond and Terranova, 1984), as reflected in the many aliases of melange (e.g., chaotic formations, wildflysch, mega-breccia, argille scagliose and friction carpets). In the geological literature there are abundant references to melanges, there being over 1950 references alone accessible from GeoRef (a geological literature database supplied by the American Geological Institute), but there are only rare references to melange in the engineering geology or geotechnical engineering literature.

Raymond et al (1989), included melanges within a larger geological family of block-in-matrix rocks. For engineering purposes, I abbreviated the term "block-in-matrix rock" to bimrock (Medley, 1994), which is defined as "a mixture of relatively large, competent blocks within a bonded matrix of finer and weaker texture", a definition that ignores rock genesis. Bimrocks similar to melange that are formed from cataclasis and fragmentation include breccias, coarse pyroclastics, lahars and tillites. Other bimrocks form from weathering (e.g.: decomposed granite) and sedimentation (e.g.: boulder conglomerates). Similarly, mixed soils such as colluvium and till, are termed bimsoils.

Melange bodies have been identified in the mountains of over 60 countries (Medley, 1994), as shown in Figure 1 and are exemplified by the Franciscan Assemblage (the Franciscan) a regional-scale jumble in northern California, the
argille scaglione of the Italian Apennines, and the extensive melanges of Turkey and Iran. Typically, melanges contain chaotic zones of relatively strong blocks of greywacke sandstone, together with scarcer chert, basalts, limestone and exotic metamorphic rocks, all embedded within matrices of pervasively sheared shale and argillite. In the Franciscan of northern California, blocks sizes range between sand particles and mountain masses, are irregularly shaped, and generally trend NNW-SSE (Figure 1). Shears are confined to the weak shale matrix, which flows around the closely jointed blocks. The sheared shale, and the sheared serpentinite bodies common in the Franciscan, are both responsible for the myriad of earth-flow landslides of the Franciscan. Also, encounters with unpredictably distributed blocks in melanges cause expensive surprises during earthwork excavations and foundation preparations. Although it is relatively easy to excavate the sheared shale using conventional earthwork equipment, blocks larger than approximately 3m in dimension generally must be blasted.

When working with bimrocks or bimsoils, it is universal geotechnical engineering practice to design using the strength of the weaker matrix. But this practice may be overly-conservative if the bimrock contains a large proportion of blocks. Extensive laboratory testing of physical model melanges (Lindquist and Goodman, 1994; Lindquist, 1994) showed that the strength of a physical model melange increased with the volumetric proportion of blocks when the proportion exceeded approximately 30 percent as a result of the increased tortuosity of failure surfaces. Similar relationships between block content and overall strength have been observed qualitatively for melanges in California.
(Bedrossian, 1980) and Italy (D'Elia et al, 1986), and in colluvium deposits in Hong Kong (Irfan and Tang, 1993).

The practical use of a relationship between block volumetric proportion and overall bimrock strength requires the determination of the in-situ volumetric proportion of blocks. In this paper, I show that much can be learned from the study of small-scale images of melanges and that stereological methods are useful in the estimate of volumetric proportions. I briefly introduce the more complex problem of the determination of block and block size distributions; and finally, I present the results of a field application of my approach to estimating the volumetric proportion of blocks at the site of a California landslide.

2 THE APPARENT SELF-SIMILARITY OF FRANCISCAN MELANGES

Melanges are commonly characterized as "chaotic". But recent work (e.g., Turcotte (1992)) showed that many chaotic geological processes have a self-similar or "fractal" nature. A structure possesses self similarity if, when fragmented, each fragment is a replica of the parent whole. The fractal organization of blocks in a melange at the outcrop scale was demonstrated by Lindquist\(^1\) when he discovered that the block size frequency distribution of the melange obeyed power laws. Plotted against logarithmic axes, the data were organized linearly: such plots are referred to here as "log-log linear". Lindquist's preliminary work has been confirmed (Medley, 1994; Medley and Lindquist, 1994; in preparation) for melanges of many scales (compare Figure 2 and Figure 3, which have a scale ratio of greater than a million), and also for other fragmented geological materials such as the clasts within lodgment (basal) till. These findings have a fundamentally practical implication because, given sufficient data to define a log-log linear plot, predictions can be made about the sizes and numbers of blocks within a melange. Any predictions, though simple, are useful to geotechnical engineering designers and earthwork contractors. Within the California geotechnical community, there has not been a method for making such predictions until recently (Medley, 1994; Medley and Goodman, 1994; Medley and Lindquist, 1994).

In a bimrock with a continuous distribution of blocks, the discrimination between matrix, and blocks, is difficult since blocks will be found at any scale. However, for any given volume, the large blocks will contribute the most to the volumetric block proportion. Recognizing this, one can select an arbitrary threshold block size, such as one percent of the maximum observed dimension of the largest block in the population being measured. Assuming spherical blocks and a fractal block size distribution, blocks with largest dimensions less than a 1 percent of the threshold size will contribute less than 1 percent of the volume and less than 5 percent of the total surface area of the blocks in the rockmass (Medley, 1994).

3 GRAPHIC MODELS

It is difficult to distinguish the difference between tracings of the blocks shown on maps of melanges and tracings from photo-microscopic images, if there are no reference scale bars. Hence, drawings and photographs at the intermediate outcrop scale are graphic models of larger scale melanges. Graphic models allowed the development of characterization methods useful in estimating the block volumetric proportion and block size distributions of melanges at engineering scales. For example, Figure 3 is the hand-tracing of a photograph of melange from Caspar Headlands, near Mendocino, California. The model cross-section is 180 mm in "depth".

Manual and computer-assisted image analysis methods were used to determine the areal proportions of the blocks and the block size distributions of graphic models, block size being characterized by the length of the maximum observable dimension of the block. The pictures were scanned and the scanned image digitized into an array of pixels (picture elements), each with a value of between 0 (black) and 255 (white) representing some shade of gray. Image analysis software measured the areas, perimeters and axial dimensions of the individual blocks in the images. In the case of the image of Figure 3, the

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\(^1\) E. Lindquist, 1991; "Fractals-Fractures and Franciscan", Term Paper for CE 280, Rock Mechanics; Dept. Civil Engineering, University of California, Berkeley, CA, 94720, USA
Figure 3: Graphic model of melange, showing scanlines. Scale bar is 20 mm long.

sum of individual block areas yields a block areal proportion of 35.6 percent. Additional data were collected by tracing the arrangement of blocks revealed at the surface of 14 cylindrical laboratory specimens (150 mm diameter, 300 mm long), part of a suite of 60 specimens of model melanges tested by Lindquist (1994). Lindquist fabricated the model melanges with a known distribution of block sizes, and varied both the block volumetric proportions and the orientation of the blocks. The actual volumetric proportion of these specimens, as determined from their weight densities, was compared to the estimates made from image analysis of the tracings. In general the actual volumetric proportion of blocks in the specimens was found to be approximately 30 percent higher than the areal proportion of blocks as exposed on the cylinder surfaces. The discrepancy was apparently due to the higher concentration of blocks toward the center of the specimens, but is still being investigated at the time this paper is being written (March, 1994). For the purposes of my work, I assumed that the areal proportions, as measured on the cylinder surfaces, were representative of the actual volumetric block proportions specimens, since I was really interested in how accurately the block volumetric proportions could be estimated by the linear block proportions measured from scanlines across the traced areas.

4 STEREOLOGICAL MEASUREMENTS

In practice one rarely has the good exposure of melange modeled in Figure 3, so outcrop mapping and drill core sampling must be performed in an attempt to characterize the rock mass. The continuous detailed geological log of a borehole

Figure 4: Unrolled tracing of surface of cylindrical triaxial specimen, showing location of scanlines. Specimen is 300 mm high. Blocks are oriented at 30 degrees to the vertical axis.
or the core itself, if available, are sampling scanlines. In the case of the graphic models, scanlines (modeled boreholes) were drawn over the graphic models of Figure 3 and the tracings of the surfaces of the triaxial specimens. Measurements were made of the lengths of chord intercepts through each block intersected by the scanline. The block lineal proportion for each scanline is the total length of chord intercepts along the scanline divided by the length of the scanline.

The equivalence of lineal, areal and volumetric proportions is a fundamental law of stereology, using which structures are characterized by zero-, one-, and two-dimensional measurements (Underwood, 1970). If an image is sampled by a sufficient total length of scanlines, the areal block proportion is estimated, and if sufficient cross-sectional areas are available, the volumetric proportion is estimated. However, it is not necessary that the linear measurements be made on common planes: the investigated volume can be sampled by an arbitrary array of linear traverses in order to arrive at the volumetric proportion. The practice of making linear traverses across microscopic images to estimate the areal and volumetric proportion of particulates within a matrix was a standard laboratory technique for petrographers, biologists and metallurgists before the advent of computer-assisted image analysis, and were commonly referred to as "Rosivahl Traverses" after the German geologist who devised the method (Rosivahl, 1898). The method was later modified to a larger scale, to allow estimate of the volumetric proportion of air bubbles in concrete test cylinders and the size distribution of those bubbles (Lord and Willis, 1951).

The areal proportion of the blocks in the image of Figure 3 were estimated by accumulating the block linear proportions of eight scanlines totaling 1684 mm in length. As shown in Figure 5, the cumulative linear block proportion is the ratio of the cumulative sum of chord intercepts measured for each scanline, to the cumulative sum of the lengths of the scanlines. Initially, the proportions will vary, but eventually the cumulative proportion converges to one value. For the image of Figure 3, the individual block lineal proportions varied between 53 percent and 30 percent. To test the influence of varying the summation order of scanline data, the incremental cumulative proportion (running average) and incremental proportion of total scanline length were computed, separately for two series of scanlines in arbitrarily chosen sequences. Figure 5 shows that the cumulative block lineal proportion converges (of the image shown in Figure 3) to be 38 percent, close to the block areal proportion of 35.6 percent. Convergence effectively occurs at 50 percent of the total scanline length, or some 840 mm of scanline measurements.

Images of the triaxial test specimens were each sampled by ten scanlines totaling approximately 3000 mm. The linear block proportions generally converged to plus or minus a few percent of the areal block proportions within approximately 40% to 60% of the cumulative scanline length. Hence, measurement of a larger suite of images confirms that at least the areal proportion (and by assumption, the volumetric proportion) of blocks exposed at a surface may be quickly estimated by scanlines.

Holmes (1921) reported a guideline to selecting a sufficient length of linear traverse for estimating the areal proportions of mineral grains in microscope images: a total traverse length of 100 times the average grain size in the area traversed was recommended. But to confuse this issue, it should be reported that Krumbein and Pettijohn (1938), stated that the "it was generally said" that a total traverse length equal to 1000 times the largest particle should give a "fair" degree of accuracy). However, if Holmes' admittedly more attractive criterion is adopted for the scanline measurements of the graphic model of Figure 3 (average block size of 8.25 mm), then a cumula-

![Figure 5: Block lineal proportion for the graphic model shown as Figure 3.](image-url)
tive length of scanlines of 825 mm would be sufficient to estimate the block areal proportion. The results shown in Figure 5 validate Holmes' (1921) guideline for the block population shown in the graphic model since the cumulative block linear proportion converges to the known block areal proportion, for both series of scanlines, within approximately 840 mm scanline measurements, or 50 percent of total scanline length. However, the guideline is of little use without prior knowledge of the average block diameter. Moreover, if the length of scanlines is reduced to more efficiently estimate the block areal proportion, then there will be poorer correlation between the frequency distributions of block intercept lengths (chords) and block diameters, as briefly discussed below.

5 ESTIMATING BLOCK SIZE DISTRIBUTIONS FROM SCANLINES

The lengths of chords across blocks intercepted by scanlines are rarely the same as the block characteristic dimensions (or "diameters"). But it can be assumed that the distribution of block cross-sections in the plane of a graphic model, or the distribution of chord lengths from scanlines, is directly related to the true distribution of the three dimensional blocks in the parent volume. The geometric probability character of this complex problem and some stereological solutions are discussed by Underwood (1970). From the geotechnical viewpoint, Tang and Quek (1986) used a statistical approach to estimate the frequency distribution of boulder diameters from a frequency distribution of borehole chord intercepts through boulders. Savely (1990) used Monte Carlo simulations to determine correction factors between the frequency distribution of boulder dimensions in a cemented boulder conglomerate, and the frequency distribution of drilled core intercepts through the boulders. Correction factors range between 1.0 and 1.6 (David Nicholas, Call and Nicholas, Inc; pers. comm). Interestingly, the eight scanlines across the Figure 3 graphic model resulted in 86 chord intercepts, with an average length of 7.29 mm, but the actual average block diameter (in two-dimensions) was 8.25 mm, larger by a factor of 1.13. However, for melanges, the notion of average dimensions is misleading, since average block sizes are greatly skewed toward the small sizes due to the fractal distribution of blocks. Hence, chord to diameter conversions may not be as simple for fractally distributed and irregular shaped melange blocks as it appears to be for normally distributed and rounded boulders in conglomerates.

The frequency distributions of chord intercepts for graphic models and field scanlines generally plot as power-law curves (log-log linear), which suggests that an empirical and general adjustment between the frequency distributions of chord intercepts and block maximum dimensions for melanges may yet be discovered. Current work shows that for any given chord intercept length, the number of equivalent block diameters is smaller; and for any given frequency, the estimates of true block dimensions will be severely underestimated if one assumes the chord dimensions to be diameters. The topic is explored further in Medley (1994) and Medley and Goodman (1994).

6 CASE HISTORY: LONE TREE LANDSLIDE, CALIFORNIA

Shortly after the Loma Prieta earthquake in October, 1989, California Highway I was closed by the Lone Tree Landslide (Figure 2). The California Department of Transportation (CALTRANS) restored access by excavating 956,000 m$^3$ (1.25 million yd$^3$) of Franciscan melange to an average depth of 37 m (Van Velsor and Walkinshaw, 1993). Relatively intact rock was excavated from behind the interpreted landslide surface to provide stable cut slopes (Figure 6). Blocks up to 30 m in exposed largest dimension were blasted during the excavation work (Michael Hobbs, Ford Construction, Inc.; pers. comm). Fill from the excavation was placed on the downhill side of the cut to buttress remnants of the landslide. It is anticipated that the fill will be removed by coastal erosion over several decades.

Prior to construction, nine exploratory borings (Figure 6) were drilled to between 37 m and 82 m deep to investigate the stability of the melange that would be exposed in the cut slopes. Some 375 m of HQ size core (61 mm diameter) was recovered, and eventually given to us for our research. Coring started some distance below the ground surface and only 82 m of the core was from those borehole segments located above the future excavated surface. The core from these segments was assumed to represent the melange excavated, and are referred to here as the
"excavation segments". Although much of the excavated melange was landslide material, Figure 6 shows that some of the melange removed from below and behind the assumed failure plane.

Since the largest blocks in the immediate vicinity of the slide are some 30 m in size (maximum observed dimension), the matrix/block threshold was chosen to be 0.3 m using an arbitrary 1 percent criterion (as described in section 2). Nevertheless, the block intercepts in the core were measured to as small as 5 cm in length. The data obtained by measuring the core are shown in Table 1 below.

The estimate of block volumetric proportion of the excavated material as measured from the excavation segments core (4.5 percent), was tested by measuring the areal proportion of blocks that are exposed in the excavation slopes. The blocks are relics of larger blocks that had been blasted or ripped during excavation. The site (37,780 m², or 8.1 acres) was divided into 21 sub-areas and the maximum observed dimension of blocks greater than 1 m in each sub-area was visually estimated. The block areas were estimated using the assumption that the maximum observed dimension was a circular diameter. The data are summarized in Table 1 below.

Some 40 percent of the core (approximately 150 ) was sufficient to show that the block linear proportion of all the core effectively converged to approximately 21 percent (Figure 6). The longest block intercept in all the core was 7.9 m, with an average length of 0.43 m. The intercepts from the excavation segments core were much shorter, ranging to 9.3 cm long, with an average length of 3.1 cm. Some 38 percent of the excavation segments core was sufficient to estimate the block linear proportion of approximately 4.5 percent (Figure 7). The block volumetric proportion of the unexcavated melange is estimated to be approximately 28 percent.

The total areal proportion of blocks exposed on the slopes of the excavation is estimated as 4.2 percent. This proportion is remarkably close to the 4.5 percent linear proportion estimated from measuring the core from the excavation segments. The contractor has estimated that approximately 5 percent of the excavation was composed of blocks greater than 1.3 m in size, (Michael Hobbs, Ford Construction; pers. comm.) which is an encouraging confirmation of the results of the study.

It appears that the material removed from the excavation was relatively deficient in blocks. Since the volumetric proportion was significantly less than 30 percent, it was geotechnically prudent and justified to assume the rockmass strength to be equal to that of the weak, sheared matrix. However, the proportion of blocks encountered was much greater than anticipated from the exploration drilling. Removal of blocks or reducing them to grade, required more effort than had originally been anticipated. But in this case, the extra blocks were welcome since they were used as protective rip-rap at the shoreline toe of the fill. Commonly, the discovery and laborious removal of blocks results in contractual disputes due to "changed conditions".
Table 1. Data obtained from measurements of Lone Tree Slide core and from field mapping

<table>
<thead>
<tr>
<th>Measurement</th>
<th>All core</th>
<th>Excavation Core</th>
<th>Field Mapping</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of boreholes</td>
<td>9</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td>length of core measured</td>
<td>375 m</td>
<td>82 m</td>
<td>-</td>
</tr>
<tr>
<td>avg. length core/borehole</td>
<td>42 m</td>
<td>10 m</td>
<td>-</td>
</tr>
<tr>
<td>number of blocks</td>
<td>191</td>
<td>44</td>
<td>117</td>
</tr>
<tr>
<td>average block size</td>
<td>0.43 m</td>
<td></td>
<td>2.7 m</td>
</tr>
<tr>
<td>block proportion (&gt;1m)</td>
<td>21 %</td>
<td>4.5 %</td>
<td>4.2 %</td>
</tr>
<tr>
<td>Convergence: (% total scanline)</td>
<td>(40%)</td>
<td>(38%)</td>
<td>-</td>
</tr>
<tr>
<td>min. block measured</td>
<td>0.05 m</td>
<td>0.05 m</td>
<td>0.3 m</td>
</tr>
<tr>
<td>max. block measured</td>
<td>7.9 m</td>
<td>0.093 m</td>
<td>15 m</td>
</tr>
<tr>
<td>max. predicted block size</td>
<td>15 m</td>
<td>1 m</td>
<td>100 m</td>
</tr>
</tbody>
</table>

There was a great difference between the maximum size of blocks predicted from the investigation of the excavation segments of core, and the maximum size of block mapped in the field. Interpretation of chord intercept data from all the core, and from the excavation segments, suggested that the maximum block sizes that could be encountered in the melange was of the order of 15 m and 1 m respectively. The maximum size measured in the field was, indeed, approximately 15 m, but the data collected from the fieldwork further suggested that a maximum block of 100 m could be expected. Several blocks of that at least that size are prominent in the general area of the Lone tree Slide. However, judging from my work with graphic models, it seems probable to me that the true distribution of block characteristic dimensions ("diameters") is significantly different from the distribution of chord lengths. Work is underway to predict a more likely distribution of block sizes.

7 SUMMARY AND CONCLUSIONS

Melanges are, generally, chaotic mixtures of hard blocks embedded within sheared shales and argilites, and are members of the large family of geological materials defined as bimrocks (block-in-matrix rocks). Recent work by others indicates that the overall strength of bimrocks is directly related to the volumetric proportion of the blocks. However, the application of a block volumetric proportion/strength relationship requires that a volumetric proportion must be estimated. The apparent self-similarity of the block distribution in melanges allowed the use of small-scale graphical models to develop a measurement technique useful for estimating the volumetric proportion of blocks by measuring the linear proportion of chord intercepts measured from drill core. Field mapping and measurement of drill core recovered from the Lone Tree Landslide, California, indicated that the technique may predict the in-situ proportion of blocks in a melange. However, more work needs to be performed to allow confident predictions to be made about the block-size distribution of in-situ melange from the same chord intercept data.

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Image processing of borehole wall
Traitement d’images des parois de trous de forage

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ABSTRACT Drilled core gives us the subsurface geological data except the orientation of the stratum, fault and crack. The borehole television system (BTS) was developed to obtain the above mentioned information, but it needs much time to observe the borehole wall. Therefore, borehole scanner system (BSS) has been recently developed to shorten the time.

Observing speed is 20 times faster in the BSS than that in the BTS, and the speed of processing the image of entire borehole wall is 50 times. Shortened time makes it possible to process the large amount of data such as cracks.

Main characteristics of the BSS are as follows.
1. The panoramic color image of the entire borehole wall is available and is processed as a color hard copy by using BWS.
2. The image data are recorded on an ordinary video cassette and digital tape cassette.
3. The scanned view has the information of azimuth and depth which accuracy is adequate.

Résumé: Les carottes de sondage nous donnent des informations sur la nature géologique du sous-sol, à l’exception toutefois des orientations des structures planaires (surfaces de stratification, fractures, failles).
Un système de télévision fond-de-trou a été développé pour obtenir des informations sur ces orientations, mais l’observation des parois du trou ne peut se faire rapidement.
C’est pourquoi, afin de réduire le temps d’observations, nous avons développé un système de scanner fond-de-trou.
La vitesse d’enregistrement des données est multipliée par 20 et celle du traitement de l’image par 50.
Les caractéristiques principales du système de scanner fond-de-trou sont les suivantes:
1. Une image panoramique en couleur de toute la paroi du trou est disponible et est analysée par BWS.
2. Les données sont enregistrées sur une cassette vidéo et sur une cassette audio digitale.
3. Les images obtenues fournissent des informations exactes sur les profondeurs et orientations des plans.

1 INTRODUCTION

Borehole TV System (BTS) was developed to obtain the information of various geological plane elements such as the dip and strick of bedding, fault, joint, and crack from the borehole wall. It is well known that it usually takes too long time to carry out the photographing and the analysing in order to obtain the above information in the BTS. This hinder the wide use of the BTS in the engineering geological field.

In order to make up for the BTS shortages, the Borehole Scanner System (BSS) which will be described in this paper, has been developed. The Panoramic image of the entire borehole wall can be obtained instanciously and the statistical processing of cracks can be simplified by using the BSS.

2 APPARATUS

An outline of the BSS, is shown in Fig.1 and apparatus are shown in Fig.2. A beam of light from a light source is reflected on the borehole wall by a mirror. Images of the spot-shaped point on the borehole wall are photographed by means of a mirror. During this process, the mirror is rotated 3,000 times per minute as the probe is inserted the borehole at a speed ranging from 1.0 to 1.2 meters per minute. This enables the device to scan the entire borehole wall. The sensor relays images of a borehole wall to the surface, where the monitor displays the images continuously. Images of the borehole wall are recorded in analog format on a video tape recorder, and also in digital format on a
Fig. 1. Diagram of Borehole Scanner System (BSS). Details see text.

Fig. 2. Photographs showing the BSS apparatus (a) and the state of field working (b).

digital recorder. The azimuth, scanning image depth, image processing, and the measurement conditions are described as follows.

2.1 Azimuth
Azimuth can be measured either by the magnetic compass or gyrocompass. The magnetic compass is easy to use and very cheap. When the magnetic mineral is present inside the borehole or a high power electric line is near the borehole, the azimuth can't be measured accurately by the magnetic compass, but azimuth can be decided accurately by gyrocompass even in the above case, which is very expensive.

2.2 Depth
The depth is determined by the rotation of a gear when the cable is moved (down or lateral motion). To increase the accuracy of depth measurements, esuron tape used for surveying inside a special testing borehole is set and photographed to determine the calibration factor. Consequently, the method is sufficiently accurate within 1 centimeter per 400 meters. In the case of horizontal motion, it is pushed down to the bottom of the borehole, then pulled up with a winch as the depth is measured. In this case, it is possible to use an inclinometer to perform scanning according to the gravitational direction (bottom of the hole).

2.3 Image Processing
The images of the borehole wall pass through a photo-electric transformer, are then digitalized by an A/D transformer, and the resulting data are recorded on a digital tape. Simultaneously, it can be recorded on a video tape as an analog image. As it is, the analog image is displayed on a monitor.

Image processing can be performed on an engineering work station by using the data recorded on the digital tape, and a panoramic image of the entire borehole wall is obtained in hard copy form. The resolution of the image is 0.1mm.

2.4 Measurement Conditions
This apparatus can be employed in boreholes with a diameter ranging from 66 to 200mm, and 1,000 meters in depth. It can normally withstand temperatures ranging from 0 to +50°C. When the entire probe is set inside a cooling device, it can be used where the temperature is between 200 and 300 degrees, but observation time is shorter because it is very difficult to cool the probe long time.
3 COMPARISON OF BTS AND BSS

The observation of borehole wall can only be made on 2–3 cm in width which is enlarged into the TV screen size in one time in the BTS. This method is applicable to observing the injected state of grout milk, characteristics of crack, the state of groundwater flow, and measuring the strike, dip of bedding, fault, and crack. But it takes too long time to do measuring and observing the images, the BTS is not used so often as so far.

For example, to obtain a panoramic image and to take a photograph of overlapping some area of a crack with a dip of 45 degrees, it is necessary to rotate the TV camera 5 circles (1.8 cm × 5 = 9 cm) for a 65 mm borehole. It is necessary to combine eight images for each rotation. This means that no more than 0.09 meters area can be photographed with one time. It takes 1 minutes and 30 second to do photographing and will take much more time for observing the images in detail.

Accordingly, to obtain the panoramic images of the borehole wall using a TV camera, it takes 15 minutes per meter and 25 hours for 100 meters. In order to capture the images on the TV monitor to a continuous picture, it is necessary to take approximately 440 pictures per meter, and it takes 4 hours to combine these pictures.

In contrast to this, with the BSS, the probe can be moved down at a speed of 2 cm per second. It only takes 83 minutes per 100 meters to move down, and 1/20 of the time required using the BTS to photograph images.

Furthermore, the image is recorded in digital format, so the image processing can be performed in an engineering work station. The color borehole wall panoramic images can be output at a speed of 20 cm per minute, which is about 1/50 of the time required in the BTS.

The photographing and image processing can be done

Fig. 3. The map showing a comparison of the geological sections drawn by the drilled core data and from the BSS image data. (a) drilled core at the depth of 33.00–37.00 m. (b) geological section drawn by the drilled core data. (c) panoramic images of the BSS inside borehole at the depth of 34.00–37.00 m. (d) the geological section, revised by the BSS image data, which is the same section location of as (b).
in a short time in the BSS, so it is very easy to obtain the geological information inside the borehole by using the BSS.

4 SAMPLE MEASUREMENT

4.1 Orientation of a Geological Structure
When observing boring cores, it is generally difficult to determine the geological structures if the core is disturbed. But it is very easy to do this inside borehole by using BSS. Fig.3a shows a sample of this use of the BSS.
In Fig.3 which is a photograph of a boring core, a boundary between dolerite and stratified tuff at 34.45 meters can be observed clearly.
In the past this dolerite was interpreted to be (judged to be) a sheet, and was represented as shown in Fig.3b but the BSS image (Fig.3c) shows clearly that it is a dyke with a boundary of N67W•73S. The sectional diagram was modified as shown in Fig.3d.

4.2 Determination of the opening cracks
It is difficult to determine the range of a baserock slippage with a deep slipping surface only from a boring core. But it is possible to specify the range of an opening crack and the slipping surface can be determined by using the BSS.
Fig.4a is part of a picture of a boring core of a certain base-rock slippage. It could not be determined whether the slipping surface was up to 68.60 or up to 71.48 by the observation of the boring core. The opening cracks, area observed in boring core below 68.60m in depth, but it can be observed clearly that these cracks below 68.60m inside borehole are not opening cracks, all of which are closed and solid (Fig.4b, c). Therefore, the slipping surface can be judged to be 68.60m in depth.

Fig.4. The map showing a comparison between the drilled core and the BSS images, both taken from the same borehole at the depth between 60.00–75.00m. Note that cracks can’t be determined whether they are open or not in the drilled core, but the BSS can determines the opening cracks.
4.3 Applicability of the BSS in the Lugeon test analysis
The permeability of baserock is generally affected by the opening cracks, but the characteristics of crack can not be observed clearly from a boring core. Fig.5 shows a such sample. Some opening cracks can be observed from the boring core, where the permeability is almost equal to zero. But, it can be observed clearly that the opening cracks observed in the boring core are not opening cracks, all of which are closed and solid. In the other way, sometimes the closed cracks can be observed in the boring core where is a area with a high permeability (Fig.6).

Fig.5. The map showing a comparison during the P-Q curve, drilled core, and the BSS images. Note that the permeability is very small (a), but a lot of cracks are observed in the drilled core (b). The BSS images also show that there is no opening crack on the borehole wall.

4.4 Hydraulic Fracturing
When a Lugeon Test is performed, occasionally an abrupt failure was recorded, as in the case of the P-Q curve shown in Fig.7a, and which cause is not clear. The base-rock in this area consist of the Miocene tuff which is soft. Its uniaxial compression strength is between 5 and 7 Mpa. It can be observed that there is no in normal line direction crack in the boring core of (Fig.7b), but it is shown clearly that there is 70~75m in depth a crack on the borehole wall in normal line direction in the panoramic images of the BSS (Fig.7c). The image processing result shows that the crack has a strike of N26°W in magnetic azimuth which is N34°W in true north.
Cracks caused by hydraulic fracturing are generally normal to 3 of initial stress. This indicates that in this area the direction of the maximum principal stress is N34°W.

Fig.8 shows the maximum principal stress direction of Japan measured using the stress relief method. The locations shown in the Fig.8 marked with a open circle are the locations of the above scanner image measurements. The results of the BSS image measurement reveal that the 1 directions determined by the BSS image measurement are very similar to that determined by the stress relief method. This shows that the initial stress can be measured in the soft baserock where the limited water pressure is about 1 Mpa during the Lugeon testing.

5 COMPARISON OF THE NUMBER OF CRACKS IN A BORING CORE AND BOREHOLE WALL
Table 1 gives a comparison of the number of cracks found in a boring core and revealed by BSS in a Miocene lapill tuff. This shows that the scanner, which faithfully represents the form of the ground, revealed far fewer cracks. The cause is thought that potential cracks were transformed to real cracks and the mechanical action during the boring and core extraction processes generated new cracks.
Fig. 6. The map showing a case in which the cracks are not seen opened in the drilled core (b) but the BSS images (c) show the existence of the opening crack, which caused high permeability.

Fig. 7. The map showing a case in which there is no crack in the initial baserock (b), but cracks (c) parallel to borehole wall were generated while the water pressure rised to about 10 kg/cm² during the Lugeon testing (a).
6 STATISTICAL PROCESSING OF CRACKS

Statistical processing of cracks and joints has been done based primarily on data at outcrops, adits, and other locations which are accessible to people. But with the BSS, their orientation can now be determined easily, permitting the statistical processing of cracks inside boreholes. Fig. 9 shows an such example.

However, while there is a high probability of a borehole drilled vertically encountering generally dipping cracks, the probability of a vertical borehole encountering a steeply dipping crack is low. And this must be kept in mind when one gives a conclusion of crack characteristics only based on the BSS images.

In such cases, the best approach is to conduct a supporting study of the data from the same number of outcrops and adits.

7 CONCLUSION

The examples stated above have shown that it is possible to obtain panoramic images of borehole walls easily by using the BSS.

In addition to the benefits shown through these examples, this approach has the following merits.

(1) It is not necessary to take oriented core sample.
(2) It is possible to correct errors in core arrangement caused by a drilling operator’s mistakes. And a sharp reduction in such errors can be achieved by pointing out these errors to the operators.

But because visible light is used, recording is, of course, not possible when (a) the water inside the borehole is muddy, and (b) when bentonite is used during drilling. In case (a), recording is possible by adding a precipitating agent, but further study will be necessary to find suitable measures to employ in case (b).

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On rock mass quality assessment
Évaluation de la qualité de massifs rocheux

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Abstract: Rock mass quality assessment, both a means and a goal of rock mass engineering study, is not only judgement on geological property but also evaluation on significance of geoengineering. This paper discusses the basic criteria for rock mass assessment, the assessment index and engineering geological parameter field based on principles of information theory. Finally, several mathematical models for rock mass quality assessment are proposed systematically.

RÉSUMÉ: L’estimation qualitative, le moyen et le but de l’étude des masses rocheuses, est non seulement le jugement du caractère géologique de la masse rocheuse mais aussi l’estimation des travaux géologiques. Dans ce texte, on discute le critère essentiel de l’estimation qualitative de la masse rocheuse, l’index de l’estimation est la place de paramètre des travaux géologiques bas sus le principe informatique. Enfin, quelques modèles mathématiques de l’estimation qualitative de la masse rocheuse ont été proposés systématiquement dans ce texte.

Assessment is defined such action as measurement of given objective based on clear aim, and change it into subjective effect to meet with requirement extent for main body, i.e., the process of evaluation. The rock mass quality assessment is the judgement, analysis of the engineering geological conditions, mechanical characteristics of rock mass to meet the requirement of different project. Rock mass quality assessment have binomial characters: on one hand a means, a judgement on geological prototype and a qualitative analysis; on the other hand reflecting a goal to some extent, a evaluation on significance of geological project and a quantitative analysis.

Clarifying the research criteria for rock mass quality assessment constitutes not only the base for study but also a necessity for rational scientific assessment. On evaluation how to assess constitutes the Meta Assessment of rock mass quality assessment. With same significances is the modification, supplementation and improvement of assessment.

Correctly recognizing and selecting as assessment standard and scientific metrology and criteria, precisely giving the mean weight of various engineering geological parameter index, rationally and scientifically evaluating and analysing comprehensively the above three factors constitute the key subjects of rock mass quality assessment.

Scarce information, chaos and uncertainty during process of rock mass quality assessment require comprehensive application of data, model and expert knowledge in handling this very complicated assessment system. The data for assessment refer to those comprehensive and analyzed judgement (evaluation conclusion) parameters on rock mass environment. The model assessments refer to those based on model selection and application, and expert knowledge assessment depend on expert knowledge, experience and intuition.

The authors of this paper were once engaged in many types of geoengineering for rock mass quality assessment, such as damsite rock mass
(Three gorges hydropower project, Gaobazhou hydropower project and Geheyuan hydropower project etc.), slope engineering (Jinchuan open pit mine slope engineering, Lianziya hazardous rock mountain harnessing engineering) as well as tunnel project (Junduoshan tunnel etc.), make systematic theoretical study on rock mass quality assessment as well as engineering application. This paper will discuss in theoretical on basic criteria parameter index as well as assessment model for rock mass quality assessment to promote the developing of the problem in engineering application.

1 CRITERIA

1. 1 Engineering Geological Mechanics Study

Rock mass quality assessment belongs to typical rock mass engineering mechanics subject. Its studying objective is geological body which has been formed and deformed. Apparently, rock mass should first be discussed in view of geology, secondly it serves for project in view of material, environment or applied function. Different type of projects have different requirement, and at the same time different rock mass condition affect the design, implementation and treatment of the engineering. Therefore, the problem of responding, adapting and coupling should be considered. Moreover, rock mass quality is a summary of rock mechanics characteristics, and we need to study the rock mass mechanical regularities of the deformation and re-deformation for deformed and destroyed geological body under the action of engineering. The mechanical nature, mechanical process and mechanical mechanism of different structure of rock mass should be further recognized, and when the mechanical characteristics are summarized, different mechanical effects should be considere, and we establish correspondent rock body structure mechanical effect rules based on different rock mass structure mechanics effects. As a result, it is suitable to apply engineering geological mechanics and its academic thoughts, research methods as well as technical mean to assess rock mass quality. Meanwhile, only under guide of Structure Control Theory can one cratches the core of problem and make deep study on it.

1. 2 Engineering Geology, Geoengineering, Rock Mass Mechanics Study as a Whole

In reality rock mass quality assessment is the comprehensive analysis on engineering geological condition, rock mass mechanics, the basis for analysis of engineering geology and study of rock mass mechanics action, meanwhile it is also a pin-point of geoengineering in service of geological body re-creation. It should be considered in connection with exploration method, calculation model, excavation treatment (or support system). Study should clarify the principle of engineering geology, geoengineering and rock mechanics as a whole.

1. 3 Objective

Assessment should be objective, reducing in every effort the effect of subjective factor. Guarantee of the certainty of assessment parameters should be based on full geological study under background of macro—geological and experimental data and monitoring results. Meanwhile, assessment result should be re—checkable.

1. 4 Systematic

The systematic principle of assessment require that all the factors must be combined as a whole to recognized the function regulated by goals. Apply the principle method, concept and means of systematics engineering taking up analyzing comprehending design and utilizing objectives and reflecting the most important function of different aspected system.

1. 5 Coordination

Firstly refer to different order coordination, the one between high and low level of assessment principle, i.e., certain method is one unite of total assessment system.

Secondly refer to different stage coordination, e.g., the different stage of project such as planning, feasibility study, initial implementation, implementation, final implementation
and operating stage. Though rock mass quality assessment ranges from beginning to end, the studying extent different greatly. However the results should not be contradicted.

Thirdly refer to different research levers and means (especially measurement methods) co-ordination.

1.6 Possibility of application and Acceptance

The assessment method should be applicable to different rock mass under the same type of engineering, and the acceptance refers to the feasibility of the method in view of mastering and utilization. It ought to be both easily applied and practical and the data can be easily collected. A complicated rock mass quality assessment, no matter how academic it is, will not be useful for its inconvenient in mastering and application.

1.7 Dynamics of Assessment

With development of science and technology, assessment improves gradually.

2 INDEX SYSTEM

Regarding the academic thought and research method of engineering geological mechanics, we should comprehensively on geological setting (based on formation and deformation), engineering geological components, structure characteristics and geophysical environment of the rock mass as well as its corresponding project action feature in order to define the index system for rock mass quality assessment.

Rock mass quality is a measuring nature of its geological setting system. The complexity and multiplicity of the system and its sub system decided that the system nature should be measured through multiple means. The varient of the system define the method of measurement, constitutes the measurement model for various regional types and judgements. As a second measurement for geological setting system, rock mass quality assessment is realized mainly through unear observation. Its overall information flow feature may be ideally explained in informaton flow theory with measurement mathematical model.

The incompleteness of significance for system event produce uncertain, consequently the index factors we require are random and unclear. The standard index of different mass factors follow certain probability distribution and status description from statistic analysis is objective. This uncertainty is created by incompleteness of system condition. Quality assessment is obtained through unclear measurement. As a result, the index factor we required is also unclear.

The mathematical model for rock mass quality assessment is random, in anotherwords, it is a random function with two ($\xi, t$) or more than two varients (in which, $\xi$ — space condition, $t$ — time coordination).

Generally, $R(\xi, t) = M[R(\xi, t)] + \Delta R(\xi, t)$

where,

$R$ — assessment parameters;
$\xi$ — multidimension vector of coordination;
$M[R(\xi, t)]$ — mathematical expectation of field (nonrandom function of time and space) 

Generally, the mathematical expectation of field includes two varients; regional relative variant and semi-cycie variant, reflecting the trend of geological action and define the Malcol interaction nature of assessment index parameters in advance.

It is special significance to decide the homogeneity area for assessment parameter field, where following formula may be applied,

$I_0 = L_0 \frac{\sigma R}{R_0 - R_e}$

where,

$I_0$ — the width of $R$ homogeneous area of parameter field;
$L_0$ — the length of parameter field along $\xi$ section;
$\sigma_0$ — the mean deviation of $R$;
$R_0$ and $R_e$ — meanvalue of parameter of field boundary examined.

The heterogeneity deviation in different of main changing trend for parameters should be considered sufficiently.

It is worthwhile pointing out that the index system for rock mass quality assessment with incomplete structure element relationship and the behavior of operation information requires us to emphasized in gray problem in index system.
In summary, we may clearly recognized that the index system of rock mass quality assessment is multi—variants, multi—goal, having random, unclear and gray characteristicity.

This requires us change the way of recognizing rock mass quality index value with clear and fixed method and analysis.

3 ASSESSMENT METHOD AND MODEL

In the past the expert evaluation method was employed in rock mass quality assessment, i. e., based on evaluators subject judgement and “grades” or “index” as assessment scale, usually including:

1. Summing evaluation, i. e., decision of good or bad using sum accumulation:
   \[ S = \Sigma s_i \]
2. Constant multiplying evaluation, multiply the fractional values of various index, then evaluate according to value product:
   \[ S = \Pi s_i \]
3. Summing and multiplying evaluation; divide each index into several groups. First calculate the sum of fractional values in each group, second, multiply the sums of each group as total evaluation:
   \[ S = \Sigma s_{iwi} \]

where, \( w_i \) — weight of i item for convience of judgement and comparation, the sum of each weight number fits induction principle.

The above expert assessment methods are simple and easily applied, but have strong subjectivity. On the basis of application and considering the random, unclear and gray characteristics of assessment criteria and index system for rock mass quality assessment as well as its multiple variant and goal characters, we develop a dynamic method using mathematical model for multiple factors described briefly as following.

3.1 Mahalanobis Distance Optimum Ordering Method Based on Variant Analysis

Assessment follows this steps:
1. input sample matrix \( S \);
3. 3 Gray—R Analysis Method Based on Gray System Theory

Apply five steps model setting method to gray R—analysis. The procedure is shown in follow.
1. input index value;
2. adjusting index structure;
3. data change 0—100 for white matrix;
4. set white function (Y);
5. based on white classification criteria;
6. calculate index R weight (w);
7. calculate unknown sample R;
8. coefficient for each group (θ);
9. judge type K=K(θK)=MAX(θk).

Where R—weight $w_i = \frac{Y_i}{\sum Y_i}$

objective R—coefficient may be obtained through this formula:
$$C_0 = \sum Y_i (X_i) w_i$$

3. 4 Fuzzy Comprehensive Judgement Based on Fuzzy Mathematics Theory

Follow these step:
1. Set fuzzy relation matrix;
2. Fuzzy reformation $B = A \ast B$
3. For weight mean type $B = A \Theta R$

The above four assessment methods and their corresponding parameter, model have all been applied to damsite, slope as well as tunnel engineering rock mass quality assessment with satisfactory effects.

4 CONCLUSION

As essential rock mass research, the rock mass quality assessment is not only a means but also a goal. There still exist a great number of theoretical and practical problems. The authors hope to strength the theoretical research of rock mass so as to serve engineering better on basis of sufficient experience.

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Quantitative measurements of discontinuities in rock masses for practical purposes: Comments on and suggestions for sampling procedures
Mesures quantitatives des discontinuités de massifs rocheux: Commentaires et suggestions relatifs aux prélèvements d’échantillons

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ABSTRACT: Despite the wide acceptance of both the GSL (1977) and ISRM (1978, 1981) recommendations for quantitative measurements and description of discontinuities in rock masses for engineering purposes, some of its suggestions need rejection. This concern especially the suggestion of both recommendations that discontinuity survey may be limited to some discontinuity parameters, a priori assumed to be relevant for an engineering problem, only. Some other suggestions are commented, too. The importance of quantitative measurements of discontinuities for applied hydrogeological purposes is emphasized and shortly discussed.


1 INTRODUCTION

The complete description of a rock mass for any practical purposes requires the characterization of both the material that constitute that mass and the discontinuities that dissect that mass. It is now generally accepted that the strength, deformability and permeability properties of rock masses are to a large degree determined by the type, spatial orientation, spacing, aperture, persistence and surface characteristics (roughness, waviness, infilling) of discontinuities, i.e., of bedding planes, faults, joints, fissures, cleavage- and foliation planes etc. Thus, accurate collection of data concerning these quantities of all types of discontinuities are of greatest practical importance.

Structural data requirements for engineering purposes and recommendations for necessary sampling procedures have been described in detail by a number of authors (e.g., Knill 1971, Pacher 1959, Piteau 1970) and institutions (e.g., GSL 1977, ISRM 1978, 1981). The latter are of special interest because of their wide acceptance. However, wide acceptance may result from unawareness or ignorance of other standards or requirement and do not mean that all suggestions or statements are right.

Based on this assumption and our experiences in quantitative measurements of discontinuities for a broad spectrum of practical purposes, we wish to present our comments to the two most accepted recommendations for measurements and description of discontinuities of rock masses for engineering purposes cited.
Finally the significance of quantitative measurements for some applied hydrogeological purposes, exactly for the prediction and/or evaluation of dispersion parameters for jointed rock masses, is briefly examined.

This contribution could be understood as exchange of both opinions and experiences.


Following the elaboration of several engineering classifications of rock masses (e.g., the RSR Concept by Wickham et al. 1972, the Geomechanics Classification by Bieniawski 1973, slightly changed in 1979 and the Q-System of Barton et al. 1974), a number of national and international institutions, like the Geological Society of London Engineering Working Group (GSL 1977) and the International Society of Rock Mechanics (ISRM 1978, also 1981), had published detailed recommendations for measurements and quantitative description of discontinuities in rock masses. As a mean of correlating experiences on rock conditions at one site to the conditions and experiences encountered at others, rock mass classifications are of high potential (Bieniawski 1976). And as a means of facilitate the communication of data and facts between interested parts, standardisation should be achieved in the description of rock masses, especially of discontinuities, too (GSL 1977, p. 356). Thus, the usefulness of rock mass classifications for practical purposes as well as the need for standardisation of discontinuity surveys and description of rock masses in terms of structural properties are unquestionable.

However, because both the GSL (1977) and ISRM (1978,1981) recommendations for measurements and quantitative description of discontinuities were published after the elaboration and extensive employment of rock mass classifications for a wide range of engineering purposes, the recommended procedures and techniques of discontinuity measurements are greatly affected by the relative importance of individual discontinuity parameters given in an accepted rock mass classification system. Thus both recommendations discussed admit that only some discontinuity parameters are exactly measured (orientation parameters, i.e. strike or dip, spacing and persistence) whereas other are visually estimated (aperture of discontinuity surfaces, roughness, weathering and waviness of discontinuity walls, and infilling). Thus both recommendations suggest that a subjective approach of discontinuity surveys will furnish the necessary data for an adequate rock mass description provided that adequate quantities of measurements are made (GSL 1977, p.379). But this subjective approach limit the number of interested parts and, if the projected engineering work is dislocated or changed and/or replaced by another project, supplementary or repeated discontinuity survey will be necessary to obtain the structural parameters needed for the new project. Based on this inconsequences, the authors disagree with this commonly practicized subjective discontinuity survey in which only a limited number of classification indices are exactly measured. Rather than making a priori assumptions about rock masses and the relative relevance of some discontinuity parameters it would be better to characterize and quantify a individual rock mass structure as complete as possible and then look for simplifying factors or complex indices to make a defined problem feasible for solution, an opinion repeatedly expressed in publications (e.g., Hudson 1988).

Another question concern the discontinuity survey technique for engineering purposes recommended. It is commonly stated that the scanline sampling method appears superior to other, as, e.g., areal sampling, methods of recording techniques, as it gives more details on the discontinuity intensity and its variability and ensure that all discontinuities that intersect the scanline, unaffected of type, orientation and dimension, are recorded. Clearly this technique offers many advantages, among others:

1. Measurements of discontinuities along a scanline are simple and
quick.
2. If line sampling on differently oriented faces or two or three mutually perpendicular axes are carried out, 2-d discontinuity parameters may be easily calculated. However, as the possibility of discontinuity measurements in three dimensions is seldom realized, some assumptions are needed to obtain volumetric jointing parameters, even if advanced geostatistical techniques, like those developed and discussed by La Pointe and Hudson (1985) are used.

But these advantages of scanline sampling technique do not justify the rejection of areal measurements simple on the assumption that the areal counting procedure is laborious and time consuming, an assumption also accepted to justify the use of a subjective (see above) rather than the - in fact - more intensive and extensive objective approach of discontinuity survey (see, e.g. GSL 1977, p.379). The real variability of areal fracture density and/or intensity within a rock mass may be obtained only if sampling and counting are carried out on fixed scanareas. The distribution and dimensions of sampling areas on the exposure walls could ensure adequate representation of the visible structural variability (see, e.g., Halstead et al. 1968, Da Silveira et al. 1966, Knill 1971, Liszkowski and Stochlak 1976). Thus area sampling could be recommended as required supplement to scanline sampling data (Liszkowski and Stochlak 1976, Baczynski 1980).

The statement that area sampling is laborious and time consuming is unjust. It is more a question of the methods used for obtaining the data. The authors stressed the usefulness of photographs of rock mass exposures not only as a method of discontinuity survey as discussed in GSL (1977) but as required document of a discontinuity survey, unaffected of the approach - subjective or objective - used. Enlarged photographs could form the base maps to locate or mark on all scanlines and areas as well as all traces of visible and measured discontinuities in the wall(-s) of the exposure. Each measured visible discontinuity could be numbered and plotted successively on both the photograph and discontinuity data sheet like those proposed by GSL (1977, p.385, Fig.4). Enlarged photographs of selected representative scanareas may be then used to measure areal densities and intensities of discontinuities directly from those photomaps. A similar but more advanced 2-d technique was proposed and successful used by Baczynski (1980).

It should be stressed that all linear and areal discontinuity quantities could be corrected or "normalized" for the angular relationship between the strike of the set and the strike of the scanline or scanareas; this is a necessary procedure to obtain true data (see also La Pointe and Hudson 1985, Liszkowska and Liszkowski 1989, Liszkowski and Stochlak 1976) contrary to the opinion of GSL (1977). The data obtained may be then used to estimate true probability distributions or density functions of trace length, spacing, areal density and intensity of discontinuities (Baczynski 1980, La Pointe and Hudson 1985).

Our next comment refer to the number and distribution of scanlines for direct discontinuity measurements in the field. It is commonly recommended to measure discontinuities be measured along scanlines that cross one another at approximately perpendicular axes (Liszkowski and Stochlak 1976, GSL 1977, La Pointe and Hudson 1985). However, the number of scanlines is not exactly defined or discussed. In our discontinuity surveys for applied hydrogeologic purposes we accepted at least two to three nearly horizontal scanlines on each (or at least on two nearly perpendicular) exposure walls to obtain quantitative structural data for each (mostly two or three) weathering grade zone (as defined by e.g., Fookes et al. 1971) distinguishable in the vertical profile of the exposed rock mass and one to two nearly vertical scanlines on each exposure wall to obtain quantitative data for the variability of, e.g., bedding plane spacings in the vertical. Despite the apparently large number of scanlines measured, the survey is not very time consuming while the benefits of the surplus data collected are unquestionable. It seems that the same method may be recommended for engineering purposes.
Despite the objections both recommendations may prove useful in achieving standardisation in description and assessment of rock masses for engineering purposes and may be the basis to establish new classifications or methods of appraisal.

There is another field of likely employment of quantitative jointing data, namely: applied hydrogeology and this will be briefly discussed below.

3 THE SIGNIFICANCE OF QUANTITATIVE JOINTING SURVEYS FOR APPLIED HYDROGEOLOGY PURPOSES

The potential significance of jointing measurements for applied hydrogeologic purposes may be easily manifested examining the simplified flow equation through a set of parallel fractures:

\[ q/J = k = (Q \eta \gamma \mu \sigma r^{-s} d^{-b})^{-1} b^2 \]

\[ = c_n b^2 \]  

(1)

where: \( q \) - the fluid flux, \( J \) - the head gradient, \( k \) - the hydraulic conductivity, \( Q \) and \( \mu \) - the fluid density and dynamic viscosity, respectively, \( g \) - the acceleration due to gravity, \( s \) - the separation index after Pacher (1959), \( \gamma \) - the tortuosity of fractures, \( r \) - the roughness of fracture surfaces, \( b \) - the aperture of fractures, \( d \) - the spacing of fractures, \( C \) - a constant including the physical properties of the fluid (the first term in brackets on the right side of equation (1)) and the physical properties of the fracture surfaces (the second term in brackets of equation (1)), and \( n_b \) - the linear fracture porosity equal to the ratio b/d.

All jointing parameters (s, \( \gamma \), r, b, and d) of equation (1) may be obtained from direct jointing measurements in rock exposures and subsequently utilized to evaluate the hydraulic conductivity of the rock mass studied. But direct measurements of fracture aperture, the most relevant jointing parameter since it occur in equation (1) to the second or third power, are not likely to provide a good estimate of the hydraulic conductivity. In situ methods for determining rock mass permeability as the packer (or lugeon) and other hydraulic tests (see, e.g. U.S.Dept.Int. Bureau of Reclamation 1974) are by far the simplest and best techniques to measure effectively of the property we are interested in.

But in the last decades groundwater contamination and prediction of contaminant transport in aquifers arised to the greatest, both theoretical and practical problem in applied hydrogeology.

A number of models for solute transport in fissured rocks are actually available (e.g., Maloszewski and Zuber 1993). The solution of each of these models need the independent and explicit knowledge of at least one, commonly more than one, jointing parameters. In most cases this is the fissure aperture (b) and/or fissure porosity (n_b); in the model of Nerotniokis (1983) it is the fissure aperture distribution (f(b)). The last model is of special interest because it relate explicitly the dispersion coefficient (D_1), the quantity describing the spreading of a conservative contaminant along the flow path of the fluid, with the fissure aperture distribution in terms of its standard deviation (\( \sigma_b \)). This solution is of great practical use, especially for predictive purposes.

For this branch of applied hydrogeology direct jointing measurements on rock exposure walls are the only method for obtaining the quantities we are interested in. However, to obtain proper mean values and distributions of fissure apertures a great number of individual measurements are necessary. We accept in our discontinuity surveys for spreading predictions (Liszkowska and Liszkowski 1989) that at least one hundred values for each set of systematic joints could be obtained. The employment of a technical feeler gauge make the aperture measurements simple and fast. Up to 500 to 600 measurements down to aperture values of 0.05 ± 0.1 mm may be obtained in 5 to 6 hours. For each single joint 3 to 5 measurements by chance along the visible - and marked on the suitable enlarged photograph (see above) - trace length of the joint were performed. At a later stage (after the number of sets and its orientation parameters were fixed) the measured apertures were corrected for the angular relationships between the strike of the set
and the strike of the scanlines (s). Our investigations prove the finding of Snow (1970) that the aperture of joints of each set and of all joints together may be best described by means of a lognormal probability density function (Liszewska 1990, p.75, fig.11). It is alike true for both spacing between adjacent joints of each set and joint trace lengths (see also Baczynski 1980).

It should be stressed that the values obtained could be careful analyzed and critically examined for possible error sources prior to calculate the quantities needed for the problem solved. However, these problems are outside the scope of this paper.

4 CONCLUSIONS

This contribution is clearly divided into two parts. In the first part comments to the two most known and accepted recommendations on methods of quantitative measurements, description and assessment of discontinuities in rock masses for engineering purposes of the GSL (1977) and ISRM (1978), are presented. If any conclusion should be drawn from this part, then it would be that direct discontinuity surveys in rock mass exposures could be as complete and objective as possible, unaffected from the particular projected engineering work for which the survey was carried out.

In the second part the significance of quantitative discontinuity surveys for applied geotechnical purposes are briefly discussed. It is stated that for the solution of groundwater pollution problems in fissured media, exactly for the analytical solution of contaminant transport and dispersion models, direct discontinuity measurements are the only method of obtaining the jointing parameters of the rock mass we are interested in. This conclusion emphasize the ones drawn from the first part because any engineering work realized on or within a rock may be at any time in the future the potential source of groundwater contamination.

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Behaviour of rigid shallow foundations on Quaternary sands
Comportement des fondations superficielles sur un sable Quaternaire

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ABSTRACT: This paper deals with behaviour of rigid shallow foundations on sand. The influence of main factors to the settlement of this foundation is considered. These factors are: bearing pressure, breadth of loading area, soil compressibility and depth of influence of the loaded area. The results of statistical analysis of settlement records of shallow foundations on sands over 200, are presented. As result of this analysis, a simple method for settlements predicting is given. Proposed method is suitable for routine design procedure.

RESUME: Cet article decrit comportement des fondations superficielles sur un sable quaternaire. De nombreuses études ont été consacrées au calcul du tassement de ces fondations. Il est nécessaire d'effectuer ce calcul, prenant en compte l'influence des facteurs suivants: la pression superficielle, la largeur de la charge, la compréhension du sable et la profondeur d'influence de la charge.
On a montre, ici, les résultats d'analyse statistique, environ 200 exemples. Le résultat de cette analyse est une équation pour calculer le tassement des fondations superficielles rigides. Elle est très simple pour l'utilisation. Les résultats obtenu jusqu'à présent en matière de ce calcul sont encourageants.

1. INTRODUCTION

Numerous methods of predicting settlement of foundations on sands have been published, many more methods than for clays. The reason lies in the extreme difficulty of obtaining undisturbed samples for the laboratory determination of compressibility under appropriate conditions of stress and strain history. Hence resort has been made to the interpretation of field in situ tests such as the standard penetration test (SPT), cone penetration test and plate loading test, and much of the literature has been devoted to such interpretations.

This paper describes the analysis of 224 record of settlements of rigid shallow foundations. The sands are of a different genesis (fluvial, aeolian, glacial, etc.). Examples are taken all around the world. The object of the study was to assemble as much data as possible on actual field observation of settlement with the minimum of interpretation to see if a simple picture emerged.

The most important factors controlling settlement (s) are: the effective bearing pressure (p), the breadth (B) of the loaded area and the compressibility of the ground within the depth of influence of the loaded area. There are many other factors influencing settlement such as depth of founding, geometry of the loaded area, depth of the water table, time, etc. These factors are secondary compared with the above three principal factors and could be examined separately after the main trends are established.
2. ANALYSIS OF SETTLEMENT ON SANDS

One of the most important factors controlling settlement is pressure-settlement relationship. Most of the current methods of settlement prediction on sands assume that the relationship between bearing pressure and settlement is linear over the working range of stresses.

A number of the case records presented by Ćorić (1991) contain complete pressure-settlement data and make possible a study covering a range of ground conditions, foundation dimensions and bearing pressures. Two examples will be given in this paper.

First example is a 16 storey reinforced concrete building in Belgrade, figure 1. It is founded on a raft (25 m x 25 m) over loose sand.

![Figure 1. A 16 storey reinforced concrete building founded on a raft (25 m x 25 m) over loose sand.](image)

Second example by Burland (1985) is nuclear reactor founded on a 3 m thick and 60 m dia. raft. The underlying ground consists of 60 m of dense sand and gravel. The net pressure against average settlement relationship for the reactor is shown in figure 2 and is, for all practical purposes, linear.

In conclusion an analysis presented by Ćorić (1991) indicates that the relationship between pressure \( p \) and settlement \( s \) may be taken as linear.

\[
\Delta s / \Delta p = K = \text{const.} 
\]

where \( K \) is "the foundation subgrade compressibility".

![Figure 2. Nuclear reactor founded on a 60 m dia. raft over dense sand.](image)

The second most important factor controlling settlement is the breadth (B) of the loaded area. The collection of a relatively large set of settlement data (224 cases) makes it possible to study statistically the influence of this factor together with the average SPT blow count \( (q_c) \) over the depth of influence of foundation.

The relationship between these factors was obtained as

\[
\bar{K} = \frac{B^{0.8}}{q_c^{0.8}} \times 1.50 
\]

where \( K \) is the average foundation subgrade compressibility.

The results of this statistical analysis are given in figure 3. The regression line for \( \log(K/B^{0.8}) \) on \( \log(q_c) \) is shown as a full line. It has slope of -0.88 and intercept on the \( q_c = 1 \) axis of \( K_e = 1.50 \). The coefficient correlation is 0.852.

This regression line agree well with the points for each compressibility grade. Mean upper and lower limit lines have also been drawn in as a chain dotted lines. It can be seen that most of the individual cases lie between these limit lines.

The regression line in figure 3 can be represented by the expression

\[
\log\left(\frac{q_c^{0.88}}{B^{0.84}} \times K \times 10^2\right) = 2.18 (\pm 0.28) 
\]

where the figure in brackets represents one standard error.
An important factor of the method for the settlement predicting is the depth of influence (z) of the foundation. There are many proposals how to take into account the depth of influence. Terzaghi (1967) recommend taking a depth equal to the breadth B. Schmertmann (1970) takes the depth of influence equal to 2B and uses a simple influence diagram to obtain the distribution of vertical strain. For a uniformly distributed circular load on an isotropic elastic half space the depth of influence is usually taken as 2B. The settlement at this depth is about 25% of the surface settlement. Hence, for practical purposes, the depth of influence may be assumed to be the depth at which the settlement is 25% of the surface settlement.

There are not many experimental data for assessing the depth of influence for foundation on sand and much of the data are from model tests. Many more measurements are needed of the distribution of settlement with depth beneath foundations on granular soils both from the point of view of establishing the depth of influence and, of more importance, for studying the in situ deformation properties. For the purpose of this study relationship in figure 4 was obtained as a guide to the depth of influence when \( q_e \) is constant or increases with depth. In a very few cases \( q_e \) decreased with depth and in these
instances the best fit to the general trends of the data was obtained by taking the depth of influence equal to 2B.

![Graph](image_url)

Figure 4. Relationship between breadth of loaded area B and depth of influence z (within which 75% of the settlement takes place)

3. CALCULATION OF SETTLEMENT

As the result of the method presented herein, the relationship has been established between the slope of the pressure-settlement relationship for the foundation, the breadth of the foundation (B) and the average SPT blow count (q_e) over the depth of influence of the foundation. The relation is shown in figure 5 where full line has been derived from a regression analysis of 224 settlement records on sand. The chain dotted lines approximate to two standard deviations above and below the mean line. Mathematically the regression line is given by

$$K_e = \frac{1.50}{q_e^{0.88}} \quad (3.1)$$

with the coefficient of correlation of 0.852.

According to the proposed method in this paper, the settlement of rigid shallow foundation on sand can be calculated by the formula

$$s = p \times K_e \times B^{0.8} \quad (3.2)$$

The value of $K_e$ may be obtained by relationship in figure 5 when the value of average SPT blow count $q_e$ over the depth of influence of the foundation $z$ is known. Depth of influence is given by relationship between breadth of loaded area B and depth of influence presented in figure 4. In numerical calculation, settlement is obtained in mm when the breadth of foundation B is in m, the average SPT blow count is in daN/cm², the compressibility index $K_e$ is in mm/(kN/m²) and the bearing pressure is in kN/m².

![Graph](image_url)

Figure 5. Relationship between compressibility index $K_e$ and mean SPT count blow $q_e$ over depth of influence. Chain dotted lines show upper and lower limits.

The probable limits of accuracy of equation (3.2) can be assessed from the upper and lower limits of $K_e$ given in figure 5 and it may be necessary to take these into account in the design.

It must be emphasized that the factor of the safety against bearing capacity failure should always be checked in addition to the settlement. If the factor of the safety is less than about 3 the pressure-settlement curve may be non-linear and the method will underestimate the settlement.

This method has been based on case studies with quartzitic sand deposits. Sites where coral (calcite) or other mineralogically unusual sand deposits are encountered should not be analyzed by this method unless the deformation properties of these deposits can be demonstrated to be similar to quartzitic deposits.

The method is well suited for routine design purposes. However, it is suggested that, for major projects, or those where proposed struc-
ture has strict permissible total of differential settlements, other well-established methods of estimating the settlement are also used as a check.

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Remote instrumentation for slopes during rainstorms in Rio de Janeiro
Auscultation à distance des pentes de Rio de Janeiro pendant de fortes pluies

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Abstract This paper describes a remote data acquisition and instrumentation system named Sigra (System for remote geotechnical instrumentation via radio) which has been experimentally used for monitoring slopes in Rio de Janeiro. Sigra allows remote, real time and continuous monitoring and data transmission to a central station. Development, instrument selection, installation and system performance are described.

Introduction
Rio de Janeiro lies between the sea and very steep mountains. Almost every year in the Summer it suffers from severe rainstorms that lead to slope failures. The most hazardous rainstorms occurred in 1966 and 1967 (Barata, 1968) and more recently in 1988, when failures were responsible for several casualties (Barros et al, 1992). Thirty years ago the Geo-Rio Foundation (formerly the Geotechnical Institute) was formed in Rio became the state authority in charge of slope stabilization works and prevention of slope failures.

Conventional techniques for slope monitoring have been used by Geo-Rio, employing piezometers, inclinometers and surface displacements gauges. They are not totally adequate because no information has ever been obtained during a rainstorm. Therefore, mechanisms during severe rainfall cannot be well studied. An advanced monitoring system was required.

Sigra was designed for continuous remote monitoring of slopes. A series of transducers is utilized for measurement of rainfall, pore pressures and water levels, soil suction, internal and surface displacements. Data are collected at the slope by a Remote Station (RS) and transmitted by radio to a central data acquisition station located at the head office of the Geo-Rio. Sigra can trigger alarms to warn authorities if pre-set threshold levels are reached by specific instruments.

Reliability in data acquisition, transmission and storage was sought during Sigra design. Data are transmitted as soon as collected. Storage only takes place safely at the central station.

Sigra has been experimentally installed in two slopes: at the Borel and the Formiga Hills in North Rio. Access these sites is difficult, and frequent visits have to be avoided. The work carried out to date was aimed at testing the system and checking its reliability. Therefore, the remote station was programmed to send continuous information on its performance, power level, temperature and other information for hardware and software checking.

Description
Sigra encompasses Remote Stations (RS) and one Central Station (CS) (Figure 1). The RS’s are located at the slopes. They have data acquisition boards that scan transducer signals, take readings, convert analogue to digital signals and send data to the transmitter-receiver units which forward data to the CS.

Each RS can accommodate up to 32 different transducers, although this number can be easily increased with minor modifications in the hardware.
The most common instruments are: rainfall meter, temperature gauge, piezometers and water level indicators, soil suction or tensiometers, flowmeters and soil displacements gauges.

The CS comprises an IBM-PC type microcomputer linked via RS-232 to a transmitter-receiver station. This microcomputer stores all data automatically and the data acquisition software allows reporting, alarms and communication with other systems.

Rainfall meter

The rainfall meter (Figure 2) comprises a 250 mm diameter funnel on the top of a 50 mm diameter 1.8 m high steel pipe. At the base of the pipe there is a water pressure sensor and a solenoid valve. The pressure sensor operates in the 0 to 10 kPa range and measures the water column in the steel pipe. As rainwater fills 80% of the pipe height, the RS automatically opens the solenoid valve and drains the water.

Piezometers and water level indicators

Piezometers and water level indicators are conventional standpipe instruments (Figure 1) with an electrical pressure transducer dipped in the access pipe. The transducers have a 0 to 300 kPa range and an output of 4 to 20 mA. It can be retrieved at any time for calibration and checking.

A mechanical cone packer, shown in Figure 1, is employed to plug the access pipe just above the piezometer sand bulb. This ensures a low flexibility of the system and short response time for pore water pressure measurements in fine grained soils.

Soil suction measurements have not yet been attempted by Sigra. However, electrical soils suction
Measurers as the one developed by Fleming et al. (1992) can be easily connected to Sigra.

**Surface displacements**

Two different surface displacement systems were developed. One is a simple mechanical system and the other is the advanced Sima. The mechanical system is a temporary device (Figure 4) installed at the slopes, until Sima can be deployed.

Surface displacements system are measured in the mechanical by an invar wire stretched between two points. One end is fixed to the ground, the other to a precision rotary potentiometer. The accuracy is close to 1 mm. Temperature effects may decrease the accuracy to 5 mm. The maximum distance between the fixed points is 20 m.

An advanced instrumentation for measurement surface displacements has been designed and tested in laboratory conditions but not yet in the field. It is due to be deployed in the Winter 1994. It is called Sima (System of Marks) and is shown in Figure 5. It has no moving parts, no temperature effects, no cables and no physical link between marks. Measurements are obtained automatically by radio. The operating principle is similar to the radar. Electronic surface marks are laid on the slope in a desired pattern. Three antennas connected to a readout system are located outside the zone affected by slope movements. Each surface mark in turn sounds

![Diagram of mechanical surface displacement gauge](image)

*Figure 4 Mechanical surface displacement gauge*

**SIMA**

![Diagram of Sima system](image)

*Figure 5 The Sima system for measuring surface movements on a slope*
a short ping (an electromagnetic wave) which is received by the antennas. The phase difference of the ping is compared with an initial reading. Therefore, the precise location of a surface mark is obtained and the difference between previous readings allows the calculation of horizontal displacements.

The electronics on board each surface mark is powered by NiCd batteries rechargeable by a small solar cell. The system was designed for an accuracy of 1 mm, although it can be increased. The accuracy does not depend on the distance between the marks and the antennas. The maximum design operating distance between marks and antennas is 1 km.

**Internal displacements**

Continuous monitoring of internal displacements in soil mass can be obtained with **Cliper (Permanent Inclinometer)** (Figure 6). It consists of a train of tiltmeters installed in a conventional inclinometer access tube. The tiltmeters are liquid level gauges used for more than 3 decades in the aeronautics industry, but only recently have found applications in geotechnical instrumentation (e.g., Campanella et al, 1994). The tilt gauges are tiny glass phials containing an electrolytic liquid and four opposite electrodes. As they tilt, the electrodes dip in or out in the liquid and give a signal proportional to the tilt. With appropriate electronics, the accuracy that can be achieved is comparable to a standard inclinometer device.

![CLIPER Permanent inclinometer](image)

**Figure 6 Cliper: The Permanent Inclinometer**

**Water flow**

Surface and internal drainage systems in slopes can be evaluated by Sigra by measurement of flow during rainstorms. Turbine type flowmeters can be installed at the outlet of sub-horizontal drains to measure water flowing out.

Water flowing on surface of a slope can be measured by a V type notch weir, as it is used in dams. Measurements can easily be automated by a water level transducer in the pond behind the weir.

These measurements however have not yet been made by Sigra.

**Characteristics of the Remote Station RS**

The RS employs a central processing unit CPU with a 80C31 microcontroller and a clock of 12 MHz. The analogue-digital converter has a 10 bit resolution and 16 kbytes EPROM. The data acquisition board has a total of 32 channels, being 28 for analogue 4-20 mA signals and 4 input/output digital ports.

Power is supplied by a no-break circuit from a set of rechargeable batteries. They are recharged automatically from solar panel energy. The CPU checks the integrity of the solar panel and the power level in the batteries. If any problem is detected a warning is given at the CS.

Before taking a reading, the CPU turns on the voltage to a single transducer, allows warm-up time, takes a reading and turns it off again. Then, it turns on the power to the transmission module and sends the data from one single channel. It repeats the procedure for all transducers in turn.

The CPU controls the water level inside the pipe of the rainfall meter. When it reaches a pre-set maximum level, it opens the solenoid valve to drain the water out.

Self-checking watch-dog routines are run periodically in the CPU. This is a procedure to detect and correct running time errors. If a fault is found, it stores all data in non-volatile memory, shuts down the system and re-initializes it again.

**Communication**

Radio and satellite communication were considered, but the first was selected for distances up to 30 km, because of its lower cost. Beyond that distance, radio wave propagation in Sigra's operating
frequency starts to deteriorate. In this case, a satellite communication system is preferred and Sigra was designed to have that option too.

Digital signals processed by the CPU are sent to the communication unit that sends and receives data. This unit modulates digital signals and transmits them via a radio circuitry. It also receives, demodulates and decodes signals sent by the CS.

The CS and RS’s communicate by radio waves in full-duplex mode at a frequency of 250 MHz and a rated power of 1 W. This is a private telephone radio channel. Therefore, at any time the CS can communicate with the RS’s or vice-versa. This also allows voice communication which is very useful during system-installation and testing.

Full-duplex mode enables hand-shaking of information, i.e., CS and RS’s exchange data in either ways. The data acquisition process has two modes: the automatic or the commanded. In the automatic mode, the CS orders data acquisition in selected intervals of 15 minutes, according to current system configuration. Alternatively, the operator can command at any time a measurement package from a RS.

The CPU of a RS can be programmed to give warnings, if a pre-set value risk level is reached. This is informed automatically to the CS which, in turn, warns the operator, send faxes to selected machines, communicates with another computer system, and can even warn a rescue brigade. This could be useful for traffic control during severe rainstorms, as it occurred in the mountains outside Rio, that roads have to be closed during heavy rains.

The control software encompasses the basic microprocessor software that runs independently at the RS’s and is responsible for the data acquisition tasks, already described.

The analysis program, filters and organizes a data bank and has graphics packages for presentation on the screen. It also exports data in ASCII format for further analysis high resolution plotting in spreadsheet programs.

**Protections**

The Remote Station is physically protected by a heavy sealed steel box. Protection against lightning is provided by efficient grounding.

Vandalism can be a problem for unattended remote instrumentation. The slopes where the RS’s are located are inhabited by poor people living in shanty towns. We found out, however, that the best protection against vandalism is to convince people that Sigra can result in their benefit. The neighbourhood association of each site was contacted and the research programme was explained in an understandable language. A flyer was produced and distributed. Therefore, the RS’s never suffered from vandalism to date.

In one occasion, however, a family of goats ate instrument cables at the Formiga Hills. Their preference for delicious cable wires was not appreciated. Since then, all cables were buried at least 80 cm deep backfilled in trenches.

**Software**

*Sigra* software that runs in the PC computer in the CS was written in Borland’s C++ language for DOS. It has the following modules: communication, presentation, control and analysis.

The *communication* software is a TSR (Terminate and Stay Resident) program. It runs in the high memory, independently of the operator. Therefore, the microcomputer can be used for any other DOS or Windows application while it receives data from the RS’s, stores in a memory buffer and eventually stores data in hard-disk.

The *presentation* software manages screens, graphics, reports and menus that help the operator to use the system.

The *control* software encompasses the basic microprocessor software that runs independently at the RS’s and is responsible for the data acquisition tasks, already described.

The *analysis* program, filters and organizes a data bank and has graphics packages for presentation on the screen. It also exports data in ASCII format for further analysis high resolution plotting in spreadsheet programs.

**Protections**

The Remote Station is physically protected by a heavy sealed steel box. Protection against lightning is provided by efficient grounding.

Vandalism can be a problem for unattended remote instrumentation. The slopes where the RS’s are located are inhabited by poor people living in shanty towns. We found out, however, that the best protection against vandalism is to convince people that *Sigra* can result in their benefit. The neighbourhood association of each site was contacted and the research programme was explained in an understandable language. A flyer was produced and distributed. Therefore, the RS’s never suffered from vandalism to date.

In one occasion, however, a family of goats ate instrument cables at the Formiga Hills. Their preference for delicious cable wires was not appreciated. Since then, all cables were buried at least 80 cm deep backfilled in trenches.

**Sigra performance**

*Sigra* started to operate in April 1992 and its performance is continuously monitored in respect to communication, operating temperature inside the RS box, grounding and electric isolation, power consumption, mechanical protection and software.

Temperature inside the box reached a level well beyond the maximum expected. It affected hardware and several electronic components were redesigned or replaced. Today the Sigra is safe against temperature up to 70°C.

A few months after the first unit became operational seven faults were detected. Two due to power failures, other two due to insufficient grounding during a thunderstorm, one due to a cable failure, others from unknown reasons.
Power failures were corrected by the installation of a no-break system. The grounding was substantially improved.

**Measurements at the Borel Hill**

The instrumented section at the Borel Hill is shown in Figure 7. Two piezometers of the type shown in Figure 1 were located close to the contact with the gneissic bedrock, since this is the most likely water seepage path. Surface displacements devices are not shown.

Examples of measurements taken during rain are shown in Figure 8. The measured accumulated rainfall is around 80 mm during a couple of days. The resulting porepressure rise in piezometers P1 and P2 is shown in the figure.

![Figure 7 Instrumented section at the Borel Hill](image)

**Figure 8 Porepressure rise during rain**

A local (within a slope) wireless communication system was developed for *Sima* and it can also be used to eliminate cables from instruments to the RS.

Long range satellite communication has also been developed and tested, although the price is slightly higher than radio communication.

**Final remarks**

*Sigr* has proved to be a reliable system and has been operating without any fault for more than a year. Maintenance is reduced to an yearly inspection.

Short range radio communication is a low cost option limited to 30 km distance between the CS and RS’s. For longer distances, it can be easily replaced by satellite communication. Local communication (within one slope) can be wireless, via FM frequency.

In-house hardware and software development allows production of a low cost system. *Sigr* software can be easily modified and be tailored to specific needs.

**Current stage of development**

*Sigr* was experimentally installed at the Borel Hill by the end of 1992 and at the Formiga Hills in 1993. The units had only a very limited amount of soil instrumentation, since the primary purpose at the current stage of our research is to evaluate performance and to correct problems.

*Sima* is in the final stages of development and is due to be deployed in the field by the Winter of 1994.

**Acknowledgments**

The development of *Sigr* was sponsored by the Geo-Rio Foundation. Support from Aldo Rosa, Willimir Barros, Efrain Akherman, Renato Cunha and Luis Odavio Vieira is appreciated. Collaboration from Sylvia Silva and Adalino Goncalves from Institutek Ltd is acknowledged. Lucia Alves and Javier Far helped in the preparation of the paper.
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New results from pluriannual observations of the displacements in Maratea Valley (Southern Italy)

Nouveaux résultats des observations pluriannuelles des déformations de pente dans la vallée de Maratea (Italie méridional)

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ABSTRACT: In the Maratea valley (Basilicata, Italy) have been singled out complex landslides phenomena, that starting from a deep gravitational deformation of sackung type in the carbonatic formations evolve to creep, and sliding movements of colluvial masses and clayey deposits filling the valley. The instrumental data collected in this decade indicate that the rate of movement are of the order of some centimetres per year for the landslide units over which the historical centre lies, as well as for the southern part of the valley, crossed by the Salerno-Reggio Calabria railway. Periodical interventions on it single out movements of the order of 2.5 cm/yr that are seriously deforming also the railway tunnel in a big "floating" calcareous block. The data collected up to day give a quasi-linear displacement not influenced by rainfall conditions.

RÉSUMÉ: Les observations instrumentales pendant la dernière décade indiquent des vélocités de l’ordre de quelques centimètres par année, soit dans les éboulis où se trouve le centre historique, soit dans la partie au Sud de la vallée traversée par le chemin de fer Salerno-Reggio Calabria. Dans la galerie du chemin de fer fondu sur un bloc de calcaire flottant on a référé des mouvements de l’ordre de 2,5 cm par année que la déforment sérieusement. Les données obtenues permettent tracer la loi de mouvement et montrent une accélération qui n’est pas influencée par les conditions climatiques.

INTRODUCTION

The study area is characterized by the outcrops of carbonate rocks, forming a part of the Monte Bulgheria-Verbicaro and Alburno - Cervati Units deriving from the Campania-Lucania Platform, and by clayey-marly flysch deposits belonging to the Liguride Unit. Recent tectonics have deeply altered the normal tectonic superposition relationships between these units. In fact in the late overthrust area there is a partial décollement of the Bulgheria-Verbicaro Unit from the substratum formed by the Alburno-Cervati Unit and the partial superposition of the former on the Liguride Unit. Ancient erosional episodes, related also to the last Holocene marine regression, have produced large deep gravitational deformations and failures in the rocky masses favoured by the above-mentioned structural setting (Fig.1). Among the large deep gravitational

deformations, some very interesting of sackung type occur in the Maratea Mountains, at the margin of a thick carbonatic sequence close to a tectonic line (Guerricchio and Melidoro, 1979; Guerricchio and Melidoro, 1981; Guerricchio 1982). Starting from this one, deep complex movements occur also in the valley filled by widespread alluvial fans covering a flysch formation with an high clayey content (Fig.2). Studies on geomorphological dynamic and historical characters allowed the reconstruction of an alternation of tectonic and gravitational mass movements phases, which still continues, and the hypothesis of a preliminary evolutionary model (Guerricchio et al., 1987 a).
Beginning from the 1983 year, in order to obtain quantitative data, systematic (annual) instrumental observations of slope deformations have been carried out. Previous
works (Guerricchio et al., 1986b; Guerricchio et al., 1988; Guerricchio et al., 1990) have pointed out the existence and the amount of large, active displacements in the lower part of the valley. New and abounding data, collected from already decennial observations, confirm those displacements. They are involving also some important infrastructures like the railway line and the railway tunnel that cross the lower part of the Maratea valley, and give us more precise indications about their development in the time and their superficial extent.

2. NEW DATA ON THE SURFACE DISPLACEMENTS

In the sackung zone, in the whole Maratea valley and in the area of the old part of the town a network of points for distance measuring was established (Guerricchio et al., 1986 b, 1987 b, 1988, 1990), (Fig 1 and 2). In addition, in the old part of the town several extensometers on the fissures of the buildings have been set (Guerricchio et al., 1988).

Electronic long-base extensometers in invar wire have provided continuous monitoring of the movements of large sliding blocks (Grabowski, 1988), at the base of the sackung and located immediately uphill from the old part of the town. For this area the movements are very small in accordance with the values monitored by the infrared distance measurements in the ten-years interval (1983-1993), (Tab.1).

The existence of movements in the old part of the town of Maratea was further confirmed by recent deformations verified in 28 fissure-meters placed on buildings showing largely signs of a progressive degradation. A deformation of 3 cm/yr recorded by periodical distance measurements has been detected along a wide area in the old part of the town and along the intensive inhabited lower southern side of the valley.

The new data of the repers' displacements collected in this last decade (Tab. 1) confirm that the landslide movements occur prevalingly in the central portion of the Maratea valley (Fig.3), where the thickness of the
continental deposits (alluvial fans and eluvio-colluvial covers) resting on the Crete Nere Formation (Black Flysch) is not too much thick (D=20-40 metres), or practically absent.

The main active movements start from the toe of the sackung area and reach the harbour, involving the old part of the town and the southern portion of the middle-lower valley, crossed by the Salerno-Reggio Calabria railway and its tunnels. In particular in the area three railway tunnels exist, the first of which (Profetti tunnel), built at the beginning of this century, was abandoned in the last 30 years in consequence of the irrecoverable conditions (Fig.4). It was successively replaced with a new tunnel near its side. The old tunnel shows, in fact, widespread, serious damages but, above all, a rough transversal displacement of the railway axis of about 0.5 metre near the northern entrance. The new tunnel too, though relatively recent, has been variously restored and, in spite of the numerous works, it shows to-day widespread microfissures.

In general the tunnel entrances, at the contact of the rigid "floating" calcareous blocks, or rock masses in place, with the landsliding colluvial and clayey bodies present an high degree of damages.
Table 1. Displacements of the moving repers in the last ten years (values obtained as a mean of six measurements for each reper).

1* Total amount of the displacements of the various repers after ten years; 2* Mean value of the yearly displacements; 3* Instrumental accuracy for the various measured repers. A,B,C are the most active areas in the valley, that beginning from the old part of Maratea reach the harbour, moving with similar rates as parts of a continuous body. D and E, respectively the "sackung" area and a big "floating" block, both bordering the previous most active body, are almost not active areas.

For what concerns the considered gauge points inside all the valley (Fig.3), they show an yearly constant activity in the time, rather high and surprising in connection with the small openings of the building fractures in the area but clearly identified, and ranging from about 0.2 to 4.4 cm/yr, with an average of 2.84 cm/yr, as reported in the column M of the Tab.1.

It represents the value to assume as the average-sliding of the instable area in the last decade.

In substance the collected data show that, from year to year, the

Fig. 4. Serious systems of microfissures in the concrete of the old railway tunnel (Profetti tunnel). The numbers indicate the conventional Italian Railways distance-points.

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...and objective measurements, in some way coincide with the informations got by the periodic restorations needful to align again the railways (Fig.8 and 9). On the base of this kind of data supplied by the Italian Railway Administration, it has been possible to obtain gross valuations that, anyway, be compared, in amount, with those obtained by the instrumental observations; besides, pluriannual accelerations of the movements seem to exist, though of very small values, as well as an exact indication that the more active portion of the whole landsliding valley is the lower southern side.

Fig. 6 Yearly displacements (M), as a mean of the values detected in the decade (column 2* in tab.1), of all the single gauge points, in comparison with the relative instrumental accuracy indicated by the white symbol.

Accelerations have been also detected by the infrared distancemeter system (Fig.7).

Fig. 7 Mean yearly displacements of all the active areas.
At present the more valid explanation for this biennial recurrence of the single out accelerations, lacking any evident relationship with the rain data, could be perhaps attributed to a periodical losses of cumulated stresses, in someway helped by the underground waters, producing also vibrations recorded in Maratea (Guerricchio et al.1986 b).

We can conclude that in this last decade in the Maratea valley the sackung area is quite firm and that all the monitored repers have shown very small or quite absent displacements. Only the central middle-lower portion of the valley shows continuous, remarkable displacements, progressive in the time, which are causing severe damages in the buildings. Finally it seems that there aren't correlations between rainfalls and displacements.

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Blast vibration control: Effects on structures/regulations

Contrôle de vibrations: Effets sur les structures/réglementations

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ABSTRACT: This paper presents comparatively some national standards and codes of practice as well as the related guideline values for safety evaluation. This knowledge allows an enlarged basis to promote monitoring and control practice by describing the procedures and criteria to consider, in a complementary way with the Portuguese standard (NP-2074).

RÉSUMÉ: Cet article present, comparativement, la réglementation et les méthodes recommandés dans quelques pays et exprime une synthèse des niveaux de sécurités adaptés. On pretende une contribution pour faciliter le contrôle des vibrations en décrivant quelques critères et procédés pour compléter la Norme Portugaise (NP-2074).

1. INTRODUCTION

The vibrations caused by blasting or other mechanical processes travel through the ground, thus affecting buildings or other structures near construction and mining works. Besides the possible material damage, there is also the problem of environmental impact which is a very sensitive one for people. More and more, these operations are done in city or suburban areas due to convenience or even necessity to utilize underground space.

Blasting is often unavoidable or economically the most advantageous method to excavate rock masses, but its direct effects and the lack of knowledge in that concerns control procedures alarm the authorities, the builders, and the local population, often giving rise to unfounded conflicts or even blocking solutions.

For the reasons pointed out, the existence of regulations and the adaption and diffusion of adequate methodologies to analyse these problems is of great usefulness.

Procedures, criteria, and safety levels adopted in various countries differ due to experience, studies performed, and local statistics. The damage and risk that are socially acceptable also vary. A comparative and current analysis of this matter is, therefore, clearly advisable.

2. VIBRATION CHARACTERIZATION

Vibration characterization is achieved through knowledge of the particle movement as a function of time, its frequency, and its duration.

If one considers that a point (P) moves in relation to its resting point (P₀) the following parameters can be defined:

\[ P₀ \rightarrow P = \overline{U(t)} \]  desloaction of point P  \[ 1 \]

\[ dU(t) = \overline{V(t)} \]  particular velocity of P  \[ 2 \]

\[ \frac{d^2 U(t)}{dt^2} = \overline{\Gamma(t)} \]  particular acceleration of P  \[ 3 \]

Knowing either one of these three vectors, the movement of point P can be defined.

The type of transitory vibration caused by blasting can be dealt with, in analytical terms, as a harmonic vibration, even though this procedure implies errors in some cases. The formula is: \[ \overline{V(t)} = \overline{V₀} \sin \omega t \] \[ 4 \]

\( V₀ \) is the amplitude of the velocity of motion and \( \omega \) is the angular velocity (rad.s\(^{-1}\)). The deslocation and the speed is obtained the formulas:

\[ \overline{U(t)} = \overline{V₀} / \omega \cos \omega t \] \[ 5 \]

\[ \overline{\Gamma(t)} = \overline{V₀} \cos \omega t \] \[ 6 \]

The velocity amplitudes, deslocation, and acceleration are the modulus of the respective vectors, and their relation to each other is used in the manual treatment of "vibrograms":

\[ \overline{V} = \overline{V₀} \] \[ 7 \]

\[ \overline{U} = \overline{V₀} / \omega \] \[ 8 \]

\[ \overline{\Gamma} = \overline{V₀} \cdot \omega \] \[ 9 \]

The particles Velocity of Vibration is considered the most representative way to assess the effect of blasting on constructions. In most norms, the values of this parameter are pointed out as a criteria for harm. It can, however, be considered in two ways: the particle velocity (in one or in any specific direction) or the velocity modulus. This last is the value of the vector resulting from the composition of the
particular vectors in three orthogonal directions [10] — which is more conservative and representative of the range of the true motion of particles.

$$V(t) = \sqrt{V_x^2 + V_y^2 + V_z^2}$$ \[10\]

The French rules and the German norm are, in this respect, the only ones to present limiting or indicative values, referring to a particular velocity, namely the vertical component. Other regulations studied use the maximum resulting velocity of vibration as their parameter for evaluation, considering various maximum values throughout time for an evaluation based on respective determinant frequencies. Such is the case of the Swiss norm.

3 MEASURING TECHNIQUE

Seismic waves or its effects over constructions differ from point to point. Consequently, in the beginning of a campaign for permanent control, it is advisable to do a group of simultaneous measurements in several locations on the construction to select the critical points for subsequent measurements.

The placement of measuring devices should be done on rigid elements of the structure, where the most important effects are expected. However, this process is not always feasible or necessary. Most of the regulations indicate, for the most current situations, a point solidarily linked with the foundations on the side where the seismic waves are produced, at a maximum height of 0.5 meters from the ground. When the expected vibration has a frequency higher than 60 Hz, places a top the foundations should be chosen.

The transducers or geophones should be firmly affixed to the construction elements, or simply set down over those elements when the acceleration is lower than 3 m/s². As an orientation, to this value correspond the following velocities of sinusoidal oscillation: 25 mm/s⁻¹ at 20 Hz, 13 mm/s⁻¹ at 40 Hz and 8 mm/s⁻¹ at 60 Hz.

In each group of register, consisting of two horizontal and one vertical transducers, the direction of one horizontal transducer should coincide with the direction defined by the measuring point of register and the origin of vibration (longitudinal direction). The other two should be placed transversally (transversal and vertical directions). The German norm mentions the placement of the longitudinal transducer as parallel to the closest walls or to the largest dimension of the structural element to be monitored but this applies to more specific studies of structures.

4 REGULATIONS EVALUATION CRITERIA

4.1 Switzerland: SN 640 312 Norm

This norm applies namely to vibrations caused by blasting or generated by machinery or motors at work, and also due to highway or railway circulation. The norm does not apply to the effects of these ground vibrations upon human beings, equipment, and particularly sensitive installations or goods. Also excluded, which is more significant, are the effects upon the terrain itself, like settlements or liquefaction, or those caused by vibrations with frequency lower than 8 Hz associated with earthquakes. The impossibility of evaluation or prediction of these types of effects using vibration monitoring is referred to in the norm.

The norm establishes: a) criteria to evaluate the effect of the produced ground vibrations on constructions; b) values or limit levels at which damage is considered improbable; c) recommendations for the execution and interpretation of measurements; d) recommendations for recognition and evaluation of cracks.

The established appreciation criteria considers the maximum value of the resultant or velocity vector of particles vibration $V_r$ (mm/s⁻¹) in function of the frequency of oscillations (Hz), the number of events, and the class of the structure sensitivity.

The vibration frequency, which is considered by means of three intervals, is that correspondent to the maximum value of the resultant of velocity $V_r$ with a precision of 10%, and is named Determinant Frequency.

Each maximum value of $V_r$ higher than 70% of the guideline value, is called an event and the norm establishes three classes, based upon the number of events ($N_s$):

- Occasional $N_s < 1000$
- Frequent $1000 < N_s < 100000$
- Permanent $N_s \gg 100000$

The norm also defines 4 classes of sensitivity in constructions that are determined by the type of construction or structure, by materials, and by the state of conservation of the buildings (Table 1).

The criteria is shown in Table 2 where the guideline values of $V_r$ are established in function of the referred parameters.

There is little probability that any damage occurs with values that do not pass the indicated ones, and there is still little risk with values occasionally higher up to 30%. A significant probability is only admitted for values equal or higher than the double of the indicated values. However, passing cracks are only expected for velocities of a higher magnitude.
<table>
<thead>
<tr>
<th>Sensitivity class</th>
<th>Buildings</th>
<th>Others</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Class 1) Very slightly sensitive</td>
<td></td>
<td>Bridges made in reinforced concrete or steel. Support structures made in concrete, reinforced concrete or stone masonry. Galeries, tunnels, caves, pits in rock or hard soil. Crane and machinery foundations. Not buried pipes</td>
</tr>
<tr>
<td>(Class 2) Somewhat sensitive</td>
<td>Industrial buildings in reinforced concrete or metallic structures, silos, towers, factory chimneys, solid structures. (Condition: constructions built according to the rules of good practice).</td>
<td>Caverns, tunnels, pits, buried pipes. Underground garages, gas and water pipes, sewerage, buried cables. Loose stone walls.</td>
</tr>
<tr>
<td>(Class 3) Normally sensitive</td>
<td>Housing in store masonry made of concrete, reinforced concrete or brick. Administrative buildings, schools, hospitals, churches made of natural stone or brick masonry. (Condition: constructions built according to the rules of good practice).</td>
<td>Water impoundings, captations, reservoirs, wrought iron pipes, caverns, structures inside tunnels, sensitive cables.</td>
</tr>
<tr>
<td>(Class 4) Very sensitive</td>
<td>Houses with ceilings of plaster, stucco buildings, new or recently remodeled buildings in class 3, protected historical monuments.</td>
<td>Old lead cables. Old wrought iron pipes.</td>
</tr>
</tbody>
</table>

Table 1 — Classes of sensitivity (adapted from Swiss Norm SN 640 312, April 1992)

<table>
<thead>
<tr>
<th>Sensitivity classes</th>
<th>Frequency of events</th>
<th>Maximum Values of Vector $V_r (\text{mm.s}^{-1})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Very slightly sensitive</td>
<td></td>
<td>Triple of indicated values for sensitivity class 3</td>
</tr>
<tr>
<td>2. Somewhat sensitive</td>
<td></td>
<td>Double of indicated values for sensitivity class 3</td>
</tr>
<tr>
<td>3. Normally sensitive</td>
<td>Occasional, Frequent, Permanent</td>
<td>$&lt; 30 \text{Hz}$ 15, 6; $30 - 60 \text{Hz}$ 20, 8; $&gt; 60 \text{Hz}$ 30, 12; Between the values indicated for class 3 and half of the same.</td>
</tr>
<tr>
<td>4. Very sensitive</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2 — Guideline values of $V_r$ in function of Sensity Class and the frequency of events (adapted from Swiss Norm SN 640 312, April 1992)

4.2 France: A.F.T.E.S. Rules

The recommendations of the "Association Française des Travaux en Souterrain" aim the preliminary study of the blasting diagrams and the monitoring of vibration effects on constructions. The term "construction" is used in a wide sense, as housing, bridges, buildings, and diverse equipment. However, when these constructions have special characteristics such as great height, great capacity or abnormal construction process, the simple compliance of the recommendations, is not intended. Instead, a specific study becomes necessary.

Regarding limit values or the establishment of an evaluation criteria, the French regulation is quite indefinite — alleging the need for such criteria to have a general nature because, on one hand, there's a lack of consensus and guarantees related to the existing information, and on the other hand, there exist a variety of situations and factors of quantification which are complex or, even difficult to detect. The intention is that the levels in Table 3 secure a simplified control, with a high safety factor, both applicable to a wide range of situations.
10 mm.s\(^{-1}\) — Under this level, the probability of slight damage is considered insignificant. Work proceeds without problems.

30 mm.s\(^{-1}\) — Above this value, the probability of some damage and the appearance of complaints exists. In the 10 - 30 mm.s\(^{-1}\) interval, the operation should go ahead, normally monitored by vibration control. Limits can be established depending on the state of the construction, the risks involved, and the general context of the operation.

50 - 70 mm.s\(^{-1}\) — Highest levels for unoccupied areas only. They represent technical limits likely to be applied only to special structures or exact objectives. These cases always undergo a specified study that will define the lower levels.

Table 3 — Safety limit values for vibration velocity (AFTES, 1981)

### 4.3 Germany: DIN 4150 Norm

This norm allows the evaluation of vibration effects in structures in function of the existing static charges. The respect for the advisable limit values (Table 4) guarantees the absence of damage in terms of reducing the use of the constructions. By reduction of use is also meant, in structures 2 and 3, the appearance and opening up of cracks in plaster, even though classified as minor damage.

In controlling vibration (Table 4) in floors or "platforms" measuring should be done at the points of greatest deslocation, generally in the middle. Damage due to vibration velocity in a vertical direction with values less than 20 mm.s\(^{-1}\) is considered improbable.

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>Vibration velocity, (V_i) (mm.s(^{-1}))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Foundation at a frequency of</td>
</tr>
<tr>
<td></td>
<td>10 Hz</td>
</tr>
<tr>
<td>1 — Buildings used for commercial purposes, industrial buildings and buildings of similar design</td>
<td>20</td>
</tr>
<tr>
<td>2 — Dwellings and buildings of similar design and/or use</td>
<td>5</td>
</tr>
<tr>
<td>3 — Structures with particular sensitivity to vibration and with great intrinsic value (e.g. buildings under a preservation order)</td>
<td>3</td>
</tr>
</tbody>
</table>

*For frequencies above 100 Hz, at least the values specified in this column shall be applied.

Table 4 — Guideline values of vibration velocity, \(V_i\), to evaluate the effects of short-term vibration (adapted from DIN 4150-P3 Norm, 1986).

\(V_i\) = maximum values of the three components of the vibration velocity at the foundation.

These guide numbers only refer to temporary vibration situations, like those derived from blasting.

In case of continuous vibrations, such as demolition hammers and other machinery, the limit number of 5 mm.s\(^{-1}\), measured at the level of the last floor, is considered safe for structures 1 and 2. If quite higher values are expected, specific studies of the structure components by means of calculation or measuring of resulting tension will be necessary. These suggested methods are not of an empirical nature anymore and their application is often complicated and approximate anyway.

The current German norm shows some changes in relation to its 1975 edition, having benefitted from experience in its application. The maximum number of the Resulting Velocity is no longer used. Instead the maximum velocity value in either of the directions X, Y, or Z, is used as a criterion for evaluation.

A complementary measuring at the level of the top floors is advisable, and limit safety values now, take into consideration the frequency factor which is, justifiably, a determinant parameter.

### 4.4 Ex-Soviet Union

In the ex-Soviet Union, Mosinets V. N. developed a method for determining the safety charges which uses a complex diagram that considers as a first entry a value called "Permissible velocity" of particles vibration for site and structure under study. This parameter, considered here also as a safety criterion, shows limit values that are tabulated according to two structure classification (Table 5). One classification considers the kind of utilization, worth and social importance and includes a class (class 1) where vibrations are not allowed. The other (Table 6)
weighs the details of construction, safety, and state of conservation. For the purpose of comparison, it can be said that the limit values for class III (small buildings for industry services, or housing) vary, in function of the construction characteristics, from 5 to 70 mm.s⁻¹. There is also considered an ultimate velocity limit, which value is 50 to 80% lower, applicable to isolated blasts with immediate proximity.

The total range of values is quite wide, and the second classification which relates to construction characteristics is also very comprehensive and detailed. It is consequently easy to assess safety levels based upon detailed site inspection.

<table>
<thead>
<tr>
<th>Buildings and structures classes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class I — Very important buildings and structures (All-Union and for the Republics), historical and architectural memorials, in whose vicinity conduction of blasting operations is permitted only exceptionally (Gosstroiy of the USSR or of the Republics have the authority to categorise buildings in this class).</td>
</tr>
<tr>
<td>Class II — Very important huge industrial structures (pipelines, plant buildings, mining headframes, water tanks of 20-30 years life, residential/office buildings in which a number of people live/work, apartments, cinemas, theatres, houses of culture etc.).</td>
</tr>
<tr>
<td>Class III — Industrial and service buildings of smaller dimensions (having no more than three storeys): workshops, compressor houses, civilian buildings in which fewer people live and work, apartments, shops service centres etc.</td>
</tr>
<tr>
<td>Class IV — Civilian and industrial buildings housing expensive and valuable machinery and instruments, damage to which could harm the life and health of people, godowns, service centres for transport, cold storages, compressor installations etc.</td>
</tr>
</tbody>
</table>

Table 5 — Classes of buildings and structures (in Kuzmenko, A. A., 1993)

<table>
<thead>
<tr>
<th>Characteristics of buildings and structures</th>
<th>Permissible velocity of ground vibration, mm.s⁻¹, as per classes of the structures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings and structures intended for civilian or industrial purposes, with reinforced concrete or metallic carcase and having antiseismic reinforcement. There are no residual deformations in the carrying elements, in the contructions and in the filler material.</td>
<td>50 II</td>
</tr>
<tr>
<td>Buildings and structures with the reinforced or metallic carcase without antiseismic safeguards. No residual deformations.</td>
<td>20 II</td>
</tr>
<tr>
<td>Buildings with carcase, filler-bricks or stone construction, cracks in the filling material. New or old large-blocked or brick building without antiseismic safeguards.</td>
<td>15 II</td>
</tr>
<tr>
<td>Carcase type buildings, having considerable damages in the filling material and cracks in the carcase. New or old large-blocked or brick building, having small individual cracks in the carrying walls and barricades. New or old building of carcase type having cracks in the carcase, damaged links between individual elements.</td>
<td>10 II</td>
</tr>
<tr>
<td>Large-block or brick building with carrying walls, having considerable damages in the shape of oblique cracks, cracks in the corners etc. Buildings and structures with damage in the reinforced concrete carcase, corrosion in the armature carcase, large cracks in the filling material.</td>
<td>5 II</td>
</tr>
<tr>
<td>Buildings with load-carrying walls, having a large number of cracks, broken links between the external and internal walls etc. Large-panelled buildings without antiseismic safeguards.</td>
<td>3 II</td>
</tr>
</tbody>
</table>

Table 6 — Permissible velocity of vibration as per classes of the structures (in Kuzmenko, A. A., 1993)

4.5 United States

American regulation exist at three levels — Federal, State, and local. In case of incompatibility between norms when applied to a particular geographic situation, the one which guarantees the highest safety level is used. Figure 1 shows a simplified diagram of safety criterium advised by the Office of Surface Mining and Reclamation (OSMRE), the responsible federal agency, and the United States Bureau of Mines (USBM RI 8507), which has done much research on the problem.
4.6 Sweden (Persson et al., 1980)

The authors allow for an evaluation in function of velocity, deslocation, and acceleration of particles by use of a three entry diagram which suggest the limit values to be considered. Limit C is probably too conservative in relation to current computer equipment. Generally, the values are high probably due to the good condition of both the rock masses and the quality of the constructions studied which led to the preparation of that diagram (Fig. 2).

4.7 Portugal: NP 2074 Norm

The aim of this norm is to establish limiting criteria of the characteristic parameter of vibration caused by blasting, pile driving and similar operations in order to avoid construction damage.

The parameter used to evaluate vibration level is the Maximum Resulting Velocity of particles vibration $V_r$ [10], admitting also the use of the Resultant of the maximums of the particles velocities in the three orthogonal directions as a form of quick calculation—quite useful when it is done manually and by direct measuring in the vibrograms. Note that the values obtained by this way have no physical meaning but are more conservative.

The limit value, established as a criterium for construction safety, $V_f$ - Limit value of the Maximum Resulting Velocity of particle vibration, is calculated in function of the characteristics of the foundation's ground ($\alpha$), type of construction ($\beta$) and the medium daily number of events ($\gamma$) (Table 7).

$$V_L (\text{m/s}) = \alpha \cdot \beta \cdot \gamma \cdot 10$$

[11] The values of $V_f$ measured in a given construction should, therefore, remain below the limit value calculated for a specific construction.

The best place for the measuring points is in structure elements solidly linked with the foundation at a maximum height of 0.5 meters atop the ground, and on the closest side to the source of the waves. A minimum frequency range of the equipment is from 3 to 60 Hz.
<table>
<thead>
<tr>
<th>Ground characteristics (α)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rocks and cohesive hard soils</td>
<td>2</td>
</tr>
<tr>
<td>Very hard uniform soils of medium consistency; coesionless compact soils; sand and fine mixtures of sand and gravel, uniform sand (1000 m.s⁻¹ &lt; 2000 m.s⁻¹)</td>
<td>1</td>
</tr>
<tr>
<td>Loose coesioless soils; sand, and fine mixtures of sand and gravel uniform sand, soft and very soft cohesive soils (&lt;1000 m.s⁻¹)</td>
<td>0,5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type of construction (β)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction requiring special care: historical monuments, hospitals, water deposits, chimneys, etc</td>
<td>0,5</td>
</tr>
<tr>
<td>Current (or new) constructions</td>
<td>1</td>
</tr>
<tr>
<td>Reinforced constructions</td>
<td>3</td>
</tr>
</tbody>
</table>

| Medium daily number of events (γ)                              |       |
| < 3                                                             | 1     |
| > 3                                                             | 0,7   |

Table 7 — Limit values for the coefficients α, β, and γ (Portuguese Norm NP-2074, 1983)

*velocity of longitudinal elastic waves

5. EQUIPMENT

Vibration is usually measured by geophones (transducers of particular velocity, Fig. 3) that furnish an electric signal created by a magnetic field proportional to the vibration velocity.

![Geophone Diagram](https://via.placeholder.com/150)

Fig. 3 — Schematic constitution of a geophone

The equipment should keep the register in memory or print the values of the vector velocity and its three components as a function of time, and should also allow the evaluation of the determining frequency. Fig. 4 illustrates the composition of a simple apparatus for measuring and logging the vibrations. The frequency interval should be between (5 to 150 Hz) (Swiss norm, 1992). Vibration can also be evaluated by means of accelerometers or transducers of displacement (SDF). These values can be converted into velocity by means of integration and derivation (numerical or analogical) respectively.

6. CRACK INSPECTIONS/CLASSIFICATION

In the making of particular study of vibration effects on constructions, it's best to begin with an inventory of existing "cracks", and then accompany their progress throughout the period of vibration, generating phenomena.

The appearance of "thin cracks" is a natural occurrence in building, independent of vibration. It's associated with factors like retraction and chemical transformation of materials, especially in the post construction period, as temperature variations, physical and biological aggression, abrasiveness, strains due to the imposed stresses, and settlement.
of the ground during the life of the construction. Thin cracks may begin during the construction itself, increasing with time especially in the first years and have a tendency to diminish afterwards. It becomes necessary therefore, to have a standard or a normalized system to classify cracks in construction, when carrying out inspections. It is clear that in some cases, or in predictable litigation, all inspection reports should include visual documentation (photographs).

Let us consider only cracks visible to the naked eye — "macro cracks" — defined in the Swiss norm as visible one meter away and that have a minimal opening of 0.05 mm. This norm proposes a classification (Table 8) that distinguishes cracks between superficially only with esthetic implications, and "gaping" which might affect durability or stability of the structures and may endanger its use. There is no reference, however, to the length or shape of the cracks.

<table>
<thead>
<tr>
<th>Crack Classification</th>
<th>Opening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine</td>
<td>≤ 0.2 mm</td>
</tr>
<tr>
<td>Medium</td>
<td>0.2 to 1 mm</td>
</tr>
<tr>
<td>Wide</td>
<td>1 to 3 mm</td>
</tr>
<tr>
<td>&quot;Gaping&quot;</td>
<td>≥ 3 mm</td>
</tr>
</tbody>
</table>

Table 8 — Crack classification in buildings (Swiss norm SN 640 312, 1992)

It is also very advisable to pay attention to the order of the size of the opening of the cracks permissible from the standpoint of esthetic beauty and durability. The Swiss norm indicated values as follows:

Strict esthetic requirements 0,1 mm
Exterior elements subject to environmental factors 0,2 mm
Exterior elements not subject to the environmental factors 0,3 mm
Interior elements 0,4 mm

The occurrence of cracking due to vibrations whose levels do not significantly overstep the ideal is probable in places where tension concentration due to already mentioned factors reach high values in such a way that even slight dynamic tension associated with vibrations are enough to open or increase thin cracks.

Therefore, this occurrence cannot be understood as a real effect of vibrations, but as an anticipation of cracking that might take place later. It must be referred to that there is no possibility to assign a particular origin to the traction cracks. There are no "typical" cracks resulting from seismic actions.

Inspections that include the number, localization, and dimension of cracks before and after vibration activity should be encouraged. It becomes justifiable, however, to waive these intensive and costly surveys (namely in concentrated urban areas) if the vibrations derived from the works are continuously monitored at the indicated points and vibration levels are kept clearly under the minimum advisable values.

7. PREDICTION

Even though the study and evaluation of each situation is left up to the specialist's criterium, procedures to predict and lower the vibration levels caused by bursts are referred to in the French regulation. Namely, two conditioning factors are weighed.

7.1 Ground influence between the blasting site and the construction

Obviously the type of ground has effects on vibration propagation, acting like a "pass-low" filter. In other words, high frequencies weaken more quickly than low frequencies. The dominant frequencies also depend on the nature of the soil, since high frequencies are better transmitted by compact rock than by soil.

For each charge, the relation of weakening is of the kind \( V = \frac{K}{D^{b}} \) or, in the case of instantaneous charge, \( V = \frac{K(D/VQ)^{b}}{Q} \), where, \( V \) equals the Velocity of Vibration of particles (mm.s\(^{-1}\)), and \( D \) the distance from the blasting point (m); the values for \( a \) and \( b \) are usually 1,5 and 2; \( K \) is a constant characteristic of the site, and \( Q \) is the explosive charge in kilograms. In spite of the complexity of the propagation factors, a satisfactory statistical prediction can be achieved using a small number of measured events.

7.2 Influence of the blasting diagram

The blasting diagram have influence both on the levels and characteristics of the vibration.

In the AFTES rules some considerations are drawn upon the effects of the levels of the charge and respective delay, the geometric parameters of the diagram, the form of charging and the frequency of the blasts. Although these aspects will not be developed here, it is important to remember that the best processes for reducing vibration levels are used in the blast diagrams: drilling adjustments, type of explosive, delay time, etc. There are ways and methods to adjust to test results that cannot be imposed by regulation because their application is selective and appropriate to their particular results. An exhaustive presentation can be found in Dowding (1985).

8. CONCLUSIONS

In general, vibration safety levels established by regulations are conservative enough for a good working development with practically no risks or probable damage. However, a precaution is due to the complexity of everything involved and the statistical or semiempirical nature of the studies and conclusions based upon them.
The relatively recent and limited treatment of these problems has justified changes in the regulations fitted to experience and available statistical information. The evolution of limit values for damage seemed to have begun conservatively and later, whenever the significance of information would justify, it went in the opposite direction in some cases. The Swiss norm is an example. A clear change is the consideration of the Determinant Frequency in the criteria and the common analysis of the frequencies in vibograms. There is also a tendency for a more exact descrimation of situations and factors, and a prolonging interval between the extreme values.

Nevertheless, frequency analysis is still not considered in the Portuguese norm, nor in the French and ex-Soviet rules, for example.

In the case of the Portuguese norm, it's thought that the limit velocity values obtained are affected indirectly, through coefficient \( \alpha \), in a way which is correlated with the influence of frequency. Note that different types of ground behave differently as vibration filters, thus conditioning the determinant frequencies. An important parameter of this behavior is precisely the velocity of longitudinal propagation of elastic waves in the ground, which is translated in some way by the referred coefficient, even though the characterization deals only with the foundation ground levels.

Figure 4 shows the range of limit safety values now in effect in several countries (excluding the Swiss and Portuguese norms for frequent or permanent events, whose values can be reduced to 1.5mm.s\(^{-1}\) and 1.75mm.s\(^{-1}\), respectively). The dark part of the bars in diagram represents the limits applicable to common situations: a construction with a structure of reinforced concrete and brick walls, of a small to medium dimension, with current foundation elements submitted to a vibration originated from blasting whose frequency is not less than 10 Hz.

In comparative terms, there's a reasonable level of similarity between the Portuguese and the German norms, even though the influence of the foundation ground in the Portuguese one opposes the analysis of the determinant frequencies in the German norm. These norms are the most conservative within the ones which are being reviewed.

![Fig. 5 — Range of limit levels for the vibration velocities based upon some Regulations and Norms.](image)

Global correlation among safety levels becomes difficult not only because of the difference in parameters and ways of discussing them, but because of different understanding and importance given to risks. For example, the ex-USSR defines classes of protection in function of the economic/social risk associated with the damage of the structure in cause (Table 5). Yet other criteria are indifferent to the functions or use of the building but stress the resistance characteristics normally inherent.

The problem of continuous vibrations, caused by mechanical tools (demolishing hammers, the running of machinery, etc.) or of transitory and frequent events, even at a low level, implies the consideration of the phenomena of resonance and fatigue in the structures. There is very little information in this area, and the opinions are divergent. The Portuguese norm establishes a reduction of 30% in the limit values while the Swiss norm prescribes a reduction of 60 - 80%. The German norm considers damage-free vibration velocity only the values lower than 5mm.s\(^{-1}\) at least in structures which aren't especially sensitive. The other regulations reviewed in this paper do not refer to these particular situations. In these cases, the elaboration of specific studies and an intensive control are recommended not only for vibration levels, but also for both stresses and displacements.

The need for vibration control and the application of regulation should be studied whenever possible at the site of operations of civil engineering constructions or mining explorations and it is the authorities responsibility to demand this in certain cases. The carrying out of previous inspections should be planned whenever possible. These procedures, besides mitigating environmental impact and increasing safety at the site, enable insurance coverage and provide a base for litigation inquiries.

Deutsche Norm DIN 4150-P3 — Structural vibration in buildings, Effects on structures, 1986.

Swiss Norm SN640312a — Les ébranlements; Union de professionnels suisses de la route, 1992.

Texte des recommandations concernant l’etude des effets sismiques de l’explosif, Group de Travail n° 3 - Travaux a l’explosif, presented by H. Bejui; Association Française des Travaux en Souterrain (AFTES), 1981.


Peculiarities of geological environment investigation in halokinesis regions
Singularités des études géo-environnementaux dans des régions de halocinèse

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Volgograd Civil Engineering Institute, Russia

ABSTRACT: The main principles of geological environmental investigation and mapping in Pricaspian basin and other halokinesis regions are presented in connection with the necessity of salt structures and salt tectonic faults register as geoeological risk zones. Contents of necessary series of analytical and synthetical maps of geological environment were established. The maps depict the modern state of territories and give the possibility of working out the recommendation for its using with minimal risk for geological environment and constructions. Geological environment investigations programme for 5 different stages of oil and gas development in halokinesis regions was composed. It is the basis of preserving and rational using of geological environment.

RÉSUMÉ: Les principes généraux des recherches ingénieur-géoécologiques et de la cartographie du bassin de la mer Caspienne et des autres régions de halocinèse sont présentés à l'occasion de la nécessité de la statistique des structures salifères et des fractures tectoniques comme des zones de risque geoécologiques. Les cartes ingénieur-geoécologiques analytiques et synthétiques sont dressées, elles reflètent un état actuel du territoire et permettent de donner des recommandations sur son étude et son utilisation au risque minime pour l'environnement géologique et des constructions. Le programme des recherches est préparé pour 5 étapes des travaux d'exploration et pour l'exploitation des gisements de pétrole et de gaz dans les régions de halocinèse.

1. INTRODUCTION

On all continents of the Earth there is a number of regions with active salt tectonics and specific complex of engineering geological and geoeological conditions, their amount is 23. The most well-known are the Pricaspian basin, the North Sea basin and the Gulf Coast basin which represent a special category of platform regions, the evolution of which in Quaternary time and in the preceding stages was characterized by a striking peculiarity. The inherited subsidence of basins in the Pleistocene and Holocene caused repeated transgressions of the sea and accumulation of thick marine and terrestrial sand clayey strata, forming low aggradational plains with a low relief, shallow occurrence of ground water and a specific complex of modern geological processes and phenomena: salt tectonics and created by it salt domes and faults, gypsum and salt karst, swell, loess subsidence etc. Moreover, all called basins are the largest oil gas regions. Last years large-scale oil and gas fields in undersilt sequence were discovered in outlying districts of Pricaspian basin near Orenburg, Tengiz, Astrakhan, Karachaenak, Zhanazhol. They are characterized by high depth (4-6 km) and area, abnormal reservoir pressure (to 80 MPa), high content of aggressive components (sulfur, carbonic acid etc.). Oil and gas development caused ecological consequences: deformation of land surface and structures, fires, ground water delevelling, underflooding and bog's formation, accidents. The main cause of them is absence of geological environmental investigations before oil and gas development. The paper deals with the main principles of geological environmental investigations and mapping in Pricaspian basin and other halokinesis regions with connection of salt structures and salt tectonics faults influence the conditions of the formation of geological risk zones. They have been worked out in connection with oil and gas development, but results of study, especially principles of geological environment analytical and synthetical maps compilation can be of wide use in connection with any other type of technogenic impact for the geological environment. The geoeological consequence estimate based on analysis and generalization of data about 50 oil and gas fields in Russia and abroad.

2. GEOECOLOGICAL CONSEQUENCES OF OIL AND GAS DEVELOPMENT

The most typical geological environment changes on the gas and oil fields usually represented by soil, grounds and ground water pollution, by natural landscapes changes, pressure decay, change of hydrogeological conditions. In Pricaspian and other
halokinethic regions exist new additional problems connected with salt structures. The salt structures are represented by positive forms (domes, anticlines) and negative forms: dividing them interdome depressions and overdome grabens and mules. There are 1758 salt domes in the Pricaspian basin, 300 in the North Sea basin, 328 in the Gulf Coast and about 300 on the shelf. The depth of them mainly changes from 0 to 1500 meters. The salt domes movements are continuing in recent time as result of salt creep. It confirmed by experimental data obtained by author on Astrakhan gas field and Svetly Yar salt dome. The dynamics of salt domes movements are studied not enough. Comparing data of levelling polygon on Svetly Yar dome in 1987-1991 a rate of salt dome rising was calculated about 3 mm/year. The maximum rate of rising (12mm/year) was fixed on the Hoskins Mount dome in the Gulf Coast (Barton 1933).

There are subsidience grabens and many faults above the salt domes. Some of faults have been active in recent time, other faults can be active as a result of strata strained state changes. This can be the reason of construction deformation and accidents. The accident of drilling situated in fault zone on Karachaganak gas field caused breaking of gas and cave-in on area with radius about some kilometers. Another drilling rig fell through the cave. Long (about 400 day) fire of drilling on Tengiz oil field was in connection with the fault too. In Gulf Coast area between Victoria and Beaumont (Texas) 10% of faults over salt domes are active and have caused dislocations of the Pleistocene and Holocene strata, deformations of structures, roads, railways etc. (Verbeek 1979). The active faults with amplitude of about 2 meters and connected with the salt tectonics are known in North Sea basin; for example on Ever salt dome (Illies 1957). Therefore, revealing of faults and monitoring of surface movements over the salt structures are necessary for safe placing of construction. Another problem is well-known and deals with landsurface subsidience caused by oil and gas extraction and consequent change of natural strained state in strata with fluid. The natural reservoir pressure created by elastic properties of fluids and soils decreases and cannot compensate pressure of weight overlying strata (ground pressure). As a result of compaction of soils it happen with consequent subsidience of surface.

Classical example of maximum subsidience (about 9,5 m) is Wilmington oil field (Walsham 1978). The subsidience in Western Venezuela oil field (5m) and delta Po gas fields (4 m) are also well known. In Russia subsidience data of South Caspian basin oil fields (2 m) and North Stavropol gas fields (0,9 m) are published. It is very possible subsidience over many other oil and gas fields in Russia, but published geodetic levelling data are absent. Gas and oil extraction can provoke induced subsidience - additional risk for construction. In Russia the weak shocks (about 3-4) are observed in Grozny, Almetyevsk and Nefteugansk oil fields in different regions of country. The intensive exploration in the Lacq gas field (France) caused 1000 induced earthquakes. 44 of them magnitude more than 3, and 4 - more than 4. Destrcutions are absent because of favorable geological structure (Maury et al.1992). The induced subsidience phenomenon is studied very scanty and their possiblility must be registered, especially in overdue fault zones. Peculiar apprehension connected unique conditions of Pricaspian largest oil and gas fields: enormous areas and reservoir pressure in the oil and gas pools, a great number of salt domes with numerous faults. In these conditions the large decrease of pressure in the oil and gas pools can cause large change of strained state in strata, large subsidence, faults activation and induced subsidience.

3. PECULIARITY OF ENGINEERING-GEEOECOLOGICAL INVESTIGATIONS.

Engineering geocological investigation represent systemic study of geological environment interaction with external natural and man - made environment (Golodkovskaya & Eliseev 1990). Term "engineering geocology" is very close to the term "environmental geology". The aim of engineering geological investigation is geological environment state estimate, prediction of it's further change under the influence of natural and technogenic factors, elaboration and realization of geocological safety system, environment convience preserving and rise. The main objects of study are natural landscapes, rocks, grounds and soils, ground water, modern geological processes and phenomena as well as technogenic objects. In oil gas fields they are presented by exploratory and oil wells, pump-rooms, pipelines, settling basins for oil and ground water, oil and gas preliminary preparation plants, different objects of oil manufacturing plants. The typical peculiarity of oil gas field technogenic impacts on geological environment is presence of two levels of this influence: on the upper part (near the surface) and on the lower part - to the bottom of borcholes. This is the fundamental difference as compared with impacts on urban territories, on roads, railways etc. Unfortunately, lower part of geological environment is never reflected on engineering geocological maps. Influence of salt structures and connected with them faults are also not taken into consideration. Meanwhile, positive and negative salt structures very actively influence the entire complex of engineering-geocological conditions: relations of thicknesses and facies of Quaternary sediments, formation of positive and negative landforms, occurrence and chemism of ground water; composition, state and soils properties; modern geological processes and phenomena (Slniakov 1984). For example, thickness...
of Quaternary soils rises on negative structures and decreases on positive structures. In relief salt domes are represented by higher areas, interdome depressions - by lowlands. A depths of ground water level on salt domes is more as compare with depressions. Properties of soils are also different: swelling and loess subsiding soils, as a rule are situated on positive structures, weak soils - on negative structures. Salt karst and faults are connected only with salt domes and anticlines. In whole, the over salt strata complex of particular importance is forming a stretching zones above salt domes and anticlines and compression zones with more favourable geoeological conditions - over interdome depressions. These zones have essential differences in jointing, permeability, strength of soils. Moreover, geoeological conditions of drilling deep boreholes are different on salt domes and interdome depressions. On positive structures boreholes are being drilled through thick sequence of salt and drilling mud is saturated by brines containing different chemical combinations. These brines can be a source of soil, water and ground water pollution. The difficulties and accidents can take place in drilling as a result of pipe compression under the influence of salt plastic deformations. Therefore, the salt structures influence very greatly on formation of the geoeological conditions and must be studied and mapped side by side as the other factors. This gives the possibility of creation the most complete geoeological models reflecting the modern state of geological environment on two depth levels.

4. ENGINEERING GEOECOLOGICAL MAPPING

Geoeological maps are presented by two types: analytical, which depict the individual components of geological environment, and synthetic, which characterize the entire geoeological conditions. Analytical maps include:

1. The map of spreading of positive and negative salt structures as the zones of different degree of geoeological risk.
2. The map of faults as zones of high risk: the drilling accident and the possible pollution of soils, grounds and ground water, the deformations of structures, induced seismicity etc.
3. The map of engineering geological zonation of upper part geological environment, on which the taxons of the same of geological structure, relief, composition and properties of soils, modern geological processes and phenomena, ground are distinguished.
4. The map of ground water natural protection degree, based on registering of the ground water level depth and thickness, lithology and permeability of soils over ground water level (Goldberg & Gazda 1984).

5. The map of soils natural protection is based on degree of soils absorption capacity (Egorova 1991).
6. The map of technogenic impact on geological environment, the aim of which is revealing all impact sources (especially large and ecologically dangerous), the ecologically conflictual nudes: urban territories, pipelines, oil and gas boreholes, quarries, irrigating systems etc.
7. The map of specific soils spreading: swelling, loess subsiding, weak, saline, artificial, residual deposits, which require the special tests and methods of foundation calculation.
8. The geochemical map.

All these maps are basis for synthetic engineering geoeological map compilation. This map depicts the borders of: the engineering geological district types; the specific soils spreading; the positive and negative salt structures. The different types of tectonic faults, the unfavourable geological processes and phenomena, the degree of ground water and soils natural protection from pollution also are shown. Different methods of portrayal have been used for different information: colour for district types, hashes and spots for salt structures and specific soils, signs for geological processes forms, and cyclograms for degree of ground water and soils natural protection from pollution. In the table-explication of the map the geological environment characteristics are given for every engineering geological district type: geologic section, the relief data, ground water data and their natural protection degree, the specific soils and unfavourable processes presence, types of soils and their natural protection degree, the salt domes and tectonic faults presence. The totality of all this and especially of unfavourable factors (dangerous processes, specific soils, positive salt structures and faults etc.) gives the possibility of ecologica risk estimate and revealing of the more vulnerable territories.

5. THE ENGINEERING GEOECOLOGICAL INVESTIGATION OF OIL, GAS REGIONS PROGRAMME

5.1 Main principles

The problems and maintenance of engineering geoeological investigations depend on two factors types: the character of technogenic impacts on geological environment and peculiarities of the territory geological environment. The complexity of investigations rises with the rising of impacts on geological environment on different stages of its using. The vulnerability of geological environment to impacts and its natural protection are not the same in different districts of perspective oil and gas areas. For example, in districts of sand soil spreading, the ground water is not protected from pollution if
accident on drilling borehole will happen. But if thick stratum of impermeable clay lies on the land surface, this risk of pollution is absent. The higher ecological risk zones exist in the places of the faults and salt domes spreading. Therefore, the most important problem is typification and zonation of the geological environment and revealing of different ecological risk zones. The technogenic impacts on geological environment difference on different stage of exploration and exploitation of oil and gas fields. On the stage of geophysical prospecting of large territory the impact is minimal, on the stage of prospect holes drilling their impacts limit on local zones around the holes. On the stage of the exploitation holes drilling the concentration of impacts rises inside the limited territory of oil and gas field. Then impact very rises on the stage of field exploitation. After all, in occasion of the oil and gas manufacturing and chemical factories building they can render the additional impacts on the geological environment. In accordance with this the geological environment reaction and geoeological consequences of every stage of oil and gas development are not the same and need the different methods of geoeological maintenance. Therefore, geoeological investigations must be connected sharply with the different stages of oil and gas development. On the one hand, they must give the information for the rational placing of the technogenic objects (boreholes, structures etc.) and also - for the geoeological monitoring on every stage.

5.2 The geoeological consequences of different stages of the oil and gas fields exploration and exploitation.

On the 1 stage (geophysical prospecting), geological environment changes are practically absent.

On the 2 stage (prospect holes drilling) the local changes in the zones of drilling of limited number of holes are possible. The probable are the pollution of the soils, grounds, water and ground water by oil, drilling mud, bore meal, oil-field brine. The sources of the pollution are the drilling rigs, pipes, pumps, reservoirs of liquids. The causes of the pollution are the accidental throughouts of the reservoir liquids, low hermetiability of borehole pipes, the filtration of liquids from reservoirs.

On the 3 stage (exploitation holes drilling) the analogous pollution of soils, ground, water and ground water are possible. The intensity of these processes changes, because the volume of drilling rises and concentrates within the territory of oil field. The most significant changes of geological environment can take place on the 4 stage (exploitation of the oil field). Like before, pollution of environment is possible. In the process of extraction and preparation of oil, the sources of pollution are presented by the oil reservoir, settling basins for underground water and brine, pipelines etc. The pollution substances are presented by oil, paraffin, products of gas combustion, reservoir water.

Besides that the processes connected with extraction of fluids are possible on this stage. Some of them arise by the "domino principle", when one of the processes is the consequence of predecessor and the cause of the following one (table 1).

Table 1. The processes caused by fluid extraction.

<table>
<thead>
<tr>
<th>Types of processes</th>
<th>Causes of arising</th>
</tr>
</thead>
<tbody>
<tr>
<td>The reservoir pressure decrease and ground water depression forming</td>
<td>Oil and gas extraction</td>
</tr>
<tr>
<td>The salt creep activation</td>
<td>The reservoir pressure decrease</td>
</tr>
<tr>
<td>The landsurface subsidence, faults activation, induced seismicity</td>
<td>The landsurface activation, induced seismicity</td>
</tr>
<tr>
<td>The deformations and accidence of drilling, pipelines, structures; fires</td>
<td></td>
</tr>
<tr>
<td>The underflooding, bog's formation, loss subsidence, swell and deformations of structures, corrosion of pipelines</td>
<td>The landsurface subsidence</td>
</tr>
</tbody>
</table>

Characteristically, the supporting of reservoir pressure by the force-pumping of the liquid into the oil pool also can cause the unfavourable change of geological environment. They are presented by the repression mounds with radius up to 100 km, ground water level rising and their pollution, the underflooding and the bog's formation, the breaking of gas and the cave information, the induced seismicity.

In the deep strata the solution of rocks and the new mineral creation occur.

On the 5 stage (oil and gas manufacturing and chemical factories building and exploitation) would be changes of environment caused by their function. The most probable are pollution of soils, grounds and ground water by the oil and sulfur and other chemical combination; ground water level rising and underflooding as on Astrakhan gas chemical factory.
In the areas of the swelling and loess subsiding soils spreading the deformations of structures are possible. If the factories will be placed in the depression or repress zones caused by reservoir pressure changes on the oil and gas field, they can have a deformations in connection with the landsurface deformation and fault activation.

5.3 The geoeological investigations on the different stages of the oil and gas fields exploration and exploitation

According to the technogenic impacts on the geological environment and it's possible changes as result of these impacts, the scheme and composition of engineering geoeological investigations must be following.

On the 1 stage the aim of investigation is the study of the background state of geological environment on the regional level, the typification and zonation of territories, the estimate of activity of the modern natural and technogenic geological processes, the revealing geological risk zones for the planning of bore holes placing. The following work must be fulfilled in this period:

1. Analysis of natural and technogenic objects geological environment changes data.
2. Compilation of middle-scale (1: 200000) engineering geoeological maps series of studied territory, which are numerated in part 4.
3. Geochemical survey working out and geochemical map of geological environment modern pollution state compilation.
4. Working out recommendations for bore holes placing out of high ecological risk zones.
5. Working out programme of local environment monitoring around prospect holes drilling.

On the 2 stage the aim of investigations is the estimate of geological environment changes in every hole sphere of impact. During drilling rig building the examples of soils are selected for determining the chemical composition, oil content and gas composition of soil air. Beside that, the examples of water are selected from streams, basins and wells for the same aim. The observation boreholes for study the dynamics of ground water pollution are drilled in several directions from prospect hole.

During the prospect hole drilling the following works are made:
1. The soil examples selection from the excavations for the chemical analysis (monthly).
2. The water examples selection from the streams, basins and wells for the same aim (monthly).
3. The ground water examples selection from the observation boreholes simultaneously with the ground water level measuring.
4. The generalization of analysis results in the form of diagrammes and special geochemical maps and sections. The hydrogeological maps are composed for the recognizing of ground water direction and rate.

On the basis of all these data the predictions of the pollution are composed and environment protect action are worked out.

At the beginning of the 3 stage the oil gas field project is worked out. The engineering geological and geoeological investigations must precede projecting and be its basis. As a result of investigations the engineering geoeological model of oil gas field must be composed in the form of large-scale (1:25 000) maps, sections and other materials. This model must depict ecological risk zones.

Besides that, during exploitation holes drilling, the monitoring of soil, grounds, water and ground water pollution are worked out. For this aim the observation holes are drilled. The placing of these holes, their depth and amount depend on the exploitation holes and other technogenic objects of field placing, as the placing for examples of soil selection. On this stage examples of soil and water are selected and analysed, as on the preceding stage. In addition, the periodical geochemical survey is carried out; then on the basis of all information the prediction of geological environment changes and protect actions are worked out.

At the same time, the geodetic levelling polygon must be created for the monitoring of surface movement on the oil gas field. It is very important 1-2 cycle of levelling to be worked out before the oil gas field exploitation because of natural movements of soil structures and land surface. It is necessary to know the rate of these natural surface displacements in order to distinguish them from the technogenic movement connected with the oil and gas extraction.

Beside that, on this stage the seismograph establishment is necessary for monitoring the induced seismicity in the period of the oil gas field exploitation.

On the 4 stage (exploitation) the geological environment investigation is carried out on some divisions of monitoring:
1. Monitoring of soil, ground, water and ground water pollution.
2. Monitoring of ground water regime and underflooding on the observing boreholes polygon.
3. Monitoring of land surface and structures movement and deformations.
4. Monitoring of induced seismicity.
5. Monitoring of reservoir pressure changes.

On this basis the maps of geological environment changes are periodically composed as well as connection between these changes and natural and technogenic factor's data are revealed. These trends give a possibility of the future dangerous changes prediction and protect action work out.

On the 5 stage, before factories building the engineering geoeological zonation of their territory must be worked out and contructions must be placed within ecological risk zones.
The geological environment monitoring on oil gas field continue. In addition to it, the new objects need in this monitoring: the factories different constructions. The components of monitoring are the same: pollution, ground water level and underflooding, induced seismicity, constructions movements and deformations. The geomonitoring data also are used for prediction of unfavourable environment changes and protect action work out.

6. CONCLUSIONS

As a result of geocological consequences analysis of exploration and exploitation of about 50 oil gas fields in different conditions, the main principles of geocological investigations in salt tectonics regions were established. The necessity of salt domes and salt tectonics faults register as ecological risk zones was revealed. The composition and contents of engineering geocological map series depicting the modern state of territories and giving a possibility of their using with minimal risk for environment worked out. The geocological investigations standard programme for 5 different stages of oil gas development was composed. The practical applications of results are also effective for prediction of the dangerous geological processes possibility, for planning of rational using of territories, choice of rational foundation types and protect actions working out with the help of maps series.

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Characterization of the erodibility of unconsolidated materials in engineering geological mapping
Caractérisation de l’érodibilité des matériaux non consolidés en cartographie géotechnique

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ABSTRACT: In this paper we present the results of the application of the empiric method used to characterize the erodibility index of the unconsolidated materials. This method uses intact samples with small dimensions and it considers 2 very simple tests: water absorption and loss of weight by soaking (Nogami & Villibor 1979). For elaboration of this work we studied about 200 samples of unconsolidated materials of the Piracicaba region (São Paulo State - Brazil). This region presents a big diversity of materials. The results obtained by this method present good correlation with the observations of erosion features in the natural landscapes.


1 INTRODUCTION

The erosion problems are among the hazards that produce more damage to the environment. Thus the identification of the erodibility potential of the unconsolidated materials and the definition of the most susceptible areas to the development of erosion are essential in Engineering Geological Mapping. The determination of the erodibility of the unconsolidated materials is based on various attributes which present difficulties to be obtained. Therefore we tried to establish a methodology in this work to determine the erodibility based in a simple test, that can be easily executed in the engineering geological mapping works.

2 CONSIDERATION ABOUT THE ERODIBILITY OF THE UNCONSOLIDATED MATERIALS

The erodibility of the unconsolidated materials is related with two principal factors: action forces that are characteristics of clayey materials and size of the solid particles which interfere in the transport of the particle (Vilar 1987). The "in situ" unconsolidated materials propriety, like moisture content and water infiltration coefficient, are important too. This way we verify that there is a complexity in the analysis of the erodibility involving the chemical and physical properties and "in situ" conditions of the unconsolidated materials. Several works present tests and correlations with chemical and physical proprieties and the erodibility, but they do not present good results. The natural
physical tests are of easy execution and have been utilised to characterize the erodibility.

2.1 Revision methods to erodibility

Various methods, mainly based in empirical tests, search to characterize the soil erodibility in man made slopes, among them we can distinguish the works of Meireles (1967), Santos & Castro (1966, 1967), Philipponat (1973), Nascentivo (1974), Nogami & Villibor (1979) and Villibor et al. (1986). The analysis of these works show that the efficiency for identification of the soil erodibility is around 80%, for the best method (Table 1).

The method proposed by Nogami & Villibor (1979) consider two factors: disaggregation and water absorption. These factors are considered very important in the soil erodibility for the majority of authors. This method utilizes undisturbed samples and it considers the act of drying; it is simple and does not utilize sophisticated equipment.

Table 1. Percentage of soil samples with high erodibility (total=21) identified by various methods (apud Santos & Castro 1967).

<table>
<thead>
<tr>
<th>METHODS</th>
<th>NUMBER OF IDENTIFIED SAMPLES</th>
<th>PERCENTAGE OF THE TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRAIN SIZE DISTRIBUTION</td>
<td>14</td>
<td>67</td>
</tr>
<tr>
<td>ATTERBERG LIMITS</td>
<td>12</td>
<td>57</td>
</tr>
<tr>
<td>INDEBITIZEN</td>
<td>11</td>
<td>52</td>
</tr>
<tr>
<td>EXPANSIBILITY</td>
<td>16</td>
<td>76</td>
</tr>
<tr>
<td>ESPECIFIC SURFACE</td>
<td>13</td>
<td>62</td>
</tr>
<tr>
<td>EQUIVALENT CENTRIFUGAL</td>
<td>08</td>
<td>38</td>
</tr>
<tr>
<td>MOISTURE CONTENT (pF=0)</td>
<td>09</td>
<td>43</td>
</tr>
<tr>
<td>SILICA/SESQU. RELATION</td>
<td>08</td>
<td>38</td>
</tr>
<tr>
<td>WATER ABSORPTION</td>
<td>07</td>
<td>33</td>
</tr>
</tbody>
</table>

3 MATERIAL AND METHOD

The materials studied were obtained in an area of almost 2,900 Km², located between the parallels 22º 30' and 23º 00' S and the meridians 47º 30' and 48º 00' W, in the center-east region of São Paulo State, Brazil.

These materials are the results of the "in situ" weathering or coluvium sediments associated with sedimentary rocks from the formation of the Paraíba Sedimentary Basin. In a few sites occur materials associated with the basic igneous rocks (basalts and diabases). The texture of these unconsolidated materials varies from sandy (> 80% of the sand) until clayey (> 60% of the clay).

The methodology employed in this work consisted in the sampling of unconsolidated materials in road cut slope of the region in sites where there are the natural erosion features. These samples were obtained according to the methodology by Nogami & Villibor (1979) in different depths in the weathered profiles. The disturbed samples were collected for the realization of characterization tests.

3.1 Determination of the erodibility index

The method is based in two simple tests: water penetration test and loss of weight by soaking. The undisturbed samples are obtained with a PVC cylindrical casing (40 mm internal diameter and 20 mm high). Before the tests, the samples (with the casing) must be dried in the air and shade for a period of at least one week. Below we describe the procedure according to Nogami & Villibor (1979):

a) Water Penetration Test (S): The procedure used for this test is similar to the one developed by Peltier (1954). The bottom of the sample (with casing) has to be perfectly adapted to a saturated porous stone placed in a horizontal position, which limits a container filled with water. This container is provided with a horizontal graduated tube that measures the volume of water absorbed by the sample through the porous stone without changing hydrostatic head. By measuring the volume of absorbed water at appropriate time intervals the values may be plotted in a graph in which the abscissa is the square root time (t) in minutes and the ordinate is the absorbed water (q) in cm² per cm² of sample section and we can calculate the slope S=q/t² of a straight line best fit (Figure 1).
b) Loss of Weight by soaking (P): After the water penetration test, the porous stone must be adequately attached to the bottom of the casing containing the sample and the assemblage should be immersed in water so that the top of the sample remains in a horizontal position and about 2 mm above the water level. Keep the assemblage thus as long as there is no more perceptible modification on the free top surface. Suspend the assemblage in such a way that the free surface of the sample is kept vertically. Then carefully immerse the assemblage in water. The lower part of the free surface of the sample must be at least three centimeters above the bottom of the water container. After 24 hours determine the loss of weight in consequence of soil detachment from the free surface of the sample; express that loss in percentage of the initial weight on a dry weight basis.

c) Calculation of erodibility index (E): The erodibility index, according to Nogami & Villibor (1979), can be calculated by the expression:

\[ E = 52 \frac{S}{P} \]  

(1)

where \( S \) and \( P \) are the values determined. If the final value \( E \) is less than one it means high erodibility.

4 DISCUSSION OF THE RESULTS

The observation of the erosion conditions in field allowed the identification of 56 sites with slope erosion problems. The results of the water penetration index (S) and loss of weight by soaking (P) obtained for the samples of these sites were plotted in the graphic (Figure 2).

According to Nogami & Villibor (1979) a straight line with the expression (1) should separate the materials with high and low erodibility potential. In the Figure 2 we can observe that only 18 samples are above the straight line (E>1) and these results are discrepant with the observations in the field. We searched to identify the possibilities of error and we verified that 8 samples presented high quantity of organic matter which results in attraction forces (Vilar, 1987), occasioning a low loss of weight in soaking test and consequently E<1. For a group of 5 samples, the problems appear to be related with a residual diagenetic force that also causes a low loss of weight.

Therefore, for 13 samples we identified the possible cause of error, but there exist 25 samples with the discrepant results yet. The analysis of the Figure 1 show that a group of samples is essentially sandy and presents high loss of weight and water absorption index. In this case the high water absorption index is not enough to impede the erosion. Therefore, the expression (1) is not adequate to characterize the erodibility of the unconsolidated materials of the region. Nevertheless a straight line with a small inclination can separate 82% of the materials with high erodibility. In this case the erodibility index can be determined by the expression:
E = 40 S/P

\[ E < 1 \text{ high erodibility} \]
\[ E > 1 \text{ low erodibility} \]

The Figure 3 presents the results of the water absorption index and loss of weight by soaking of 209 samples analysed, where we verified that the straight line separate the majority of materials of erosion different behaviour. This results allow us to conclude that the methodology is adequate to identify the erodibility potential, but the expression (2) proposed in this work permitted to separate a larger number of materials than the expression (1).

![Figure 3 Water absorption index versus loss of weight - 209 samples.](image)

5. ANALYSIS OF THE RELATIONSHIP BETWEEN THE ERODIBILITY INDEX AND OTHER ATTRIBUTES OF THE UNCONSOLIDATED MATERIALS.

The knowledge about erodibility of the unconsolidated materials is very important and we tried to establish the relations between the erodibility index and index property one.

5.1 Erodibility index and specific gravity of solids

The majority of the materials studied presents specific gravity of solids varying between 2.55 and 2.80 g/cm³ independently of the erosion behaviour (Figure 4). Therefore the specific gravity of solids is not a good indicative to erodibility of the unconsolidated materials.

5.2 Erodibility index and dry density

The analysis of the Figure 5 show that there is not relation between natural dry density of the unconsolidated materials with high or low erodibility potential.

![Figure 4 Erodibility Index versus Specific Gravity of solids.](image)

![Figure 5 Erodibility Index versus Dry Density](image)

5.3 Erodibility index and natural void ratio

The majority of the unconsolidated materials that present high erodibility potential presents natural void ratio range 0.50 - 1.00 (Figure 6), however there exist various materials with low erodibility index that are in the same interval. Therefore the natural void ratio appear to have influence in the erodibility of the unconsolidated materials, but its utilisation as
indicative of the erodibility conditions must be avoided.

Fig. 6 Erodibility Index versus Natural Void Ratio.

5.4 Erodibility index and clay contents

The clay contents is related to the erodibility of the studied materials, as we verify that the majority of the unconsolidated materials with high erodibility presents clay contents inferior to 40% (Figure 7) and occurs mainly below 20%. However, the clay contents shouldn't be utilized separately as index property to estimate the erodibility of the unconsolidated materials without the knowledge about clay mineralogy once the clay contents of the materials with low erodibility vary between 10% and 70%.

Fig. 7 Erodibility Index versus Clay Contents.

6 CONCLUSION

The results obtained in this work allow the conclusion that the erodibility index is adequate to predict the erosion behaviour of the unconsolidated materials in man cut slopes in Engineering Geological Mapping, since the following observations are considered:

- Don't utilize samples with high quantity of organic matter
- Utilize at least three samples in such sites to verify the validity of the results
- Be careful with the sampling and transport to avoid structure and texture of unconsolidated materials breakage.

In Engineering Geological Mapping work this erodibility index should not be utilized as quantitative measure of the erosion potential, but only as qualitative measure.

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Significance of discontinuity survey and physiographical study in engineering works

Importance de la surveillance des discontinuités et des études physiographiques pour les travaux de construction

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ABSTRACT: Topographic and groundwater divides are assumed to coincide in the conventional method. However, they keep apart in tilted and stratified units. Based upon the effect of persistent discontinuities governing the groundwater movement, a new method is proposed. It is named here as Structural Method. Because of this phenomenon the Nurmountain (in Southern Turkey) which is characterised by northwest dipping homoclinal structures, has quite different hydrogeological conditions over its flanks: wet over the western flank and dry on the other side. The prevailing wet ground conditions adversely affect the stability of the existing highway and railway and also the motorway being constructed over the western flank.

Annual precipitation, snowcover duration, and groundwater recharge increase with altitude while temperature, atmospheric pressure, and evaporation decrease. Groundwater movement, runoff, and porewater pressure are the common factors which are taken into account in geotechnical designs. These factors are essentially dependent on discontinuities and physiographical features.

RESUME: Les séparations topographiques et celles de l'eau souterraine sont considérées comme ils se coïncident en "métodé conventionnelle". Pourtant, elles se sont séparées dans les unités stratifiées, et pliées. Une nouvelle méthode est présentée, qui se base sur des effets des discontinuités persistantes, qui dirigent le mouvement de l'eau souterraine. On la présente ici comme "la Méthode Structurelle". A cause de ce phénomène, la Montagne Nur (en Sud de la Turquie), qui est caractérisée par ses structures homoclines qui s'inclinent vers Nord-Ouest, montre des conditions hydrogéologiques très différentes: le flanc en Ouest est humide et l'autre du côté opposé, est sec. Les conditions dominantes de la terre humide, affectent gravement la stabilité d'autoroute et le chemin-de-fer qui existent déjà et aussi la stabilité de route qui est encore en construction sur le flanc en Ouest.

La précipitation annuelle, la durée de couverture de neige et la recharge par l'eau souterraine s'augmentent avec l'altitude pendant que la température, la pression atmosphérique et l'évaporation diminuent. Le mouvement de l'eau souterraine même sous la terre, la pression de l'eau dans les pores, sont les facteurs communs qui sont tenues compte dans les projets géotechniques. Ces facteurs dépendent surtout des discontinuités et des propriétés physiographique.

1 INTRODUCTION

Discontinuities from a simple crack to an extensive fault and landforms from a peneplain to a mountainsous topography develop under the influence of geological processes. In this respect, a discontinuity survey and a physiographical study can provide convenient data to grasp the actual ground model and thereafter to carry out engineering works.

Kinematical approaches based upon discontinuity surveys, are commonly employed to analyse stability of slopes in tilted sequence or dissected (nonmassive) units. Moreover, persistent discontinuities govern position of Ground Water Recharge Area Boundary (GRAB) and thus, hyrogeological characteristics of the catchment area (Yılmazer 1991). In the conventional method, it is assumed that GRAB and topographic divide coincide with each other although they keep apart in tilted nonmassive units which constitute more than 95% of the rock units encountered in engineering works. Hence, a new approach (Structural Method) based on discontinuity data, is proposed to locate GRAB.

Landforms, altitude, sides of a mountain facing northward or southward, and distribution of large surface water bodies have great influence on precipitation and groundwater occurrences. Turkey is characterised by east-west trending mountain belts, numerous large water bodies including seas, lakes, and dams, and extensive rugged terrains (Fig. 1). Altitudes vary in a wide interval of 0-5000 m. Meteorological stations are usually at lowlands nearby towns. Precipitation, snowfall, snowcover duration, and consequently, groundwater recharge increase with
2 GROUNDWATER RECHARGE AREA BOUNDARY

As introduced above, GRAB deviates from the topographic divide depending on persistent discontinuities. Bedding planes are the common and distinct discontinuity types which are essentially taken into consideration in constructing GRAB (Yilmazer 1985). Figure 3 is given to demonstrate the method and its procedure is itemised below.

- Carry out the discontinuity survey.
- Plot prevailing groundwater table contours at a desired interval.
- Draw the topographic divide (TD).
- Draw tangents touching the points on TD.
- Draw strikes of bedding planes at the related tangent points.
- Project the bedding plane onto the horizontal and draw the structural contour intersecting the groundwater table.
- Draw a perpendicular line to the strike from the intersection point (SW) of the structural contour and the tangent line to find a point on GRAB.

An actual catchment area and groundwater divide could be determined approximately by adopting Structural Method. In addition to bedding planes, close-to-widely spaced tensioidal and cooling joints, en echelon faults, and schistosities are also effective on GRAB. The conjugate and compressional joints are less effective because of their

Fig. 1 Simplified physiographic features over the Anatolia.

altitude. In this connection, meteorological data obtained from lowland stations could not be used without a correction for engineering works executed at highlands (Fig. 2). The correction factor changes from basin to basin depending on the other factors influential on climate. A precipitation versus elevation graph has to be plotted for the basin under study and then, the slope of the best fitting line \((S = 0.001\tan \alpha)\) can be found to calculate annual precipitation \((P, \text{mm})\) at the midpoint of the study area as

\[
P = P_m + S \times \Delta H
\]

where

- \(P_m\) is the annual precipitation at the nearest meteorological station, \(\text{mm}\)
- \(\Delta H\) is the elevation difference between the midpoint of the study area and the station, \(\text{m}\).

Fig. 2 Dependence of precipitation on altitude.

Fig. 3 The structural method to construct groundwater recharge area boundary in a tilted nonmassive geological unit.
low permeability. Structural Method is applicable in a catchment area where above discontinuities are dominant. Hydrogeological features, which play significant roles in the design of engineering structures, and groundwater exploration are noticeably affected from prevailing major discontinuities.

Figure 4 is prepared to depict the significance of structural elements on the hydrogeology of a watershed area. A plunging syncline is one of the major structural elements which can divert groundwater toward the axial plane in the plunge direction. For instance, an enormous landslide (2.5 million m$^3$) due to a shallow excavation (5 m) at downstream part of a plunging syncline occurred at KM 63+000 of the Ankara-Göreme Motorway in 1991. It was predicted and reported in 1990 (Yılmazer 1991). On the contrary, anticlines cause groundwater to move away from the axial plane. In a basin characterized by an anticline, groundwater efficiency reduces considerably while ground stability increases appreciably. Strike slip faults which are subvertical are less effective than thrusts and gravity faults on hydrogeology. Faults and joints are the common discontinuity types which control groundwater movement in intrusive igneous rocks. Schistosity, bedding, and imbricated thrusts are particularly effective on hydrogeology when they are subjected to compressional stresses due to major tectonic activities.

The resulting homoclinal structures cause seepages, springs, and shallow groundwater conditions to form in the basin where structural elements dip towards downstream (see Fig.4). The three adjacent basins shown at the bottom of the figure are in Thrace. The groundwater in this area is diverted towards southeast by schistosity and accompanying bedding planes while the N-S trending normal fault and joints conduct it southward. Some of the groundwater, controlled by discontinuities, is extracted at a rate of >10 l/s which is an appreciable quantity for bottling near Istanbul, a large city of 10 million consumers. It is usual that groundwater in metadetrinitic rocks is slightly acidic (pH<6.5) and poor in mineral content. The pH value for drinking water should be around 8. However groundwater extracted from major deformation zones with a high level of alteration in metadetrinitics can be used directly as drinking water.

One can deduce from the figure and above explanation that the basins labelled with II have respectively more

- groundwater potential,
- seepages and springs,
- perennial streams,
- vegetative cover,
- potential ground instability problems, and
- flood hazards, particularly where the land left barren by man.

3 PHYSIOGRAPHICAL STUDY

Physiography of a basin is governed mainly by distribution of geological units bearing different engineering properties, neotectonic activities, Late Tertiary-to-Quaternary igneous activities, its distance to surface water bodies including large dams, the direction toward which the majority of the basin is facing, the overall topographic slope, and vegetative covers. More than six percent of the geological units in Anatolia is constituted by Triassic and Cretaceous melanges which comprise a small block to mountain diverse in origin and
engineering properties. It is distinct that top of hills and mountain ridges are characterized by stronger units. A typical example has formed along the northern side of the Karasu Valley, 20 km north of Erzurum (in Eastern Turkey). An extensive and thick thrust slab of a strong limestone has been dissected by creeks forming mesa features. 500- to 700 m high hills with cap rock of limestone align along the valley. The chain-like course, composed of very high mesa plains, are distinguishable from a few ten kilometres away. They are the unique source of crushed rocks in the region. Numerous springs and seepages formed at their lower tectonic contacts (thrust plane). Longitudinal valleys and plains have developed along the active strike slip faults namely North Anatolian Fault (NAF) and East Anatolian Fault (EAF) and grabens perpendicular to the Aegean Coastline (see Fig.1). Potential hydrothermal energy sources are clustered along these tectonic zones. Additionally, mild and humid climatic conditions prevail. Most of these plains are filled with recent deposits. Shallow groundwater condition is frequent. Areas of marsh and swamps constitute main types of unstable grounds.

The regions characterized by Miocene to Quaternary igneous activities include high mountains such as Mount Ararat (5165 m) and highly dissected topography (see Fig.1). Geothermal gradient over these districts are noticeably high (>0.06 C/m) and thermal springs appear often. Hence, hydrothermal alteration is widespread and very poor underground conditions are observable. However, some dykes and the unique vents with accompanying lava flows appear as strong rocks. Persistent dykes with high gorges provide favourable locations to construct thin walled arch dams. Dodurga Dyke (in Middle Black Sea Region) with several gorges stands up like a high castle and the Great Wall! It forms a natural boundary between the mountainous region and the lowland with a hummocky topography. Both sides differ in climate, hydrogeology, vegetative cover, and hence engineering geological characteristics.

Large surface water bodies, natural or artificial, increase humidity and turn the climate into mild. Thus, a thick weathering zone may develop over bedrocks. Most of the mountain belts in Turkey trend in east-west direction with flanks facing northward and southward. Snowcover over northern sides has longer duration which recharges groundwater at higher rate. This is one of the main reasons why highways and similar engineering structures, especially where cold climate prevails, are preferably located over southern flanks in Northern Hemisphere. Unstable grounds due to relatively abundant groundwater appear often at northern sides. However, higher daily fluctuations of ambient temperature at southern sides cause deterioration, disintegration, and formation of thick sheetwash deposits: talus derived from crystalline rocks and colluvium from others. Therefore, unstable grounds are often seen along toes of southern flanks whereas they may extend uphill over northern flanks.

Altitudes and overall topographic slopes are also influential on the climate and engineering designs.
Fig. 6 The relationships between precipitation and the topographic elevations of the meteorological stations in the Cukurova Basin.

\[ Q_n = C \times \frac{I \times A}{3600} \]

where

- \( Q_n \) is the rainfall intensity, mm/h
- \( I \) is the rainfall intensity, mm/h
- \( A \) is the area of the concerned basin, m²

Some hydrologists of Turkish State Water Works Directorate have also noticed that the actual \( Q_n \) value could not be found by this equation (B. Congar, personnel communication, 1994). This discrepancy is commonly observable in the Black Sea Region where mountainous terrains are widespread and rainfall is usual on snowover since the northern side of the Pontid Mountain Belt faces Black Sea. Thus, it is exposed to direct atmospheric circulation coming over the sea. In this connection, Figure 7 is presented as an example in which physiography and structural elements are distinct. The ratio of the actual flood rate and the \( Q_n \) value based upon the conventional formula, given above, is found greater than 1.5 in this area. Some of the culverts constructed in 1992 along the motorway could not transmit flood. Because of this improper calculation the famous historical bridge (Pozanti Silifke Bridge) across the Cakıt stream was destroyed in 1992 by the jetting action of a culvert constructed for the Ankara-Adana highway, 50 m upstream part of this historical bridge. The area as depicted in the figure is characterized by metamict rocks, recrystallized limestones, ophiolitic melange, recent deposits, and homocline structures including imbricated thrusts, schistosity, bedding, recumbent folds, and compressional joints. The nearest meteorological station, where the data about time (t, min) versus rainfall intensity (I, mm/h) are obtained, is at an elevation of 40 m. Hence, it was recommended that the \( Q_n \) value should be multiplied by a correction factor \( F_c \), based on a discontinuity survey and physiographical study. The \( F_c \) value was found as 1.5 across this mountain, which affected the design of engineering structural elements of the motorway significantly. The correction factor \( F_c \) can be simplified and expressed as

Fig. 7 Geological map and section depicting major homocline structures and distribution of accompanying unstable hydrogeological features over the Nurmountain Belt crossed by the motorway.
\[ F_c = [1 + \frac{a}{A} + \frac{(S^*P/P_m)}{\text{ altitude snowcover}} + \frac{(S_{cd}/12a^2/A)}{\text{Physiography}}] \]

where

- \( a \) is an area of increase (+) or decrease (-) obtained by Structural Method, m²;
- \( A \) is the concerned area found by the conventional method, m²;
- \( S \) is the slope of the best fitting line of the P-H plot, and \( S_{cd} \) is the annual duration of the snowcover at the midpoint of the strip area.

The snowcover component of the equation can be omitted if \( 1/2 \times S_{cd} \leq 7 \) months.

4 CONCLUSIONS

A large number of basins and sites of major engineering structures over Turkey have been surveyed in order to specify the effects of discontinuities and physiography on engineering geological works. It is deduced that persistent discontinuities play significant roles in the determination of Groundwater Recharge Area Boundary (GRAB) and groundwater movement. Springs, seepages, perennial streams, and unstable grounds appear often over the flank of a mountain range where majority of discontinuities slope out and/or slope into the ground in the direction of topographic slope. Atmospheric pressure and annual evaporation decrease with elevation above mean sea level while annual precipitation and snowcover duration increase. The northern side of a mountain range especially the one facing sea and/or an extensive plain can produce flood at an unexpectedly high rate.

The study provided some actual cases to emphasize the significance of discontinuity surveys and physiographical studies in engineering works executed in mountainous terrains.

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Cartographie géotechnique et zonage du Rio de Janeiro

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ABSTRACT: Severe environmental problems and increasing housing in vulnerable locations stress the necessity of considering engineering geological knowledge when administrating present and planning future in Rio de Janeiro. This paper presents new data gathered in natural hazards studies and how engineering geological maps prepared to help solving urban problems have been considered by city planners.

RÉSUMÉ: Les sérieux problèmes de l'environnement et l'augmentation des logements en localités dangereuses mettent en évidence la nécessité de conséder les connaissances en géologie de l'ingénieur dans les décisions administratives actuelles et dans la planification du futur du Rio de Janeiro. Cette contribution expose des nouvelles données récoltées par les études des accidents naturelles et comment les cartes géotechniques, préparées pour aider à résoudre des problèmes urbains, ont été considérés par les planificateurs urbains.

1 INTRODUCTION

An Analysis of World Population (UNO, 1988) indicates that Rio de Janeiro will be at the end of this century the eighty world largest metropolis. The accumulated urban-provoked geoenvironmental problems are expected to increase quickly together with the development of urban facilities and if housing in vulnerable locations continue.

Both accumulated and foreseeable urban problems have so far stressed the necessity of considering engineering geological knowledge in land-use plans and hazard mitigation activities. The application of available maps faces however constraints due to planners’ lack of background in geoscience and due to intensive relationship between man and nature, which creates new risk areas quite everyday.

This reality requires the development of appropriate engineering geological / hazard mapping procedures and new alternatives to forster political determination to use of existing knowledge.

2 NATURAL HAZARDS IN RIO DE JANEIRO

Sugar loafs and scarpments constitute the conspicuous relief features of Serra do Mar belt along the brazilian southeast coast where Rio de Janeiro is situated. The region receives more than 1500mm annual rainfall. Mountains up to 1000m and mounts composed of igneous and metamorphic rocks build the landscape together with alluvial unconsolidated sediments. The landforms are represented by rather steep slopes which can show either thick residual soils, or large outcrops or thick colluvial masses.

The encroachment on the city slopes by urban land-use and favelas grew fast after the 50's and is practically complete at the end of this century. The favelas creep along stream channels, increase vertically and comprise 35% of the city inhabitants.

Two major hazards affect Rio de Janeiro - landslides and floodings - and the continuous development of inappropriate areas increase their risk and consequences.
Landslides are so far the most destructive disasters in Rio de Janeiro and have occurred since the city’s foundation. In the years 1966 and 1967 - when landslides disasters occurred in many hundred of locations during heavy rainfalls (484mm/10-12.01.1966 and hourly rainfalls higher than 100mm in 19.02.67), 2500 houses were destroyed, the number of fatalities was over 200 and 9000 people led homeless. In February 1988, when rainfalls occurred in 3 main events (200mm/02-06.02; 150mm/10-13.02; 430mm/19-22.02), the number of fatalities reached 80 and 4200 people led homeless.

From 1988 till 1992, 420 slope stabilization works were executed with a cost of US$ 130M. The number of stabilization works accomplished in the most critical landslides areas since 1966 amounts 3500.

The landslides mostly occurred in unsaturated colluvial and residual soils. It is noticeable however that in the last years the number of landslides affecting garbage deposits placed on the slopes increased dramatically, particularly inside the slum settlements.

Landslide Hazard Maps (Barros et al.,1992; Amaral,1993) show that a large part of Rio’s 1400km2 area is highly susceptible to landsliding and that many hundred thousand inhabitants are directly threatened by landslide hazards. Table 1 presents landslide information obtained from the recently developed Local Landslide Inventory (Amaral,1993).

2.2 Floodings

Floodings are not so destructive as landslides but are also very common in Rio de Janeiro. Inundations occur along the drainage network crossing the most densely occupied districts whereas natural overfloodings take place in the west and south zones. In these flooding areas sand-clayey and organic clayey sediments show water table close to the surface and bad drainability. These overfloodings assume major proportions when the precipitations concur with high tides. Overfloodings sustained considerable damages in 1975 and 1988, leaving 2000 people homeless at Campo Grande, Sepetiba and Santa Cruz towns.

2.3 Development of vulnerable locations

The development of vulnerable locations is a 24-h round process in Rio de Janeiro. The result is a host of problems for the task of hazard prevention. The 3500 slope stabilization works undertaken till now, for example, have to be continuously supplemented by new projects. They control natural slopes and prevent accidents in many man-made cuts but never come as fast as the interrelation between man and nature. Many concrete walls, which reduced the debris flow hazards by controlling the surface water discharge during heavy rainfalls, were followed in dry periods by rapid occupation downstream, since the dwellers think that no more risk remain at these sites.

<table>
<thead>
<tr>
<th>Year</th>
<th>Significant Landslides</th>
<th>Fatalities</th>
<th>Destroyed Houses</th>
<th>Roacs Affected</th>
<th>Main Rainfall Events / Nq of Landslides</th>
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<tbody>
<tr>
<td>1986</td>
<td>95</td>
<td>10</td>
<td>87</td>
<td>22</td>
<td>29-31.12 / 24</td>
</tr>
<tr>
<td>1987</td>
<td>39</td>
<td>1</td>
<td>36</td>
<td>9</td>
<td>26.01 / 9</td>
</tr>
<tr>
<td>1988</td>
<td>200</td>
<td>80</td>
<td>-</td>
<td>-</td>
<td>19-22.02 / 180</td>
</tr>
<tr>
<td>1989</td>
<td>61</td>
<td>17</td>
<td>32</td>
<td>14</td>
<td>10-13.06 / 39</td>
</tr>
<tr>
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<td>22</td>
<td>4</td>
<td>21</td>
<td>4</td>
<td>18-20.04 / 15</td>
</tr>
<tr>
<td>1991</td>
<td>25</td>
<td>3</td>
<td>32</td>
<td>3</td>
<td>27-30.03 / 8 / 07.05 / 7</td>
</tr>
</tbody>
</table>
3 ENGINEERING GEOLOGICAL MAPPING AND ZONNING

Developments in engineering geological mapping in the large urban area of Rio de Janeiro involve the use of appropriate mapping procedures. Of great importance is the use of pedological information when preparing regional maps, the choice of a correct scale to map the hazards and the continuously updating of the risk maps.

3.1 Medium-scale maps for land-use purposes

Rio de Janeiro was partly covered by synthetic engineering geological mapping (1:50,000 scale) in the eighties. These maps were prepared to act as a guide to areas that could be orderly occupied in the west and southwest zones of the city, where a urban expansion was expected. The mapping procedures followed the guides published by the IAG (1976) and used results of interpretation of pedological informations to geotechnical purposes as described in Barroso et al. (1987). The soil surveys undertaken in the 70's, which resulted in good quality pedological maps for agricultural purposes, constitute an enormous amount of analytical information and description of soil profiles, which were appropriately employed for geotechnical characterization.

Amaral (1988) prepared an engineering geological map for the 470km2 area of Sepetiba Bay. The map contains 9 geotechnical units developed over Pre-Cambrian metamorphic crystalline rocks and 5 units developed over fluvial and marine sediments. In order to facilitate its application a special land-use map was prepared based on each unity properties (fig.1). The residual soils were indicated as suitable for the installation of urban infrastructure. Sandy soils were precluded as environmental preservation areas. Areas with lower water table were considered less suitable to urbanization. Areas with compressible organic soils were indicated as susceptible to settlements.

3.2 Medium and Large-scale Hazard Maps

Hazard maps have once been produced in 1984 when the landslide hazard inside the Favela do Pavao-Pavaozinho was assessed in response to demand by the Major of the City, who chose that favela to be the first one to receive special urban infrastructure.

In 1988 a landslide hazard mapping program was set down. First a medium-scale Landslide Susceptibility Map (LSM), scale 1:25,000, was prepared by means of a first generation GIS (simple combination of geological, slope, surficial deposits and land-use thematic maps) and assessed by landslide occurrence informations from administrative narratives and field control.

Hazards are represented very easily in the LSM. The high susceptible areas in red, the moderate in yellow and the low susceptible areas in green. This information is stored in data files inside a very simple GIS (fig.2). The map was intended to decision-makers to have an idea how landslides hazards are distributed and served as a guide to areas that received special attention.

Large-scale hazard maps started then to be prepared for the highly susceptible areas to landsliding. Map scales varie from 1:2,000 to 1:250 and updating of their content is needed every six months. Mapping procedures use informations from Local Landslide Inventories and are based on systematic inspections of individual slopes, in which their instability features, geometry, probability of landsliding and risk potential are detached. Hazards are represented in color as in the LSM. These special maps are intended to relief office individuals who are concerned to the implementation of emergencial disaster relief measures. In order to facilitate the identification of most critical risk areas these maps are accompanied by so many aerial photos as possible taken by helicopters.

The very large map scales and the continuously updating are the unique possibility to fit users' requirements and map hazards in very densely occupied slopes of brazilian large cities (Sao Paulo, Rio, Salvador, Recife and Belo Horizonte).
Fig. 1: Special engineering geological map for land-use purposes

- inadequate areas for housing development
- clayey soils prone to settlements
- clayey and sandy soils with low water table prone to floodings
- residual soils on steep relief, moderately susceptible to landsliding
- swampy and sandy soils with great environmental significance

- adequate areas for housing development
- sandy soils with deep water table and good bearing foundation conditions
- residual soils on gentle sloping, very low susceptible to landsliding

- bad developed areas calling for control measures
- urban agglomerations with problems of sewage discharge
- environmentally degraded by quarries activities
In the last ten years the consequences of natural disasters confirmed that vital information flow to solve the city problems in Rio de Janeiro is not satisfactory. Actually, the fact that 1988 landslides and floods caused 80 fatalities and led 4200 homeless people, a garbage deposit was located over a flooding area and that several buildings were located above swampy soils show that relations between scientists and planners have not improved too much in the last decade. Examination of available records shows that it was not ever the case.

The troublesome geological hazards in Rio de Janeiro have been known internally and worldwide since the end of the 60’s (Costa Nunes et al., 1973). After the creation of the Geotechnical Institute (IG) in 1966, it was common to find scientists, urban engineers and private companies participating together in landslides studies, which supported the implementation of successful methods of slope stabilization.

Misunderstanding of hazard potential took place however between 1974 and 1988, due perhaps to the reduction in extension and economic consequences of landslides and floodings hazards in this period. The cooperation between scientists and planners went down and the IG transformed to a simple department. The single trial to demonstrate political determination in mobilizing the technical knowledge was made in 1985, when engineering geological maps were presented to city planners. It went no longer because little has been done in bringing any background knowledge in geosciences.

In February 1988 torrential rains met a vulnerable city, as the encroachment upon the slopes during the 70’s and 80’s further aggravated unstable conditions. Hazards alarmed the society and pressured local administrators to improve the capacity of assessment of environmental problems. The necessity of establishing warning systems and hazard mapping was pointed out, the connections between urban geologists and planners were facilitated.
4.1 Urban Geology

The Land-use Plan for Rio de Janeiro City, voted in July 1991, directly benefited from the provoked interdisciplinary work. During its preparation city planners and geologists joined in Working Parties and examined the available hazard maps. Urban problems related to geology were outlined in several articles and technical products that should be produced in the future by state agencies were also included as a request for reducing the risk of hazards and avoiding development of unsuitable areas.

Examples of such procedures fostered by the Land-use Plan follow:

1- due to restriction for the exploitation of construction materials inside the city limits, the planners should establish a mineral zonation based on the informations from the medium-scale engineering geological maps;

2- landslide hazard maps are assumed as the basic criteria to assess decisions like house removing;

3- the Secretary of Social Affairs is responsible for improving the common understanding between city branches, urban geologists and planners;

4- flooding risk areas in the west zone have been attended by urgent control measures.

A Land-use Plan itself, despite its stimulating articles, is only a regulation and cannot occlude the historic disregarding of precautions and hazard triggering factors. The most important prerequisite to an effective hazard reduction is the involvement of public planners, decisionmakers and residents.

In order to reach this requirement urban geologists have tried to involve themselves in the transfer of expertise and to participate in every urban project in which specialist input is required, varying from the assessment of the environmental impacts from a planned hotel enterprise to the definition of the best localization of a recreational area. Of great importance has been the explanation of mapping results in the field to increase public awareness about the benefit of such information for reducing the hazard risk.

5 CONCLUSIONS

Engineering Geology is only a part, although important, of a multivariable process of administrating a large city like Rio, where social-claims are quite urgent. To solve all urban problems is utopic. What can be done and sounds a realistic goal to pursue is the reduction of the consequences of hazards and the restriction of creating new hazardous areas.

This paper shows the local capability for recognition of hazards and that the available material provides enough information for city planning. The next step is to reach a more satisfactory relationship between city planners and scientists.

REFERENCES


Engineering geological mapping of rock slopes using a laser-transit

Cartographie géotechnique des pentes rocheuses à l'aide d'un système 'laser-transit'

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ABSTRACT: Engineering geologists are often faced with the task of evaluating rock slope stability on inaccessible rock slopes. Ideally, for a complete evaluation, scale maps are needed, upon which the relevant features are portrayed. The laser-transit is a total station surveying instrument which projects a pulsing laser beam at the point of interest, and receives back sufficient reflected energy to enable the instrument to record the three-dimensional coordinates of the point. No reflective target is necessary. The data is then downloaded to a conventional CAD program and three-dimensional maps of slopes produced. These preliminary base maps are used for the preparation of slope stability zonation maps.

RESUME : Les ingénieurs-géologues sont souvent chargés d'étudier la stabilité de pentes rocheuses inaccessibles. Idéalement, pour obtenir une évaluation complète, il est nécessaire d'employer une carte à l'échelle sur laquelle toutes les caractéristiques sont indiquées. Le "laser-transit" est un instrument topographique complet en lui-même qui projette un rayon d'impulsion laser sur la région à étudier et reçoit une quantité d'énergie suffisante pour permettre à l'instrument d'enregistrer les coordonnées en trois dimensions de la région en question. Aucune cible réfléchissante n'est nécessaire. Les données sont alors transférées à un programme conventionnel CAD et des cartes en trois dimensions du relief de la pente sont obtenues. Ces cartes de base d'avant-projet sont utilisées pour la préparation des cartes de zonage indiquant la stabilité de la pente.

1 INTRODUCTION

Engineering geologists are often faced with the task of evaluating the stability of rock slopes which are inaccessible for the purposes of engineering geological mapping. The laser-transit allows the fast and safe acquisition of the data necessary for such evaluations. It also has the advantage that the engineering geologist can be trained in its use and that a rod-man (chain-man) is not necessary. Therefore, while the engineering geologist is conducting the survey, he can also perform his geological field observations. This is a distinct advantage over the use of photogrammetric techniques where the mapping activity is performed by a specialist consultant, leaving the geological field observations still to be done as a separate activity.

1.1 Laser-Transit

The Quarryman Laser-Transit is a total station surveying instrument and was used for engineer-
neously, vertical and horizontal angles are measured electronically, with tilt and turn sensors. Power is provided from a portable power pack and the data logger is mounted upon a leg of the tripod. Later models incorporate the data logger and battery into the body of the laser-transit.

The range is up to 400 m, with an accuracy of ± 20 cm. Options are available with accuracies of ± 1 cm and ± 10 cm. The accuracy of vertical and horizontal angle measurements are respectively 0.1 degree and 0.01 degree. The data logger electronic notebook can store up 1600 data points between downloads to a personal computer.

Surveying techniques are the same as those used for any other type of full station surveying instrument, excepting that a person holding a reflective device is not required.

1.2 Computer software and outputs

Any computer-assisted design (CAD) software packages such as Auto CAD or EM X S Surface Compiler, which has advanced digital terrain modeling capability, is suitable. The x-y-z coordinate data may be imported using DXF or ASCII files.

The desired computer output is an isometric view of the rock slope or face which is to scale. The isometric view is selected to enhance the three-dimensional effect. This allows the viewer to locate zones where features of interest such as talus, colluvium, planar surfaces, faults, bedding planes, joints, water inflow, vegetation, toppling blocks, etc. The resulting drawings may be described as rock slope stability zonation maps.

Sections can be produced at any desired location which yield the slope angle and width of benches. These sections can be used in rockfall simulation programs such as the Colorado Rockfall Simulation Program. (Pfeiffer 1989)

The engineering geologist can uniquely identify a set of coordinate data relating to a series of planes which potentially could form kinematically feasible wedges. These data sets can then be imported into a stereonet projection program for further analysis.

2 CASE HISTORY

An abandoned quarry, formerly used for the production of crushed limestone aggregate, was identified by the City of Bristol, Tennessee, as a potential site for a sanitary (household garbage) landfill. A consulting engineering company was retained through a competitive qualification process in order to conduct preliminary feasibility studies which included cost and constructability evaluations. Subsequently, more detailed site exploration was conducted, including hydrogeological studies and engineering geological mapping of the quarry walls.

2.1 Geological setting

The quarry is approximately oval in shape, with the long axis lying north-south as shown in the generalized site diagram (Fig. 1). The approximate dimensions of the quarry are 580 m long, 70 to 200 m wide, with sub-vertical to vertical walls 80 to 100 m high. The quarry was mined in Ordovician age limestone and dolomite rock (Bartlett 1971). A thrust fault, which is expressed as a thin, planar to slightly undulating surface, traverses the quarry. The thrust fault strikes at about 026° to 044° and dips range from 20° at the north end to 25° at the south end of the quarry. The thrust fault intersects the ground surface at the north end of the quarry rim. The formations below the thrust have essentially horizontal bedding in the north increasing to a 20° dip to the south. Two joint sets are present with orientations of 65°/56° (dip and dip direction) and 86°/34°

Above the thrust, the bedding is not readily apparent due to the folding which developed during the emplacement. Joint surfaces are very apparent in the southern half of the quarry walls, and are generally random in orientation with surfaces daylighting into the quarry on both east and west walls. Initial studies identified the quarry walls above the thrust fault as problematic with respect to stability, access, and location; particularly as the landfill operation would require that work crews would spend long periods directly beneath these walls.

2.2 Rock slope stability

Rockfalls from the quarry sidewalls are caused by several external and, in some cases, internal forces. The behavior of the falling or sliding rock is dependent upon several controlling factors. In this section, rock slope stability is considered to consist of two basic failure modes, that of rock falls and rock slides. The causes of both failure modes are
listed in Table 1. The factors influencing rock fall behavior (McCaulley 1985) are presented in Table 2. Those factors influencing rock slides are presented in Table 3.

Table 1. Causes of rock falls and rock slides

<table>
<thead>
<tr>
<th>Factor</th>
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<tbody>
<tr>
<td>Rainfall</td>
</tr>
<tr>
<td>Geologic Features</td>
</tr>
<tr>
<td>Differential Erosion/Weathering</td>
</tr>
<tr>
<td>Swelling Clay</td>
</tr>
<tr>
<td>Freeze-thaw</td>
</tr>
<tr>
<td>Wind</td>
</tr>
<tr>
<td>Channeled Runoff</td>
</tr>
<tr>
<td>Snowmelt</td>
</tr>
<tr>
<td>Springs &amp; Seepage</td>
</tr>
</tbody>
</table>

2.3 Rock sliding potential

Rock slope stability analysis considered two basic failure modes; that of sliding and toppling. The two types of potential sliding failures noted on the panels are wedge and planar. Wedge failure indicates that a rock mass defined by two intersecting planes has the potential of sliding into the quarry. Planar features, which may create this condition, include joints, bedding planes, or other planar discontinuities. Planar failure indicates that a rock mass, defined by a single discontinuity surface which intersects the quarry wall, has a potential of sliding into the quarry. To ascertain the potential for sliding, limit equilibrium analyses (Hoek 1977) must be performed considering the shear strength and the presence of water pressure in the discontinuity. The discontinuities observed on site appear to be primarily healed with calcite, which would result in a stable condition as long as the calcite remains unweathered. The greatest potential for sliding failure occurs above the thrust fault on the east, west and southeast walls in the south half of the quarry.

Toppling failure generally results from a sub-vertical to vertical pile of rock blocks becoming kinematically unstable. Examples of this are found near the quarry’s necking feature from the floor up to the first bench and along the quarry rim on the east wall where a weathered zone is present.

2.4 Rock fall potential

The following five classifications of zones were used on the maps for potential rock falls.

Colluvium: In most cases the zones of colluvium were covered with grass, weeds, and small shrubs. Unless the colluvium had formed into a slope greater than the angle of repose of the soil material, then slope failure was unlikely.

Talus: Where talus was present in the quarry, it was usually a stable mass except where perched near the rim of a bench. It can, however, be a source of rock fall comprising single or several
blocks. The largest amount of talus had collected on the abandoned west access road.

Rock Blocks: Rock blocks may be considered as talus, however it was one of the most significant sources of potential rock fall. Rock blocks were scattered on the colluvial slopes, primarily above the thrust fault. A large pile of rock blocks was present in the southeast corner upon the wide bench. The source of these rock blocks was from the above sub-vertical to vertical slope which was being progressively undermined by the weathering out of the thick lens of the Athens Formation shale. The shale is present just above the bench and extends for several hundreds of feet sub-horizontally.

Weathered Rock: Weathered rock observed on the quarry walls may be caused by solutioning. The most significant zones of weathered rock occur around the rim of the quarry where it was formed in the Chepultepec Formation. This appears to be a major potential source of high energy rock falls on the eastern rim of the quarry.

Rock Fill: Loose rock was placed as a rock fill to construct access roads to grade. There also appear to be areas of rock fill on benches which are no longer related to access roads. Large boulders are often located along the edge of the access roads and are sitting on this loose material. Storm water runoff on the access roads has created washed out gullies in the rock fill. Because of its location, looseness, and extent, the rock fill provides a source of rock falls to significant sections of quarry wall.

2.5. Engineering geological mapping

Two engineering geologists were trained in the use of the laser-transit in one day at the quarry site. Three known points had been established on the quarry floor for an aerial survey. These points of known coordinates were then used as a reference to establish twenty three points on a closed traverse around the quarry. The objective was to provide stations of known coordinates from which surveying of the walls could be done. This effort also required the delineation of sections of wall (panels) which could be easily viewed from a single location. The extent of each panel was marked with red paint where accessible at its base.

As with any total station survey instrument, the laser-transit was set-up over a known point, the system "zeroed" and a back-sight taken. The coordinate data acquisition of the rock face simply required pointing the laser (which is not visible) using the cross-hairs on the mounted telescope and pressing a button on the data logger. An audible signal indicates that the point has been recorded. Nothing can be allowed to obstruct the line of sight (e.g. tree branches) as this will interfere with the laser beam. The operator can simply traverse horizontally and vertically across the panel with data points at say, 2.0 m centers which allow coverage of a slope 60 m wide x 100 m high, with approximately 1,600 points, before downloading is necessary. A lap top computer was used for this purpose.

The data logger allowed the operator to uniquely identify significant features such as the thrust fault, benches, a shale bed, large planar surfaces, etc. with its own set of data points.

As the work proceeded, the engineering geologists took photographs and conducted field observations of the rock slope conditions: e.g., the rock type present; the degree of weathering and solutioning; the character of joints, bedding and faults; and the nature of groundwater seepage and flow. The field mapping effort required a total of thirteen, ten-hour man days to map approximately 1400 m of quarry wall with an average height of 90 m.

2.6 Production of rock slope stability zonation maps

The following uses were planned for the engineering geological maps produced. They would provide the basis for recording the nature and extent of potential rockfalls and slope instability; the necessary mitigating measures and their extent would be shown on the plans, thus providing the basis for cost estimates and production of working drawings for actual slope stabilization work.

Because the landfill would necessarily come into operation incrementally, it is important to relate the costs of slope stabilization and revenue stream. Therefore, slope monitoring of the initially unused areas of the quarry would be recorded using the maps. This would provide the data for the design of future remediation measures.

Examples of the maps produced are shown in Figures 1 and 2. Figure 1 shows a map of Panel 2
which was where the quarry wall was inset at right angles, as shown on the Generalized Site Diagram. Also shown is a section profile of the slope. Figure 2 shows a map of Panel 4 located at the north end of the quarry which is planned to be the initial site of landfill operations. Initial slope stability evaluations indicated that the slopes above the thrust fault would be the most problematic as they could potentially provide high energy rock falls to the floor of the quarry. A review of Figure 2 verifies this and provides the bases for design of remediation measures. Also apparent is the extent of rock fill used to construct the haul road. This was subject to wash-out and subsequent undermining of boulders placed as barriers. Therefore, it was apparent that this fill would need to be removed and properly compacted, graded and drained.

The first part of the permit application process was approved. The next phase of the design is currently underway in which the remediation measures will be identified and detailed on the plans.

3 SUMMARY

The use of the laser-transit has proved to be a safe, quick and cost-effective tool when used by the engineering geologist for field mapping of otherwise inaccessible rock slopes. The x-y-z coordinate data gathered may be readily transferred to existing CAD programs enhanced with advanced Digital Terrain Modeling (DTM) capability to allow production of isometric base maps upon which engineering geological data can be portrayed. These base maps then become the design and construction drawings for the necessary remediation and monitoring measures.

The method would be useful for highways in mountainous terrain, subject to slope stability problems. The laser-transit mapping could be done from the highway. Mapping of tens of kilometers of highway in this manner is feasible when viewed as the first step in a long-term maintenance program, the aim of which would be to budget and rationalize expenditures over a five- to 20-year time span.

REFERENCES


ACKNOWLEDGMENTS

The author wishes to acknowledge the project design consultant, STS Consultants, Ltd., and their project manager, Mr. Douglas Herman. Also, Mr. William Dennison, Director of Public Works of the City of Bristol, Virginia, whose imagination and drive made the project possible.
Sedimentary history and geotechnical characteristics of Umeda Formation (Holocene) in the central Osaka Plain, Southwest Japan

L’histoire sédimentaire et les caractéristiques géotechniques de la formation Umeda (Holocène) dans la plaine d’Osaka, sud-ouest du Japon

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ABSTRACT: This paper presents results of a recent geological study of the Holocene deposits in the Osaka Basin, Southwest Japan, especially at the central area around Uemachi Upland. The study was based upon the analysis of a large number of borehole data compiled in the computer database. $^{14}$C datings of fossils sampled by boring and excavation sites were carried out at several key sites and used to interpret the processes of deposition. The Holocene deposit, named the Umeda Formation, is subdivided into three layers by facies. Based upon $^{14}$C dating, the marginal facies of sand and gravel layers around the Uemachi Upland were identified as Holocene deposits. The subground structures in the Osaka Plain were obtained in vertical sections; they show not only Holocene but also Upper Pleistocene structures in the area. Other tectonic structures like flexures as well as geotechnical aspects are also discussed.

RESUME: Cet article présente les résultats d’une étude géologique récente sur les Dépôts Holocène du Bassin d’Osaka, en particulier dans la zone centrale autour des collines de Uemachi. L’étude est basée sur l’analyse d’un grand nombre de données de sondages compilées dans une base de données. Des datations $^{14}$C de fossiles échantillonnés par sondages et excavations ont été réalisées dans divers sites clés, et utilisées pour interpréter les processus de mise en place. Les dépôts Holocène, intitulés Formation Umeda, sont subdivisés en trois couches selon les facies. Selon les datations $^{14}$C, les facies marginales de couches de sables et de graviers autour des collines de Uemachi ont été identifiés comme étant des dépôts Holocène. Les structures du sous-sol dans la Plaine d’Osaka ont été représentées en sections verticales; elles révèlent non seulement des structures Holocène mais aussi du Pléistocène Supérieur dans le secteur. D’autres structures tectoniques telles que des flexures ou certains aspects géotechniques font également l’objet d’une discussion.

1. INTRODUCTION

The underlying, unconsolidated formation in the Osaka Plain was defined as the Umeda Formation (Holocene) by Yamane (1930). A great deal of effort has been made to detamine the stratigraphy of the formations underlying the Osaka Plain (Ikebe et al., 1970; Maeda, 1976; Furutani, 1978). This formation has been recognized as a problem from the standpoint of the civil engineering; its identification was based on standard penetration tests (N-value) (The Publication Committee of the Ground of Osaka, 1966; Miki et al., 1987).

The author has investigated the formation in the West Osaka area of the Osaka Plain (Figure 1, 2) with the database system of borehole data (Hayashida et al., 1987a; Iwasaki et al., 1990; Yamamoto et al., 1992) and $^{14}$C dating (Mitamura, 1991; Mitamura et al., 1992; Mitamura and Miki, 1992). This paper presents the results of detailed studies of the stratigraphy and the geological characteristics of the Holocene formation in this

Figure 1. Generalized geologic map around the Osaka Plain (adapted from Ithara et al., 1991 and Tanaka et al., 1982).
2. ALLUVIAL FORMATION

The Alluvial Formation was at first defined as Holocene and named the Umeda Formation by Yamane (1930). Since then, the Publication Committee of the Ground of Osaka (1966) sorted the Alluvial Formation with the civil engineering properties of the strata, for example, sands of less than 30 in N-values and unconsolidated clays. Hence, the formation is included with the Upper Pleistocene deposits from the Würm Maximum Stage (about 20000 years ago in age). Almost all the $^{14}$C ages of the formation are younger than 10000 years, with the exception of the peaty beds which are from 20000 to 10000 years old (Kajiyama and Ithara, 1972). From this viewpoint, the Alluvial Formation in the Osaka Plain was defined as the Namba formation (Kajiyama and Ithara, 1972) and the Osaka Bay formation (Maeda, 1976), which are included in the uppermost Pleistocene.

2.1. Stratigraphy

The Umeda Formation comprises the Holocene deposits which are piled up in the bay under coastal sedimentary environments by the Holocene transgression. The type locality of this formation is the area from the OD-1 site (Minato-ku Tanakamotomachi) to Kita-ku Umeda and ranges less than 30 m in depth. The formation is divided into three members: the Lower alternation (the alternation of peaty clays and sands), the Middle clay bed (soft marine clay bed), and the Upper sand bed (well sorted loose sand bed), in ascending order (Figure 4). These typical facies extend in the western area of a line from Nishiyokobori to Umeda. In contrast to this, the Middle clay bed changes into gravels and sands in the marginal area of the Uemachi Upland. So, the formation is not able to be divided in this area. The formation overlaps the Upper Pleistocene Formation (including the Tenma Formation, Ma11, and Ma12 bed; Ikebe et al., 1970) and the Osaka Group (Plio - Pleistocene; Ithara et al., 1984) in the lowland area, and abuts the basement formation in the Uemachi Upland (Figure 5).
facies. In this paper, the facies in the type locality is defined as the main facies, the gravelly facies is the marginal facies.

2.2.1. Main Facies
The main facies of the Umeda Formation is characterized by the Middle clay bed which is more than 10 m in thick.

The Lower alternation of the Umeda Formation unconformably overlies a gently rolling upper plane of the Tenma Formation, the alternation varies markedly in thickness. On the other hand, the boundary between the Lower alternation and the Middle clay bed is a flat plane, which forms OP-20 m in height (OP: abbreviation for Osaka Peil, altitude above standard sea level in Osaka Bay) and dips to the coastal area. The distribution of the Lower alternation and its peaty facies shows the brackish and coastal sedimentary environments at 7000 years B.P.

The Middle clay bed horizontally overlies the Lower alternation and ranges from OP-20 m to OP-8 m. The clay bed thickens to the coastal area, but thins towards the marginal area of the Umemachi Upland and intertongues with gravels and sands of the marginal facies.

The Upper sand bed conformably overlies the Middle clay bed and is 10 m in thick. Along the Muko River, the Kanzaki River, and the Aji River, this bed shows an increase in grain size. In the Teama area, especially, the Upper sand bed becomes a gravelly bed.

Figure 6, which is compiled from borehole data at 13000 sites in the Osaka Plain, shows the contour map and the isopach map of the Umeda Formation. The contour map of the basal plane of the Umeda
Figure 5. Geological profile of the Osaka Plain.

Figure 6a. Contour map of the base of the Umeda Formation (altitude from OP in meters, adapted from Matsuyama et al., 1992).

Figure 6b. Contour map of the base of the middle clay bed (altitude from OP in meters, chain line: marginal line of the middle clay bed, adapted from Matsuyama et al., 1992).

Figure 6c. Contour map of the base of the upper sand bed (altitude from OP in meters, chain line: marginal line of the middle clay bed, adapted from Matsuyama et al., 1992).

Figure 6d. Isopach map of the Umeda Formation (thickness in meter, adapted from Matsuyama et al., 1992).
Formation (Figure 6a) signifies the paleotopography before the deposition of the Umeda Formation. The paleotopography shows a broad, southwest-trending valley of the Paleo-Yodo River, the broad north-trending Uemachi Upland, and valleys of the Neya River, Nagase River, and Hirano River in the eastern part of the Osaka Plain. The contour map of the basal plane of the Middle clay bed shows the Paleo-Yodo valley across the northern area of the Uemachi Upland, a broad, gently undulating valley in the east area. The valley of the Paleo-Yodo River is shifted to the north by a bank running north and south at the stage of the deposition of the Upper sand bed (Figure 6c). In this stage, the valley in the east area is reduced.

2.2.2. Marginal Facies
The Umeda formation, which is composed of three facies, is modified at the marginal zone near the Uemachi Upland. Figure 6e shows the distribution of the Middle clay bed. This distribution is in accord with the isopach contour of the Umeda Formation's 10 m in thickness (Figure 6d). Hence, the distribution of the marginal facies, which is the coarse facies of the Umeda Formation less than 10 m in thick, is consistent with the outside of the middle clay distribution. In the marginal zone, the middle clay begins to inter-finger with sand and gravel layers, and finally fades out. The depository environment is also shown as intertidal to the sandy shore-face zone, based upon fossil considerations.

2.3. Geological Structure
The investigation about the distribution and facies of the underlying formations in the Osaka Plain with the boring database system has revealed the geological structure in detail, so that the two flexures which accompany the Uemachi Fault were confirmed. Figure 8 summarizes the recognized geological structure in this study.

Uemachi Fault The Uemachi Fault runs north and south under the ground. The east side of the Uemachi Fault is elevated. So, along this fault, the Pleistocene formations steeply dip west and are cut by the fault. The deep boring investigation revealed the vertical displacement of the Azuki volcanic ash layer in the Osaka Group by the fault to be about 400 m ( Ikebe et al., 1970 ). The recent investigation with the seismic reflection prospecting and the data of the deep well revealed that the vertical displacement of the basement rocks is about 800 m. This fault tilted the Ma12 in this area. In the northern part of the Uemachi Upland, the clay bed correlated with the Ma12 is interbedded by the Uemachi Formation (Furutani, 1978). The vertical displacement between this bed and Ma12 is about 50 m. These facts indicate the continuous activity of the Uemachi Fault from the Pliocene to the Upper Pleistocene.

Sakuragawa Flexure This flexure runs northeastward from Sakuragawa to Tenjinbashi. Along this flexure, the Ma12 bed dips northwest, and the main facies of the Umeda Formation becomes the marginal facies. In Tenjinbashi, this flexure joins the Uemachi Fault. In Sakuragawa, the vertical displacement of the Ma12 by the flexure is about 20 m. In this area, Ma12 and the underlying Pleistocene dip into a poorly delineated northeast trending anticline. The displacement appears to diminish southwestward, and it may die out in this direction.

Suminoe Flexure This flexure runs NE-SW, from Tamade to Hirabayashi. Along the flexure, the Ma12 bed dips northwest. The vertical displacement of the Ma12 is more than 25 m in the northern part, and increase southwestward; in the Hirabayashi area it is about 50 m. The northeastern part of the flexure joins the Uemachi Fault in the Tamade area.

3. PALEOTOPOGRAPHY
The marginal facies of the Umeda Formation occurs along the Uemachi Fault and the two flexures. The author thinks that the tectonic landscape resulting from the activities of these faults and flexures affected the sedimentary environment of the Umeda Formation. On the other hand, the Holocene transgression was the direct factor of the deposition of the Umeda Formation. The relative sea-level change in Holocene times ( from 10,000 to 6000 years B.P.), was a rapid rise, which came to the peak at 6000 years B.P. ( Ota et al., 1990 ). In the Osaka Plain, the sea level in Holocene times was at a peak from 6000 to 5500 years B.P. ( Maeda, 1976 ). The result of the investigation about the distribution and lithofacies of the Umeda Formation,
Stage IV (2000 years B.P.): The sea level descended to the Recent sea level. The bay mouth bar of the north side of the Uemachi Upland closed the inner bay, so that the inner bay area changed gradually to a lacustrine environment. This stage is correlated with the stage of the Kawachi Lagoon (Kajiyama and Itihara, 1972).

4. GEOTECHNICAL CHARACTERISTICS AS URBAN GROUND

The Uemachi Formation, which forms the alluvial plain, is composed of soft clays and loose sands, which are recognized as weak materials. The flat ground formed by the deposition of the Uemachi Formation is used by man. The ground water, for example, which is present in the sandy aquifers, is used as drinking water and industrial water. Buildings of less than four stories are supported by direct foundations in the Uemachi Formation.

However, the Uemachi Formation is a normally consolidated clay bed, so that this layer easily contracts in volume by dehydration. Hence, the excessive use of ground water has caused ground subsidence. The geotechnical properties of the Uemachi Formation such as the ground subsidence and the consistency, are discussed below.
4.1. Ground subsidence

In the early 1930's, ground subsidence was recognized in the western part of Osaka City. Since then, the level of the ground-water table has gradually been lowered and the ground-subsidence area has expanded to cover the entire urban district of the Osaka Plain. In the early stage of the subsidence, the excessive use of the ground water from the shallow aquifers such as the Tenma Formation induced the shrinkage of the Middle clay bed in the Umeda Formation. Later, the use of the deep aquifers by the high-performance pumps produced the shrinkage of the Pleistocene clayey formations, so that the ground subsidence area was expanded (Ikebe et al., 1970). The cumulative subsidence of the ground surface for 50 years, from 1935 to 1985, reached about 3 m. Figure 8 shows the distribution of this cumulative subsidence and tectonic lines in Osaka City.

The subsidence in the western area of Osaka City is larger than that on the east side of the Uemachi Upland. These areas correspond to the distribution of the middle clay bed of the Umeda Formation. In the Uemachi Upland, a relative small shrinkage zone extends north and south. The sudden dip zone of the contour map on the west side of the Uemachi Upland runs along the Uemachi Fault and the flexures.

The distribution of the subsidence is affected by the distribution of the Middle clay bed and the Pleistocene clayey formation, the deposition of which was regulated by the geological structure.

4.2. Grain size and consistency properties

The alternations of water content influence the properties of sediments. The Atterberg limits, given by the consistency tests such as the liquid limit, plastic limit and shrinkage limit, are the indices which indicate clayey sediments. These factors are closely related to the grain-size distributions of the sediments.

The tendency of clay contents, water contents, and liquid limits increase in common to the middle part of the clay bed. Figure 9 shows the distributions of
Umeda Formation of the Holocene deposits around Uemachi Upland in central Osaka Plain were obtained, described below.

*Stratigraphy:* The Umeda formation was formed by the Holocene transgression and is represented by bay mud and littoral sandy deposits. The total thickness of the Umeda Formation is about 30 m in this zone, which can be divided into three parts, based upon geological facies. The lowest member, about 3 m thick, is composed of sand and clay with organic matter; its $^{14}$C age is 7000 to 10000 years B.P. The middle member is marine clay, 10-15 m thick with its $^{14}$C age of 2000 to 7000 years B.P. The uppermost member is loose and coarse sand, 5-10 m thick, with its $^{14}$C age of less than 2000 years B.P.

The Umeda Formation becomes shallower in the marginal zones around the Uemachi Upland area. In the marginal area, sand and gravel layers were deposited at the surface. The main facies of clay was shown to inter-finger with sand and gravel layers in these marginal zones. The fossils in the marginal zones of the Umeda formation indicate that the depository environment was intertidal to sandy shore-face. The sand and gravel layers in the marginal zones were deposited during the Holocene transgression and belong to the Umeda Formation.

*Geological Structure and Paleotopography:* Along the western margin of the Uemachi Upland, the Uemachi Fault extends north to south; but its location was not identified in detail. In this study, many geological profiles were investigated with the urban ground database system.

The Uemachi Fault extends northwards to the Butunenji-yama Fault in the Senri Hills and southwards to the western area of Sakai City. In addition to this, two NE-SW flexures were identified in the at Suminoe and Sakuragawa areas. Along these flexures, the Ma12 bed dips steeply west to northwest, and the marginal facies of the Umeda Formation abuts on the tilted Pleistocene formation.

By the detailed investigations using many geological profiles and $^{14}$C dating, the paleotopographical maps ranging from the lower part to the upper part of the Umeda Formation were constructed. The paleotopographical maps of the lower part (stage I to II in Figure 7) were described for the first time. Tectonic lines have affected the paleotopography at the preceding stage of the Umeda Formation (10000 years B.P.). At the stage of the middle clay member of this formation, a coastal zone was formed along this zone.

*Geotechnical Characteristics as Urban Ground:* In the main urbanized area of the Osaka Plain, the ground surface had subsided 1-3 m, in 30 years (between 1930's and 1960's) by the excessive pumping of ground water. Using the database system about the ground subsidence in Osaka City,
the distribution of the cumulative displacement was arranged on a map. This distribution is in harmony with the distribution of the middle clay member. The inflection zone of the cumulative displacement extends along the above tectonic lines.

The consistency properties and the clay content of the middle clay member decrease eastward. These distributions show the effects of the sedimentary environments in this area during the Holocene transgression.

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REFERENCES


Engineering geological mapping at Leme region (State of São Paulo, Brazil) based on Zuquette (1987) methodological proposition

Cartographie géotechnique de la région de Leme (État de S. Paulo, Brésil) fondée à la proposition méthodologique de Zuquette (1987)

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ABSTRACT: The Zuquette (1987) methodological proposition was a trial for establish engineering geological mapping methods for Brazilian conditions. This paper focuses the first effort to apply this methodological proposition on regional scale, and was developed in "Leme Region" (in the central-east part of São Paulo State, Brazil) at 1:50,000 scale on an area that measures 717,43 km².

RÉSUMÉ: Zuquette en 1987 a proposé une nouvelle technique dont l’objectif est établir des méthodes de cartographie géotechnique pour les conditions brésiliennes. Le but de cet article est de présenter, dans une échelle régionale, une première tentative d’application de cette nouvelle technique. Le site proposé a été celui du Leme, situé dans le centre-ouest de l’état de São Paulo - Brésil, à une échelle au 1:50.000 sur une surface d’environ 717km2.

1 INTRODUCTION

This study was the first effort to apply the Zuquette (1987) methodological proposition on regional scale, and was developed in "Leme Region" (in the central-east of São Paulo State, Brazil) at 1:50,000 scale on an area that measures 717,43km².

The scope of this study was test the efficacy of the methodological proposition for the regional conditions and the chosen scale, and to present comprehensive special purpose maps that shows the conditions under some obligation in front of the works kind and considerable occupation.

Observational maps (such topographic, bed rock, soil units, surface drainage, documentation and steepest slopes) were used for prepare seven comprehensive maps (erodibility, excavation difficulty, waste disposal suitability, construction natural resources, slope stability, subsurface installations suitability, and road installation suitability). Among this maps, will be here presented only the maps of bed rock, soil units, steepest slopes, erodibility, excavation difficulty, and waste disposal suitability, that which be considered the most important in this area.

2 METHODOLOGICAL PROPOSITION

The Zuquette (1987) methodological proposition was a trial for establish engineering geological mapping methods for Brazilian conditions, were it stands the knowledge of engineering geological mapping in other countries and the difficulties that appeared during their adaptation to Brazilian territory.

2.1 Principles

The fundamental principle of the methodological proposition is the utilization of objective, low cost and rapid methods for information survey, and the attribute classification through naturalistic characteristics.

The obtention and observation of this attributes was based in three kinds of works: (1) foregoing geological and geotechnical works; (2) remote sensing; and (3) field works. The basic documents is that ones consulted (or produced)
for the obtention of environmental information, permitting the elaboration of interpretative maps.

2.2 Methods of work

The work sequence include four steps: (1) obtention and analysis of basic maps; (2) elaboration of auxiliary maps; (3) production of interpretative maps; and (4) presentation of geotechnical conditions in comprehensive maps. The basic maps are topographic, geology, surface drainage, pedology, geophysics, geomorphology, climate, and occupation maps. The really elaborated maps auxiliary map is the steepest slope map, according to gauge methods of De Biasi (1970), that represents the inclination of terrain in terms of percentage, considering five classes (for regional surveys): <2%, 2-5%, 5-10%, 10-20%, >20%. Other auxiliary map is the documentation map in which is presented the information about foregoing geological and geotechnical works, include laboratory and "in situ" tests. The groups of comprehensive map considered in the methodological proposition are General Purpose Geotechnical Zoning Map, Specific Purpose Geotechnical Zoning Map, General Planning Map, and Construction Conditions Map. Among this four types, the most applicable are Specific Purpose Geotechnical Zoning Maps, that represents the evaluation of the area based on an specific use proposition.

For the geotechnical condition analysis will be necessary foregoing information, and others information obtained from laboratory tests and estimate properties (in the case in which extensive laboratory tests were expensive).

For the soils units the methodological proposition foresee the following tests: granulometric distribution, atterberg limits, proctor normal compaction, mini-MCV, and mineralogy. For rock materials: density, mineralogy and resistance. The estimate properties are: permeability, compressibility, expansibility and resistance.

3 APPLICATION

3.1 The Leme Region

The Leme Region in localized in the central-east part of São Paulo State between the meridians 47°15' and 47°30'W and the parallels 22°00' and 22°15'S, corresponding to the "Folha de Leme" at 1:50,000 scale (IBGE 1971). The climate is, according DAEE (1981), wet subtropical, and the area is ecologically localize (Setzer 1945) in the "Hot Palaeozoic Depression". The relief is wavy and gentle.

According Mello (1979) the hydraulic regime is characterized for to present three typical periods during the year: (1) rain period between October and march; (2) arid period between June and September; (3) and transition period April and may. The natural vegetation is composite for savanna and tropical forest, but more than 95% of this vegetation was removed for agricultural activities.

The rock units presents pertain to the sedimentary sequences and basic intrusives of Paraná Basin (Kaefer 1979).

The basic intrusive rocks are basalt, and the sedimentary units are sandstone (Botucatu and Pirambóia formations), silt stones (Corumbatá and Tatui formations) and mudstones (Iráti formation). Due the tropical weathering and sediment transportation processes the area have three sediments units (two arenaceous and one argilaceous) and six soil units (three arenaceous and three argilaceous).

3.2 Results

The area zoning in terms of bed rock units are presented in the map 1. The represented units are: basalt; Botucatu formation (fine sandstone); Pirambóia formation (argilaceous sandstones); Corumbatá formation (silt stones); Iráti formation (mudstones); Tatui formation (silt stones).

In the map 2 could be observed the soils and sediments units: three sediment units (arenaceous alluvial sediments, arenaceous colluvial sediments, and argilaceous colluvial sediments) and six soil units (argilaceous
residual soil from basalt, Irati and Corumbataí formations; and arenaceous residual soils from Botucatu, Pirambóia and Tatuí formations).

The steepest slopes (map 3) shows the zoning of the studied area in terms of five classes: <2%, 2-5%, 5-10%, 10-20%, >20%.

For elaborate the erodibility map will be considered the attributes: nature of soil materials, slope profile, slope form, slope extension, steepest slope, and occupation. The area was divided into three classes: high erodibility (soft arenaceous soils in long and convex slopes occupied for annual agricultural activities or natural vegetation); medium erodibility (structured arenaceous or argilarenaceous soils in convex medium slopes, occupied for annual agricultural activities); low erodibility (argilaceous soils in convex and concave slopes, occupied for annual agricultural activities). The representation of this zoning is presented in map 4.

For excavation difficulty analysis was considerate the attributes: nature and thickness of soils and sediments, subsurface deep water, steepest slope, and bed rock characteristics. For this zoning were considered three classes: suitable (soil thick larger than 5m, steepest slopes lesser than 10%, subsurface level water larger than 5m); reasonable (soil thick between 2 and 5m, steepest slope between 10 and 20%, and subsurface level deep water between 2 and 5m); and inadequate (soil thick lesser than 2m, steepest slope larger than 20%, and subsurface level deep water lesser than 2m). This zoning is presented in map 5.

For waste disposal suitability analysis the considered attribute are subsurface deep water, bed rock deep, nature and thickness of soils and sediments, and steepest slopes. Three classes of suitability was considered: suitable, reasonable, and inadequate. No suitable areas were identified, but the reasonable ones may be used for this finality using corrective techniques. The reasonable class include areas with the characteristics: subsurface water and bed rock deep bigger than 5m, steepest slopes between 2 and 10%, and soil permeability between 10<sup>3</sup> and 10<sup>-5</sup> cm/s. The inadequate class includes areas with at least one of the attributes restrictive. The representation of this zoning is presented in map 6.

4 CONCLUSIONS

The application of this methodological proposition, in the studied area, produce good results in terms of geotechnical zoning, but some observations may be done, for better future use: (1) the adopted scale (1:50.000) would be consider administrative or phisiogetic limits, in terms of facilitate the information access; (2) the used attributes in suitability analysis would be hierarqueze for permit more rapid analysis; (3) some interpretative maps proposed in the methodological proposition (such foundation suitability) only could be elaborated in major scales.

5 REFERENCES


6 GRATEFULNESS

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Engineering geological mapping of São Carlos city – State of São Paulo (Brazil) at 1:25,000 scale

Cartographie géotechnique à l’échelle 1:25,000 de la ville de São Carlos (São Paulo, Brésil)

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ABSTRACT: In Brazil most of urban centers present geological and geotechnical problems, and, most of them, are a result of unsuitable establishment of different occupations. If there were engineering geological maps that oriented the establishment of occupations, as well as the necessary specific investigation, these problems could be avoided. In Brazil there are few areas where engineering geological maps were executed. As an example of their importance there is the work done in the region of São Carlos (State of São Paulo-Brazil).


1 INTRODUCTION

Engineering geological mapping has been used as essential tool in various areas in the late years, specially with respect to geographical space ordination. Zuquette (1987), by critically analyzing the methodologies used in developed countries in Europe, the USA, Canada, Australia and others, proposed the adequation of one of them which shows to be compatible with Brazilian social, economical and geographical conditions.

The aspects related to the physical environment and to the situations arising from antropic action where verified during the execution of the work in order to supply geotechnical subsidies to accomplish an adequate planning or a specific utilization of urban areas. The fourteen documents which were prepared with this purpose and performed by Aguiar (1989) in a 1:25,000 scale, had as basic objective to contribute for the improvement of the methodology used and to assist the previous orientation for the expansion of São Carlos Urban area.

Due to space limitation only two cartographic documents are presented as examples.

2 METHODOLOGY

According to the principles of the current methodology, evaluation studies and classification of the geotechnical units were based on the utilization of all elements given by maps and other available parameters, in order to assist the consecutive stages which are essential to the work: previous information survey and
analysis, attribute recognition and final identification of the homogeneous units.

The attainment of attributes represents an essential stage of the work, planned and accomplished in sufficient number and duration in order to solve all doubts. According to Zuquette (1987) the most important ways to obtain information are: published works, utilization of aerial photography and others images, field work, laboratory tests and material geotechnical properties estimation.

The acquired data in previous works of the specific field should be analyzed with special care, first of all because they condition almost the whole geotechnical mapping and because of the criterious choice of available data (reliance), in order not to prevent the definition of homologous units when final interpretation takes place. Having this information the operalization of the processes for the elaboration of a preliminary map could be achieved. This processes is different from those usually realized in conventional geological mapping, presenting two stages, one of photo identification and the other more elaborated, the photo interpretation. Afterwards, it was attempted to catalogue, in the field, the attributes, realizing surface investigations (profiles/pathways) as the most usual way to obtain them, according to Aguiar (1989).

By the next probation, with the entirety sight obtained after the field works along with the photo interpretation, it could be defined previous characteristics of the geotechnical preliminary units, managing in this way, the sampling stages and new observations to points that kept obscure.

The material properties, in depth, were only obtained in natural or antropic slopes (road cross-section, and digs, etc.) and in the few simple recognition explorations, that took over an important distinction the study.

All the soil samples were classified according the rules adopted by the Highway Research Board (H.R.B.) and the Soil Unified Classification System (S.U.C.S.) - ASTM, D2487/83 - and by the systematic proposed by Villibor (1981) for compacted materials used in the pavement basis.

These samples tested in minis-MCV and "Immersion loss" in MCT classification (Nogami & Villibor 1981) were considered.

When both evaluation and classification were concluded, followed the interpretation of the available data (explorations, analytical results from other works and those ones found in this study), for the identification of the geotechnical units of homogeneous behavior by using, once more, the aerial photography, ending with the elaboration of the basic maps, of the thematic charts and of the current text.

It is useful to point out the importance of using computers, during the biggest part of the work, giving distinction to the utilization of the geographical information system - GIS that permits fast and accurate alterations inclusions, participating as in the simple typing of the cartographic documents as in the inter crossing of the various discussed attributes.

3 STUDIED AREA

3.1 Localization

The parallels 21°57'01" and 22°05'00" south and meridians 47°49'24" and 47°57'13" west are the area limits whose engineering geological maps were realized. It is part of SF-23-V-C-IV-3 S0 & SE) and SF 23-Y-A-I-1 (N0 & NE) sheet municipal district. It is totally located in São Carlos, region of the São Paulo State and covers 186 Km² surface.

3.2 Geotechnical units map (Figure 1)

This map includes the various types of unconsolidated materials identified in the area, defined from the laboratorial and field works (including the previous ones), the existent exploration and also by the photo interpretation. Besides the contacts among the different units, represented by distinct hachures, there are
approximate limits of substrate isoprodun-7ity, either geological or rock; the latter referring to the treatment of residual products.

Types of material - Making use the obtained information, it was possible to set up the nine units of unconsolidated materials, that were subdivided in two groups:

1. Residues material - correspond to materials deriving from rock alteration, without having suffered any kind of transportation, which kept themselves on the matrix rock. They are Botucatu residuals, Basic magmatites residuals and Bauru residuals.

2. Transported materials - are those present reworking and significative transportation evidences and the organic; they were subdivided in sandy material I, II and III, alluvial material, colluvial material and organic material.

It is possible to observe in Table 1 the variation of several characteristics of each one of the main homogeneous geotechnical units defined in the area.

3.3 Chart to orientation (Figure 2)

Resulting from the inter crossing analysis of the basic documents, this kind of thematic chart shows the characteristics of the physical environment from the geotechnical point of view.

The potential evaluation and/or area geotechnical restrictions, seeking the best utilization of its resources, constitute one of the forms to enable the use and rational occupation of the space when joined to a politics of global planning.

According to the methodology proposed by Zuquette (1987), this chart retracts the most adequate units to the possible types of use and occupation, based on the several cartographic documents elaborated, and, in a complemental way, on the irrigation ability, foundation and slope stability as well as on the intrinsic costs to each occupation and utilization form.

To represent the units, it was adopted the combined solution of hachures (to the recommended areas for the extraction of minerals and those suggested to be studied aiming to the preservation and/or to tourism potential) and of numbers, producing 4 digitals, where each one of them can vary in a crescent order, from 0 to 2, which indicate the adequability degree to the different forms of utilization (0 - inadequate, 1 - reasonable and 2 - adequate).

The sequence of the digitals registrate, by their positions, the urban and rural residential occupation, the rural occupation (agriculture, cattle breeding, vegetal extraction, etc.) and the industrial areas respectively so, for example, the digital order "1.2.2.0", designates that the area appears to be reasonable for the urban residential occupation (1), adequate for rural residential (2) and rural occupation (2), being inadequate for the industrial activities (0).

Considered the criteria mentioned by Zuquette (1987), it is noticed the clear predominance, in São Carlos region, of areas with rural indication, still noticing a better adequation for the urban expansion to the northwest square and some areas close to the east limit of the city.

It is also important to see that the limitations resulting from the various considered factors, the most promising areas for industry installation correspond in great part, to those more indicated for urban domestic occupation.

The primitive jungles, the hills and the dense regions, were treated with an important attention, because of the importance they play in different aspects of the physical environment.

The adequation classes located close to the margins of superficial drainages of the area are inadequate for activities that use chemical products and which can compromise their quality, mainly those used as water supply of the city.

4 CONCLUSIONS

Considering the technical subsidies provided by the basic principles of the methodology used along this work, it was possible to realize this study as a contribution form to the local geological and geotechnical knowledge, trying to
help the urban expansion planning, seeking a more reasonable geotechnical of the physical environment of São Carlos.

It was identified unconsolidated materials which, due to their behavior and geotechnical properties, could be grouped in 9 homogeneous units. The sandy materials I, II and III occupy the largest areas.

Considering the several forms of use and occupation on can see that there is a clear tendency for rural activities in São Carlos; after the filling of the unoccupied spaces in the current urban perimeter, the northwest square appears to be the best for industrial and urban utilization.

It is vital to have more studies about the preservation and fiscalization the of the rare nucleous of primitive grounds, ciliar grounds and the dense vegetation which contribute a lot to the geotechnical balance of the local physical environment.

It is necessary to have more geotechnical studies in larger scales than the present work (1:10.000 or more), involving mainly the current urban spaces, in order to provide more adequate rules for the occupation of open areas still existing in the city, as well as to contribute to the detection of the risky areas for urbanization.

REFERENCES


Zuquette, L.V. 1987. Engineering geological mapping critical analysis and methodological
Figure 1 - Unconsolidated materials map - São Carlos city (Brazil)
<table>
<thead>
<tr>
<th>Units</th>
<th>$\rho_s$ (g/cm³)</th>
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<th>G (%)</th>
<th>F (%)</th>
<th>LL (%)</th>
<th>IP (%)</th>
<th>$\rho_d$ (g/cm³)</th>
<th>$\epsilon_0$</th>
<th>C.E.C e.mg/100g</th>
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<td></td>
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<td>75</td>
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<td>Ar. I</td>
<td>2.63</td>
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<td>5</td>
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<td>np</td>
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<td>Ar. II</td>
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<td>3.5</td>
<td>64</td>
<td>15</td>
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<td>27</td>
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<td>i</td>
<td>9</td>
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</tr>
</tbody>
</table>

R. JKB - residual of the Botucatu  
R. Jkg - residual of the basic rocks  
R. KB - residual of the Bauru  
Ar. I - sand materials I  
Ar. II - sand materials II  
Ar. III - sand materials III  
$W_{Hig}$ - hygroscopic water content  
G - coarse particles (>0,074 mm)  

$\rho_s$ - specific weight  
$\rho_d$ - dry unit weight  
$\epsilon_0$ - void ratio  
LL - liquid limit  
IP - plastic index  
np - not plastic  
C.E.C - cationic exchange capacity  
F - fine particles (<0,074 mm)  

K - coeff. of the permeability  
Cc - compression index  
$\phi$ - angle of friction  
$\epsilon$ - Young's modulus  
SUCS - Unifi. classification  
HRB - HRB classification  
MCT - MCT classification  
SPT - standart penetration test  
i - impenetrable
Figure 2 - Urban planning chart - São Carlos city (Brazil)

LEGEND

UNITS
- Preservation area
- Mines
- Unit boundary

Adequability class
0 - adequate
1 - moderate
2 - inadequate

IET
RU - Urban Residencials
RR - Rural Residencials
AR - Farm areas
AI - Industrial areas
Utilization of the engineering geological mapping for the urban territorial ordering in Brazilian cities

Utilisation de la cartographie géotechnique pour la planification territoriale des villes du Brésil

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Institute of Technology Amazons/Manaus-AM, Brazil

Nilson Gandolfi
USP/EESC, São Carlos-SP, Brazil

ABSTRACT: Recently, specially on the more developed Brazilian regions, the progress of geotechnical cartography as an essential tool within the modern territorial planning/ordering concept, has been demonstrating the increasing preoccupation with adequate and balanced occupation of urban spaces (mostly on expanding areas).

RÉSUMÉ: Récemment, au Brésil, particulièremment dans les régions plus développés, l'avance de la cartographie géotechnique dans le sens moderne du concept de planification territoriale, montre une croissante souci avec l'occupation adéquate et équilibré des terrains urbains (surtout dans les areas en expansion), et il permet aussi un contrôle systématique de l'environnement, d'une manière à compatibiliser l'usage et l'occupation du sol à ses limitatios et potentialités.

1 INTRODUCTION

The settlement of a planning base-system, either regional or urban, whereby the mapping appears to be of vital importance, supports adequate and balanced space development; the permanent data utilization, based on the dynamism of the physical environment elements, biotic and abiotic, also provides the representation and systematic control of the landscape, offering in such a way, better technical and administrative tools for public and private organs, devoted to a variety of objectives, such as, engineering, agriculture, ambiental environment, planning, and others.

The realization of studies about the physical environment of a determined place, involves the development of a series of successive stages, which, in general, begins with the scale choice and work objective and ends with the characterization and classification of the researched area. The physical environment dynamics, related to the interdependency among its properties, generates the necessity of developing mechanisms which lead to simple and fast filing and handling of the information arising from several sources, to attend the mapping basic principles, such as: reliance, updating and compatibilization of different attributes, what is also stated by Bell et al. (1987), Hearn & Fulton (1987), Culshaw et al. (1990) and Hermelin (1991). Attribute refers to the characteristic, either qualitative or quantitative, which identifies a physical environment component.

From the 1970's on, when the geotechnical cartography began to be slowly developed in Brazil, in a few research and higher learning centers, in which experiments for the utilization of international methodologies began, the researchers started to suggest adaptations to the Country's peculiarities.

The genercal operational sequence of work in scales 1:50.000 and 1:10.000 includes the evaluation of some topics, aiming at attending the mentioned objectives; in this context one can distinguish the area dimension, the available study time, the physical and economical-social
aspects, the relationship between occupation and environment, the urban structure growth tendencies and the related rules involved. In such case, the exact understanding of these subjects leads to a better definition and execution of half-detailed surveys, due to the existence of few experiments in the country on systematical geotechnical cartography, seeking the urban planning. Therefore, as soon as such an adjustment brings about, the mapping progressively loses its purely scientific characteristic and takes a basic element character, what makes it possible to be understood, in its cartographical expression, by technicians and geotechnical inexpert users. The work "Geotechnical Cartography Critical Analysis and Methodological Proposition for Brazilian Conditions", by Zuquette (1987), is considered a remarkable step in the evolution of the systematic studies on geotechnical survey. In this work, one can see the clear methodological tendency towards the elaboration of charts dedicated, specially, to the urban planning, scales between 1:50.000 and 1:10.000, being respected the economical-social situations and the Brazilian territory dimension. The current methodological proposition tries to attend, although subjectively, a favorable cost-benefit relationship, with no damages to the technical/technological level to be used.

2 METHODOLOGY

It was based on the principles established by Zuquette's methodology (1987 and 1993) and takes the following execution order: survey and analysis of available information, identification of attributes to be used in the final product of the work and definition of geotechnically homogeneous units. At a first stage, the data resulting from previous works developed in the area must be criteriously evaluated and only those which present proved reliability should be selected for use, as stated by Dearman (1991) and other researchers. Among others, which can be adequate, there are: topographical maps, geological maps, hydrological maps, pedological maps, geomorphological maps, geophysical maps, climatological maps, soil occupation maps, as well as sounding profiles, results of laboratorial tests and remote sensor products. The attribute collection, second and principal work stage, according to Aguiar (1989), shows as principal ways of collection: publicized works, use of aerial photographs and other remote sensor products, field works, laboratory tests and estimation of material geotechnical properties. The inventory of such attributes, at the field, primordially occurs by surface investigations (profile/pathway), according to the established limits in Table 1.

<table>
<thead>
<tr>
<th>Objective</th>
<th>Scale</th>
<th>OBSERVED PLACES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Qualitatively</td>
<td>Quantitatively</td>
</tr>
<tr>
<td>Urban and/or Expansion Area</td>
<td>Crystalline</td>
<td>Sediment</td>
</tr>
<tr>
<td>50.000</td>
<td>3</td>
<td>2/5km²</td>
</tr>
<tr>
<td>25.000</td>
<td>5</td>
<td>3/ km²</td>
</tr>
<tr>
<td>10.000</td>
<td>15</td>
<td>10/ km²</td>
</tr>
</tbody>
</table>

Source: Adapted from Aguiar (1989)

Concluding this stage, it is possible to elaborate a preliminary geotechnical map, different from those realized in the conventional geological mapping, starting from the summation of data existing previously to the photo interpretation. Afterwards, still in the second work stage, with the field results, added to the photo interpretation data, it is defined preliminary geotechnical units, so that the sampling stage and new observations only concentrates on doubtful points. The field results are restricted to physical and/or chemical analysis of samples systematically collected and normatively classified. Having accomplished the former stages, the collected data are interpreted, being used to identify geotechnical units of homogeneous behavior, having for this the availability of the remote sensor products and of a geographical information system, resulting in the elaboration
of the basic maps, thematic charts, all of them in a presentation scale equal to or higher than 1:50,000, and elucidative texts.

2.1 Document Classification

Like other systems, geotechnical mapping aspects related to the scale, to the objective and form of presenting the results, play a significative role on the development of the work, because of the necessity of defining all the procedure for which the research objectives may be reached. About the scales, Zuquette (1987) proposes the division in: General Scale - smaller than 1:100,000, Regional Scale - from 1:100,000 to 1:25,000, and Half-detailed Scale - from 1:25,000 to 1:10,000, not recommending the ones over 1:10,000, due to high costs and excessive number of observations, with the risk of inadequate substitution of local investigations. Referring to the objectives, closely related to the scale to be used, the mapping offers support to the complex ordering of the physical environment as well as to specific situations. In general scales, directed to regional planning, the attribute groups which better participate in the study are: geomorphological conditions, rocky and/or unconsolidated materials, current soil occupation and climatological data for hydrographic basins. For the regional and half-detailed scales, mainly for those over 1:50,000, the following physical environment components get in the analysis: unconsolidated and rocky materials, geomorphological elements, surface and underground water, climatological factors, and also occupation forms and antropic actions.

Another important aspect in this methodology, is the presentation form of the information at the various stages. There are three types of documents used for the representation of the geotechnical mapping products: Geotechnical Condition Map - characterizes the physical environment without delimiting the area geotechnical similarities, being more indicated for general scales; Geotechnical Zoning Map - separates portions of the soil which are geotechnically similar, without, however, considering their purposes; and, finally, Specific Geotechnical Zoning Map - indicated for works in scales over 1:50,000, whereby the geotechnical conditions are grouped in such a way to attend specific purposes. Based on the combined use of the elements resulting from the maps, from the charts and other available information, Zuquette methodology (1987) foresees four classes of cartographical documents, where the acquired and/or evaluated attributes may be represented: Fundamental Basic Maps, Optional Basic Maps, Auxiliary Map, and Derived or Interpretative Charts.

3 UNITS DEFINITIONS

To determine the units, a correlation between the subjective crossing of attributes which were taken in the analysis for the elaboration of the documents, according to the adopted methodology, and the criteria of attribute pondering (also subjective) from 0 to 2, was performed. Depending on the higher or lower level of benefit to the purpose of the document and taking in account the relative importance of each attribute in the completion of the engineering geologic map. The summation of ponds gave some values that have allowed the classification of units according to their average natural potential that is similar to the system used by Froelich et al. (1976, in Zuquette, 1987) and based in the models reported by Farias et al. (1984).

\[
P_t = \frac{\sum P_{aj}}{P_{aj\,\max}} \times N
\]

Class inadequate reasonable adequate

<table>
<thead>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>----------</td>
<td>----------</td>
<td>----------</td>
</tr>
<tr>
<td>0,00</td>
<td>0,35</td>
<td>0,70</td>
</tr>
</tbody>
</table>

\(P_t\) = total unit pond; \(P_{aj}\) = attribute j pond; \(P_{aj\,\max}\) = maximum attribute j pond; \(N\) = attribute number.
4 CONCLUSIONS

The criteria for the settlement of preliminary homogeneous or heterogeneous units, normally unclear, make it difficult to choose the best places for qualitative and/or quantitative observations, seeming to be convenient to use the procedure presented in item 3 of the present work.

Not having a definition of the limits for each attribute for some interpretative charts induces to the application of very subjective principles, enabling, then, the adoption of conflicting parameters, mainly when the mapping is carried out by professionals without specific knowledge of the different geotechnical fields.

The qualitative observations, contrary to the surveys in other scales, must deserve the right importance, since if they are carried out in different stages and in a higher number, they help picking up the sampling points, as well as the division of the final geotechnical units, mainly in the regions where there is a lack of previous works, and without causing significative increase in the costs and time of the research.

From the samples collected for laboratory tests, only those determined as being representative must be submitted for longer and more expensive analysis, such as consistency limits, sedimentation, compaction and mini-MCV; consequently, it is necessary to establish the sample selective parameters, which could be the simple characterization tests, which are faster and cheaper, besides being sources of good indicatives about the material general characteristics, solid specific mass, sieving granulometry, hygroscoical moisture, and others.

For works in scales between 1:50,000 and 1:10,000, carried out by a team which is not really multidisciplinary, the elaboration and credibility of the interpretative charts become prejudiced, being more advisable to elaborate Geotechnical Condition Maps or even General Geotechnical Zoning Map.

Based on the circumstances verified and presented along the present work, it seems to be a good solution for scales 1:10,000 or higher, the world tendency of adopting the risky charts with specific purposes, mainly those which deal only and exclusively with problems directly related to geotechny.

REFERENCES

Zuquette, L.V. 1987. Engineering geological mapping critical analysis and methodological proposition for Brazilian conditions. EESC/USP, S.Carlos-SP, Doctoring thesis;
Zuquette, L.V. 1993. Geotechnical mapping importance in the physical environment use and occupation: fundamentals and elaboration guide. EESC/USP, São Carlos - SP, Professorial thesis.
Figure 1 - Definition of units from the basic and/or thematic map crossing

Explanation:
- Pond 0
- Pond 1
- Pond 2

Units Final Pond

Definition of Units

\[ P_i = \frac{\sum_{j=1}^{n} P_{ij}}{P_{max}} \times N \]

- \( P_i \) = Total unit pond
- \( P_{ij} \) = Attribute \( j \) pond
- \( P_{max} \) = Maximum attribute \( j \) pond
- \( N \) = Attribute number

<table>
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<tr>
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<tr>
<td>3</td>
<td>0.70</td>
</tr>
<tr>
<td>4</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Explanation:
- Adequate
- Reasonable
- Inadequate

Final Chart

From Aguiar (1989)
Engineering geological mapping of Cosmópolis Area, Brazil: A tool for land use planning
Cartographie géotechnique de la région de Cosmópolis (Brésil): Un outil pour la planification de l'occupation du sol

G.A.G. Gruber & J.E. Rodrigues
University of São Paulo, São Carlos, Brazil

ABSTRACT: Engineering Geological Mapping is an important tool in land use planning and site assessment of hazards and risks. In this paper the methodology used and documents obtained from an engineering geological mapping (scale 1:50.000) performed in the Cosmópolis area (Brazil) are shown. The documents include six interpretative charts concerning soil erodibility, excavability, waste disposal, underground works and road construction.

RESUMÉ: La cartographie géotechnique est un important moyen pour la planification de l'occupation et l'utilisation du milieu naturel et pour l'évaluation des aléas géologiques et des risques. Cet article concerne la méthode utilisée et les documents obtenus (Échelle - 1:50.000) par l'étude effectuée dans la région de Cosmópolis (Brésil). Les principales cartes interprétatives obtenus par ce projet sont: d'erodibilité des sols, d'excavabilité des sols, pour stockage de déchets, d'ouvrages souterrains, d'ouvrages routières.

1. INTRODUCTION

Engineering geological mapping is of great concern to technicians involved in land use planning and in environmental studies. This paper presents the methodology used and the resulting documents obtained in an engineering geological mapping performed at a scale of 1:50.000 in Cosmópolis area, situated in the State of São Paulo, Brazil. The work was developed by collecting and analyzing the existing data such as geologic and soil survey maps, aerial photos, deep bore-holes, SPT and laboratory tests.

After that, field work was developed and samples were collected for performing laboratory tests. The analysis of these information allowed the elaboration of various maps and interpretative charts such as erodibility, excavability and waste disposal charts.

2. METHODOLOGY AND CHARACTERISTICS OF THE AREA UNDER STUDY

2.1. Geology

Geological studies were conducted to obtain two basic geological maps: one grouping the oldest lithologies in the area (granites from rock substrate and sedimentary rocks) and an other grouping both residuals and transported unconsolidated materials.

2.1.1. Bedrock

Cosmópolis area is situated on the east border of the sedimentary Paraná basin. This basin lies on a rock substrate of pre-Cambrian age constituted by leucocratic granites. Over these rocks, there are the sediments of Itararé Group, which was subdivided in five main units: Unit 1: sandstones; Unit 2: siltstones; Unit 3: diamicites; Unit 4: claystones and Unit 5: intercalations of sandstone/siltstone/ claystone. Intrusive rocks (sills and diques) are randomly distributed all over the area and are represented by diabases of fine to medium texture.

2.1.2. Unconsolidated Materials

Texture and genetic characteristics have been used to group different units of unconsolidated materials. Materials were divided in Residual and Transported. The Residual Material was subdivided in those originated from Pre-Cambrian rocks and from Itararé Group (Unit 1: Sandstone; Unit 2: Siltstone; Unit 3: Diamictites; Unit 4: Siltstone and Unit 5: intercalations of sandstone, siltstone and claystone and from basic intrusive rocks (see figure 1). Transported materials were subdivided in sandy,
Figure 1 - Unconsolidated materials map

EXPLANATION

TYPE OF MATERIALS

- RESIDUALS
  - Basic Int. Rocks
  - Sandstone
  - Siltstone

- TRANSPORTED
  - Diamictite
  - Rock Substrate
  - Intercalations of Sandstone/Siltstone/Claystone

- Alluvium

- Sandy
- Clayey

- Urban Area
- Dam
- Limits between Units

UNCONSOLIDATED MATERIALS MAP

Figure 1
From Gruber, 1993

NO SCALE GRAPHIC
0km 1 2 3 4km

1160
clayey and alluvium. Physical characteristics of these materials are listed in table 1.

2.2. Geomorphology

By using aerial photos, topographic maps, field survey and based on Ponçano et al. (1974) the area was divided in three geomorphological units as listed:
Wide Hills: flattened and wide tops interfluves, convex or linear slopes of over 1000 m long and up to 15% declivities. Drainage concentration is low with subdendritic pattern, wide open valleys with perennial or intermittent lagoons.
Medium Hills: short and flattened interfluves, concave and/or convex slopes not longer than 1000 m and declivities of 15% or over. Drainage concentration is from medium to high subrectangular pattern with open to closed valleys.
Alluvial Plain: level or gentle slope, declivities of less than 2%, adjacent to river banks that are periodically subjected to floods.

2.3. Soil Survey

Soil survey maps are very important when performing an engineering geological mapping since they contribute in different stages for data acquisition and interpretation of physical, chemical and genetic characteristics of the materials. Based on pedologic maps elaborated by Oliveira et al. (1979) the following units, in order of importance, can be found in the Cosmópolis area: red-yellow podzol; red-yellow latosol; dark-red latosol; red latosol; structured red latosol; lithic soil; cambisol and hydromorphic soil.

3. DEFINITION OF HOMOGENEOUS UNITS OF AN INTERPRETATIVE CHART

To define homogeneous units for elaborating an interpretative chart a subjective comparison and crossing of attributes of the area was performed. To elaborate the Erodibility, Excavability; Waste Disposal; Underground Works and Road Construction Charts all the existing information was used. The crossing of these charts has originated the Orientative General Chart for Land Use and Planning.

The principle of homogeneous unit definition (Zuquette, 1987; Farias et al., 1984) deals with the superposition of attribute with different weights and the assignment of subjective values (0, 1, 2) to the class of the attributes.

The sum of the weights for the assigned values to the classes is obtained by the following equation:

\[ V_t = \frac{1}{N} \sum_{i=1}^{N} \frac{P_{ai} \cdot V_{ci}}{P_{a\ max} \cdot V_{c\ max}} \]  \hspace{1cm} (3.1)

Where:
- \( V_t \) - total value of unit
- \( P_{ai} \) - weight of attribute \( i \)
- \( P_{a\ max} \) - maximum weight of an attribute
- \( V_{ci} \) - value of class attribute \( i \)
- \( V_{c\ max} \) - maximum value of classes
- \( N \) - number of attributes

This equation was used when elaborating the Orientative General Chart for Land Use and Planning. For the other interpretative charts, equal weights for the attributes were considered and the following equation was used:

\[ V_t = \frac{1}{N \cdot V_{c\ max}} \sum_{i=1}^{N} V_{ci} \]  \hspace{1cm} (3.2)

Using the scale proposed by Aguiar (1989) the total value of unit for the different suitability classes is obtained. Figure 2 shows an hypothetical crossing of maps that represent three certain attributes and how to use equation 3.2.

To weigh the attributes used for elaborating the Orientative General Chart for Land Use and Planning it was considered the farming activities in the area, the erosion problems and road preservation. The weights were:
- Road Construction Chart: 10
- Erodibility Chart: 10
- Waste Disposal Chart: 8
- Underground Works Chart: 4
- Excavability Chart: 4

The scale proposed to define this Chart, considering Total Value of Unit (\( V_t \)) was:
- 0 - 0,25 - Inadequate
- 0,25 - 0,50 - Reasonable
- 0,50 - 0,75 - Less Adequate
- 0,75 - 1,00 - Adequate

4. LOCATION OF THE AREA

The area under study corresponds to 712 Km² and is situated in the northeast of São Paulo State (Brazil),
### Table 1 - Physical Characteristics of Unconsolidated Materials.

<table>
<thead>
<tr>
<th>UNITS</th>
<th>TRANSPORTED</th>
<th>RESIDUALS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sandy</td>
<td>Clayey</td>
</tr>
<tr>
<td>AREA %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MIN</td>
<td>30.4</td>
<td>38.1</td>
</tr>
<tr>
<td>MAX</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COARSE SAND</td>
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</tr>
<tr>
<td>MEDIUM SAND</td>
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</tr>
<tr>
<td>FINE SAND</td>
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<td>73</td>
</tr>
<tr>
<td>SILT</td>
<td>11</td>
<td>43</td>
</tr>
<tr>
<td>CLAY</td>
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<td>43</td>
</tr>
<tr>
<td>LL (%)</td>
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<td>42</td>
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<tr>
<td>LP (%)</td>
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<td>29</td>
</tr>
<tr>
<td>IP (%)</td>
<td>NP</td>
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<td>SPECIFIC GRAVITY</td>
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<tr>
<td>DC (%) **</td>
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</tr>
<tr>
<td>WEIGHT LOSS FROM IMMERSION (%)</td>
<td>0</td>
<td>154</td>
</tr>
</tbody>
</table>

* Intercalations of sandstone, siltstone and claystone.
** Degree of compaction.
Figure 2 - Definition of units

**EXPLANATION**
- Attribute class
- Units values
- Value 0
- Value 1
- Value 2

**Formula**

\[ V_l = \frac{1}{N \cdot V_{cm\max}} \sum_{i=1}^{n} V_{ci} \]

- \( V_l \) = total value of Unit
- \( V_{ci} \) = value class attribute \( j \)
- \( V_{cm\max} \) = maximum value of classes
- \( N \) = number of attributes

**Suitability class interval for Interpretative Charts**

- 0.00
- 0.35
- 0.70
- 1.00

- inadequate
- reasonable
- adequate

**Data of example**

<table>
<thead>
<tr>
<th>( \frac{1}{n} V_{ci} )</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_l )</td>
<td>0.00</td>
<td>0.17</td>
<td>0.33</td>
<td>0.50</td>
<td>0.67</td>
<td>0.83</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**Figure 2** (Gruber, 1993)

**DEFINITION OF UNITS**
between 22°30' and 22°45' south parallels and 47°00' and 47°15' west meridians (Figure 3). Ten towns are included in the area with a total population of about 300,000 inhabitants.

5. INTERPRETATIVE CHARTS

Tables 2, 3, 4, 5 and 6 list limit values of attributes used for establishing suitability charts. Figures 4, 5, 6, 7 and 8 show the distribution of homogeneous units of Erodibility, Excavability, Waste Disposal, Underground Works and Roads Construction Charts. Orientative Chart for Land Use Planning and Management is showed in Figure 9. Adequate Unit is located in areas with declivity ranging from 2 to 5% on sandy and clayey unconsolidated materials. Inadequate Unit is confined to areas with higher declivities over residual materials of Itararé Group, pre cambrian rocks and basic intrusive rocks which show boulders. The other two units rest on various material types and various physical land forms.

6. FINAL CONSIDERATIONS

Engineering geological mapping is an important tool in planning and land use management. Information put forward by different interpretative charts allow planning staff to identify which areas deserve further studies and attention.

Orientative General Chart for Land Use and Management is an important document for deciding about the different ways of land use and terrain occupation.

7. ACKNOWLEDGMENTS

The authors are indebted to Programa de Apoio ao Desenvolvimento Científico e Tecnológico (PADCT) and to Financiadora de Estudos e Projetos (FINEP) for financial support and to SOFPLAN - Planejamento e Sistemas Ltda for allowing the use of equipment to draw maps and charts.

8. REFERENCES


Table 2 - Limit values of attributes used for establishing class erosion susceptibility to Erodibility Chart.

<table>
<thead>
<tr>
<th>ATTRIBUTES</th>
<th>LOW ERODIBILITY</th>
<th>MEDIUM ERODIBILITY</th>
<th>HIGH ERODIBILITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials</td>
<td>residuals clayey materials from basic and Itararé</td>
<td>Clayey sand materials</td>
<td>sandy materials: residuals from Itararé Group</td>
</tr>
<tr>
<td></td>
<td>Group (Unit 4) rocks</td>
<td></td>
<td>(Units 1, 2, 3 and 5) and from rock basement</td>
</tr>
<tr>
<td>Erodibility factor (USLE)</td>
<td>&lt; 0.25</td>
<td>0.25 - 0.45</td>
<td>&gt; 0.45</td>
</tr>
<tr>
<td>Declivity (%)</td>
<td>0 - 5</td>
<td>5 - 10</td>
<td>&gt; 10</td>
</tr>
<tr>
<td>Land form</td>
<td>concave or convex lesser than 200m in length</td>
<td>convex to straight lesser than 1000m in length</td>
<td>convex larger than 1000m in length</td>
</tr>
<tr>
<td>Present and foresen</td>
<td>natural vegetation and large reforesting trees</td>
<td>seasonal cropping</td>
<td>annual cropping with some bare soil and disordered occupation</td>
</tr>
<tr>
<td>occupation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>hill erosion</td>
<td>no marks</td>
<td>few marks</td>
<td>some marks</td>
</tr>
</tbody>
</table>

1164
Figure 3 - Location of Cosmópolis

Figure 4 - Erodibility chart

EXPLANATION UNITS

- Low erodibility
- Medium erodibility
- High erodibility
- Limits between Units
- Urban Area
- Dam

SCALE GRAPHIC

0km 2 4 6 8km

From Gruber, 1993
Table 3 - Limit values of attributes used for establishing class of suitability to Excavability Chart.

<table>
<thead>
<tr>
<th>ATTRIBUTES</th>
<th>ADEQUATES</th>
<th>REASONABLES</th>
<th>INADEQUATES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Declivity (%)</td>
<td>&lt; 10</td>
<td>10 - 20</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Bedrock depth (m)</td>
<td>&gt; 5,0</td>
<td>2,0 - 5,0</td>
<td>&lt; 2,0</td>
</tr>
<tr>
<td>Unconsolidated materials</td>
<td>Residual and transported materials (without boulders and/or blocks)</td>
<td>Residual materials from soft to medium resistance rock</td>
<td>Residual materials with boulders and/or blocks.</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>&gt; 5,0</td>
<td>1,0 - 5,0</td>
<td>&lt; 1,0</td>
</tr>
</tbody>
</table>

Table 4 - Limit values of attributes used for establishing class of suitability to Waste Disposal Chart.

<table>
<thead>
<tr>
<th>ATTRIBUTES</th>
<th>ADEQUATES</th>
<th>REASONABLES</th>
<th>INADEQUATES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Declivity (%)</td>
<td>2 - 5</td>
<td>5 - 10</td>
<td>&lt; 2 e &gt; 10</td>
</tr>
<tr>
<td>CTC (mg/100g)</td>
<td>&gt; 15,0</td>
<td>5,0 - 15,0</td>
<td>&lt; 5,0</td>
</tr>
<tr>
<td>Coefficient of permeability (cm/s)</td>
<td>4.10^-3 - 7.10^-4</td>
<td>10^-2 - 4.10^-3</td>
<td>&gt; 10^-2 e &lt; 7.10^-4</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>&gt; 5,0</td>
<td>1,0 - 5,0</td>
<td>&lt; 5,0</td>
</tr>
<tr>
<td>Bedrock depth (m)</td>
<td>&gt; 20,0</td>
<td>5,0 - 20,0</td>
<td>&lt; 5,0</td>
</tr>
</tbody>
</table>

Table 5 - Limit values of attributes used for establishing class of Underground Works Chart.

<table>
<thead>
<tr>
<th>ATTRIBUTES</th>
<th>ADEQUATES</th>
<th>REASONABLES</th>
<th>INADEQUATES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedrock depth (m)</td>
<td>&gt; 5,0</td>
<td>2,0 - 5,0</td>
<td>&lt; 2,0</td>
</tr>
<tr>
<td>Unconsolidated materials</td>
<td>sandy, clayey and residual materials from granites and basic rocks (without boulders and/or blocks)</td>
<td>residual materials from Itararé Group, granites and basic rocks (with boulders and/or blocks)</td>
<td>alluvium</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>&gt; 5,0</td>
<td>1,0 - 5,0</td>
<td>&lt; 1,0</td>
</tr>
</tbody>
</table>

Table 6 - Limit values of attributes used for establishing class of Road Construction chart.

<table>
<thead>
<tr>
<th>ATTRIBUTES</th>
<th>ADEQUATES</th>
<th>REASONABLES</th>
<th>INADEQUATES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Declivity (%)</td>
<td>&lt; 5</td>
<td>5 - 10</td>
<td>&gt; 10</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>&gt; 1,0</td>
<td>---</td>
<td>&lt; 1,0</td>
</tr>
<tr>
<td>Bedrock depth (m)</td>
<td>&gt; 10,0</td>
<td>2,0 - 10,0</td>
<td>&lt; 2,0</td>
</tr>
<tr>
<td>Superficial drainage (channels/km²)</td>
<td>&lt; 4,0</td>
<td>4 - 5</td>
<td>&gt; 5</td>
</tr>
</tbody>
</table>

1166
EXPLANATION UNITS

- adequate  - reasonable  - inadequate

\(\checkmark\) - Limits between Units  \(\text{\textcircled{\text{\text{-}}}\text{\textcircled{\text{-}}}}\) - Dam

From Gruber, 1993

SCALE GRAPHIC

0km 2 4 6 8km

- Urban Area
Figure 9 - Orientative chart for land use planning

EXPLANATION UNITS

- adequate  - less adequate  - reasonable  - inadequate  - Dam

- Urban Area  - Limits between Units

ORIENTATIVE CHART FOR LAND USE PLANNING
Engineering geological zoning mapping of the Ribeirão Preto city (Brazil) at 1:25,000 scale
Carte de zonage géotechnique de la ville de Ribeirão Preto (Brésil) dans l’échelle 1:25,000

L.V.Zuquette, O.J.Pejon & O.Sinelli
USP/FFCLRP, Ribeirão Preto, Brazil
N.Gandolfi
USP/EESC, São Carlos, Brazil

ABSTRACT: The area delimited by parallels 21° and 21°15'S, and meridians 47°45' and 48°W, in São Paulo State (Brazil), is also constituted by diabases or basalts of the Serra Geral Formation and Sandstones of the Botucatu Formation. The urban center covers 400 km² and has a population of 500,000 inhabitants. In the courses of engineering geological works were elaborated several graphic documents: declivity chart, unconsolidated materials map, rock substrate map, overland flow chart and engineering geological zoning chart. These documents allowed the division of the area into different basic engineering geological units. The most common problems that occur in the area are: collapsivity of the soils, declivities higher than 20%, pollution of the aquifer, flood zones, low depth of the bedrock, recharge zone and adequate sites for waste disposal.

RESUMÉ: La région étudiée est localisée entre les parallèles 21° et 21°15'S, et les méridiens 47°45' et 48°W, dans l'Etat de São Paulo (Brésil). Cette région est constituée par diabases ou basaltes de la Formation Serra Geral et arènes de la Formation Botucatu. Les centres urbaines recouvrent 400 km² avec une population d'environ 500,000 habitants. Dans le travail de cartographie géotechnique ont été élaborés plusieurs cartes: carte de déclivité, carte de matériaux non consolidés, carte du substratum, carte d'écoulement superficiel des eaux et carte de zonage géotechnique. Ces documents ont permis la division de la région en unités géotechniques basiques. Les problèmes plus communs que la région présente sont: déclivité plus grand que 20%, pollution des eaux souterraines, petite profondeur du substratum, zone de recharge du aquifère et zone de stockage de déchets.

1. INTRODUCTION

The Ribeirão Preto city and expansion areas are located in the Northeast region of the São Paulo State (Brazil), between the parallels 21° and 21°15'S and meridians 47°45' and 48°W, covering more than 400 km², with 500,000 inhabitants (Figure 1).

The region presents accelerated growth and several constructions are being fixed, such as: industries, sanitary landfills, residential centers, gas stations, wells, electricity and transportation services.

Due to these constructions, several problems are occurring in the region, for example: pollution of aquifers and soils, flood, erosion, changes in the infiltration ratio and damage of the buildings due to the collapsivity of the soils.

An engineering geological mapping was elaborated for the region at a scale 1:25,000, and reproduced in this work at a scale 1:200,000, for urban and regional planning purposes.

2. GENERAL CHARACTERISTICS

The region is constituted by:

- Geology: 2 formations from Mesozoic Era:
  - Serra Geral Formation - diabases or basalts (Triassic-Jurassic);
  - Botucatu Formation - sandstones (Triassic).

Climatic conditions: The predominant climatic type is Aw (Koppen's classifications) with rainy summer, dry winter, average annual temperature is
Figure 1 - Map of the drainage net, boundary of the urban area, drainage basins and expansion zone of the city.

Figure 2 - Declivity chart.
22\degree C and annual rainfall index is 1400 mm.

**Morphology:** The region can be divided into 2 morphological zones, which are separated preliminarily by altitude and declivity levels:
- The flat area is located from 500 to 600 m above sea level and declivities between 1 to 10%. The other area is constituted by the altitudes and declivities higher than 600 m and 10%, respectively.

**Hydrogeology:** The sandstones of Botucatu Formation are a good aquifer and there are wells that can produce about 300 m³/h, and part of the region is characterized as recharge zone. In the urban area, there are 400 wells producing 80,000 m³/h.

3. METHODOLOGY

The work was elaborated through methodology proposed by Zuquetto and Gandoiffi (1990) and basic principles of Fookes (1987), Matula (1979) and Dearman *et alii* (1979). In this work were prepared the basic graphic documents: rock substrate map, unconsolidated materials map, declivity chart (Figure 2), hydrograph basins map (Figure 1) and interpretative document, the engineering geological zoning chart (Figure 3) that was elaborated by matrix and logic tree mechanisms.

These mechanisms permitted the association among the rock substrate and unconsolidated materials maps and the results from in situ and laboratorial geotechnical tests, that resulted in the basic engineering geological units (Table 1).

4. RESULTS

The engineering geological zoning permitted to define a group of 35 basic engineering geological units (Table 1), that were used as basis for the division of the region into 200 subareas (Figure 3). Each unit is characterized by basic profile of the unconsolidated materials, lithology of the bedrock, class of depth of the bedrock and some index properties such as void ratio, S.P.T. (Standard Penetration Test) index and mineralogy of the different levels of the unconsolidated materials.

These characteristics of the units permitted a good knowledge of the region in this scale. By the engineering geological zoning chart we elaborated an analysis of adequabilities of each unit for several occupation factors, such as: sanitary landfill, excavability, foundation, erosion, aquifer and recharge zone, residential and industrial zones and agriculture potential. These factors have influence in the occupation process and urban planning, and the results are presented in the Table 2, where the units were classified as favorable, moderate and restrictive.

**Favorable units** — The natural attributes, present adequate levels for occupation factor.

**Moderate units** — Less than 30% of the natural attributes present non adequate levels for occupation factor. The correction is possible with low costs and common technological mechanisms.

**Restrictive units** — More than 30% of the natural attributes present not adequate levels for occupation factor.

After the analysis of the units we verified that region presents at least one limitations in the total extension, then we elaborated the graphic document named engineering geological limitations chart with the unfavorable attributes (Figure 4).

5. CONCLUSION

The engineering geological zoning mapping allowed the assessment of the region:
- there are several units that have adequate conditions for sanitary landfills, septic tank, cesspool, residencial and industrial occupations;
- a large number of units presents limitations as flood, erosion, foundation, pollution of aquifers and soils, low depth of the bedrock, low ground water level and agriculture.

The results obtained from engineering geological zoning chart show that the region present high grade of diversity and that more detailed studies must be elaborated on the specific zones.

The units 24, 26 and 27 are characterized as the principal recharge zones of the regional aquifer.

BIBLIOGRAPHY


FCOKES, P.G. 1987. Land evaluation and site
Figure 3 - Engineering geological zoning chart.

Figure 4 - Engineering geological limitations chart.
### Table 1 - Matrix of the basic engineering geological characteristics of the units.

<table>
<thead>
<tr>
<th>Depth of the bedrock (m) (d)</th>
<th>Unconsolidated materials</th>
<th>S.P.T. (o)</th>
<th>Mineralogy (b)</th>
<th>Basic profile (texture)</th>
<th>S.P.T. (o)</th>
<th>Mineralogy (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unconsolidated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Basic profile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(texture)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandy or clayey</td>
<td>&lt; 5</td>
<td>1, 3, 4</td>
<td>clay (with silt and clay &lt; 10%)</td>
<td>2</td>
<td>1, 3, 4, 6</td>
</tr>
<tr>
<td></td>
<td>texture</td>
<td></td>
<td>1, 0</td>
<td>texture</td>
<td></td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Sandy</td>
<td>5</td>
<td>1, 4, 6, 7</td>
<td>texture</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>texture</td>
<td>25</td>
<td>1, 0</td>
<td></td>
<td></td>
<td>15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lithology of the bedrock: Basalt or sandstone</th>
<th>&gt; 25</th>
<th>Sandstone</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 2</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>&gt; 2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>&gt; 5</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>&lt; 10</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>&gt; 10</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>&lt; 15</td>
<td>29</td>
<td></td>
</tr>
<tr>
<td>&gt; 15</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

* Basic engineering geological units.

### Table 1 (cont.)

<table>
<thead>
<tr>
<th>Depth of the bedrock (m) (d)</th>
<th>Unconsolidated materials</th>
<th>S.P.T. (o)</th>
<th>Mineralogy (b)</th>
<th>Basic profile (texture)</th>
<th>S.P.T. (o)</th>
<th>Mineralogy (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unconsolidated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Basic profile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clayey texture (clay &gt; 70%)</td>
<td>&lt; 5</td>
<td>1, 3, 4, 5</td>
<td>Clayey texture (clay &gt; 70%)</td>
<td>&lt; 6</td>
<td>1, 5, 4, 3</td>
<td></td>
</tr>
<tr>
<td>&gt; 1,5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&gt; 1,5</td>
</tr>
<tr>
<td>Stone line</td>
<td>&gt; 1,0</td>
<td>1, 3, 4, 5, 8</td>
<td>Stone line</td>
<td></td>
<td>&gt; 1,0</td>
<td></td>
</tr>
<tr>
<td>Weathered basalt</td>
<td>&gt; 30</td>
<td>1, 2, 4, 6, 7, 8</td>
<td>Silty clay texture</td>
<td>7, 15</td>
<td>1, 2, 4, 6, 7, 8</td>
<td></td>
</tr>
<tr>
<td>&gt; 1,0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&gt; 1,0</td>
</tr>
<tr>
<td>Lithology of the bedrock: Basalt</td>
<td>7, 8</td>
<td>Weathered basalt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 2</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>&gt; 2</td>
<td>4</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 5</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 5</td>
<td>17</td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>&lt; 10</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 10</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 15</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 15</td>
<td>31</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 20</td>
<td>32</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Basic engineering geological units.

a - Standard penetration test (minimum and maximum index).
b - Mineralogy: 1- kaolinite; 2- illite; 3- Fe and Al oxides; 4- quartz; 5- gibbsite; 6- other oxides; 7- weathered feldspars; 8- weathered plagioclases and amphiboles.
c - eo - Natural voids ratio (average value).
<table>
<thead>
<tr>
<th>Depth of the bedrock (m)</th>
<th>Unconsolidated materials</th>
<th>Basic profile (texture)</th>
<th>S.P.T. (ø)</th>
<th>Mineralogy (b)</th>
<th>Basic profile (texture)</th>
<th>S.P.T. (ø)</th>
<th>Mineralogy (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>eo (ø)</td>
<td></td>
<td></td>
<td>eo (ø)</td>
<td></td>
</tr>
<tr>
<td>Clayey sand texture (with silt and clay &gt; 20%)</td>
<td>&lt; 7</td>
<td>1, 4, 6</td>
<td>&lt; 6</td>
<td>1, 4, 6</td>
<td>&gt; 1,0</td>
<td>&gt; 1,2</td>
<td></td>
</tr>
<tr>
<td>Fine sandy texture (sand &gt; 85%)</td>
<td>7 15</td>
<td>1, 4, 7</td>
<td>Stone line</td>
<td></td>
<td>Fine sandy and gravels</td>
<td>&gt; 10</td>
<td>1, 4, 6</td>
</tr>
<tr>
<td>Weathered sandstone</td>
<td>&gt; 15</td>
<td>4, 7</td>
<td>Weathered sandstone</td>
<td>&gt; 15</td>
<td>4, 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lithology of the bedrock</td>
<td>Sandstone</td>
<td>Sandstone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 2</td>
<td>5'</td>
<td></td>
<td>6'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 2</td>
<td>12</td>
<td></td>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 5</td>
<td>19</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
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<td>&gt; 10</td>
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<tr>
<td>&lt; 15</td>
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<td>34</td>
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* Basic engineering geological units.

<table>
<thead>
<tr>
<th>Depth of the bedrock (m)</th>
<th>Unconsolidated materials</th>
<th>Basic profile (texture)</th>
<th>S.P.T. (ø)</th>
<th>Mineralogy (b)</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>eo (ø)</td>
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<td>Sandy clay texture</td>
<td>&lt; 5</td>
<td>1, 5, 4, 6</td>
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<td>Clayey texture</td>
<td>5 20</td>
<td>1, 3, 4, 5</td>
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<td>Weathered basalt</td>
<td>&gt; 30</td>
<td>1, 2, 7, 8</td>
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<td>Lithology of the bedrock</td>
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<td>&lt; 2</td>
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<td>&lt; 15</td>
<td>35</td>
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</tbody>
</table>

* Basic engineering geological units.

a - Standard penetration test (minimum and maximum index).
b - Mineralogy: 1- kaolinite; 2- interstratified; 3- Fe and Al oxides; 4- quartz; 5- gibbsite; 6- other oxides; 7- weathered feldspars; 8- weathered plagioclases and amphiboles.
c - eo - Natural voids ratio (average value).
Table 2 - Adequabilities of the basic engineering geological units for some factors of occupations.

<table>
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<tr>
<th>Factors of occupations</th>
<th>Favorable units</th>
<th>Moderate units</th>
<th>Unfavourable attributes</th>
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<td>Units</td>
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<td>Moderate units</td>
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<td>Excavability conditions</td>
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<td>8, 15, 10, 11, 14</td>
<td>Low ground water level, Depth of the bedrock</td>
<td>3, 4, 7</td>
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<td>Foundation conditions</td>
<td>3, 4, 7, 10, 11, 14</td>
<td>1, 16, 17, 18, 19, 20, 21, 22, 24, 25, 26, 27, 28, 30, 31, 32, 33, 34, 35</td>
<td>Low ground water level, Collapsible material in the surface</td>
<td>8, 15, 22, 29</td>
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<td>Sanitary landfills</td>
<td>31, 35</td>
<td>24, 25, 28</td>
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<td>Septic tank and cesspool</td>
<td>24, 25, 26, 28, 31, 35</td>
<td>10, 11, 14, 16, 23</td>
<td>Low ground water level, High permeability coefficient, Low retardation factor</td>
<td>8, 15, 3, 4, 7</td>
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<tr>
<td>Erosion (potential)</td>
<td>3, 10, 17, 24, 31, 4, 11, 18, 22 (low potential)</td>
<td>7, 14, 24, 35, 12, 19, 26</td>
<td>Sandy texture</td>
<td>16, 23</td>
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<td>Aquifer (potential)</td>
<td>16, 23, 30, 19, 20, 26, 27, 33, 34</td>
<td>1, 8, 15, 22, 29</td>
<td>Low ground water level</td>
<td>3, 4, 10, 11, 17, 18, 24, 25, 31, 32, 33, 34, 7, 14, 21, 28, 35</td>
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<tr>
<td>Contamination of aquifer (potential)</td>
<td>12, 19, 26, 33</td>
<td></td>
<td>Low ground water level</td>
<td>9, 16, 6, 13, 20, 23, 30, 27, 34</td>
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<tr>
<td>Flood (potential)</td>
<td>All units</td>
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<td>Low ground water level</td>
<td>1, 8, 15, 22, 29</td>
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<td>Agriculture potential</td>
<td>10, 11, 17, 18, 24, 25, 31, 32, 31, 114, 21, 28, 35</td>
<td>15, 12, 19, 26, 33, 3, 4, 7, 1, 18, 15, 22</td>
<td>Low ground water level, Low permeability coefficient, Low retardation factor</td>
<td>6, 13, 20, 27, 34, 2, 9, 16, 23, 30</td>
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<td>Urban (residential, industrial) zones</td>
<td>17, 24, 31, 18, 25, 28, 35</td>
<td>12, 19, 16, 23, 26, 10, 11, 14</td>
<td>Low ground water level, Low permeability coefficient, Low retardation factor</td>
<td>3, 4, 8, 7, 15</td>
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Gravitational morphogenesis of the Apennine chain in Central Italy
Morphogenèse gravitationnelle de la chaîne Apennine dans l'Italie centrale

B. Gentili & G. Pambianchi
University of Camerino, Italy

ABSTRACT: The present work investigates the oriental portion of the central Apennines. This is formed prevalently of calcareous and (in the southernmost part) of arenaceous terrain, characterized by a complex arcuate belt of folds and thrusts with north-east vergence, as a consequence of the Neogene compressional tectonics. In this area large-scale landslides and deep-seated gravitational deformations frequently occur in the slopes. The distribution and typology of the gravitational phenomena are correlated with the lithostratigraphic and structural features of the terrains, and above all with the Quaternary tectonic and geomorphological evolution of the area, the main factors responsables for the evolution of the landscape. Seismic activity appear not infrequently to be the essential factors in the genesis of the phenomena studied.

RÉSUMÉ: Cette étude concerne la partie orientale de l'Apennin Central formée essentiellement de roches calcaires et arénacées, caractérisées par une enchâinement arqué complexe présentant des plis et des glissements orientés vers nord et nord-est, résultat de la tectonique compressive du Neogene. Dans cette zone se produisent souvent sur les versants des amples éboulements importants et des déformations profondes dues à la force de gravité. La distribution et la typologie des phénomènes gravitationnels sont corrélées aux facteurs lithostratigraphiques et structurels des terrains, et, surtout, à l'évolution géomorphologique et tectonique quaternaires de la zone, facteurs principaux pour l'évolution du paysage. L'activité sismique semble être souvent les facteurs essentiels dans la genèse des phénomènes étudiés.

INTRODUCTION

Many Authors from different parts of the world have evidenced how large-scale landslides and deep-seated gravitational deformations of slopes frequently affect, in addition to the prevalently clayey slopes also those formed of strong rocks, which are generally considered to be stable (Harrison & Falcon, 1934; Stini, 1941, 1952; Jahn, 1964; Zischinsky, 1966, 1969; Nemcok, 1972; Radbruch-Hall et al., 1976; Mahr & Nemcok, 1977).

In the mountainous areas of Italy over the last two decades, phenomena of this type have been ever more frequently found, their genesis being essentially connected to particular geomorphological conditions, to the complex litho-structural framework and to the intense tectonic and seismic activity (Guerricchio & Meldoro, 1981; Sorriso-Valvo Ed., 1984, 1987, 1989).

Also for the area under study, a large stretch of the Central Italian Apennine chain (the Umbro-Marchean-Sabine ridge to the west, the Marchean ridge to the east, and the Latiun-Abruzzi ridge to the south; Fig. 1), formed of
Fig. 1 - Geological-structural and geomorphological sketch of the central Apennines: 1) Apenninic chain mainly calcareus; 2) turbiditic sediments; 3) overthrust; 4) reverse fault; 5) back-thrust; 6) main transversal antiapenninic ridges; 7) large landslide; 8) deep-seated gravitational slope deformation.
limestone, marly - limestone, and subordinately of arenaceous turbidites, the important role played by the gravitational morphogenesis in the Quaternary evolution has been evidenced (Coppola et al., 1978; Carraro et al., 1979; Crescenti et al., 1987; Dramis et al., 1987, 1988, and in press; Blumetti et al., 1990; Barchi et al., 1993; Gentili & Pambianchi, 1993). Altogether about 500 phenomena (average extension approx. 2 km²; frequency 0.06 phenomena per km²; density 0.12) have been recognized, of which about 10% are represented by deep-seated gravitational deformation of the slopes and the rest by various typologies of landslide phenomena.

In the course of the research, landforms and structural elements have been identified affecting an area of more than 20 km² of the Marche ridge. These are distinguished, by their considerably larger size, their less clear geomorphological and geological picture and more complex kinematics, from the numerous gravitational phenomena to have been recognized. Furthermore, their genesis could not be inserted in the "classical" tectonic evolution of the Apennines, and thus an interpretative model was proposed (Gentili et al., 1992) in which the gravitational factor was assigned a role of importance in no way inferior to the tectonic one.

The search for these particular tecton gravitational landforms was extended over a larger stretch of the central Apennines and evidenced an unexpectedly frequent recurrence. The main purpose of the present work is to illustrate them.

GEOLOGICAL FEATURES AND TECTONIC EVOLUTION OF THE AREA

The stratigraphic sequence of the Umbro-Marchean-Sabine Apennines (Centamore & Deiana Eds., 1986; Bigi et al., 1991) includes: at the base a powerful and massive calcareous complex carbonatic platform (Lower Lias) about 800 m thick, followed by a pelagic sequence of limestone, marly limestone, cherty limestone and marls, all well stratified and with an overall thickness of about 1400 m (Middle Lias - Lower-Middle Miocene). Turbidite associations (Tortonian-Messinian) characterize the areas between the two northern ridges and those to the south and east of them (Fig. 1).

The Latium-Abruzzi domain instead is characterized by a more powerful calcareous platform sequence (with thicknesses of 2000 m and more) which laterally becomes scarp and basinalcarbonaceous facies intercalated with levels of marly limestone and marls (Mesozoic-Palogene). Above, marly and turbiditic facies are found (Oligocene-Miocene) similar to those of the Umbro-Marchean-Sabine domain.

At the surface in the northern portion of the area, the above-described terrains give rise to two anticline structures which merge towards the south and generate the Monti Sibillini massif. The slope angles of the strata vary from 15° to 30° with notable local variation on the western slopes of the structure; instead, slopes of 20° to 30° which progressively increase until becoming vertical and/or overturned characterize the eastern slopes. Folds and faults forming on the whole a vast synclinorium characterize the structural framework of the Miocene turbidite sediments of the area lying inside the Apennine ridges and the eastern one.

Also the Latium-Abruzzi ridge oriented approximately E-W forms a vast anticlinorium where the strata are characterized by a clearly asymmetrical arrangement: vertical or overturned strata on the northern slope, but generally on the southern one undulating and slightly deepening southward.

Whereas the folds are the most evident structural element, thrusts and, subordinately, backthrusts are the main ones. They have in fact determined the superposition of the several layers of differing stiffness of the powerful sedimentary cover which forms the arcuate Umbro-Marchean-Sabine Apennine chain and that of the Gran Sasso d'Italia, with eastward
and northward vergence, respectively. The most important thrusts are those of the Gran Sasso d'Italia and the Monti Sibillini (Fig.1). The disappearance of the latter to the north of the River Chienti should be seen in connection with the presence of backthrusts and with the marked NNW-ward axial depression of the entire structure (Calamita & Deiana, 1988).

This structural framework has been determined by the intense compressional tectonics (Oligocene-Lower Pliocene) which, in the outer area, has continued active until the whole of Pliocene (Bally et al., 1986; Calamita & Deiana, 1988). Direct faults, mostly Apenninic ones, connected with the generalized uplifting and with the consequent extensional phase, which reached their peak in Lower-Middle Pleistocene (Dementjeva, 1965; Ambrosetti et al., 1982; Colliorti et al., 1991), have in many cases reactivated the discontinuity produced by the preceding compressional phase. The transverse faults, already activated in Messinian and which have continued to act subsequently as well, are essentially connected with the setting up and deepening of the hydrographic network (Boccaletti et al., 1983; Dramis et al., 1991).

To the diverse intensity along anti-Apenninic and Apenninic fault lines of the abovementioned Lower-Middle Pleistocene uplifting and, subordinately, to the direct faults are attributable the highest points of the Apennine chain (M. Vettore, 2475m; Gran Sasso d’Italia, 2912m) and the genesis of the important ridges transversal to it (Fig. 1), of which the southern one is the highest (Dramis et al., 1991).

THE GRAVITATIONAL PHENOMENA OF THE CENTRAL APENNINES

Of the approximately 50 deep-seated gravitational deformation phenomena of slopes, the most frequently found typologies are lateral spreading, deep-seated block slides and sackungs (Jahn, 1964; Zischinsky 1969; Varnes, 1978; Agnese et al., 1987; Dramis et al., 1987; Gentili & Pambianchi, 1993; Dramis et al., in press). The most frequent landslide phenomena are collapses and translational slides; the former are localized essentially on the eastern slopes of the ridges, the latter on the western slopes corresponding with the valley gorges.

The gravitational phenomena occur with varying frequency along the anti-Apenninic fault lines: on the western slopes of the structures the number is reduced, whereas the frequency increases on the slopes of the deepest intramontane tectonic depressions and along the valley gorges transversal to the chain. The greatest number of events occurs on the eastern slopes of the ridges (Fig. 1). This differentiated spatial distribution of the phenomena is above all connected with the different tectonic evolution of the area along the anti-Apenninic fault lines. The intense uplifting of Lower-Middle Pleistocene has in fact produced, on the Adriatic side (still undergoing compression), deep cuts of the drainage systems and thus high relief values, with a consequent triggering of a large number of gravitational phenomena. On the western slopes (which are extensional) the direct faults have considerably reduced the relief, thereby favoring the setting up of better conditions for stability (Dramis et al., in press).

In various cases, the movements have occurred along thrust and/or backthrust planes, with kinetics similar to those of the low-angle direct faults; in other cases, their setting up is connected with the direction of their planes and/or shear zones, due to the rapid deepening of the hydrographic net or the action of the direct faults (Dramis, 1992; Gentili et al., 1992; Dramis et al., in press). Other important genetic factors of the phenomena in question are the lithostructural features of the terrains, characterized by the superposition of powerful and strong calcareous or, subordinately, arenaceous masses over the more plastic pelitic levels, and the seismicity of the area. The last of these was probably very intense in the past and is still active along the Apennine chain and the
Adriatic coast (Postpischi, 1985).

THE CASES STUDIED

Aerial photography studies of the area followed up by investigations in the field have evidenced the recurrence of narrow longish depressions and of cuts which are prevalently arcuate in their overall shape (Fig. 2), and have adifferent arrangement with respect to those of the main structural elements. They cut the anticline structures transversally, especially in the vicinity of the deepest transversal valleys, whereas in other cases the cuts run obliquely and/or parallel to the structures. Analogous landforms can frequently be observed in the steep and extensive slopes of the valleys following anti-Apenninic tectonic fault lines of regional importance, and along the upper fault slopes bordering on the main intra-Apenninic tectonic depressions. The origins and evolution of portions of the hydrographic net seem to be connected directly or indirectly with these landforms.

![Map](image)

Fig.2 - Main morphological and structural features: 1) anticline axis; 2) sincline axis; 3) overthrust; 4) main normal fault; 5) arcuate valley; 6) trenche or elongated depression; 7) tectonic depression; AN= Antrodoco; MC= Calvo Mt.; MM= Moricone Mt.; MP= Patino Mt.; MT= Terminillo Mt.; MV= Vettore Mt.; VI= Vigliano.
A detailed description of the most representative morphological features of the territory studied will now be given.

Particularly evident are those at the summit of the ridge between the River Chienti and Monte Vettore (Monti Sibillini), where several generations of trenches can be observed, transversal to the anticline structure axis; these are often continued in the cuts of the lesser hydrographic net which occurs on the slopes. The size of these trenches is extremely variable, reaching and sometimes exceeding 500 m in width, 40-50 m in depth at the summit of the ridge, and overall lengths of 6-8 km (Fig. 2).

The mostly concave bottom of the summit portions of the trenches is filled by debris, sometimes strongly cemented and attributable to periglacial climatic conditions (Coltorti & Dranis, 1988).

The described landforms have the same orientation as the main anti-Apenninic tectonic fault lines and cut the continuity of the oldest morphological element, the summit erosion surface modelled in calcareous terrains. In fact, this element has a stepwise pattern towards the Chienti valley not reflected in any evident dislocations of the bedrock. Inside the bedrock there are several levels of intensely tectonized belts, connected with the compressional tectonics, and folds with reduced curve radius whose approximately anti-Apenninic axes are not congruent with the orientation of the main tectonic stress fields.

These arcuate landforms decrease in frequency towards the south in the highest portion of the structure, where the two antithetic arcs of the Tenna and Ambro valley gorges transversally cutting the ridge are particularly evident and interesting (Fig. 2). The two main branches of the Fiastrone torrent are analogous in form: they cut the western slope of the anticline structure, which is characterized by a regular arrangement of the strata dipping out of the slope.

A second, but smaller, area where morphological features analogous to those described can be observed is that lying on the anti-Apenninic ridge between M. Patino and M. Moricone, to the west of M. Vettore (Fig. 2). Its structural edifice is characterized by the superposition determined by thrusts of a tectonic wedge of limestone and calcareous marls, with maximum thickness of about 700 meters, over an analogous lithological sequence slightly folded in an anticline (Calamita et al., 1990).

Connected with the direct Apenninic faults on the western slope of this structure is the genesis of the extensive steep slopes characterized by high relief values. The lowering associated with these has led to the laying bare of the gently westward sloping thrust planes.

Finally, in the southern portion of the area under study, a thrust structure has been analyzed where calcareous and calcareous-marly sequences are superposed; the contact between them is marked by a convex asymmetric surface, the southwestern branch of which has the greatest angle of slope. This branch contains approximately Apenninic direct faults that, towards the south, lower the structure, thereby laying bare the thrust plane (Fig. 2) on the southern slopes (between Antrodoco and Vigliano).

Further north, the unit has approximately Apenninic fractures and faults as well as narrow lengthy depressions congruent with it.

DISCUSSION

The characteristic arcuate landforms and the geometry of the rock bodies not congruent with the general structural features are interpreted as indicators of a gravitational activity that has remodelled the morphostructure in question, in all probability in several different phases.

The factors leading to this are to be seen firstly in the lithostructural features of the terrains, characterized by the superposition of strong and often thick limestone masses over the more brittle marly layers, then Oligocene-Pliocene compressional tectonics responsible for
the genesis of the thrusts and of the vast and extensive tectonized belts inside the limestone, and finally in the intense Quaternary tectonic activity. The last of these factors is of particular importance in that the following phenomena are connected to it: Apenninic and anti-Apenninic discontinuity (fractures and faults) of the strong rock bodies; a tilting of the anticlinal structures, connected to the marked differential uplifting; a rapid and intense deepening of the hydrographic network. To the last of these phenomena, and sometimes to the activity of the faults with greatest displacement, is connected the laying bare of potential thrust planes and/or zones: pelitic intercalations and tectonized zones inside the limestone masses; main and/or secondary thrust planes.

The particular tectono-structural-geomorphological features described have favored the recurrent activation of gravitational phenomena in the area, very probably coinciding with particularly intense seismic events (IX degree MCS; Monachesi et al., 1985; Gentili & Fambianchi, 1993). These have produced shear planes, or remodeled the preexisting ones of tectonic origin, in the portion of the bedrock nearest the surface, generating arcuate trenches at the top and bulging at the foot of the slope. In depth, the brittle deformations have lost their individuality in plastic deformation zones (Mahr & Necok, 1977), or else they are continued in the pelitic layers of the rock mass or in the deepest shear planes and/or zones of the thrusts.

Connected with a vast gravitational deformation occurring, in all probability, in the brittle layers underlying the thrust plane there should be the antithetic valley arcs of the Tenna and Ambro. In fact, this deformation probably produced the southward rotation of the slightly northward-dipping thrust plane (Fig. 3.a). Instead, for the antithetic landforms of the Fiastrone, gravitational collapses have been hypothesized (Harrison & Falcon, 1934; Ollier, 1981), with a prevalently translational element of the slope of the anticlinal structure, due to stratigraphic (pelitic intercalations in the limestone) and/or structural discontinuity (thrust planes dipping westward).

The numerous trenches occurring stepwise in the ancient summit erosion surface to the south of the River Chienti (Fig. 2) mostly follow anti-Apenninic fractures which represent lines of weakness in the rock mass and have favored the activation of gravitational deformations. The kinematics of these has been similar to that of the faults with listric geometry along the discontinuities made up of pelitic layers, tectonized belts, and thrust planes (Fig. 3.a).

The thrust planes dipping out of the slope, generally having a lesser angle than that of the slope, and which on the slopes of the structure are subject to the action of direct faults, should be considered the main factors controlling the deformation phenomena observed in the area between M. Moricone and M. Patino (Fig. 3.b).

The landforms characterizing the southern slope of Monte Calvo (between Antrodoco and Vigliano) are associated with analogous mechanisms, whereas the vast longish depressions bordered by antithetic scarps, which can be observed on the northern slope of the same structure, are in all probability connected with expansion phenomena (Fig. 3.c) of the limestone masses as a result of the decline in the mechanical characteristics of the prevalently pelitic terrains underlying them.

CONCLUSIONS

The present study has evidenced landforms and structures, preserved in the calcareous terrains of the central Apennines, which cannot be explained by hypotheses based on the action of faults, thrusts and backthrusts. Their genesis is instead attributed to the gravitational collapse of whole geological structures, or to portions of them, where the stratigraphic-structural features exercise a passive control while the seismic activity is an important active factor.

A systematic study of these impressive phenomena, reported here for the first time, is
thus required, given that they can be considered valid indicators of neotectonic and seismic activity. Moreover, the great importance of these implications in the field of seismic zoning and the projecting of large-scale civil engineering works is only too clear.

Fig. 3 - Block-diagram showing the gravitational deformation phenomena: a) Vettore Mt.-Chienti river area; b) Patino Mt.-Moricone Mt. area; c) Calvo Mt. area.
1) Limestone; 2) pelitic sediments; 3) overthrust; 4) normal fault; 5) trench or elongated depression; 6) tectonized zone.
REFERENCES


Dramis, F., Farabollini, P., Gentili, B. & Pambianchi, G. Neotectonics and large-scale gravitational phenomena in the Umbria-


Engineering geological zoning of São Paulo State (Brazil) – Scale 1:500,000
Zonage géotechnique de l’état de São Paulo (Brésil) à l’échelle 1:500.000

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ABSTRACT: This study presents a preliminary assessment of an engineering geological mapping in terms of geotechnical problems, construction materials, aquifer contamination and probability of occurrence of hazards. The São Paulo State, located in the southeast region of Brazil, has 248,898 km² of area and it is divided into 750 cities and is delimited by parallels 20° and 25° S and by meridians 45° and 52° W. The studies were based in the geological characteristics (rocks, relieves, unconsolidated materials, groundwater) and evidences of geotechnical problems that are now occurring. Lithologies of the three rock groups that are covered by unconsolidated materials (residual and transported) with thickness between 0.5 and 30 m, occur in the region studied. São Paulo State was divided into 18 regions that are constituted by different geological geotechnical characteristics and presents probabilities of occurring different kinds of hazards with various intensities.

RÉSUMÉ: Cet étude présente une évaluation préliminaire de la cartographie géotechnique avec rapport aux problèmes géotechniques, matériaux de construction, contamination des eaux souterraines et probabilité d'occurrence d'événements dangereux. L’état de São Paulo est localisé dans la région sud-est du Brésil et présente une superficie de 248.898 km², est divisé en 750 villes et est délimité par les parallèles 20° et 25° S et pour les méridiens 45° et 52° W. Les études ont été basés sur les caractéristiques géologiques (roches, relief, matériaux non consolidés, nappe phréatique) et sur les problèmes géotechniques que se rencontrent actuellement. Dans la région se rencontrent lithologies appartenant: à trois groupes de roches qui sont recouvertes par matériaux non consolidés avec épaisseur entre 0,5 et 30 mètres. L’état de São Paulo a été divisé en 18 régions qui sont constituées par des caractéristiques géologiques et géotechniques différentes et qui présentent des probabilités d'occurrence de types différents d'événements dangereux avec plusieurs intensités.

1. INTRODUCTION

São Paulo State is located in the southeast region of Brazil and is delimited by parallels 20° and 25° S and by meridians 45° and 52° W with a total surface area of about 248,000 km². The most developed and industrialized region of Brazil, the São Paulo State has 750 urban areas with population ranging from 1,500 (in the little cities) to 11,000,000 inhabitants in the city of São Paulo. The increasing development in the last decades and sometimes the irrational occupation of some areas has led to some problems as, for example, soil erosion, landslides and water pollution. These problems are not extended to the whole State, but occur mainly in some sub-regions where the intensity of occupation is accelerated and where the environment geology limitations are large.

As to know the needs of the region and to furnish guides to prevent the referred problems, several studies of engineering geological mapping, in various scales, were developed in the last ten years and were adapted in an unique map that is presented in this paper.

GEOLOGY

Figure 1 shows geological constitution of the region studied and Table 1 lists the Geologic Formations
EXPLANATION

1, X, A - Geological formations and Groups (see Table 1).
C6, C7 - Principal corridors of occupation
- Cities
- Geologic boundary

Figure 1 - Geologic and principal corridors map - São Paulo State (Brazil).
<table>
<thead>
<tr>
<th>GEOLOGY</th>
<th>TIME</th>
<th>GROUP OR COMPLEX</th>
<th>Codex</th>
<th>FORMATION</th>
<th>LITHOLOGIES</th>
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<td>Quaternary</td>
<td>Maciço Pequeno Group</td>
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<td>Varginha Complex</td>
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and their lithologies.

METHODOLOGY

The documents were elaborated based in existing geologic maps and reports (IPT, 1981; Brasil, DNPM, 1979), in engineering geological mapping (Paraguassú et alii, 1991; Lollo, 1991; Nishiyama, 1991; Zuquette et alii, 1992; Zuquette & Gandolfi, 1992) that were elaborated at scales larger than the this work, and in fields investigations specially performed for this map. These informations allowed the recognition of zones that were delimited by the following aspects: a) the predominant lithology; b) the occurrence of natural events (soil erosion, landslides, subsidences and flood); c) the occurrence of other geotechnical problems, such as soil collapse or soil expansion; d) the presence of unconsolidated materials; e) the presence of material for use in landfills and f) the presence of aquifers and its susceptibility of contamination.

Figure 2 shows the engineering geological zoning map and Table 2 summarizes the correspondent engineering geological units.

CONCLUSION

According to the methodology used in this work, a synthesis map was elaborated for the engineering geological units a scale 1:500,000 and reproduced at a scale 1:5,000,000, allowing for global assessment of the geotechnical characteristics of the studied area. This work can be used in the orientation of more detailed studies in this regions.

BIBLIOGRAFY

EXPLANATION

2.5...8 - Basic engineering geologic units (see Table 2)

Figure 2 - Engineering geological zoning map - São Paulo State (Brazil).
<table>
<thead>
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<th>Units</th>
<th>Basic characteristics</th>
<th>Class</th>
<th>Lithologies or textures</th>
<th>Characteristics</th>
<th>Sub-class</th>
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<th>Declivity (%)</th>
<th>Unfavourable attributes for occupation</th>
<th>Potential hazards</th>
<th>Others informations</th>
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<td>&gt; 10</td>
<td>0-2</td>
<td>- low ground water level; slope instability</td>
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<td>&gt; 10</td>
<td>0-2</td>
<td>- settlement; low ground water level; slope instability</td>
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<td>&gt; 15</td>
<td>- low ground water level; slope instability</td>
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**Notes:**
- conglomeratic sandstones and silts
- fine sandstones
- coarse sandstones
- sandstones with clayey cement
- silts, sandstones
- fine and coarse sandstones
- conglomeratic sandstones
- sandstones with clayey cement
- silts, sandstones
- fine sandstones
- conglomeratic sandstones
- sandstones with clayey cement
- silts, sandstones
- fine sandstones
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- fine sandstones
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- sandstones with clayey cement
- silts, sandstones
- fine sandstones
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<td>dominant is higher 20%</td>
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A study of hidden active faults in Quaternary deposits: Implication for the construction of large-scale engineering projects

Un étude des failles actives cachées dans des dépôts Quaternaires: Implications pour la construction de grands projets de génie civil

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ABSTRACT: Active faults need to be incorporated into major engineering construction designs such as nuclear power plants, dams, and high-building. When active faults are inferred to exist beneath the surface of Quaternary deposits, they can be difficult to reveal. Some geophysical exploration methods, such as the controlled sourced audio-frequency magneto telluric method, the shallow seismic reflection methods, the resistivity electrical methods and the geotomography methods among boreholes and surface, can be applied in order to detect faults under the Quaternary deposits. These geophysical results are compared with observations made in trench excavations and borehole logs. The trench excavations also reveal the latest activity and any reactivation of the faults.

Resume: Lors de l'élaboration des grands projets de génie civil tels que la construction des centrales nucléaires, des barrages et des grands bâtiments, il est important de ne pas ignorer les failles actives. Lorsque l'on suppose que des failles actives existent en dessous d'une surface couverte de dépôts quaternaires, elles peuvent être difficiles à détecter. Certaines méthodes d'exploration géophysique telles que la méthode magnéto-tellurique d'audio-fréquence à source contrôlée, les méthodes sismiques de petite profondeur, les méthodes électriques et les méthodes géotomographiques entre les trous de sonde et la surface peuvent être utilisées pour détecter des failles sous les dépôts quaternaires. Ces résultats sont comparés avec les observations faites dans les creusements des fossés et les trous de sonde. Les creusements des fossés révèlent aussi la plus récente activité ainsi que toute réactivation des failles.

1. INTRODUCTION

Because the Japan island arc is located at the junction of three tectonic plates, with the Pacific and the Philippine plates subducting beneath the Eurasian plate, earthquakes are common phenomena along the length of the Japan island arc. Studies of earthquake have been made in Japan since the 1960's utilizing such technique as the study of geomorphology, geology and volcanology, geodetic measurements, and the application of seismology. More than 3000 active faults are described in "The Active Faults In Japan" (Research Group for Active Faults in Japan, 1990), and about 1800 faults of them are thought to be present certainly.

The design of an important structure, such as nuclear power stations and dams, needs to incorporate the effects of earthquakes from active faults. The criteria, or guidelines, for earthquake design for various structures are detailed in various Japanese building codes.

Fig.1 Localitions of the studied faults
Table 1 Geological settings of the studied faults

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Locality</th>
<th>Vertical Displacement (m)*</th>
<th>Fault Dip</th>
<th>Thickness of fractured zone (m)</th>
<th>Thickness of overburden (m)</th>
<th>Facies of overburden</th>
<th>Facies of bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fukouzu</td>
<td>Tokouji</td>
<td>1</td>
<td>40-61</td>
<td>4-5</td>
<td>5</td>
<td>gravel, sand</td>
<td>schist, diorite</td>
</tr>
<tr>
<td></td>
<td>Nishi-</td>
<td>fukouzu 2 56</td>
<td>5.5-8</td>
<td>3.8-6</td>
<td></td>
<td>gravel, silt</td>
<td>schist</td>
</tr>
<tr>
<td>Kawafune</td>
<td>Ourasawa</td>
<td>1.5?</td>
<td>?</td>
<td>?</td>
<td>27</td>
<td>gravel, sand</td>
<td>mudstone, conglomerate, andesite</td>
</tr>
<tr>
<td></td>
<td>Yatoma</td>
<td>9 35-60</td>
<td>4-13</td>
<td>4-13</td>
<td></td>
<td>gravel, sand, silt</td>
<td>tuff breccia, mudstone</td>
</tr>
<tr>
<td>Umehara</td>
<td>Jjira</td>
<td>?</td>
<td>?</td>
<td>15-70</td>
<td></td>
<td>gravel, silt</td>
<td>sandstone</td>
</tr>
<tr>
<td>Nagao</td>
<td>Shin-</td>
<td>hwaskibashi 63 40-60</td>
<td>15+</td>
<td>45-105</td>
<td></td>
<td>gravel, sand, silt</td>
<td>granite</td>
</tr>
<tr>
<td>Yamasaki</td>
<td>Furumachi</td>
<td>less than 1** 90?</td>
<td>?</td>
<td>4.5</td>
<td></td>
<td>gravel, sand</td>
<td>slate, tuff</td>
</tr>
<tr>
<td></td>
<td>Nishimati</td>
<td>less than 1** 70?</td>
<td>?</td>
<td>?</td>
<td></td>
<td>gravel, sand</td>
<td>slate</td>
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<tr>
<td>Morioka</td>
<td>Hanamaki</td>
<td>?</td>
<td>30-60</td>
<td>?</td>
<td></td>
<td>gravel, sand</td>
<td>silt</td>
</tr>
</tbody>
</table>

* fault vertical throw of upper part of bedrock
** mainly strike slip component

In areas covered by alluvial deposits, the presence of faults is particularly difficult to discuss whether or not they are active. The presence of active faults is confirmed through the survey of lineaments, fault outcrops and so on in mountain areas. The evaluation of activity of faults hidden under the Quaternary deposits, which consists of gravel, sand and silt, is very important for engineering application.

This paper reports a systematic procedure for detecting active faults that may be hidden under the Quaternary alluvial deposits. In order to standardize the surveying method, case studies were made of active faults that are compared to active faults having varying thickness of Quaternary deposits. The locations of the studied faults are shown in Figure 1 and their geological settings are described in Table 1.

2. THE METHOD FOR THE DETECTION OF THE ACTIVE FAULT

The method used in the detection of active faults hidden under the Quaternary deposits is essentially the same as used in mountain area (Figure 2).

The main difference between these methods is thickness of deposits covered faults. In mountain areas, deposit that covers faults is very thin or not present. The existence of active faults beneath the Quaternary alluvial deposits can be confirmed by utilizing the following criteria (Figure 3).
In order to detect

1. Surface Geomorphological Displacement
   - Lineament
   - Side Scan Sonar

2. Dislocation in the Quaternary deposits
   - Precise Electrical Survey
   - Resistivity Tomography
   - Seismic Reflection in Marine
   - Seismic Reflection on Land

3. Dislocation at the Uppermost Basement
   - Refraction Method

4. Distraction in the Basement Stratum
   - Geochemical Survey
   - CSAMT Method
   - Seismic Tomography

5. Fractured Zone or Facies Boundary

Fig.3 The relationships between available survey methods and corresponding fault characteristics of geological phenomenon for detection of active faults under the Quaternary deposits.

1) Survey of the fault position beneath the Quaternary deposits using geomorphological information obtained through historical records, aerial photographs and field surveys.

2) Noting any dislocation of either the Quaternary deposits at surface, the contact between deposits and basement rocks, or within basement rocks themselves. Application of geophysical methods, drill holes and logging of trenches can be used to detect any dislocation.

3) The presence of faults can also be achieved through interpreting information related to fractured zones by geophysical methods.

The likelihood of active fault existing in an area covered by alluvial deposits can be evaluated by the following means; (1) literature survey of historical record detailing information on active faults, (2) extrapolating faults from area not covered by alluvial deposits to those deposits that are, (3) signs of fault activity seen on the surface of the alluvial deposits. An approximate estimation fault position, the scale of the fault, the thickness of the Quaternary deposits and the deformation of the deposits can be shown through application of geochronological and geophysical surveys such as the controlled sourced audio-frequency magneto telluric method (CSAMT), the precise electric survey, the shallow seismic reflection survey, and the seismic reflection survey. The thickness of the alluvial deposits and the form of the underlying bedrock are examples of the type of information that can be derived from the above surveys. These results are compared to information obtained from boreholes. Orientation of fault planes, character of faults and orientation of bedding planes in both the alluvial deposits and bedrock is measured by direct observations made by borehole television system.

Evaluation of fault activity is very important, and, in Japan, is discussed mainly on the basis of the last time of faulting. Commonly, geological methods, in particular, the investigation of relationship between faults and overlying layers, are used to date fault movement, and 

14C method using organic materials in the Quaternary deposits is used for dating. If need, trench excavation is carried out for this purpose. However, it is very difficult to date it by geological methods at the site where overlying young layers are not distributed. For this case, new fault dating methods are expected to be developed (for example, Electron Spin Resonance, Theraloluminescence methods using quartz grains in the fault gouge).

3. APPLICATION OF DIFFERENT TECHNIQUES USED IN DETECTING ACTIVE FAULTS

3.1 Controlled Sourced Audio-frequency Magneto Telluric method (CSAMT)

When a fault with a fractured zone of several ten to a hundred meters' lengths occurs beneath the alluvial deposits, the CSAMT method is able to detect it, due to the difference in resistivity between fractured zones and surrounding unfractured rocks. The CSAMT method works by measuring the resistivity of rocks by sending a current of several different frequencies (4.2-8700 Hz) from site a few kilometers apart, through the ground from a measuring point. The different frequencies correspond to the different resistivities of strata at various depths. Resistivity values of fractured zones are very low, while those of surrounding...
Fig. 4 Results of the CSAMT method at the Umehara fault site, Gifu Pref., Central Japan. The upper shows the location of supposed fault that is derived from the lineament and historical earthquake records. The lower shows the result of the CSAMT survey of the A-line of the upper figure. The resistivity of the ground, where the Umehara fault is estimated, is lower than the surrounding rocks.

(Topographic maps are parts of 1/25000 map sheets of Geographic Survey Institute.)
unfractured rocks are much higher.

An example of the CSAMT method to
detect the Umehara fault, active fault in
Central Japan, is shown in Figure 4.

3.2 Shallow seismic reflection survey

Originally the seismic reflection
method has been used for the deep part
(several kilometers) exploration (e.g.,
oil-exploration).

The shallow reflection method is an
improved seismic reflection method with
high resolution, and enable to detect
fault position that is at the depth of
less than 100 meters. This method also
enables to distinguish strata having
different sonic impedance, and is
especially applicable for detection of
contacts between basement rocks and the
overlying Quaternary alluvial deposits,
due to the large sonic impedance
difference.

As a result of this method, an
example of the Morioka fault is shown in
Figure 5. In this exploration, reverse
displacement of the Morioka fault in
Northeast Japan was detected.

3.3 Resistivity tomography method

The principal of geotomography
method is based on that of the X-ray CT
scanner. This method has the advantage
that results of a geological cross-section
between boreholes can be displayed as a
visual image. The principal behind the
resistivity tomography is to send an

![CMP NUMBER](image)

Fig.5 The result of shallow seismic reflection survey at the Morioka fault, in Iwate
Pref., Northwest Japan. The Morioka fault is estimated as a reverse fault to be upper
left of the figure between E and A layer. A is correlated to the Quaternary deposits, B
is the Neogene sedimentary rock, and E is the Miocene sedimentary rock.
Fig. 6 The result of the resistivity geotomography method at the Nagao fault, Takamatsu Pref., Southwest Japan. The left shows the geological cross section from boring log. The basement rock consists of Mesozoic granite. The lower part of the Hitoyo Group is composed of gravel of lower Pleistocene. The upper part of the Hitoyo Group is composed of gravel and sand of late Pleistocene. Both parts of Hitoyo group are deformed by the reverse fault of 40 to 60 degree dip. The right is the result of the resistivity geotomography. It shows that the resistivity of granite is high, and that the deformation of the layer of low resistivity is clear. The fault position is vague due to the small resistivity difference of layer between faults.

electric current from one point of the borehole, which is then recorded at various points of another boreholes, enabling the resistivity of the ground between the two boreholes to be measured. The electric current point is then moved to another position a few meters from the previous position and the whole procedure is repeated. These data are then analyzed by computer in order to find the most suitable condition.

As a result of this method, an example of the Nagao fault, located in Southwest Japan, is shown in Figure 6.

4. SUMMARY

A systematic approach for the detection and evaluation of active faults hidden beneath Quaternary alluvial deposits has been described in the paper. Recently new or improved geophysical methods have enabled these faults to be detected. Some other methods have also been applied in such as the seismic refraction method, the geochemical method, the ground radar system, the P-wave geotomography method and the precise electrical exploration method. This paper was compiled from the results of case studies of the Fukouzu, Kawafune, Yamasaki, Neodani and Umehara earthquake faults and the Nagao and Morikawa faults that are not earthquake faults but still active faults.

The study was performed by active fault group of CRIEPI (The Central Research Institute of Electric Power Industry), Japan. The authors are indebted to many CRIEPI colleagues who shared data and observations concerning the various applied techniques. Special thanks go to Messrs. K. Miyakoshi, S. Toda, K. Suzuki and Dr. Y. Fujimitsu, with whom we worked in the field.
REFERENCES

Engineering geological zonation and the related evaluation in Pearl river mouth basin of South China Sea

Le zonation de géologie de l’ingénieur et l’évaluation sur le bassin de l’embouchure de la rivière des Perles de la mer du sud de la Chine

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ABSTRACT: This research is based on the data of Project CFR/85/044 supported by UNDP. According to the similarities and diversities of engineering geological conditions, the Pearl River Mouth Basin can be divided into 2 zones, and 5 subzones further. The comprehensive evaluation of water depth, hydrodynamic environment, topography, soil mechanics, gravity and tectonism shows that the engineering geological conditions are best on the middle shelf with mixed soil, good on the outer shelf with coarse soil, poor on the inner shelf with fine soil and the upper slope with rock and reef, worst on the upper slope with soft soil.

RÉSUMÉ: Cette recherche se base sur les valeurs du projet CFR/85/044 supporté par UNDP. D’après la similitude et la diversité des conditions de la génie géologique, le bassin de la bouche du fleuve des Perles est partagé en deux zones et cinq sous-zones, dont la meilleure condition de la génie géologique est le support milieu avec une mélange de sol, la moyenne condition est le support intérieur avec fin sol et la pente supérieur avec roche et seif, la plus mauvaise condition est la pente supérieur avec doux sol. Cette recherche offre un fondement scientifique à l’exploration océanique et au plan général de la Mer Sud. Au même temps, cette recherche permet aussi le développement de la géologie de la génie océanique.

INTRODUCTION Pearl River Mouth Basin, the largest sedimentary basin in the northern part of South China Sea, is rich in oil and gas resources. The oil exploration has led to an increased awareness of the importance of engineering geological investigation data of Project CFR/85/044 supported by UNDP. The classification principle and evaluation method of the marine engineering geology in the Pearl River Mouth Basin is put forward.

1 CLASSIFICATION PRINCIPLE

The marine engineering geological zones are demarcated according to the similarities and differences of geotectonic units and by comprehensive conclusion of engineering geological characteristics. The research area is divided into 2 zones, the shelf zone and the slope zone, which determine the basic features of regional engineering geology. Further, 5 subzones are carved out, which are the fine soil subzone (Ia) of inner shelf, the mixed soil subzone (Ib) of middle shelf, the coarse soil subzone (Ic) of outer shelf, the soft soil subzone (II A) and the rock and reef subzone (II B) of upper slope, according to geomorphology, soil composition, grain size, physical mechanics which reflect the complexity of engineering geological conditions.

2 BASIC FEATURES OF ENGINEERING GEOLOGY

2.1 The shelf zone (I)

The northern shelf of South China Sea is a natural extension of continent, slants smoothly down from NW to SW with the average dip angle 0° 03'–0° 04' and water depths of 0–220m, having 4 terraces with the depth of 15–25m, 40–80m, 80–100m and 110–130m respectively. The 80–100m terrace is smooth and wide. The total thickness of Mesozoic and Cenozoic strata is up to 10km, transferring upward from terrestrial facies to transitional and marine facies.

The structure of Pearl River Mouth Basin is characterized by the intersection of EW tectonic blocks and NS tectonic belts. The EW tectonic blocks are related to the Mesozoic tectonic activities, and the NS tectonic belts result from
Fig. 1 Zonation map of engineering geology in the Pearl River Mouth Basin
the expansion of oceanic plate in later period. These two tectonic units have great effects on engineering geological environments and potential geological hazards such as active fracture, landslides, seismicity. The shelf structure assumes depression-upheaval-depression with corresponding fine soil—mixed-soil—coarse soil from inner shelf to outer shelf from north to south. So, 3 subzones are carved out.

2.1.1 Fine soil subzone(I₁) of inner shelf

This subzone, lying in the northern part of Pearl River North Basin, is a shoaling part of the shelf, stretching east-westward. Following is its major features.

A. High hydraulic dynamics Strong longshore current, tidal currents and bottom currents lead to instability of sea-floor with erosion and sedimentation. The biggest wave in a century will cause landsliding on sea-floor and lead to severe destruction of sea-floor foundation, especially by storm surge.

B. Poor soil mechanics High plastic muddy soil covers this zone. The soil mechanics properties (listed in Table 1) show that the water content is beyond the liquid limit, the muddy soil assumes liquid-plastic form, and most of void ratios are greater than 1.5. So it is unfavourable for engineering foundation.

2.1.2 Mixed soil subzone(I₂) of the middle shelf

This subzone, lying in the middle shelf, is a mixed sedimentation area with present and ancient remnant sediments.

A. Complicated sea-floor relief Field investigation shows the sea-floor is ornamented by valleys, channels, depressions, ancient shoals, river beds, banks and dunes having elevation differences of 0.5~3.0m. The fluctuating assemblage of positive and negative bedforms is a direct hazard to undersea tunnels and pipelines.

B. Sedimentary faults The loose Quaternary sediments, with a great thickness and differences of sedimentation rates and compressibilities, often lead to differential sedimentation faults. There are 38 faults in NW or NWW orientation, found between 30 and 100m beneath the sea-floor. Although such faults extend in small scale, their accident occurrence will bring unexpected hazards to sea-floor engineering.

2.1.3 Coarse soil subzone(I₃) on the outer-shelf

This subzone lying in the outer shelf is an ancient remnant sediment area. The main features are:

A. Paleo-delta sediments This subzone has several multistage deltas developed during late Pleistocene. The paleo-delta at the water depth between 80 and 100m is the largest one, which can
be divided into east and west parts. The east part, projecting southward in fan shape, about 5000km², has radial channels and sand bars on the outer fringe with elevation differences of 2-3m. The west delta about 1000km² has secondary sand bars and shallow channels on its surface. At the depth between 100 and 180m, there is a palaeodelta. The south-west delta about 4800km² has an abrupt slope on its outer fringe with the elevation difference of 2-3m. The north-east delta about 80-1000km², has a great slope on its outer fringe. The delta sediments are mainly fine silty sand with partial silty or clayey soil. The sediment structure is typified by S shap e cross shape and compound S-cross shape. The complexity of sediment composition and bedding structure is unfavourable for sea-floor engineering.

B. Seismic activities. This subzone is located in the seismic zone at the north fringe of slope, with seismic activities. According to historical records, earthquakes with magnitude 8 or greater occurred in the vicinity of Xisha Channel, earthquakes with magnitude 5.0 occurred in the south-west area of Dongsha Islands, and the earthquakes with magnitude 6.25 and 5.5 occurred in the north-east area of Dongsha Islands. An earthquake with magnitude 5.5 occurred in this research area (20°50' N, 118°30' E) on September 14th, 1992.

Fig. 2 Buried palaeo-rivers and palaeo-deltas

2.2 The slope zone (II)

This subzone is referred to Nanwei Shoal and Beiwei Shoal where the base rock is exposed out of sea-floor or buried at the depths of several or tens meters beneath sea-bed. A. Intensive variation of sea-floor topography. The base rock in Nanwei and Beiwei shoals is exposed, bulging in NW with 380km². The Nanwei Shoal is 3.5-50km wide and 12ka long swelling from 285-303m from sea bed, having a smooth top surface and fault terraces at the north and south flanks with the maximum fault width of 20m. The Beiwei Shoal is 5-8km wide, bulging upward 187-202m from

2.2.1 The soft soil subzone (I1) in the upper slope

This subzone lies in the upper part of the slope from the folding line between the shelf and the slope to the depth of 400m. A. Tectonic fault development. This subzone is characterized by the intensive neotectonic movement with 2 fault groups in NW-NNE, which occur in the density of 1-3 faults per km, with grabens and horsts buried at the depth from tens to hundreds meters. The neotectonic movement is a major geological hazard in this subzone.

B. Instability of soil mass. According to the theory of unlimited soil slope stability, the soil mass on the sea-floor is unstable in this subzone, because its safe coefficient is less than 1 under gravity control. Turbidity current, wave and earthquake are prone to causing landslide in this research area. The field investigation shows that the sliding bodies are distributed in stripe form in NW with a great scale and most of them occurred recently or are taking place now because of the clear sliding faces and V-shaped slump valley without any sediment filling. The repetition of old and new sliding in a same place makes up a multistage structure of sliding. Slump-mass is a bulging body which is the product in the stage after a landslide. Sliding, slumping and turbidity current are likely to occur together during any single event of mass gravity transport. The process is sliding—slumping—turbidity current, which is a potential hazard for sea-floor engineering.

2.2.2. The rock and reef subzone (Ib) on the upper slope
seabed; having fault terraces on the north and south flanks with slope angles of 15° -30° and the maximum up to 65°.

A. Mud-diapir development The soft intercalations may migrate destroy and intrude into the overlying layers or out of the sea-floor in the form of mud diapirs (Fig. 3) A round-shaped diapir in the south part of Beievi Shoal rises 10m out of seabed; a tower-shaped diapir in the south part of Nanwei Shoal rises 40m out of seabed. They are considered as geological hazards.

![Fig. 3 Single-channel seismic profile showing a mud diapir](image)

3 EVALUATION OF ENGINEERING GEOLOGICAL CONDITIONS

3.1 Geological environments

A. Water depth Shoaling area is favourable for seafloor engineering. The water depth condition is good in the inner and middle shelf, and is bad in the outer shelf and the slope.

B. Hydrodynamic condition The field investigation shows that wave erosion occurs to the depths of 10m and sediment transport by wave currents occurs to the depths of 30m in the inner shelf. In muddy current area the seabed is eroded and washed out, in wave shadow area such as Wanshan Islands-Dagan Islands is accumulated by sediments rapidly.

In the middle and outer shelf, the wave effects on the seabed are very weak but there are bottom currents. According to the data of hydrometric station C and station D, the bottom currents is to and fro in NW-SE with low velocities of 3-7cm/s, which reform the bedform slowly.

In the slope, turbidity currents erode the seabed, which are great hazards to oceanic oil engineering.

C. Sea bed topography The shelf is smooth and broad with the depth contours parallel to the coast line. The slope is abrupt with crowded and irregular contours, which is unfavourable for sea-floor engineering.

D. Sea bed geomorphology The fine soil subzone on the inner shelf is a smooth sedimentation plain without abrupt upheaval and depression. There are valleys shallow channels, shoals, coaxes and depressions with elevation differences of 0.5-3m in the mixed soil subzone on middle shelf. There are palaodeltas and active sand waves in the outer shelf. Turbidity channels occur in the soft soil subzone on the upper slope with high dip angle abrupt upheavals, fault cliffs, scarps and slumping valleys occur in the rock and reef subzone on the upper slope. The georographical analysis shows that the inner shelf is most favourable for engineering, the middle and outer shelf is favourable; the soft subzone on the slope is unfavourable and the rock and reef subzone is most unfavourable for engineering.

3.2 Soil properties

A. Soil types Field investigation shows that the sediments in the fine soil subzone of the inner shelf are accumulated during the last 8000 years. Abundant terrestrial sediments are provided to this subzone extensively, forming silty clay and clayey silt layers with the thickness of more than 5m and the maximum up to 28m being thinner from north to south. There are storm sediments occurring in small scale with various thicknesses between 0.15-5.5m, composed of fine sand, silt and silty sand, even gravel and mud pellets, which makes the soil properties more complex.

In the mixed soil subzone of middle shelf, the sediments are composed of remanent and present sediments, which are silt, clayey silty sand, sand and gravelly sand in some parts with poor sorting and double or multi-peaks of frequency. The frequency cumulative distribution curve is deep in lower and upper parts and gentle in middle part shows the sediments are provided from various aspects. The soil properties vary intensively in vertical and horizontal directions because of sea-level and palaeo-environmental changes.

The coarse soil subzone of outer shelf is typified by palaeo-shore sediments with coarse-middle sand, middle-fine sand, fine sand and gravel sand. The mineral composition of hornblende, pyroxene, feldspar and quartz is from acid igneous rock. Single peak of grain-size frequency shows the soil is good sorted.

The soft soil subzone of upper slope is mainly neritic soil with foraminiferas and globigerina, some of which is deposited by turbidity currents.

The rock and reef subzone of upper slope is granite and biohera exposed out of seabed or buried under sea-bed. The soil typic analysis shows that the mixed soil subzone of middle shelf is best, the coarse soil subzone of outer shelf is good, the fine soil of inner shelf is bad; the soft soil...
the zone of upper slope is worst in engineering condition.

B. Soil mechanics The soil in the inner shelf is high plastic marine mud with more than 50 percent of soil grains less than 0.1mm, water contents of 37.2-58.2 percent, void ratios of 0.94-1.44, liquid limits of 30-40 percent, plastic limits of 20-30 percent, plastic index of 5.56-2.28, plastic index of 6.40-20.50, cohesion of 11-23KPa, internal friction angles of 0.29°-1.15°, compressibility modulus of 1.2-3.3 MPa and compressibility coefficients of 0.55-2.27 MPa⁻¹. The soil mechanic properties are poor in this subzone.

In the middle shelf, the mixed soil (clayey silty sand, middle-fine sand) has liquid gravitations of 8.4-9.3 kN/m³, water contents of 24-40 percent, void ratios of 0.76-1.09, porosities of 44-49 percent, cohesion of 14kPa and internal friction angles of 23°.

In the outer shelf, the coarse soil (fine sand, middle-fine sand, coarse sand) has liquid gravitations of 7.0-8.5kN/m³, water contents of 23-28.5 percent, void ratios of 0.54-0.30, specific gravities of less than 40 percent, and internal friction angle of 30°-37°.

The soft soil (mud and silty mud) in the upper slope has a liquid gravitation of 5.3-3.7kN/m³, water contents of 38.6-74.5 percent, void ratios of 1.034-2.069, plastic index of 12.7-33.0, active index of 0.75-1.57, the sensitivities of 1.7-1.9, the coefficients compressibility of 0.078-0.210MPa⁻¹, compressibility modulus of 7.823-2.839MPa⁻¹.

The rock and reef in the upper slope are broken by the neotectonic movement or covered by silty mud with various thicknesses. The soil mechanic condition is poor.

3. Stability of soil mass on slope sea-bed

1. Instability caused by hydrodynamics and shallow gas activities. Wave erosion occurs in the inner shelf. The major storm surge causes the sand wave and barrier eroded filling depression rapidly, and rebuilding up new sand barrier, sand wave and sand mound. There are gas reservoirs buried shallowly under sea-floor. The gas spills out of seabed through faults, void spaces, fracture openings. The mighty hydrodynamics and active shallow gas are important factors causing sea-floor instabilities.

Shallow gas activities and hydrodynamic regimes are weak in the middle shelf.

In the outer shelf, the shallow gas occurring in deeper palaeo-delta sediments is CH₄ and C₈H₈ decomposed from organic materials by bacteriology or related to the deeper oil and gas reservoirs. The survey and calculation on the sand waves at the depths of 100m, 150m and 200m shows the migration of sand bodies is very slow at the velocity of 0.267-0.534 cm/yr for fine sand, 0.118-0.233 cm/yr for fine-middle sand and 0.101-0.152 cm/yr for middle sand which have no direct effects on the sea-floor stability.

Shallow gas has not been found in the soft soil subzone and the rock and reef subzone on the upper slope. The bottom currents are strong, playing an important role in sediment migration and deposition.

B. Instability caused by gravity. The field investigation, theoretical calculation and mathematical simulated test show soil masses on the shelf are stable under gravity regimes, the slope is unstable where sliding occurs under gravity regimes.

3.4 Neotectonic movement and seismic activities

A. Fault The inner shelf lying in the north fault terrace zone of the Pearl River Mouth Basin, has intensive neotectonic activities with successional faults in NEE orientation. In the middle shelf the neotectonic movement is weak with fewer active faults, but there are little interlayer faults formed by differences of sedimentary compression.

Active faults increase in the outer shelf. In the upper slope the neotectonic movement is intensive with abundant active faults.

B. Seismic activities. Incomplete statistics show 273 earthquakes with Ms>4 occurred from 1067 through 1988 in the northern South China Sea, among which there are 1 earthquake with Ms 8, 11 earthquakes with Ms between 7.0-7.3, 28 earth-quakes with Ms between 6.0-6.5, 135 earthquakes with Ms between 5.0-5.9, 88 earthquakes with Ms between 4.0-4.9. Few earthquakes occur in the middle shelf. Earthquakes increase in the outer shelf. According to historic record there were many great earthquakes occurring in the upper slope, which are the dangerous geological hazards.

The above analyses are listed in Table 2. The engineering geological conditions are best in the middle shelf, good in the outer shelf, poor in the inner shelf with fine soil and the upper slope with rock and reef, worst in the upper slope with soft soil.
Table 2. Comprehensive evaluation of regional engineering geological conditions in Pearl River Mouth Basin

<table>
<thead>
<tr>
<th>Geological Zone</th>
<th>Soil Type</th>
<th>Soil Mechanics</th>
<th>Stability</th>
<th>Geotectonic Activity</th>
<th>Seismic Activity</th>
<th>Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geological Condition</td>
<td>Hydrodynamics</td>
<td>Topography</td>
<td>Geomorphology</td>
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<td></td>
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<tr>
<td>Continental Shelf (I)</td>
<td>(Ia)细土亚层</td>
<td>00</td>
<td>XX</td>
<td>00</td>
<td>X</td>
<td>XX</td>
</tr>
<tr>
<td></td>
<td>(Ib)混合土亚层</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>I</td>
<td>00</td>
</tr>
<tr>
<td></td>
<td>(Ic)粗土亚层</td>
<td>X</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Continental Shelf (II)</td>
<td>(IIa)软土亚层</td>
<td>XX</td>
<td>X</td>
<td>XX</td>
<td>IX</td>
<td>XX</td>
</tr>
<tr>
<td></td>
<td>(IIb)刚土亚层</td>
<td>XX</td>
<td>X</td>
<td>XX</td>
<td>IX</td>
<td>XX</td>
</tr>
</tbody>
</table>

00 most favourable 0 favourable XX most unfavourable X unfavourable

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Pre-glacial river channels in the Tees Basin, UK
Des lits fluviaux préglaciaires dans le bassin de la Tees en Angleterre

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Abstract: The glacial deposits of the Tees estuary are products of the successive late Devensian ice sheets. The course of the present River Tees has varied since the end of the ice age both naturally and by the action of man. The old river valleys are now buried under many metres of boulder clay and laminated lake deposits. The variation of deposits and their thickness together with rock head levels are important for many construction projects. Available data was used to determine rock head contours. A preliminary investigation of rock head depths has led to the identification of the course of the pre-glacial River Tees.

Resume: Les dépôts glaciaires de l'estuaire de la Tees résultent de couches de glace successives du Devensian tardif. Le lit du fleuve la Tees a beaucoup changé depuis la fin de la période glaciaire, à la fois de façon naturelle et par intervention de l'homme. Les anciennes vallées du fleuve sont maintenant enterrées sous plusieurs mètres d'argile à blocs et de dépôts lacustres laminés. La variation des dépôts et leur épaisseur, ainsi que les niveaux supérieurs de la roche, sont des facteurs importants pour beaucoup de projets de construction. On peut utiliser les données disponibles pour déterminer les contours des niveaux supérieurs de la roche. Une étude préliminaire des profondeurs de la roche a permis l'identification du lit préglaciaire du fleuve la Tees.

1 Introduction

The River Tees in the North of England is one of the three major rivers in the region, but the only one with an extensive well-developed estuary flat. Prior to the eighteenth century the population of the Tees Basin was sparse. During the nineteenth century Middlesbrough and its surrounding area was extensively urbanised as the Iron and Steel manufacturing, Ship building and Chemical Industries developed.

The Tees estuary is covered with deposits of late Pleistocene in age; tills and laminated clays. The pre-glacial geomorphology of the area was very similar to that of the present day with the flat low lying estuary encompassed on three sides (north, west and south by higher ground). The bedrock stratum in the basin range from Lower Carboniferous to Lower Jurassic in age.

The majority of the glacial deposits are underlain by the soft Triassic sandstone's and marls into which the pre-glacial river Tees and its tributaries carved their river valleys. Adjacent to the river are the alluvial deposits of the Tees formed since the retreat of the last ice sheet. The low lying ground and urban development mean few exposures, consequently the character of the glacial deposits depends largely on evidence from boreholes and excavations for civil engineering works.

2 Glacial history and Geology of the deposits

Kendal (1902) described a tripartite division of the glacial deposits of the Tees estuary that he divided into Lower Boulder Clay, Middle Sands and Upper Boulder Clays. Both boulder clays have been recognised in east Northumberland, Durham and north east Yorkshire and are regarded as late Devensian in age.

Smith (1981,1982) supported the tripartite nature of the deposits and demonstrated that the two tills had significantly different erratic suites and clast fabrics. He considered that both were products of successive late Devensian ice sheets. The first deposited lodgement till and outwash as it retreated and these deposits were subsequently over ridden by
the second ice sheet that left behind its lodgement till and outwash deposits.

The Lower boulder clay was deposited in the pre-glacial Tees estuary by ice moving eastwards from the Lake District and south eastward moving ice from the western Southern Uplands. Ice moving down the Tees estuary was obstructed by Scandinavian ice which occupied most of the present North Sea and so the ice was diverted southwards, round the existing higher ground of the North Yorkshire Moors.

A warmer interglacial period followed and the ice retreated with sands formed as outwash deposits.

The Upper Boulder Clay, red-brown in colour, is assumed to have been deposited by ice from the Cheviot Hills and the eastern Southern Uplands. Ice collecting here moved first eastward into the Tweed lowlands, but was again obstructed by the Scandinavian ice and consequently deflected southwards along the north-east coast of England into the Tees estuary. Francis (1972) has reported up to three horizons within this red-brown boulder clay in the lower Tees valley.

Within the estuary the tills are thickest in the north and west and thin out to the east and south. At Norton the tills are 30m thick, 15m at Billingham and 10m in the centre of Middlesbrough and only 3m at Lakenby in the south of the region.

When the Cheviot ice retreated the drainage was blocked by the Scandinavian ice that lay just off the north east coast of England. This led to the formation of proglacial lakes, one of which occupied the low lying Tees estuary. It was in this lake that the Tees Laminated clay was deposited resting
directly upon the Upper Boulder Clay (Fig. 1). Agar (1954) and subsequent mapping by the Geological Survey have defined the outcrop of the Tees Laminated Clay as roughly oval in area about the estuary of the river Tees.

It has been suggested that the lake surface reached a height of 91m and more above ordnance datum (AOD), and that at that stage it was linked with lakes Humber and Wear (Radge 1939). Agar (1954) suggested that the sands that occur at the margins of the Laminated clay between 16.8m and 26.5m AOD represent shorelines. Agar demonstrated that at Park End, Middlesbrough the sands formed a shelf, 183m to 274m in length and were notched into the till below with which they had a sharp contact. The main deposits of these marginal sands are found on the southern margin of the proglacial lake against the Eston hills, which are the beginnings of the North Yorkshire Moors. On the northern margin the sands have only been seen at one or two localities.

It is known that an alluvial fan formed on the northern edge of the Tees estuary valley (Raistrick 1931). This formed when streams charged with sediment were prevented from flowing down the Wear valley, and were then diverted through a gap in the adjoining hills (Stevens et al 1991). This could have resulted in the marginal sands on the northern edge becoming reworked and mixed with the fan deposits producing a disjointed outcrop. The thickness of the sands on the southern edge is variable. This could be due to their origin being associated with the streams that must have flowed from the adjacent Eston Hills at the time the lake existed, with coarser material being rapidly deposited at the lake edge and finer silts and clay settling out in calmer deeper water.

The laminated clays have their greatest thickness of 9.1m approximately in the area of central Middlesbrough and thin laterally towards the margins of the outcrop. The Laminated clays consist of alternating layers of clay and silt-fine sand, the latter generally being very thin. In the central area the Tees Laminated Clay has some of the highest clay content and silt-fine sand partings are very thin. The laminations on occasion may be contorted or even absent with the clay size material consisting dominantly of the minerals illite and kaolinite with lesser amounts of quartz and chlorite, (Bell and Coulthard 1991).

The lacustrine environment is also demonstrated by the inclusions of small stones, balls of sand or masses of red boulder clay within the Tees Laminated Clay, where the laminations flow around them. These would have been deposited from rafts of ice.

Towards the margins of the deposit the silt-fine sand partings thicken whilst the clay layers become thinner. Frequently they grade upwards from coarse to fine material and occasionally are finely layered and can exhibit micro cross bedding.

The last ice sheet began retreating during a period 18000 to 13000 years BP and removal of the Scandinavian ice barrier enabled emptying of the lake and the re-establishment of the river Tees in its valley. The river flow would have been relatively high compared to present day levels, thus it was able to cut through the weak deposits very quickly. This was subsequently infilled through natural sedimentation helped by the changing sea levels and isostatic uplift. At present we therefore have a well-developed estuary with extensive alluvial deposits adjacent to the present course of the river, covering infilled channels and glacial sediments. The recent alluvial deposits of silts and clays also contain thick peat horizons that present difficult ground problems in areas that are being and have been recently developed.

3 Course of River Tees -- 13000 BP to present day

The River Tees and its associated deposits have been a central factor in the development of the urban areas within the Teesside area.

The first development in the region took place in the Middle Ages at the village of Yarm on the banks of the Tees upstream from Stockton. The site was dictated by the fact that it was the first place that a bridge could be built across the Tees with the technology available at that time, however, it was still 16 km from the sea.

An act of Parliament in 1762 allowed a bridge to be constructed across the river at Stockton. This provided a better port for shipping but navigation was difficult due to the meandering of the river, principally two large loops known as the Mandale Loop and the Portrack Loop, Fig 2. These loops and the associated sand banks became too much of a problem and in 1908 the Tees Navigation Company was formed. This instigated a new course for the river by cutting off the Mandale Loop and deepening the river in the process. The final straightening of the river was made in 1830 when a second act of Parliament enabled the construction of the Portrack Cut, 1006m long and 68m wide. The river was also enclosed within training walls, with the adjacent land being reclaimed and eventually used for wharves.

These constructions assured the development of the area, however, despite these two river cuts the depth of water at Stockton was insufficient for ships
loaded with coal and so the port was moved further down river. The new site where coal was shipped from became known as Port Clarence. Due to the proximity of the coal exporting port and its railway the area that became Middlesbrough developed attracting other industries including potteries, iron works, shipbuilding, engine works and foundries.

![Diagram of the River Tees](image)

**Fig. 2. Re-alignment of the River Tees**

4 Course of River Tees -- Pre 13000

Middlesbrough was primarily developed in the late nineteenth and early twentieth century. Major industrial development in the area started in the 1840's when the first iron works were constructed. In comparison to present day standards on the Tees these were small and it was not until the late 1880's that they expanded to become a major industrial area in Britain. During this time any deep borehole drilled was usually associated with the provision of a water supply.

A visual examination of the buildings in Teesside demonstrates the traditional box construction, very rarely of greater than three stories with low foundation loads. It is only in the last forty years that buildings have been constructed having larger foundation loads and requiring piling.

The development of industry along the river Tees has provided a comprehensive source of ground investigation material. Unfortunately the borehole data is not available in the form of an easily accessible database. All the data used in this study was obtained from the individual records of local and national civil engineering construction and ground investigation companies.

As a preliminary investigation the area chosen was either side of the river Tees from a point west of Stockton to the reclaimed estuary area west of Seal Sands, approximately seventeen kilometres in length, Fig. 3. On examination of the borehole records it was noticed that most did not contain any assessment on the origin of the materials, only a description, and in many instances the description was of extremely poor quality. Assessments had to be made regarding the origin of the sediments recorded from their descriptions consequently many records were eliminated and not used. Bedrock levels were more easily recognised and in conjunction with the outcrop pattern as mapped by the Geological Survey data could be obtained. Weathered bedrock levels were taken as the bedrock surface and not the alternative definition used by engineers when assessing the load carrying capacity of the strata.

The location of some of the borehole used are shown in Fig. 4, not all positions are recorded due to density and the confidential requirements requested by several companies. At each location there were usually more than one borehole in proximity reaching
down to bedrock and depending on the area covered by the construction site and the nature of the bedrock surface a representative log was used in the mapping.

The data for the bedrock surface was digitised and contoured, Fig. 5. The contours shown have been modified by hand after considering the glacial deposits in specific areas. Also computer contouring often produces closed contours in areas where it is obvious that they must be left open. This occurs where data is either lacking or on the edge of the mapped area.

5 Pre-glacial channel

The course of the pre glacial River Tees as indicated by the bedrock surface and the subsequent deposits is shown in Fig. 5 and Fig. 6. The river has generally followed a similar line but clearly shows steeper southern banks, than the northern ones. Boulder clay deposits thicken to the north and the bedrock surface shows an overall gentle slope in the same direction.

The river must have crossed this area during interglacial periods when the region was free of ice. River channels could have been carved in any previously deposited material which could have been buried by later glacial or lacustrine deposition. Buried river channels are known to exist and are at present under investigation.

The possibility exits that the channel as identified could in fact be carved after the start of the glacial period by the river cutting through the deposits to expose the bedrock again. The deposits immediately above the bedrock along the line shown in Fig. 5 are
thought to be boulder clay (apart from any river gravel) and therefore the channel is thought to be that of the pre-glacial River Tees.

Acknowledgements

The authors would like to thank Ferguson and Mellven for digitising the data and J. Rome for his help in the data collection and all companies who allowed us access to their records.

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Evaluation of individual effects of engineering-geological parameters for urban planning and construction

Évaluation des effets individuels des paramètres de la géologie de l’ingénieur pour la planification et construction urbaine

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ABSTRACT: Classical Engineering-geological maps, however, cannot meet the needs of designers in many cases for investigation of the reconstruction of old cities and development of new towns. Presented methodology of area evaluation trough grade of membership by Fuzzy sets theory so called Synthetic Engineering-Geological maps is even more evident in making comparison of concurrent variants in urban planning. This method is adequately adopted for computer elaboration.

RÉSUMÉ: Les cartes classiques ingénieurs-géologiques ne peuvent pas souvent satisfaire les besoins des designers pour la rechec de la reconstruction des villes anciennes et le développement des villes nouvelles. La méthodologie présentée de l’évaluation de la surface par le degré de la faîabilité de la théorie de Fuzzy sets, soit disant les cartes synthétiqes ingéniers-géologiques sont même plus évidentes, faisant comparaison des variantes concurrentes de la planification urbaine. Cette méthode, y compris l’application de Fuzzy sets est préparée d’une manière appropriée pour le traitement à computer.

I DISCUSSION AND NEW OBSERVATIONS

The systematic and detailed Engineering-Geological study is one of the basic factors in the investigation and appraisal of the both the reconstruction of old cities and the development of new towns. For this purpose Engineering-Geological maps have been used. As an example of that Engineering-geological map is shown in fig. 1. These maps, however, cannot meet the needs of designers in many cases for following reasons:

a) Presented data are not classified for the purposes of designing.

b) The synthesis of the overall Engineering-Geological effects has not been performed. That is way the engineer-designer cannot gain the complete insight into the characteristics of the terrain.

c) The computer elaboration is one of the basic factor in modern design. By this reason Engineering-geological maps must bee adopted for computer elaboration, and digital shown.

In order to improve these maps many authors proposed the methodologies for reorganisation a terrain by Engineering-Geological aspects.

The computer elaboration is one of the basic factor in the modern design.

For this reason Engineering-geological maps must bee adopted for computer elaboration and digital technics.

The best tool for the engineering-geological mapping of terrain aimed at urban planning and construction, appears a book of Leget P. F. (1973). In this book the groups of engineering-geological facts have been systematized, being important for urban construction and illustrated by numerous historic cases from all over the world. It is useful as well to adopt terrain categorization after Sutic J. (1969) and Sutic J. at all (1972) based on complexity of construction criteria. Interesting methodology for making so called Synthetic Engineering-Geological maps have been used by Jovanovic V. et all (1976), Bottino C. and Civet M. (1986), Vlahovic M. (1989). Some of possible approaches for computer use in making Synthetic Engineering-Geological maps have been work out by Styles K. A. et all (1986) and Vlahovic M., Stula M. (1988). Use of Fuzzy sets in this paper is based on publications of Zadeh L. (1973) and Teodorovic D., Kikuchi S. (1991).
Fig. 1 - ENGINEERING-GEOLOGICAL MAP
Presented methodology of area evaluation through assures degree of Fuzzy set theory is not perfect and could be improved especially in choosing representative engineering-geological characteristics and their interpretation. It is necessary as well, to make some modifications of proposed methodology under complicated engineering-geological conditions or complex urban demands.

2 WORK METHODOLOGY

The proposed work methodology comprises several successive stages as:

a) Classification of competent engineering-geological characteristics (by one criterion analysis),
b) Categorization of competent engineering-geological characteristic according to complexity of construction conditions and cost, classified as two criterion analysis,
c) Drawing a network on the map of urban area,
d) Calculation individual indicators of quadrants of the urban area,
e) Total indicators summed up, and
f) Zone delimitation due to use categorization for urban planning and construction.

3 ONE CRITERION ANALYSIS

Basic engineering-geological characteristics influenced planning and construction in the urban areas are, as follows: morphology, petrology, laye ring, terrain stability, tectonics, hydrogeological and seismics charateristics of terrain.

3.1 Classification of competent engineering-geological characteristics

Aim ed at obtaining maximum of terrain selectivity, a detailed classification of individual parameters is needful. However, a very detailed classification slowdown map preparation. In this light a useful validation proposed by Sutic (1969), and Sutic et al (1972) is as follows:

- four points - optimum favorable terrain (construction is possible without any problem),
- three points - favorable terrain (construction is possible without great problems),
- two points - less favorable terrain (problems in foundation appear but can be solved economically),
- one point - unfavorable terrain (great problems are mit during construction asking for expensive technical interventions).

Introducing principles of fuzzy sets theory into this criterion effectiveness of selectivity and objectivity are rather improved easily proved in further explanations.

Morphology of terrain

Morphology can be classified as follows:
- four points - slope 2 - 5 %,
- three points - slope 0 - 1% and 5 - 10%,
- two points - slope 10 - 20% and
- one point - slope > 20%.

For the same slope terrain undulation could make construction conditions more heavy. Because of that, a correction of mentioned parameter is made in two directions in relation to terrain undulation, using fuzzy sets.

Petrology

Classification of petrological composition was made under the following criteria:
- four points - rock masses, GW, SW (USCS classification system) and their mixture,
- three points - GC, SC and their mixture, CI, CI,
- two points - CH, M and
- one point - less soluble rocks (salt, gypsum, anhydrite) and Pt. With this classification as well, the fuzzy sets have been used.

Layering

By using proposed classification, layering can be evaluated as follows:
- four points - massive rock,
- three points thick bedded rock,
- two points - layered rock and
- one point - shist.
Terrain stability

four points - terrain cannot be disturbed even under the influence of the most intensive natural processes,
three points - terrain can be slightly disturbed only under the influence of some intensive changes of natural processes,
two points - terrain in which only infrequent and significant engineering-geological processes (landslides, liquifaction etc) can occur, and
one point - terrain in which numerous active significant engineering-geological processes are present.

Tectonics

The following classification is adopted:
four points - terrain with no tectonic activity,
three points - terrain with minor faults without tectonic zones or mild tectonics deformations,
two points - narrow tectonic zones partially saturated with moderate tectonic deformation,
one point - thrusts, active faults, wide tectonic zones saturated and fold with various petrology composition.

Hydrogeological characteristics

Hydrogeological characteristics are important in evaluating water supply, foundation conditions and dewatering.

Water supply is complex and asks for separate Hydrogeological study. For foundation the most important parameters are: water permeability of rock (coefficient of permeability k- cm/sec) and depth to the water level (h).

The criteria are, as follows:
four points - k> 10^{-3} and h> 12m,
three points - k = 5 x 10^{-3} - 8 x 10^{-4} and h = 3 - 12m,
two points - k =8 x 10^{-4} - 4 x 10^{-4} and h = 1,2 - 3m,
one point - k < 4 x 10^{-4} and h < 1,2m.

Seismic

Classification is based on degree of seismicity (I - MCS) that is, on coefficient of seismicity degree (Ks), as follows:
four points - I < 6^{o} MCS, or Ks < 0,01,
three points I = 6 and 7^{o} MCS, or Ks = 0,01 - 0,02,
two points - I = 8 and 9^{o} MCS, or Ks = 0,05 - 0,1,
one point - I > 9^{o} MCS, or Ks > 0,1.

3.2 Drawing a network

Before starting the job on Synthetic map preparation it is necessary to impose a square mesh over engineering-geological map. Mesh dimensions deepen on map scale also of engineering complexity of engineering setup.

3.3 Calculation individual indicators

Each mesh element possesses number of points in accordance with criteria adopted for classification of composite engineering-geological factors. Sum of individual number of points (based on characteristics from classical Engineering-geological map) within each mesh element, represents parameter showing terrain usability for urban planing and construction.

3.4 Total factor of usability

Total factor of usability can be limited by mesh element or partial mesh area. Define of individual number, or sum of individual number of points possesses subjectivity of interpreters. Fuzzy sets theory is favourable mathematical tool by treat of subjectivity. For this reason the grade of membership $\mu(x)$ of Fuzzy sets theory is adopted for area evaluation. By sum of maximal theoretical number of points $\mu(x) = 1$, and by sum of minimal theoretical number of points $\mu(x) = 0$. 

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Legend

- $\mu_{1} = 0.83-1.00$  
  Optimum favorable terrain

- $\mu_{2} = 0.50-0.83$  
  Favourable terrain

- $\mu_{3} = 0.33-0.50$  
  Less favourable terrain 1

- $\mu_{4} = 0.17-0.33$  
  Less favourable terrain 2

0.57 - Grade of membership by one criterion analysis
10.57 - Grade of membership by two criterion analysis

Fig. 2 - SYNTHETIC ENGINEERING-GEOLoGICAL MAP
3.5 Zone delineation

Zone delineation is performed in accordance with adopted criteria. In some common cases four categories are separated as shown in chapter 3.1. If necessary, by applying fuzzy sets a number of zones can be increased. On the Synthetic engineering-Geological map (fig. 2) calculation, zoning, drawing and plotting was worked out by computer programme: "Sigmud 1" - for one criterion and "Sigmud 2" - for two criterion analysis.

4 TWO CRITERION ANALYSIS

4.1 Competent Engineering-geological characteristics

In specific conditions more over basic engineering-geological characteristics influenced planing and construction in the urban areas (as shown in chapter 3.1) some other characteristics can be implemented. Also, evaluation of individual influence within engineering-geological characteristics can be worked out. For instance, influence of petrologic composition is 30%, of morphology 20%, terrain stability 15%, hydrogeology 12%, layering 11%, tectonics 3%, and seismic 9%. Significant improvement in selectivity of this evaluation can be succeed by usage of fuzzy sets.

4.2 Classification of competent Engineering-geological characteristics

Classification of competent engineering geological characteristics by two criterial analysis comprises several successive stages as:

a) calculation basic number of points in accordance with criterion adopted in chapter 3.1,
b) define characteristics of specific conditions (for instance, influence of petrologic composition is 30%, of morphology 20%, terrain stability 15%, hydrogeology 12%, layering 11%, tectonics 3%, seismic 9%), and
c) calculation of competent Engineering-geological characteristics (basic number multiplicated by characteristics of specific conditions).

Drawing a network, calculation individual indicators of quadrants of the urban area and zone delineation is the same by two criterion analysis and one criterion analysis (as shown in chapters 3.2 - 3.5).

5 CONCLUSIONS

Users of Synthetic Engineering-Geological maps are enabled to get more insight about terrain suitability for urban planing than from Classical Engineering-Geological maps. Importance of Synthetic Engineering-Geological maps is more evident in making comparison of concurrent variants of urban planing and construction. This method including use of the fuzzy sets theory, calculations, zoning, drawing, and plotting is adequately adopted for computer elaboration.

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Developments in engineering geological mapping in Slovakia
Développements dans la cartographie géologique et géotechnique en Slovaquie

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ABSTRACT: The classification scheme of engineering geological maps compiled in view of new trends is presented and the developments in engineering geological maps constructing in Slovakia, as well as some new map types are described.

RESUME: Le système de la classification des cartes de géologie de l'ingénieur compilées du point de vue des tendances nouvelles est présenté et le développement dans le domaine des cartes de géologie de l'ingénieur créées en Slovaquie ainsi que quelques nouveaux types de cartes sont décrits.

1 INTRODUCTION

About two decades have gone since the Guide to preparation of engineering geological maps was published by IAEG & UNESCO (1976) where several examples of engineering geological maps differentiated according to purpose, content and scale were analyzed and discussed. The time has verified many of methodological approaches in preparing engineering geological maps laid down in the Guide (1976), however, some new trends in engineering geological cartography have been developed since that time.

As far as the presentation and interpretation of engineering geological data on engineering geological maps is concerned we may distinguish three basic types of maps recently prepared in our country:
- maps of engineering geological conditions depicting all relevant components of the geoenvironment by means of superposition,
- maps of engineering geological zoning where territorial units are characterized by a certain degree of homogeneity of engineering geological conditions and,
- maps of engineering geological suitability providing in a understood form the information on engineering geological features with regard to intended land use in terms of land suitability.

Each of them can be prepared as the analytical or comprehensive map and as the multi purpose or special purpose one (Table 1).

A special type of engineering geological maps are prognostic maps of anthropogeneous changes in the geoenvironment due to engineering activities. In these maps the expected changes (positive or negative) are evaluated either on qualitative or quantitative basis.

2 DEVELOPMENTS IN MAP COMPILATION

Slovakia is a country with reach tradition in preparation of various types of engineering geological maps at different scales. In the fifties and sixties the map-makers focused their attention on medium and large-scale maps, multipurpose and special ones serving various planning and engineering purposes. In 1965 a comprehensive multipurpose engineering geological map of Slovakia at a scale of 1:500 000 (Matula et al. 1965) was compiled, subdividing the rocks and soils (rock
<table>
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<th>Maps</th>
<th>Multipurpose</th>
<th>Special</th>
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**Comprehensive**

- Provide information on all components (rocks and soils, groundwater, geomorphic and geodynamic phenomena) of the Geoenvironment

  - for a variety of planning and engineering purposes
  - specific land use (industrial, residential,...) or specific interventions in Geoenvironment (siting of wastes)

  Geological features are evaluated

  - according to multipurpose classifications
  - by delimitation of homogeneous units in terms of engineering geological conditions
  - by delimitation of land units in terms of their suitability
  - according to special classifications
  - by delimitation of special zoning units
  - by delimitation of land units in terms of their suitability

---

**Analytical**

- Provide information on individual component (its characteristics) of the Geoenvironment

  - for a variety of planning and engineering purposes
  - specific land use

  Geological features are evaluated

  - according to multipurpose classifications
  - by delimitation of zoning units with approximately homogeneous character of selected components
  - by delimitation of land units in terms of the component's suitability
  - according to special classifications
  - by delimitation of zoning units with approximately homogeneous character of selected component
  - by delimitation of land units in terms of the component's suitability

---

Table 1.
Classification of engineering geological maps
environment) into lithological suites and delimiting on the basis of zoning engineering geological regions and areas.

Following the publication of this map a series of multipurpose engineering geological maps at a scale of 1:25 000 has been prepared, covering the most important economic, industrial and urban centres of the Slovak territory (Fig. 1). Within this programme the mapping activities concentrate to 6 more urban areas at the same scale. Each of these maps consists of 2 parallel map sheets. The first map sheet documenting engineering geological conditions describes lithological complexes and lithological types, the second sheet represents an engineering geological zoning map (zones and subzones are delimited). Both types of maps illustrate hydrogeological and geodynamic conditions and topographic features, as well. Examples of these maps have been published in the Guide to Preparation of Engineering Geological Maps (IAEG & UNESCO 1976).

In addition, within the period of 1982 to 1987 a set of multipurpose engineering geological maps at a scale of 1:200 000 was compiled, recording on 12 map sheets the entire territory of Slovakia (Matula 1982; Matula, Hrašna & Vlčko 1986). These maps prepared by the Department of Engineering Geology at the Comenius University Bratislava were published in 1990. In the process of their compilation the same methodology was used as in the case of the 1:25 000 scale maps. With regard to the smaller map scale the rock environment was classified at the level of lithological complexes and the territory at the level of zones. In compliance with the scale of the map the topographic features, groundwaters and geodynamic phenomena were depicted with detailness corresponding to it.

The multipurpose engineering geological maps at the scale of 1:200 000 represent a unifying base for the mosaic of the 1:25 000 scale maps. They encompass the important urban areas in which the more detailed maps have been prepared with adjacent territory which is in harmony with its land use documented in a smaller detailness. Thus a complex information system providing general information on the entire state territory has been created which may also utilize substantial details concerning important urban areas presented in the large-scale maps.

As the gravitational slope movements represent the most important geodynamic phenomena in the Slovak territory some sets of maps dealing with slope stability at a scale of 1:25 000 and 1:10 000 were prepared for the most endangered areas, as well. These maps consist of some parallel map sheets (Malgot & Balak 1993).

The experience of map users indicates that maps of engineering geological zoning are more accessible and useful to a wide public than maps of engineering geological conditions since they record the same information in a rather compact way, concentrating on a specified land unit. The need has arisen to produce besides the widely used multipurpose engineering geological maps also special purpose maps of engineering geological zoning, predominantly maps of engineering geological land suitability, on which land units are classified in terms of their suitability for a specific purpose. These maps, prepared usually by a transformation of multipurpose engineering geological maps, are intended primarily for constructional purposes (civil engineering and industrial structures), for the route selection of road and motorway alignments, and, lately even for hazard assessment in relation to special kind of structures, mainly waste disposals. The unfavourable situation in waste disposal practice in Slovakia required the compilation of maps assessing the suitability of the entire state territory from the viewpoint of waste disposal siting. These maps appeared in 1990-1991 at a scale of 1:200 000 and in 1992-1993 at a scale of 1:50 000 (Hrašna 1993).

Despite a comprehensive information maps of land suitability are not likely to provide a satisfactory answer to the question which land unit is within the area of interest most suitable for a particular land use and neither can they help to identify the most suitable land use within a particular land unit. (The land suitability is determined by various geofactors or their combination in many cases displaying the same suitability class.) The real influence of the selected geofactors and their combination on the intended land use can be stated only by the help of their quantitative evaluation. Such an
Fig. 1 Medium scale multipurpose engineering geological maps of Slovak territory:
1 - completed, 2 - prepared

Fig. 2 Map of additive typological zoning
evaluation forms the basis for introducing engineering geological optimization analysis in map preparation procedure and thus producing the map of engineering geological optimization providing full information related to the land suitability for a particular land use. The methodology implemented in the process of data acquisition and the method adopted to compile medium-scale maps is described by Vlčko (1982).

A special type of zoning maps is represented by maps of engineering geological prognosis in which the delimited land units are defined in terms of the potential kind and intensity of changes of the geological environment resulting from different engineering activities. Up to the present time just a few maps of this type have been prepared in Slovakia, mainly in connection with the EIA process. The multicomponental engineering geological map is the basis for the generation of this new type. The method used in the process of compilation of this map will be discussed later on.

3 METHODOLOGY OF SOME NEW MAP TYPES COMPILATION

3.1 Maps of engineering geological zoning

On the map of engineering geological zoning territorial units are defined on the basis of a certain degree of homogeneity of engineering geological conditions. There exist two different ways of applying the concept of zoning to engineering geological maps: individual (regional) and typological zoning.

Individual (or regional) zoning is based on the delineation of units demonstrating engineering geological conditions which are unique within the mapped territory (they do not repeat). In typological zoning principles of typology are applied. In the latter type of map the term "homogeneity" is not understood so strictly and better fitting term would probably be "similarity" (quasi-homogeneity) of fundamental engineering geological features. Typological zoning results in dividing any mapped territory into a "mosaic" of certain types of zoning units which repeat irregularly and alternate with other types of zoning units. Typological zoning gives the opportunity to generalize and to transmit the experience concerning the behaviour of geoenvironmental components (rocks and soils, groundwater, land forms and geodynamic phenomena), to efficient investigation methods and foundation techniques in typologically similar engineering geological conditions, i.e. to apply the methods of engineering geological analogy. Two principal methods, the additive or the multicomponental typological zoning, may be applied. In additive zoning territorial units are delimited according to the similarity (quasi-homogeneity) of one component of the geoenvironment (e.g. similarity in arrangement of rock units). The other geoenvironmental components are in the map depicted traditionally using isolines, symbols, etc. The map with an extended legend provides a detailed information on engineering geological conditions covering many aspects of planning and engineering activities. An example of such map from Zázorská nízina Lowland is given in Fig. 2. The symbols of zones and subzones correspond to GUIDE (1976).

Multicomponental typological zoning is concerned not only with the evaluation of rocks and soils (lithology) but also with other fundamental geocomponents (or their attributes). The evaluation is done according to particular classification criteria. Each zoning unit delimited by this method reflects the quantitative characteristics of four fundamental geocomponents either for multipurpose or specific land use. The structure of a symbol of a zoning unit may be expressed in the following form:

F_n.k2g3I - M.1
V1.N - 0

meaning that the superficial fluvial deposits (Fn) are built by 2 to 5 m thick (2) alternating layers of loams and sands (k) underlain by gravels (g) 5 to 10 m thick (3), the pre-Quaternary bedrock is built by clays (l) occurring at a depth of 10 to 15 m below the surface (3); the slope with an inclination from 5 to 12 % (M) is slightly dissected (1). The depth of the free groundwater table (V) varies between 1 and 2 m (1), the groundwater is non-corrosive (N) and geodynamic phenomena do not occur (0).
3.2 Maps of engineering geological suitability

The concept evaluating the mapped area in terms of land suitability involves three steps:
1. selection of geological factors (geofactors) having a significant effect on land use for a given purpose,
2. ranking of geofactors into classes according to their influence on land suitability,
3. re-grouping of the geofactors into land suitability classes and their presentation on a land suitability map.

Geoenvironmental factors, as mineral resources, water resources, natural reserves etc. can be evaluated and depicted on these maps, as well.

The map of land suitability shown in Fig.3 was derived from the multipurpose engineering geological zoning map presented in Fig.2. The most relevant attributes of individual geofactors (bearing capacity, slope, depth to groundwater table, etc.) which have a direct bearing on urban development (residential, commercial and transportational constructions) were evaluated and ranked into two to four different classes (Table 2).

The following step was the re-grouping of the selected attributes of geofactors into three land suitability classes (suitable, moderately suitable and non-suitable). The land suitability units delimited on the map are indicated by capital (non-suitable) and small letters (moderately suitable). Areas suitable for urban development are without any designation. The evaluation procedure does not include geoenvironmental factors (mineral resources, water resources, fertile soils, etc.) which in fact represent constraints to urban development. These were evaluated in the preliminary stages of the decision-making process. The land suitability map for urban development can be prepared for different foundation depths. In Fig.3 is given an example of the map prepared for the foundation depth of 2 m below the surface.

3.3 Maps of anthropogeneous changes in the geoenvironment

For the preparation of these maps it is necessary to set up three different classifications:
1. classification of urban construction activities into geoenvironment,
2. classification of significant geoenvironmental changes due to urban construction activities

<table>
<thead>
<tr>
<th>Geo factors</th>
<th>Land suitability classes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>suitable</td>
</tr>
<tr>
<td>Bearing capacity (MPa)</td>
<td>&gt; 0,35</td>
</tr>
<tr>
<td>Ease of excavation class</td>
<td>1-4</td>
</tr>
<tr>
<td>Depth to water table (m)</td>
<td>&gt; 2</td>
</tr>
<tr>
<td>Corrosiveness of groundwater</td>
<td>-</td>
</tr>
<tr>
<td>Angle of slope (%)</td>
<td>&lt; 12</td>
</tr>
<tr>
<td>Seismic intensity (*MSK)</td>
<td>1-5</td>
</tr>
<tr>
<td>Slope movements</td>
<td>none</td>
</tr>
<tr>
<td>Gully erosion (km/km²)</td>
<td>&lt;1,5</td>
</tr>
</tbody>
</table>

Table 2. Classification of geofactors influencing urban development
Fig. 3 Map of land suitability for urban development

Fig. 4 Map of anthropogenic changes in the geoenvironment
3. Classification of different intensities of the geoenvironmental changes (inclusive the cost for corrective measures).

These classifications formed basis for the compilation of the map shown in Fig. 4 covering the same territory as the map examples presented before.

The evaluation of geoenvironmental changes due to urban activities was prepared by the help of a cross-impact matrix and enables their assessment in terms of technological and economic feasibility. The symbols assigned to zoning units express intensive (extensive) geoenvironmental changes associated with current remedial measures (capital letter) or contemporary geoenvironmental changes or changes with a lesser extent (small letter). The numbers in the symbol indicate the type of the urban construction activity (1 -foundation pits and road cuts deeper than 5 m, 2- earthfills greater than 5 m, 3- residential buildings up to 5 floors without basement, 4-residential buildings more than 4 floors with basement).

4 CONCLUSION

The use of engineering geological maps in Slovakia shows that multipurpose maps of engineering geological zoning prepared by implementing the method of additive typological zoning are most informative for current map-users. Urban planners and designers may derive the information they need predominantly from land suitability maps and prognostic maps of changes in the geoenvironment. Maps of this kind are compiled on the basis of multipurpose or special maps of multicomponental typological zoning. They can be easily digitized and used for the generation of different types of engineering geological maps.

REFERENCES


Matula, M. 1965. Synoptical engineering-geological map of Slovakia 1:500 000. Slovak Cartography Bratislava
Land suitability for ecologically hazardous developments location
Évaluation du terrain pour l’emplacement des activités écologiquement hasardeuses

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Comenius University, Bratislava, Slovakia

ABSTRACT: Main principles of land suitability assessment and suitability maps compilation are described. The relevant factors of geoenvironment and ladscape influencing the location of ecologically hazardous developments are defined and classified. In maps also those factors are evaluated which enable another way of land use.

RESUMÉ: Les principes essentiels de l'évaluation de la convenance du terrain et la compilation des cartes de la convenance sont décrits. Les facteurs relevantes de l'environnement géologique et du paysage qui influencent la localisation des structures hasardeuses sont définis et classifiés. Les cartes évaluent aussi les facteurs qui favorisent une autre possibilité d'usage du terrain.

I THE PRINCIPLES OF LAND SUITABILITY ASSESSMENT

Ecologically hazardous developments (chemical industries, petrol stations and warehouses, waste disposal sites, etc.) are a potential source of environment pollution, including the geoenvironment, and, particularly of groundwater pollution. Undesirable influence of such developments upon the environment may come suddenly either by failure of structures and technological crash or gradually by infiltrating of pollutants into geoenvironment.

The first way of environment endangering, which can have catastrophic consequences, can be prevented by a thorough selection of building sites, and, by a proper proposal of foundation and construction of structures. At the same time it is necessary to take into consideration, beside of physicomechanical properties of foundation soils, the tectonic and gravitational stability of the territory and other geodynamic phenomena (subsidence, flooding, sagging, etc.), as well.

While the solution of this problem has been quite well known for a long time, the second way of environment endangering, i.e. by gradual pollution of geoenvironment, has only began to receive the appropriate attention in recent years. From this standpoint the suitability of the geoenvironment for such structures siting is determined mainly by the structural arrangement of strata, characterized by various permeability and transmissivity, which occur near the surface of the land. They can be constructed with minimum environmental risk and at low cost on geoenvironment structures in which a relatively impermeable stratum (an aquiclude) of a sufficient thickness is near the land surface. Where a permeable stratum (an aquifer) is located near the land surface, protective technical measures are always necessary and their type and cost depend on the thickness and transmissivity of the aquifer.

In view of this a limited number of structural arrangement types of aquicludes and aquifers can be created (Table 1) which are characterized by a certain degree of the danger of groundwater pollution and by the cost of necessary protective
Table 1. Evaluation of the ecologically hazardous developments subsoil

<table>
<thead>
<tr>
<th>Danger of groundwater pollution +</th>
<th>Structural arrangement of aquicludes and aquifers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very high</td>
<td>1A4</td>
</tr>
<tr>
<td>High</td>
<td>1A3 2A3 2A4 1B3 1B4</td>
</tr>
<tr>
<td>Medium</td>
<td>1A2 2A2 3A3 3A4 1B2 2B2 2B4 1C3 1C4 2C3 2C4</td>
</tr>
<tr>
<td>Low</td>
<td>1A3 4A4 1B1 3B3 3B4 1C2 2C2 3C3 3C4 1D3 1D4 2D3 2D4</td>
</tr>
<tr>
<td>Very low</td>
<td>4B4 1C1 4C4 1D1 1D2 2D2 3D3 3D4 4D4 E</td>
</tr>
</tbody>
</table>

Transmissivity of the aquifers

- A \( \geq 10^{-3} \text{m}^2\text{s}^{-1} \)
- B \( = 10^{-4} \text{m}^2\text{s}^{-1} \)
- C \( = 10^{-5} \text{m}^2\text{s}^{-1} \)
- D \( = 10^{-6} \text{m}^2\text{s}^{-1} \)
- E \( < 10^{-6} \text{m}^2\text{s}^{-1} \)

Thickness (depth to the surface) of aquicludes

- 1 - 0 - 2 m
- 2 - 2 - 5 m
- 3 - 5 - 10 m
- 4 - > 10 m
- \( k_f \leq 10^{-7} \text{ms}^{-1} \)

Structural arrangement

- aquifer expressed by transmissivity
- thickness of the aquiclude
- depth to the underlying aquiclude surface

+ and/or costs of protective measures

measures to prevent it. In addition to the aquifers transmissivity the type of their permeability (I - porous, III - fissurcous, V - karst and their combinations - II, IV) may be expressed in the symbols of the structural arrangement of the strata (Hrašna 1993).

Besides factors mentioned above mineral resources, water resources and storages, protected forests, fertile soils and nature reserves should be taken into account in the site selection process, as well.

All factors which are under consideration can be classified in three categories: of informative, limiting and excluding ones. The incorporation of certain factors into the latter two categories is prescribed by law. This relates e.g. to nature reserves, protective zones of water resources, etc. The incorporation of other factors, especially of geodynamic phenomena and the danger of groundwater pollution, is the problem of professional assessment of their influence on land suitability for the given purpose. The informative factors do not limit the location of ecologically hazardous developments. They are the source of new information about further (legally unprotected) potentials of the territory and about the geological conditions of the construction, as well.

With respect to great amount of factors, which must be evaluated in spatial correlation, the best way of land suitability estimation is to compile a map. The map gives its users the spatial image of a territory which practically precludes to overlook the suitable localities and enables to avoid the unsuitable ones. So it save time and money connected with a wrong oriented investigation.

While in the maps of a small scale only the basic information can be given in the maps of a medium scale the geoenvironment can be divided into structural units according to danger of groundwater pollution and the full information above the majority of the factors mentioned above can be presented, as well. However, the maps of large scale in which all factors can be evaluated in a sufficient degree of detailness offer the best information for the given purpose.

In our paper, we demonstrate an example of medium scale map (at a scale of 1:50 000) in
Fig. 1  Land suitability map for ecologically hazardous developments
Documentation map I

Table 2. Explanation to the Documentation map I

<table>
<thead>
<tr>
<th>Water protected areas</th>
<th>Natural reserves</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Protective zone of mineral water resources of I., II. degree</td>
<td>19. National park</td>
</tr>
<tr>
<td>2. Protective zone of mineral waters resources of III. degree</td>
<td>20. Protected landscape area</td>
</tr>
<tr>
<td>3. Temporary protective zone of mineral waters</td>
<td>21. State nature reserve</td>
</tr>
<tr>
<td>4. Hygienic protective zone of surface and ground water resources of I. and II. degree</td>
<td>22. Protected finding place</td>
</tr>
<tr>
<td>5. Hygienic protective zone of water resources of III. degree</td>
<td>23. Protected park and garden</td>
</tr>
<tr>
<td>6. Catchment area of water courses used for water supply</td>
<td>24. Protected study site</td>
</tr>
<tr>
<td>7. Protective zone of groundwater storage</td>
<td>25. Natural monuments</td>
</tr>
<tr>
<td>8. Reservoir used for water supply</td>
<td>26. Protective forests</td>
</tr>
<tr>
<td>9. Reservoir used for water supply under construction</td>
<td>27. Special purpose forests</td>
</tr>
<tr>
<td>10. Floodland</td>
<td>28. Other productive forests</td>
</tr>
</tbody>
</table>
| 11. Spring used for water supply                                                    | Protecte
d areas of mineral deposits |
| 12. Hydrogeological well used for water supply                                       | 29. Exploitation area of surface deposit |
| 13. Mineral and thermal spring                                                      | 30. Exploitation area of surface deposit |
| 14. Mineral-thermal hydrogeological well                                             | 32. Protective area of subsurface deposit |
| Other hydrogeological and hydrological phenomena                                     | 33. Mining area under investigation |
| 15. Area with important groundwater storage                                           | Other large mineral deposit        |
| 16. Water - logged territory                                                         | 34. Unprotected surface deposit under exploitation |
| 17. Depth to groundwater table                                                       | 35. Surface deposit with terminated exploitation |
| 18. Direction of groundwater flow                                                    | Other small mineral deposits       |
|                                                                                      | 36. Quarry: A-operating, B-abandoned |
|                                                                                      | 37. Sand pit, gravel pit: A-operating, B-abandoned |
|                                                                                      | 38. Clay pit: A-operating, B-abandoned |
Fig. 2 Land suitability map for ecologically hazardous developments
Documentation map II

Fig. 3 Land suitability map for ecologically hazardous developments
Zoning map
which the danger of groundwater pollution and other factors of geoenvironment limiting the location of ecologically hazardous structures are presented. The map was compiled on the computer PC 80486 50 MHz, 16 MB RAM, using a PC ARC/INFO ver. 3.4 D plus.

2 THE MAP COMPILATION METHOD

The land suitability map for ecologically hazardous developments consists of three parallel map sheets. Two of them are documentation maps which comprise all data needed. The third one is the proper land suitability map elaborated on the base of the two previous maps using the zonation method.

The first documentation map (Fig. 1) includes all legally established limiting and excluding factors (which can delimit small or relatively large legally protected territorial units) and some informative factors concerning groundwater and mineral resources. The real types of factors are indicated by the numerical symbols (according to Table 2) in the map.

The second documentation map (Fig. 2) represents the degree of groundwater pollution expressed by the symbols of the structure and transmissivity of the geoenvironment (according to Table 1), as well as the geodynamic phenomena affecting the land-use. In harmony with Table 1, the map divides the whole territory according to the expected degree of groundwater pollution and the costs of protective measures (to prevent the pollution) into three types of territorial units which are indicated by patterns in the map. The influence of geodynamic phenomena upon the realization of structures (informative, limiting, excluding) is marked by different type of pattern in the map, as well.

The zoning map (Fig. 3) divides the mapped area according to its suitability for ecologically hazardous developments location into suitable, moderately suitable and unsuitable territorial units (zones). The moderately suitable and unsuitable zones are indicated by patterns which, at the same time, express if the reason of their incorporation into these two categories is given by law or by geological conditions. In the latter case the factual reason is expressed by alphabetic symbol. In the case of legally protected areas one can find the real reason in the Documentation map I.

3 CONCLUDING REMARKS

The development of ecologically hazardous structures in unsuitable territorial units is either prohibited or would require changes in land-use planning schemes, legalization, or costly technical measures preventing environmental impact (Hrašna and Matys 1993). Their location in moderately suitable zones is a question of consensus and/or of technological and econmic considerations. Problems may arise even in suitable territorial units, e.g. in relation to protection zones of various objects (such as motorways, etc.) and opposition of local residents. Further problems may occur with the sufficient distance from water courses, urban areas, etc. These limitations may, of course, occur in the two previous categories, as well.

The maps at a scale of 1:50 000 make it possible to evaluate all relevant factors of land-use for the given purpose in a sufficiently large area and in a reasonable detailed way. A more detailed dividing taking into account the specific features of various types of ecologically hazardous developments can only be performed in the maps of a large scale.

At compiling the maps of various scale it is useful to employ the computer technique which enables by automatic counting of particular factors digitalized layers to construct various types of synthetic maps, and, to change the criteria for their compilation at the same time.

REFERENCES

Bearing capacity in engineering geological mapping
Capacité portante utilisée en cartographie géotechnique

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L.M. Ferreira Gomes
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ABSTRACT: The utilization of the bearing capacity as a systematical factor for engineering geological mapping based on a standard foundation composed by a strip flexible footing with 2.0 metres width, at 1.5 metres depth and submerged is proposed for discussion. The theoretical way to get the bearing capacity by the generalized Terzaghi equation or other processes is referred and a classification for this factor is pointed forward.

RESUMÉ: On a proposé pour discussion la utilization de la capacité de charge comme un facteur systématique en cartographie géotechnique basé sur une fondation standard qui est composé pour une semelle continue avec 2.0 metres de large, à la profondeur de 1.5 metres et submergée. La equation généralisé de Terzaghi est présentée tel comme les autres façons de obtenir cette capacité de charge; pour ce facteur on propose une classification pour les sols et les roches.

The bearing capacity is an important factor for engineering geological propose when the area is mapped at scales above or equal to 1/10,000, because it can be interesting to make some decisions concerned with urban and regional planning.

Pasek and Ribar (1961), Simek (1962), Coelho (1980), Li Shenglin et al (1990) and Ferreira Gomes (1992) amongst others have used the bearing capacity in engineering geological mapping; these investigators have not used the same methodology to determine the bearing capacity, concerning the depth, footing shape and water table position. The standardization in engineering geological mapping is very important, but this is a non pacific subject; in the 1st IAEG Congress (1970) in Paris started some discussion and the "Guide for preparing geotechnical maps" published by Unesco/IAEG (1976) is a consequence of the discussion at that Congress. However, the bearing capacity is not referred in that guide, either in others subsequently; probably this occurrence is due to money consumption to obtain bearing capacity for large areas, by field methods or laboratory tests.

There are several field processes to get directly or indirectly the bearing capacity of the materials or their shear parameters (c and $\phi$):
• SPT (Peck et al, 1953; Meyerhof, 1956 and 1974); Parry (1977); Skempton;
• CPT (Sangerlat, 1967; Meyerhop,1956and 1974); Folque, 1976; Meigh, 1987; Robertson et al.1983), Schmertmann 1978; Villet et al.1981);
• DPL (Ladeira and Ferreira Gomes, 1994);
• Vane Test (Sangerlat, 1967, Bowles, 1988; Walker, 1986).

With laboratory tests one gets the same parameters; amongst several tests should be referred the following: direct shear, triaxial and vane test.
The shear parameters \( (c \text{ and } \phi) \) can be obtained by other relationships such as:

- Void ratio (Girandet, in Costet and Sanglerat, 1981; Lambe and Withman, 1979);
- Unit weight (De Beer, 1970, D'Appolonia et al., 1968; Peorir III, Moline, III, Plantema, 1957 in Bowles, 1988);
- Grain size parameters (Ladeira and Ferreira Gomes, 1989, and Ferreira Gomes and Ladeira 1989);
- Atterberg limits (Skempton, 1953);
- Compacity, form, rugosity and grains dimension (Costet and Sanglerat, 1981).

To determine the bearing capacity \( (Q_{ult}) \) for footings is recommended to apply the generalized Terzaghi equation (Vesic, 1973), with the water table at the surface:

\[
Q_{ult} = c N_c S_c d_c + \gamma_d N_d S_d d_d + \frac{1}{2} \gamma B N_r S_r d_r
\]

(1)

where:

- \( c \) is the cohesion of the soil;
- \( \gamma \) is the unit weight of the submersed soil i.e. \( \gamma = \gamma_s + \gamma_w \);
- \( \gamma_s \) and \( \gamma_w \) are the unit weights of the soil and the water respectively;
- \( D_f \) is the depth of the footing;
- \( B \) is the width of the footing;
- \( N_c, N_d \) and \( N_r \) are bearing capacity factors (see Table 1);
- \( S_c, S_d \) and \( S_r \) are shape factors (see Table 2);
- \( d_c, d_d \) and \( d_r \) are depth factors (see Table 2).

For rock masses the bearing capacity is obtained by other ways such as:

- ultimate strength of the rock taking in account with spacing and aperture of the discontinuities; when the footing is placed in a slope where movement along their discontinuities is possible it must be analysed the rock slope stability and to admit the foundation as a surfage.

\[
Q_{ult} = K_s Q_r
\]

(2)

where \( Q_{ult} \) is the bearing capacity and \( K_s \) is a coefficient given by:

\[
K_s = \left( 3 + \frac{d}{D} \right) / \left[ 10 \left( 1 + 300 \frac{\phi}{d} \right)^{0.5} \right]
\]

(3)
where "d" and "e" are the spacing and aperture of discontinuities and B is the footing width.

The bearing capacity of rocks can be calculated by using the angle of internal friction and cohesion of rocks from high pressure triaxial tests; according to Stagg and Zienkiewicz (1968), the bearing capacity factors for sound rock are approximately:

\[
\begin{align*}
N_c &= 5 \tan^2 (45 + \phi/2) \\
N_q &= 6 \tan^2 (45 + \phi/2) \\
N_f &= 2 \tan^2 (45 + \phi/2) + 1
\end{align*}
\]

(4)

The bearing capacity \(Q_{ult}\) can be got by the RQD (Bowles, 1988) given by:

\[
Q_{ult} = Q_r (RQD)^2
\]

(5)

where RQD is the rock quality designation. When rock coring produces no intact pieces, RQD → 0 one should treat as a soil mass.

In order to standardize the elaboration of engineering geological maps according to bearing capacity at scales of 1/10,000 its it put forward for discussion the following "standard foundation": strip footing with 2 meters width at 1.5 meters depth and submerged.

There are several reasons for this recommendation:

1. in general footings are square, rectangular or strip; the calculations of bearing capacity is lower for strip footings than for other shapes, because of the bulb stress distribution under this footing and everybody can use it without problems;
2. the width of 2 meters is in general a conservative value for any footing for today buildings;
3. the depth of footing placed at 1.5 meters can be a punitive depth for economical reason, but can avoid superficial organic matter, decompressed and altered soil or rock masses; it is important each place be prospected to assure this depth is reasonable;

4. to consider the submerged soil is a punitive parameter but it is of great importance for security.

For engineering geological mapping is useful a classification for bearing capacity, but in terms of allowable bearing capacity \(Q_{seg}\), dividing "ucht" by a security factor (\(F = 3\)), avoiding rupture.

<table>
<thead>
<tr>
<th>(Q_{seg}) (KPa)</th>
<th>Allowable bearing capacity description</th>
<th>Litology</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 50</td>
<td>extremely low</td>
<td>silty-clayey or clayey-silty very soft and organic soils</td>
</tr>
<tr>
<td>50 - 100</td>
<td>very low</td>
<td>silty-clayey or clayey-silty soft soils or loose sandy soils</td>
</tr>
<tr>
<td>100 - 200</td>
<td>low</td>
<td>silty-clayey or clayey-silty soils of medium consistence and sandy soils of medium compacity</td>
</tr>
<tr>
<td>200 - 300</td>
<td>medium</td>
<td>hard corent soils and sandy soils of good compacity; sands and gravels well compacted</td>
</tr>
<tr>
<td>300 - 500</td>
<td>high</td>
<td>very hard corent soils and sandy soils very compacted; sands and gravels very well compacted</td>
</tr>
<tr>
<td>500 - 800</td>
<td>very high</td>
<td>extremely hard corent soils and sandy and gravelly soils extremely compacted</td>
</tr>
<tr>
<td>&gt; 800</td>
<td>extremely high</td>
<td>soft rocks and other non cohesive soils extremely compacted</td>
</tr>
</tbody>
</table>

The systematical utilization of a standard foundation for calculating bearing capacity will allow a better uniformization and universalization of this characteristic and the use of the above classification gives a good understanding of the soils foundation and will direct the user for a good and adequate regional and urban planning; the bearing capacity map of Aveiro is a good example of this; as it can be seen it should not recommend to build in the area where the bearing capacity is extremely low or very low, because this geotechnical unities (alluvial muds) can have 6, 10 or more metres of thickness.
REFERENCES


Susceptibility for settlements in engineering geological mapping

Susceptibilidade aux tassemens en cartographie géotechnique

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University of Beira Interior, Covilhã, Portugal

ABSTRACT: Based on "standard" foundation, composed by a flexible rectangular footing, with 2.0 metres width, 10 metres length, at depth of 1.5 metres, with a static charge of 100 KN/m², it is proposed for discussion, the use of the susceptibility for total maximum settlements as a useful parameter in engineering geological mapping. One presents the classification of soils to settlements.

RESUMÉ: La utilization de la susceptibilité aux tassemens est proposée pour discussion, comme un facteur très important en cartographie géotechnique; la base des calculs est une fondation "standard" constituée par une semelle rectangulaire flexible (L = 5B, avec B = 2.0 mètres), à la profondeur de 1.5 mètres et surchargé avec 100 KN/m². La classification des sols aux tassemens est présentée.

The susceptibility for total maximum settlements has already used in engineering geological by different investigators (Reuter and Thomas 1961; G.S.N, 1986: Mulder and Hillen, 1989, Philippart, 1990; Ferreira Gomes, 1990), but its systematical utilization has not been a success because the methodology has been so diversified that have originate some user problems.

As it is known settlements are function of the soil characteristics, footing shape, depth of foundation, load on footing, time involved, etc; usually investigators do some edometers tests and only present the compressibility parameters; these parameters for specialized geotechnicians are quite useful, but for the great number of non specialists are usefulness.

In order to visualize the susceptibility to settlements in maps, some symbology with shadows must be created like that in the map presented here; for preparing this map it was used for settlements calculation a standard foundation which it is advanced as a proposal for discussion:

"a rectangular flexible footing with L = 5B, with B = 2 meters, at 1.5 meters of depth and with a pressure at the contact footing/soil of 100KN/m²". This proposal is similar to that presented by the same authors for bearing capacity in engineering geological mapping (Ladeira e Ferreira Gomes, 1994).

For settlements computation it should be useful to consult the books: Bowles (1988, chapt.5) or Das (1985, chapt.6) which present a very comprehensive sequence.

It will be important for engineering geological mapping to present a soil classification of susceptibility for settlements as the following. This classification is based on the works of Bjørn (1963), Feld (1965), Golder (1971), Skempton and MacDonald (1956).

In the map of settlements presented here, there are only two symbologies: one of them is concerned with the muddy complex and the fluvial alluvions and the total maximum settlements varies from very high to extremely
elevated; the other is concerned with sands or overconsolidated clays and the total maximum settlements is very low. The user of these maps can adequately decide the urban and regional planning according to the susceptibility for settlements of the geotechnical units.

Table 1. Classification for settlements susceptibility.

<table>
<thead>
<tr>
<th>Total maximum settlement (mm)</th>
<th>Type of litology</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>0...1.5</td>
<td>Rocks, overconsolidated clay and very compacted sands</td>
<td>very low</td>
</tr>
<tr>
<td>1.5...2.5</td>
<td>Clayey-silty soils of very high consistency, sands and gravels of high compacity</td>
<td>low</td>
</tr>
<tr>
<td>2.5...5</td>
<td>Clayey-silty soils of high consistency, sands and gravels of medium compacity</td>
<td>medium</td>
</tr>
<tr>
<td>5...10</td>
<td>Clayey-silty soils of medium consistency, sandy soils of low compacity</td>
<td>high</td>
</tr>
<tr>
<td>10...20</td>
<td>Clayey-silty soils of low consistency, loose sandy soils</td>
<td>very high</td>
</tr>
<tr>
<td>20...30</td>
<td>Soft clayey-silty soils</td>
<td>elevated</td>
</tr>
<tr>
<td>&gt;30</td>
<td>Organic soils</td>
<td>extremely elevated</td>
</tr>
</tbody>
</table>

REFERENCES


The importance of the engineering geological mapping around Oliveira do Bairro for regional planning

La cartographie géotechnique de la région d'Oliveira do Bairro et son importance dans la planification régionale

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C.M.G. Rodrigues
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ABSTRACT: The engineering geological mapping around Oliveira do Bairro is presented herein, based on the lithogenetic character of the bed formations and on their geotechnical parameters (particle size distribution, shear strength, compressibility and compaction).

Other maps, besides the geotechnical units such as the raw materials, permeability and bearing capacity show the importance in regional planning.

RESUMÉ: C’est présentée la cartographie géotechnique de la région de Oliveira do Bairro, qui a comme base le facteur litologique et les paramètres géotechniques (distribution granulométrique, résistance au cisaillement, compressibilité et compactation) et qui fournissent la carte de unités géotechniques. Outre cartes sont présentées (matériaux, perméabilité et capacité de change) pour montrer leur importance dans la planification régionale et urbaine.

The Oliveira do Bairro council is crossed by several roads including EN1 and motorway A1. There are some local industries such as: ceramics (bricks, tiles, glazed tiles, pottery and artistic works), exploitation of sands for civil works, exploitation of underground water (the council is self-sufficient), agriculture (specially the wine fields where Bairrada wine is the speciality).

Land planning has been chaotic and this paper pretends to be a small contribution for the revision of the Municipal Director Planning.

The geotechnical study of the council was done at a scale of 1/10,000; to do this were collected representative specimens of each geotechnical unit for laboratory tests (grain size distribution; Atterberg limits; expansibility (Exp), permeability (K); natural water contents (Wn); density of the soils (ρs); parameters of compaction: maximum dry density (ρdmax) and optimum water contents (wopt); shear strength parameters: cohesion (c) and angle of internal friction (φ); modulus of deformability (Et); compressibility parameters: compression index (CRef) and coefficient of swelling (Cw); all specimens where classified in the Unified and AASHO classifications and grouped in to geotechnical unities; the basis of geotechnical mapping for each unity has a lithogenetic character.

The geotechnical characteristics of each unity is shown in Table 1 and the Map of Geotechnical Unities is presented in MAP 1.

For regional land planning is has been produced different maps: permeabilities (MAP 2), raw materials (MAP 3) and bearing capacity (MAP 4).

The Permeability Map is very important for municipal waste disposals planning; the waste is a very big problem for environmental controling and so, the knowledge of these
zones in each council get an important rule in the life of the population.

The Raw Material Map is a quite useful one, specially for those technicians who do land planning without the knowledge of the council. The Bearing Capacity Map is not lesser important than the others. It is necessary to be very prudent to use it because the heterogeneity of soils reserve plenty of surprises to those who use the soils for project implementations; besides this it gives a very good understanding of the subsoil in terms of foundations. The bearing capacity calculation was based on a submersed strip flexible footing with 2.0 metres width, at 1.5 metres depth, using the generalized Terzaghi equation.
<table>
<thead>
<tr>
<th>GEO-TECNICAL</th>
<th>LITHOLOGY</th>
<th>THICKNESS (m)</th>
<th>CLASSIFICATION</th>
<th>GENERAL CHARACTERISTICS</th>
<th>PHYSICAL PROPERTIES</th>
<th>SHEAR STRENGTH</th>
<th>COMPRESSION</th>
<th>Cc</th>
<th>Cc</th>
<th>Cc</th>
</tr>
</thead>
<tbody>
<tr>
<td>UNITS</td>
<td></td>
<td></td>
<td>UNIFIED</td>
<td>AASHTO</td>
<td>&lt;200 (%)</td>
<td>LL (%)</td>
<td>DP (%)</td>
<td>Exp (%)</td>
<td>K (cm³)</td>
<td>Wp (%)</td>
</tr>
<tr>
<td>ALLUVIAL COMPLEX</td>
<td>sandy-clayey and silty sands, sometimes with muddy interbeds</td>
<td>15 (max.)</td>
<td>CH</td>
<td>CL</td>
<td>A-7-4 (16)</td>
<td>68.3</td>
<td>0</td>
<td>0</td>
<td>29.3</td>
<td>7.2</td>
</tr>
<tr>
<td>GRAVELLY FORMATIONS</td>
<td>gravel and sands of large grain size</td>
<td>2</td>
<td>GM</td>
<td>SP</td>
<td>A-1-a</td>
<td>14.0</td>
<td>0</td>
<td>1.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SANDSILTY COMPLEX</td>
<td>silty-clayey sands, graded, sometimes with gravels</td>
<td>25 (max.)</td>
<td>SC</td>
<td>SP-SC</td>
<td>A-1-b</td>
<td>25.0</td>
<td>0</td>
<td>0</td>
<td>46.8</td>
<td>26.3</td>
</tr>
<tr>
<td>COMPLEX</td>
<td>course, medium and fine sands with some clays</td>
<td>10 (median)</td>
<td>SP</td>
<td>SM</td>
<td>A-1-b</td>
<td>30.0</td>
<td>0</td>
<td>0</td>
<td>35.0</td>
<td>NP</td>
</tr>
<tr>
<td>CRETACEOUS SANDSTONES</td>
<td>coarse and fine sands with clayey matrix</td>
<td>60</td>
<td>SP</td>
<td>SM</td>
<td>A-7-4 (6)</td>
<td>97.7</td>
<td>0</td>
<td>0</td>
<td>22.7</td>
<td>17.5</td>
</tr>
<tr>
<td>&quot;VAGOS CLAYS&quot;</td>
<td>clays of low to high plasticity</td>
<td>50</td>
<td>CH</td>
<td>CL</td>
<td>A-7-4 (20)</td>
<td>97.7</td>
<td>0</td>
<td>0</td>
<td>72.4</td>
<td>46.2</td>
</tr>
<tr>
<td>TRIAS</td>
<td>clayey-sandy silts and silty clays</td>
<td>50</td>
<td>ME</td>
<td>CL</td>
<td>A-7-4 (13)</td>
<td>88.1</td>
<td>0</td>
<td>0</td>
<td>43.4</td>
<td>19.5</td>
</tr>
</tbody>
</table>
Regional planning in Águeda country through engineering geological mapping
Planification régionale dans la région de Águeda par la cartographie géotechnique

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I.M. R. Duarte
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ABSTRACT: The engineering geological mapping around Águeda is presented with its geotechnical unities; each unity is characterized by its physical and geomechanical parameters. Regional planning can be done by using the geotechnical maps of permeability, raw materials and bearing capacity.

RESUMÉ: La cartographie géotechnique de la région de Águeda est présentée avec les unités géotechniques; chaque unité est caractérisée pour ses paramètres physiques et géomécaniques. La planification du territoire est très facilitée quand l’on use les cartes géotechniques de la permeabilité, des matériaux de construction pour génie civil et de la capacité de charge.

Águeda is a council with great urban development and industrialization; its enviable geographical situation, centre of important highways confluence, such as IP5, motorway A1, EN1, EN333 and EN230; the river Águeda is very important for riverside populations, as well as the Vale do Vouga railway.

This industrialized centre, have grown very quickly and without organization; land planning started with the new Municipal Director Planning in 1994 but without information about the subsoil. This work can be a contribution for future revision of land planning of the country around Águeda.

The engineering geological mapping started at a scale of 1/10.000; the geotechnical unities had been as base the lithological character associated to other characteristics obtained in laboratory tests (grain size distribution; Atterberg limits; (l.l and IP); permeability (k); water content (Wp); maximum dry unity (ydmax); optimum water content (wop); coesision (c); angle of internal friction (ϕ); elasticity modulus (Ei); Poisson coefficient (ν); compression index (Cc) and swelling coefficient (Cs). These characteristics are presented in Table 1 and the maps for regional land planning has been produced:

MAP 1 - Geotechnical Unites;
MAP 2 - Permeability;
MAP 3 - Raw Materials;
MAP 4 - Bearing Capacity.

The Permeability Map can be useful for municipal waste disposals planning although some of the zones needed to be studied in detail because fracturing permeability; the waste nowadays is one of the biggest problems and to solve it waste disposals will be the best solution, because of its minimum environmental contamination.

Materials for construction are every day so scarce that it will be very important to look for some of them in the marked areas on the Raw Material Map. Foundations sometimes is money consumption because of lack of knowledge of the soils characteristics; with the Bearing Capacity Map it will be easy to project any foundation. The reader must be aware it will be necessary to do some geotechnical investigation to know exactly the allowable bearing capacity. These values are based on a submersed strip flexible footing with 2.0 metres width, at 1.5 metres depth and for calculations was used the generalized Terzaghi equation.

MAP 2 - PERMEABILITY OF ÁGUEDA REGION

Legend:
- Dark grey: $>10^{-4}$ cm/s
- Light grey: $<10^{-4}$ cm/s

Scale:
- 0
- 500
- 1000 m
Effect of main engineering-geological factors on selection of sites of large dams
Effet des facteurs géologiques et géotechniques dans la sélection de sites de barrages

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'Hydrop project Institute, Moscow, Russia

ABSTRACT: The report describes the results of analysis of experience of engineering-geological substantiation of site selection for more than 120 dams. The effect of the following main natural factors was analyzed: relief composition of rocks, instability of slopes, structural, hydrogeological and seismotectonic conditions. In the process of dam sites selection different structural factors were analyzed in more detail: bedding, folds and injective (forms of magmatic bodies) structures, regional faults, faults and (background) joints.


1 INTRODUCTION

Study of the engineering-geological literature for the last 20 years points to inadequate attention drawn to the procedure of geological substantiation of selection of sites and wide experience gained in hydropower engineering. Of fundamental importance in the solution of the given problem is the revealing and reliable assessment of main engineering-geological factors exerting influence on selection of optimum project sites.

The purpose of the given report is to perform analysis of wide experience gained by "Hydrop project Institute (Moscow) for more than 60 years of its history and supplemented by some examples of dam construction in other countries. For revealing and systematization of main geological factors determining the site selection more than 120 cases of their pronounced effect on design approaches were studied. In the course of analysis the factors exerting effect in comparison of engineering-geological conditions of alternative sites irrespective of negative or positive nature of their effect were taken into account. Special attention was drawn to the cases of misjudgement of natural factors responsible for the abandonment of the previously selected sites. It might be well to point to some "inaccurate assessment" of this or that factor because these values are representative of only monitoring of these factors in design materials and in the geological literature.

2 MAIN ENGINEERING-GEOLICAL FACTORS DETERMINING SITE SELECTION

Study of experience of homeland and foreign hydropower engineering shows that the following main geological factors exert influence on selection of sites:
- structural conditions (~ 52%);
- relief (~ 17%);
- composition of rocks (~ 16%);
- instability of slopes (~ 9%);
- hydrogeological conditions (~ 3%);
- seismotectonic conditions (~ 3%).

Structural conditions in their turn are subdivided into such factors as injective (form of magmatic bodies) structure (~ 9%), regional faults (~ 9%), faults (~ 8%), (background) joints and schistosity and cleavage (~ 7%), fold structure (~ 4%) and karst cavities (~ 4%).

Ecological conditions in spite of their pronounced effect on site selection which is particularly urgent nowadays are considered in the given report as an effect of the project on the environment and are not included in a set of natural-geological factors. Geomechanical properties of rock masses are considered herein as derivatives of rock composition and different structures of the rock mass.
2.1 Relief

A prerequisite to site selection is a favourable relief which on the one hand should provide minimum dimensions of the dam and on the other hand should provide possibilities of the reservoir of the capacity required for the given project creation. Narrow sections of valleys favourable for construction of the dam are very often associated with intersection by the river at the given site of stronger rocks forming a stable foundation for hydraulic structures (Bratsk, Zeya - both in Russia - and some other dams). Sometimes a favourable factor at location of the dam is the configuration of the valley in plan. For instance, a sharp bend of the Daugava river made it possible to adopt a very economic arrangement of the Plyavinsk dam (Latvia). The presence of islands in the river channel is an important factor as these islands make construction activities much easier, e.g. Otvažhensky site of the Samara dam (Russia). The same is also true for selection of the site for the Dnieper HES (Ukraine), Nizhnekamsk dam (Russia) and some other dams.

However many sites favourable from geomorphological standpoint are unfavourable from other geological viewpoints. For instance, at selection of sites of Charvak (Uzbekistan) (Seliameov, e.a. 1984), Fodda (Algeria), Foum El Ghera (Algeria) and some other dams geological considerations made a suitable relief to be ignored and resulted in location of the dams on the basis of the rock mass geological structure. Sometimes the relief determines the only possible site of construction of the dam. Normally this case it is impossible to avoid this or that unfavourable geological factors to which the design of the structure should be adapted. For instance, at the site of the Roqun dam (Tadjiikistan) it is impossible to avoid the fault and landslides at the site of the Zardezas dam (Algeria).

2.2 Petrographic composition

Petrographic composition of rocks is often considered as one of the main factors exerting some effect on site selection though the composition alone does not allow the conditions and properties of rocks to be characterized unambiguously. For instance, dolomites in the foundation of the Dokan dam (Iraq) are so strong that withstand the load of the arch dam of 11 m in height but dolomites in the foundation of the Haditha dam (Iraq) represented by weathered varieties may be readily crushed by hand. In these cases engineering geological properties are controlled by other factors determining the conditions of rocks. Important initial data, for concrete dams in particular, include the thickness of Quaternary deposits and the relief of bedrocks. For instance, thick alluvial deposits resulted in abandonment of the 4th site at selection of the Dneprodzerzhinsk dam (Ukraine) and absence of such alluvial deposits resulted in the selection of the Taburiskchensk section of the Kremenchug dam (Ukraine) site. The presence of hard rocks assisted the selection of location of the Tyumeyun dam (Uzbekistan) (Glidey 1986) and some others. The presence of highly soluble rocks such as salt, gypsum, anhydrite may have a pronounced effect on selection of the proper site. The location of Kama, Nizhnekamsk and Votkinsk dams (Russia) is confined to the sections where gypsumiferous and saliferous rocks occur at a definite depth protecting these rocks against solution. The presence of relatively harder rocks compared with the alternative site was taken into account at selection of Kurpsai (Kyrgyzstan), Verkhne-Tulomskaya (Russia), Inguri (Georgia), Charvak and some other dams.

2.3 Landslides

Landslides occur widely as a manifestation of gravitational phenomena and very often they have a pronounced effect on selection of the site. In rock masses instability of slopes may be closely connected with bedding structures, faults, schistosity (and cleavage) and folds. Normally at selection of the site it is common practice to avoid landslides at abutments. Thus at the selection of the site of the Namakivani dam (Georgia) the occurrence of a large ancient landslide resulted in the abandonment of a high arch dam at the site situated downstream of this landslide. In an effort to avoid the "revival" of the landslide by underflooding a two-step scheme with two small dams located upstream and downstream of the landslide was adopted. One of the reasons of location of the Cheboksary dam (Russia) at the Elnikovsky site is the absence of landslides in dam abutments (Larionov, e.a. 1986). At the selection of the dam site of the Koteshwar dam (India) the designers had to abandon the upstream site because of potential instability of the right-bank dam abutment slackened by schistosity (cleavage) and fractures occurring close-parallel to the slope. However sometimes exceptions take place. For instance, the Irl Enida dam (Algeria) was constructed on a sliding section and in this case the dam "supports" landslides and the drainage facilities are provided to raise up the stability.

The Vaint dam (Italy) may serve as an example of a poor choice of the site due to underestimation of landslides in combination of effect of technogeneous factors. Underestimation of landslide movements on the right bank of the reservoir resulted in the well-known fatal accident. A sudden downfall of the landslide of about 250 million m³ overfilled the reservoir, resulted in overtopping of the dam and killing of about 3000 people.

The construction of the Zardezas dam (Algeria) provoked the development of the landslide on the left-bank abutment which caused the axis of the dam to be changed. At the selection of the site of the Meatlinskaya
dam the stability of a steep talus slope above the right-bank abutment was assessed incorrectly. The landslide of 20 million m$^3$ in volume formed under the effect of technogeneous factors caused the hydropower plant to be relocated to the downstream reach and delayed the construction for several years.

2.4 Hydrogeology

Sometimes a hydrogeological factor is essential to the selection of the site: favourable - in case of water impermeable rocks and unfavourable at high permeability which requires extensive seepage-control activities. The abandoning of the lower site of the Irkanaiskaya dam (Russia) and Joiskoy site of the Sayano-Shushenskaya dam (Russia) (Baduvin, e.a. 1986) was caused mainly by the danger of water seepage from the reservoir towards the nearby valley.

At location of the Samara dam the alternative sites were confined to deep ancient ravines on the right bank filled with a thick strata of neogenic clays which served as a natural barrier against seepage. At the selection of the site of the Vologograd dam (Russia) a favourable factor was the occurrence of water impermeable rocks at the section of concrete structures protecting the construction pit against pressure water. An example of the site selected incorrectly due to underestimation of a hydrogeological factor is the Lone Pine dam (USA).

Numerous fractures in the foundation of the dam are responsible for significant water leakage from the reservoir which failed to be filled at all and because of this a new dam was to be constructed at another site.

2.5 Seismotectonics

Seismotectonic conditions are very important factors in selection of the site in case alternative sites differ in seismic risk. At seismicity of intensity VIII and larger one should take into account the possibility of occurrence of abrupt seismogeneous movements in active faults and regional faults when siting the dam. Such displacements may reach 12-14 m (Assam earthquake of 1897 with magnitude 8.7). Examples of a poor choice of selection of dam sites are San Andreas and Upper Crystal dams (USA) constructed on the well-known San Andreas regional fault. At earthquake of intensity X a horizontal displacement 1.5 m took part in this fault resulted in the damage of both dams.

Another example of a site selected incorrectly is the Anburn arch dam (USA) during construction of which on August the Ist, 1975 an earthquake took place at a distance of 80 km to the north near the Orovil dam. By that time 300 million dollars out of estimated cost of the project 425 million dollars had been already spent. This earthquake revealed seismotectonic activity of the system of faults extending to the construction site of the Anburn dam. Additional detail investigations disclosed 6 active faults with possible earthquake with magnitude M = 5 - 7 and maximum displacement up to 1 m. The construction of the arch dam was suspended and it was decided to change the design of the dam for the type which is more stable to seismotectonic movements (Davis & Bacon 1979).

In case of danger of seismogeneous fault movements it is desirable to locate the dam in the center of the tectonic block confined by potentially active faults. An example of a correctly selected site is the Chirkey dam of 232 m in height (Russia) which was located so that large tectonic faults are avoided in the foundation and abutments. In 1970 during construction an earthquake of intensity VIII-IX points took place resulted in displacements of some centimeters in small faults without any damage to the dam.

3 MAIN STRUCTURAL FACTORS EXERTING EFFECTS ON SELECTION OF HIGH DAM SITES

Structural factors determining the location of dams are characterized by a wide occurrence (50%) and consequently are analyzed in more detail.

3.1 Injective structures

At the analysis of the effect of various injective structures on the selection of dam sites first of all it should be pointed to the role of such structural characteristics as a form, sizes and occurrence of magmatic bodies. Normally more or less large intrusive bodies are favourable for foundations and abutments of dams because of their hardness, lower water permeability and due to the fact that very often narrowings of river valleys and minimum thickness of riverbed alluvial deposits are associated with them. It should be noted that under different geotectonic conditions various types of intrusive bodies are most favourable for location of dams. For instance, in folded regions these are granitoid intrusions of various sizes and in Phanerozoic platform cover -these are discordant sills. The sites of Krasnoyarsk, Kolyma, Zeya, Kuvandyk (Russia), Ust-Kamenogorsk (Kazakhstan), Dnieper (Ukraine), old Aswan (Egypt) and other dams are confined to unconformable intrusive masses. Bratsk, Ust-Il'm, Viluf, Ust-Khantaysk, Boguchan (Russia) (Smul'skiy 1992) and other dams were built on stratified conformable intrusions.

However it is impermissible to consider intrusive masses as most favourable and completely reliable foundations of hydraulic structures without a thorough check-up analysis. For instance, visibly hard and thick intrusive bodies may unexpectedly branch into a series of small veins at some sections (the right-bank section of the Ust-Il'msk sill and some others) or may contain
large pockets of weathered rocks (Dnieper I, old Aswan and some other dams) and lenticular inclusions of graphite with low shearing characteristics (Kureiskaya HPS) and may be broken by faults (Krasnoyarsk HES), subhorizontal relief fractures (Krapivinskaya HES, Kazakhstan) and other structural defects.

Dikes widespread particularly in folded rocks may have the dual effect. In some cases hard dikes occurring crosswise the river are considered as a favourable factor in the dam foundation since they serve as a natural water-tight barrier. In other cases the dikes altered by tectonic and hypogene processes are weakened zones resulting in an increased volume of rock excavation and requiring special strengthening (Geksekaya dam, Turkey; Pongolapoort dam, South Africa; Bau Madgeris dam, Italy and some others). An interesting design approach was adopted at location of the Talbingo dam (Australia) where a grout curtain was confined to a fractured dike because this dike provides better conditions for efficient grouting compared with the enclosing rocks. Solt and clay diapir folds related to injective structures may have a negative effect in the selection of the site because of danger of development of leaking processes and recent deformation. An example of recent deformations of a rock mass is the left abutment of the Nurek dam (Tadjikistan) where slow movements in faults crossing diversion tunnels are associated with a salt dome occurring at a certain depth. It should be noted that such recent movements may take place even at a considerable depth of a salt diapir. At the section of the Yerevan dam (Armenia) a salt dome is responsible for an intensive fracturing of the overlying basalt cover and lowered stability of natural valley slopes.

3.2 Bedding structures

The important elements of the bedding structure having an effect on the selection of the dam site are thin clay or schistous interlayers, some layers with specific properties and weakened contacts of layers and benches as well. A typical example of a consideration of the bedding structure is the location of the dam in fractured or karsted rocks along the impermeable layer oriented across the valley. In this case a seepage-control element of the dam links up with this layer which allows the conventional grout curtain to be abandoned. For instance, the Bolgenach dam (Australia) is confined to an impermeable layer of marls the continuation of which is the dam core. The site of the Bekme dam under construction in Iraq was selected in such a way that the upstream face of the concrete buttress dam touched impermeable layer of clayey limestone and marl. In this case a lower site in karsted limestones which is more favourable from topographic point of view had to be rejected. The multiple-arch Nebeur dam (Tunisia) was located on a narrow bench of clayey limestones which were harder compared with surrounding rocks and this bench was accepted as the foundation of the dam the upstream face of which links up with a layer of impermeable marls.

The disclosure of two thin subhorizontal interlayers of bentonite clay with low strength characteristics in Cretaceous rocks of the foundation of the Euphrates dam (Syria) required a large volume of additional drilling to define more exactly the position of retaining concrete structures. As a result of detailed investigations a spot was revealed where bentonite was replaced by marl. At this section characterized by higher shear resistance the powerhouse integrated with the spillway was constructed (Papabuche 1987). At the originally selected site of the Dniester Project (Ukraine) a series of sub-horizontal clayey interlayers was responsible for numerous landslides which was the main reason of relocation of the dam. At excavation of the construction pit for the Zadezas dam (Algeria) a clay layer gave rise to a landslide of about 100,000 m³ in volume in the dam abutment. This situation necessitated the axis of the dam to be moved but since the right-bank section of the dam had been already built, the axis of the dam was turned.

3.3 Faults

Regional faults i.e. large tectonic dislocations with a zone of crushed and weak rocks the thickness of which is comparable or exceeds the sizes of the dam foundation are often characterized by recent seismotectonic activity. Regional faults produce a pronounced effect on formation of other structures: injective (magma) bodies, fracturing and schistosity and also weathering of rocks, formation of relief and other exogenic processes. Consequently at the selection of the site one endeavours to avoid both the regional fault and its surroundings.

In the zones of regional faults the processes of tectonic crushing, dynamic metamorphism and intensive fracturing decrease considerably the strength of the rock mass. For instance, at selection of the site of the Bansagar dam (India) the designers had to reject the Demba gorge because of the regional fault under the channel which deteriorates the stability of the proposed dam and increases permeability of the foundation. Near the Maina HES (Russia) there is Arbatoko-Misky regional fault accompanied by a thick zone of crushed, ground and intensively altered metamorphic schists. Therefore at the section of the upper center line of the fault it was necessary to remove a great volume of broken rocks, to take considerable consolidation, seepage-control and drainage measures. This resulted in the selection of the lower site which is more distant from the regional fault.

The importance of regional faults as a geological factor having an effect on selection of the dam site often increases by the danger of recurrence of seismotectonic fault movements which was attested by abandonment of
the Lower-Khudoni site (Georgia), the upper site of the 
Chavrik dam (Seli'ametov, e.a. 1984) and by suspension 
of construction of the Anburn dam (USA). Sometimes 
regional faults may have a positive effect on selection of the site. For instance, steeply dipping 
regional faults on the area of the proposed dam on the 
Gordon river (Tasmania) break the bench of intensively 
permeable limestones exposed in the upstream and 
downstream reach. The revealing of these faults with 
amplitude of displacement from 0.5 to 2.5 km elimi-
nated the original danger of large filtration from the 
reservoir and determined the final selection of the given 
site in spite of its initial rejection.

3.4 Minor faults and big fractures

Usual faults with displacements are widely spread 
and the dam foundations free from these faults are consid-
ered as exceptions only. For instance, the occurrence of 
the fault weakening the foundation of the Infernilllo 
dam (Mexico) resulted in rejection of change to the new 
(lower) site. At designing of the Sayano-Shushenskaya 
dam the occurrence of tectonic faults was also the 
reason of abandonment of the Joisky site in favour of 
Karlov site (Baduhn e.a. 1986). However more fre-
quently tectonic faults produce an effect on location of the 
dam axis within the limits of the selected site, on the 
design and layout of structures and on methods of 
reinforcement of the rock mass. In the left-bank abut-
ment of the Santa Eulalia arch dam (Spain) a fault with 
a clay filling dipping towards the valley at an angle of 70 
deg. was encountered. For an increase of stability of the 
dam the position of the arch was selected to ensure its 
maximum support on the lying side of the fault. In this 
case the fault material was removed by the shaft and 
substituted by concrete. After stripping of the pit the 
axis of the Angkhakot dam (Armenia) was shifted 
towards the downstream reach in consequence of reveal-
ing of a fault.

As the height of the Aramine gravity dam (Japan) 
increased the upper portion of the left-bank abutment 
appeared loosened by the tectonic fault. It resulted in a 
shift of the dam axis in plan to avoid the site in its 
abutment. At the final selection of the site of the Kassem 
arch dam (Tunisia) the axis of the dam had to be shifted 
by 12 m compared with the original attitude for better 
location with respect to three systems of tectonic faults 
developed at this site. In consequence of a regional 
fault revealed primarily by aerial photographs the axis of 
the Tumut Pond dam (Australia) had to be shifted 
downstream.

3.5 Joints

Joints are normally an important factor at engineering-
geological comparison of alternative sites. Their effect 
on strength and deformability of the rock mass is often 
"intermingled" with the effect of regional faults. At the 
selection of the site of the Hoover arch dam (USA) the 
Boulder canyon was rejected in favour of the gorge 
whereless jointy igneous rocks occur. At the selection of 
the Gyrmush dam (Armenia) the intensive jointing of 
effusive andesite-basalt was responsible for "blocky" 
structure of the rock mass. At the selection of the site 
preference was given to a less jointy foundation com-
pared with other sites. The Kurpsai dam is located at the 
"upper" site first of all in consequence of its less 
jointing compared with the alternative "lower" site. 
The intensive jointing of limestones in the foundation of the 
Fourn El Ghera made the axis of the dam to be 
shifted from a narrow inlet portion of the gorge to a 
wide downstream section of the canyon where less 
jointy limestones occur.

During construction of the Lone Pine dam (USA) 
numerous joints were found in limestones and sand-
stones forming the bottom and banks of the reservoir. 
This jointing was responsible for such a great water 
leakage that it was impossible to fill the reservoir and a 
new dam at a new site had to be constructed. During 
construction of the Santet arch-gravity dam (France) 
the intensive jointing resulted in a cave-in during exa-
vation of the construction pit which required the 
change of location of the powerhouse. The effect of 
jointing on location and on orientation in particular of 
underground workings is worth noting. The location of 
the Tumut-I underground hydropower plant (Australia) 
was selected in granites but not in gneisses because the 
jointing in the latter is less favourable for stability of 
walls and the roof of workings.

3.6 Folds

A fold structure produces an effect on the selection of 
project location normally not separately but in combi-
nation with other geological factors. In many cases 
folds determine the formation of the relief and the 
following dams e.g. Chirkey (Russia), Dokan (Iraq), 
Kasseb (Tunisia) and some other are confined to can-
yons cut through mountain ridges - anticlines. Often the 
decisive moment at selection of the site is the occurrence 
of layers and benches with particular properties gov-
erned by folds: impermeable or permeable, strong or 
soft, highly soluble (the Mosul dam in Iraq, Miatla dam 
in Russia, etc). The site of the Akstafa dam (Azerbaijan) 
is confined to a relatively narrow section of the Kura 
river valley resulted from the brachy-anticline fold and 
concrete structures are confined to the core of the fold 
where harder and impermeable rocks expose.

On the area of the Genissiat hydropower plant 
(France) the syncline determine the occurrence of 
impervious layers and the crown of the anticline is 
characterized by intensive jointing and less hardness of 
the rock mass. A consideration of folds at the selection
of the site ensured adequate water retaining capacity of
the reservoir in spite of occurrence of karsted limestones and sufficient strength of the dam rock foundation. At the section of the Fergug (Algeria) hydropower plant the anticline from the side of the upstream reach governs the location of 30m bench of hard sandstones selected as the foundation of the dam. However in the course of investigations the form of the fold and its position accordingly were determined incorrectly. Therefore in the process of construction the dam had to be shifted after revision of configuration of the fold.

3.7 Karst

The occurrence of karst cavities may have a pronounced effect on the selection of the site and in particular on permeability properties of the rock mass. Thus sometimes it is difficult to separate the effect of karst as a factor from the hydrogeological factor. The occurrence of karsted carbonate rocks resulted in rejection of lower sites of Miatla and Turukhanske dams (Russia) and the middle site of the Charvak dam (Seliametov, e.a. 1984), etc.

A classical example of underestimation of karst at the selection of the site is the Keban dam (Turkey). During construction of the powerhouse karst cavities were encountered which required relocation of the powerhouse. Subsequently large solution channels were found in the left bank abutment. The large one was 100,000 m$^3$ in volume. This required the construction of a large underground concrete diaphragm and an additional grout curtain. Nevertheless during the reservoir filling seepage in the volume of 24 m$^3$/s started which required reservoir emptying and new grouting. As a result the construction was delayed for several years and the overdraft reached 3 billion Turkish lire (at the cost of the project of some 4 billion Turkish lire). At some hydropower plants (Escale, Spain; Jerome, USA; Condo, USA) because of karst presence it was impossible to rise up the upstream level and fill the reservoir.

It should be noted that the occurrence of karst only may not be the reason of rejection of this or that site. Karst may have no pronounced effect on water retaining capacity of the reservoir, for instance, with the presence of a clay filling, at a certain orientation of solution channels, and displacements of karsted layers along tectonic faults and in case of possibilities to fill them up.

CONCLUSIONS

The analysis of experience of homeland and foreign hydropower engineering shows a complicated multifactor character of selection of the site at which of maximum importance are relief (~17%), composition of rocks (~16%) and injective (forms of magmatic bodies) structures (~12%). Of minor importance are instability of slopes (~9%), bedding structures (~9%), regional fault zones (~9%), faults (~8%) and joints (~7%). Fold structures (~4%), karst cavities (~4%), hydrogeological (~3%) and seismotectonic (~3%) conditions are of minimum importance.

It should be noted that the composition and relative importance of these factors are determined by specific geotectonic conditions and they depend in particular on attribute of the studied region to a fold region, a platform cover or a crystalline foundation.

Engineering geological substantiation of the site selection may not be separated from the selection of the structure type, sizes and layout and methods of the foundation consolidation. Hence a successful selection of the dam location depends on joint operation of a geologist and a designer at all main stages of solution of the given problem beginning from selection of alternative sites up comparison of results of investigations and surveys carried out at them and active participation of the geologist in the site selection.

Further updating of the multipurpose procedure of engineering-geological substantiation of site selection is required with a maximum use of such progressive methods as geophysical, remote (Peterson, e.a. 1982, Strom 1984, Clement e.a. 1990), with the use of empirical classifications, etc. and acceptance of design approaches in multilevel systems under conditions of some uncertainty of geological parameters and the engineering risk. In this case the latest advances in the field of the theory of information and creation of automatic control system shall be used.

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The 7th International Congress of the International Association of Engineering Geology is held in the year of the 30th Anniversary of the Association and it is organized under the motto ‘Turning the Century with Engineering Geology’. The broad range of themes reflects the wide spectrum of activities within Engineering Geology as well as its concern with the most actual matters which affect mankind like Natural Hazards and Environmental Protection. The Proceedings, printed in six volumes, include 641 papers submitted by authors from 70 countries. Each theme and workshop is preceded by a Keynote Lecture or Coordination Report written by international reknown authors. The methods and techniques of ground study, the construction materials, the surface and the underground works as well as the information technologies and the teaching, training and professional practice of Engineering Geology are also extensively covered by the authors of the papers. The Congress Proceedings can thus be taken as representative of the state of development of Engineering Geology in 1994 and demonstrate the current interest of the engineering geological international community and the ability of engineering geologists to help in the solution of some of the most important problems which affect the humanity.