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Erosion of rock masses subject to flow action: Some geomechanical and hydraulic aspects

Quelques aspects géomechaniques et hydrauliques de l'erosion des masses rocheuses soumises à l'action de l'eau

José Antunes Sobrinho & Nelson Infanti Jr., ENGEVIX SIA, Estudos e Projetos de Engenharia, Florianópolis, Brasil

ABSTRACT: The paper reviews the available information about erosion of rock masses such as: empirical observations in natural water-falls; the performance of the rocks downstream of hydraulic structures in operation as well as the informations obtained in hydraulic scale models.

A theoretical approach for block dislodgement mechanism is presented and the main geomechanical parameters involved are discussed. An attempt is made to classify rock masses according to their erosion resistance.

Finally, the positive experience acquired with unlined hydraulic structures in high quality rock masses is summarised and practical recommendations for the design and construction of such structures are proposed.

RÉSUMÉ: Les informations disponibles sur l'érosion des massifs rocheux sont révisées dans cet article. Les observations empiriques sur les chutes d'eau, l'analyse de la performance des prototypes et les résultats obtenus sur des modèles hydrauliques réduits sont aussi discutés. Une analyse théorique sur le déplacement des blocs rocheux est présentée, ainsi qu'une discussion sur les paramètres géo-mécaniques affectés. Un essai sur les classifications des massifs rocheux est présenté suivant leur résistance à l'érosion.

Finalement, l'expérience positive obtenue avec structures hydrauliques sans revêtement est résumée, et sont proposés des conseils pratiques concernant le projet et la construction de telles structures.

1 INTRODUCTION

Energy dissipation on rock masses downstream of hydraulic structures is probably one of the least known aspects of hydromechanics and engineering geology. John (1978) proposed an interdisciplinary task force to assess and to handle rock mass erosion in hydropower engineering. So far, the geomechanical approach was rarely tried.

The energy dissipation depends on the criteria adopted for designing the structure as well as on the rock mass quality. For a safer and more economical design full attention should be given to the rock mass properties and its behaviour.

The analysis and interpretation of scouring problems observed at hydraulic structures and the evolution of natural water-falls are extremely important to understand the phenomenon and must be a guide for the design of new structures.

This paper presents a comprehensive review of the available information on rock erosion from both engineering geology and hydraulics point of view; a discussion on the main parameters that govern the erosion process is reported; practical guidelines for designing hydraulic structures in hard rock are proposed.

2 EVOLUTION OF NATURAL WATER-FALLS

The geological qualitative knowledge derived from the observation of natural water-falls may be summarized as follows:

2.1 Geological features controlling water-falls in Southern Brasil

The rivers of Paraná Sedimentary Basin, with normal discharges varying from 1,000 to
Figure 1. Location of water-falls and hydro-projects in Southern Brazil

Table 1. Main characteristics of water-falls in basalts (from Bartorelli, 1984)

<table>
<thead>
<tr>
<th>Water-Fall - River</th>
<th>River</th>
<th>Width (m)</th>
<th>Canyon Head Length (km)</th>
<th>Mean Discharge (m^3/s)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sete Quedas - Paraná</td>
<td>u/s 4,000</td>
<td>80</td>
<td>40</td>
<td>70</td>
<td>9,000</td>
</tr>
<tr>
<td>Uruê - Paraná</td>
<td>d/s 1,000</td>
<td>140</td>
<td>1</td>
<td>10</td>
<td>4,000</td>
</tr>
<tr>
<td>Yucumã - Paraná</td>
<td>500</td>
<td>330</td>
<td>2</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>Estreito - Uruê</td>
<td>350</td>
<td>100</td>
<td>0.02</td>
<td>10</td>
<td>3,000</td>
</tr>
<tr>
<td>Iguazu - Iguazu</td>
<td>1,500</td>
<td>80</td>
<td>2.7</td>
<td>80</td>
<td>3,000</td>
</tr>
<tr>
<td>Salto Osório - Iguazu</td>
<td>700</td>
<td>200</td>
<td>1</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>Avanhanda - Tietê</td>
<td>450</td>
<td>45</td>
<td>2.5</td>
<td>-</td>
<td>500</td>
</tr>
<tr>
<td>Indios - Grande</td>
<td>1,000</td>
<td>120</td>
<td>3.0</td>
<td>20</td>
<td>2,000</td>
</tr>
<tr>
<td>Marimbondo - Grande</td>
<td>1,700</td>
<td>80</td>
<td>2.0</td>
<td>20</td>
<td>2,000 (Florence, 1905)</td>
</tr>
<tr>
<td>São Simão - Paranaiba</td>
<td>1,000</td>
<td>80</td>
<td>2.0</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>Dourada - Paranaiba</td>
<td>500</td>
<td>60</td>
<td>0.15</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Victoria - Zambesi</td>
<td>800</td>
<td>100</td>
<td>1.0</td>
<td>120</td>
<td>4- Florence, 1905</td>
</tr>
<tr>
<td>Uruê - Paraná</td>
<td>2,500</td>
<td>70</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Itapura - Tietê</td>
<td>500</td>
<td>100</td>
<td>0.39</td>
<td>-</td>
<td>500</td>
</tr>
</tbody>
</table>

u/s = upstream of falls;  
d/s = downstream of falls.
9,000 m³/s, present several water-falls and rapids in basalts (Figure 1 and Table 1) have been subject of many geological studies, from which the following main evidences were outlined:

1. According to Florence (1905) the origin of water-falls is related to linear geological structures like faults, contacts or major joint systems. Washburne (1930) stated that some of the water-falls developed where the rivers cut the sides of low dip anticlines.

Figures 2 and 3, drawn based on drilling and outcrop data, show that both may be right. As illustrated in Figure 2 the rapids of the Uruguay River at Ita are directly related to the chief structural alignments, along which differential movements occurred. In Figure 3 it is possible to observe the low dip anticline structure that controls the sill formation, at Sete Quedas Falls.

Bartorelli (1984) concluded that the carving of the falls might have been facilitated by minor geological features (sheared zones, fractured dikes), not necessarily preserved nowadays.

![Geological sketch at Ita rapids in Uruguay River](image)

![Geological section across Sete Quedas Falls at Paraná River](image)

2. Several authors observed that the mechanism of water-falls evolution is related to the geological features within the basaltic lava flows. Florence (1905), Maack (1968) and Ruiz (1984) observed that the sub-vertical joints that cut the rock into prismatic columns are very unstable near the vertical faces of the falls; the sub-horizontal jointed zones are also important in the erosion process as long as they separate the rock mass into thin slabs making possible to develop the cascade steps.

As illustrated in Figure 4, Maack (1968) also recognized the importance of vesicular basalts and breccias to maintain the crest and steps of the cascades.

Although more susceptible to abrasion, these rocks are less fractured and, therefore, are more capable of withstanding erosion of large size blocks. This aspect is also shown in Figure 2, where the long contour line at el. 260 m, between the rapids, corresponds to the top of a lava-flow.

3. According to Maack (1968) and Bartorelli (1984) the tributaries of Paraná river along the stretch from Sete Quedas to Iguazu mouth, reach the canyon level through cascades. The distance of the cascade with respect to the canyon varies according to its discharge: cascades of small rivers are situated few hundred meters far away, while in medium rivers they are situated at 4 to 7 km from the canyon. Iguazu Falls on the main tributary, is at the longest distance: 21 km.

4. In Sete Quedas Falls, Washburne (1930) observed that the upstream erosion along the canyon was faster than below the falls, because the river penetrates
upstream of the falls. Also Maack (1968) described that there was an exceptional turbulence and great variation of the riverbed level (from 20 to 75 m) downstream of Sete Quedas. These facts lead to another mechanism to explain the rock erosion within canyons that is discussed in the next item.

2.2 Stress relief in river valleys

Parsons & Hendon (1974) explained the phenomena related to stress relief in deep narrow canyons and in weak flat-lying rocks. Paes de Barros & Guidiçini (1981) found at Itaipu (situated in the Paraná river canyon) evidences that basaltic rocks behave like proposed by Parsons & Hendon (op.cit.) for sedimentary rocks: slight bulging of valley floor, with increase in hydraulic conductivity of the rock mass and occurrence of shearing displacements along discontinuities (up to 0.37 m); slight raising of valley wall lip, which directs the secondary drainage paralell to it, forcing the tributaries to reach the main river through cascades; and other minor features in the basaltic flows at the valley walls.

Marques Filho & Levis (1981) also present evidences of stress relief at the river bed in Foz do Areia Dam, with increase in both fracturing and hydraulic conductivity.

These facts have shown that once the canyon formation starts, due to any geological phenomenon that allowed a reduction of the base drainage level, the process of riverbed erosion is self-sustained. While the river carves the valley the consequent stress relief provokes fractures in the rock mass. This allows more blocks dislodgement and more carving, that in turn releases more stress and so on. As the geology and stress relief are not uniform along the canyon, great differences in the riverbed level may occur.

This mechanism may explain the exceptional deep scouring at Kariba’s free overfall spillway (Bartung & Häuser, 1973). The depth of scouring reached 50% of the head in this arch dam situated in a narrow valley with metamorphic rocks. The seismic induced activity that followed reservoir impounding clearly demonstrates that there was stress concentration beneath the valley.

2.3 Apparent rate of water-fall regression

The regression of water-falls is illustrated by a simple graphic from Table 1 data, in which it is compared the ratio between the river width upstream and downstream of the falls, to the length of the channel carved by the falls. It is surprising to note that the width ratio, which is an indirect measure of the specific discharge, showed a good correlation \((r = 0.90)\) with the carved channel length.

For two of these falls it was possible to calculate their regression rate, according to available geological data (Bartorelli, 1984 and Washburne, 1930). Though this rate might have been affected by several factors (lithology and degree of fracturing of rocks; climatic changes and isostasy) it can be said that the falls moved at an apparent rate about 3.5 to 7.0 m/century (400-800 m³/year) at
Figure 5. Ratio between the width and length of water-falls channel.

Victoria Falls (Africa) and 0.4 to 4.0 m/century (30-300 m³/year) at Sete Quedas (Brazil). These data stand in the middle between those available for gneiss (0.2 m/century at the Nile River) and for Etna lava flows (7.0 m/century).

2.4 Summary of geological evidences from water-falls

1. The depth of the water pool beneath the falls is proportional to the hydraulic head (H). Actually, the scour is stabilized in most waterfalls at a depth equal to about one third of this head (Davis & Sorensen, 1970).

2. The regression of the water falls, as a measure of erosion volumes, is directly proportional to the discharge, as observed in Figure 5 and at the tributaries of the Paraná river.

3. Falls in basaltic are located at the dense central portion of the rock flow, where columnar fractures and highly jointed sub-horizontal zones allow dislodgement of rock blocks.

4. Basaltic breccias and vesicular-amalgaldoidal basaltic, though more susceptible to abrasion by the water flow, sustain the steps of the falls because they are less prone to block dislodgements.

5. In canyons like the Paraná river between Sete Quedas (Brazil) and Posadas (Argentina) the stress relief due to river carving seems to play an important role in the mechanism of rocky riverbed erosion.

6. The apparent rate of upstream regression of basaltic water falls varied from less than one to few meters per century (tenths to hundredths cubic meters per year).

3 ROCK SUBJECT TO WATER FLOW: EROSION PROCESS

3.1 Hydraulic Aspects

The erosion of rock masses subject to water flow will depend chiefly on the rock classification, as will be demonstrated in item 4. The type of rock, its hardness, fracturing system, spacing of joints, dimensions of blocks and interlocking are some of the aspects to be considered.

From the hydraulic point of view the rock has to be able to withstand water flow in two distinct situations:

1. Large pressure fluctuations
   This phenomenon, which tends to dislodge large rock blocks, is well observed in plunge-pools downstream of free over-fall spillways. Large pressure fluctuations also occur in the core of a hydraulic jump stilling basin, where the eddies formed in the transition of supercritical to subcritical flow are responsible for the energy losses.

2. High velocity flow on rock
   Steep channels excavated in rock or inclined chute spillways are very often used where the rock is considered to be good and sound. The flow velocity implies in two combined effects (see Figure 6):
   (i) pressure fluctuations induced by the bed rock roughness;
   (ii) shear stress in the rock bottom;
   The first one is due to the turbulent flow in a rough surface and its magnitude will depend on the relative roughness k/y; the higher the water depth (y) with respect to the roughness height (k), the lower will be the pressure fluctuations induced by the roughness. However, observations of flows on an excavated rock surface have shown that the flow seems to be quasi uniform with hydrostatic pressure distribution.
On the other hand, the physical effect of a high velocity flow in a bed rock may be described by the shear stress (τ) which is represented by the following equation:

\[ \tau = \gamma \cdot \frac{V^2}{C^2} \]

where: \( \gamma \) = specific weight of water; \( V \) = water depth; \( C \) = Chezy roughness coefficient.

\( \tau \) is the main parameter in sediment transport as demonstrated by Shields (1936). Thus, the practical effect of \( \tau \) or \( V \) (see Figure 6) on a bed rock is either to carry loose rock blocks (from the excavations) or to wash away free granules in weathered rocks.

Actually for sound rocks it is understood that the velocity effect causes less erosion than large pressure fluctuations.

3.2 Rock erosion in plunge pools

As already explained, scour hole downstream of free overfall spillways are due to large pressure fluctuations. Because energy dissipation by means of plunge pools is widely used in dam engineering, it will be reviewed, in short, the main parameters that play an important role in the scour process.

1. Scour hole mechanism: As confirmed by several authors through hydraulic model tests and field observations, the mechanism of scour hole formation in plunge pools may be explained as follows:

(i) the large turbulence in the impact zone of the jet originates a very wide field of pressures with respect to the mean hydrostatic pressure. Yudtskii's (1963) model tests clearly confirmed this fact;

(ii) such varying pressures acting on the rock joints are able to dislodge blocks as long as the instantaneous upward pressures are much higher than the own-weight of the rock blocks; therefore an erosion process takes place;

(iii) the dislodged blocks are taken away from the scour hole by the effect of upward currents derived from the dispersion of the main jet in the pool;

(iv) as the scour hole develops, the pressure fluctuations decrease either by the effect of drainage between rock blocks or by the higher tailwater level;

(v) at this stage, the effect of the upward currents is important to the stabilization of the process, as its energy might be not enough to lift up the blocks from the hole.

2. Evaluations of the scour hole

(i) Equations with 3 or 4 parameters: In the last 50 years many equations were developed correlating the depth of scour hole (\( h \)) with the main hydraulic parameters:

- \( h \) = total available head (H);
- \( q \) = specific discharge (Q);
- \( d \) = rock block characteristic diameter (D).

However, the majority of these equations (see Table 2) were obtained from studies in hydraulic scale models with granular material. Most of the equations take the following appearance:

\[ h = k \cdot (q^x \cdot H^y / d^z) \]

where \( k \) is a coefficient and the exponents \( x, y \) and \( z \) vary according to the data adjusted by each author.

Mason (1985) presented a formula in which the exponents better fit several prototype and model data. This author also introduced the tailwater depth (\( h \)) and the gravity acceleration (g) for dimensional balance:

\[ h = k \cdot (q^x \cdot H^{1.15} / g^{0.33} \cdot d^{2.16}) \]

where: \( k = (6.42 - 3.10 \cdot H^{0.19}) \);

\( x = (0.60 - H/300) \) and \( y = (0.15 + H/200) \).

It is important to note that no geological characteristic of the bed rock is considered in these formulae, except by the rock block diameter. However, good and sound igneous rock as available in many dam sites in Brazil, Canada, etc... (see item 5) have shown exceptional resistance to erosion and does not follow any of the formulae. At Salto Osório and Salto Santiago spillways (Brazil) specific discharges up to 100 m³/s/m for approximately 80 m head were released during weeks; the observed erosion was minimal.

(ii) Complex equations: Other authors have considered additional parameters in their equations, such as: angle of the lip in the bucket (\( \alpha \)); water depth at the lip; dimensions of the rock blocks, tailwater depth at the beginning of operation, etc...

In this sense, the laboratory studies performed by Soleleva (1966) and Rui Martins (1973) are of interest; it is also worth mentioning the following works of Soviet authors:
Table 2. Equations to evaluate the erosion downstream of spillways

<table>
<thead>
<tr>
<th>Author- (Year)-Country</th>
<th>Parameters</th>
<th>Equation</th>
<th>Remarks</th>
<th>Sketch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schoklitsch, A. (1935) USSR</td>
<td>q, H, d_{90}</td>
<td>( h_e = 4.75 \times q \times \frac{0.57}{d_{90}} - 0.20 )</td>
<td>Model tests with granular material</td>
<td><img src="image" alt="Sketch" /></td>
</tr>
<tr>
<td>Patronev, A.M. (1937) URSS</td>
<td>q, H, dm</td>
<td>( h_e = 3.90 \times q \times \frac{0.50}{d_{m}}^{0.25} )</td>
<td>Model tests with granular material</td>
<td></td>
</tr>
<tr>
<td>Veronese, A. (1937) Italy</td>
<td>q, H dm</td>
<td>( h_e = 3.68 \times q \times \frac{0.54}{d_{m}}^{0.225} )</td>
<td>Model tests with granular material</td>
<td></td>
</tr>
<tr>
<td>Veronese, A. (1937) Italy</td>
<td>q, H</td>
<td>( h_e = 1.9 \times q \times \frac{0.54}{H}^{0.225} )</td>
<td>Limit of erosion for fine sand</td>
<td></td>
</tr>
<tr>
<td>Enggenberger &amp; Mueller (1948) Switzerland</td>
<td>q, H, d_{90}</td>
<td>( h_e = 22.88 \times q \times \frac{0.60}{d_{90}}^{0.40} )</td>
<td>Model tests with granular material</td>
<td></td>
</tr>
<tr>
<td>Meyer-Peter &amp; Mueller (1948) Switzerland</td>
<td>q, H, d_{90}</td>
<td>( h_e = 1.44 \times q \times \frac{0.60}{d_{90}}^{0.40} )</td>
<td>Model tests with sand</td>
<td></td>
</tr>
<tr>
<td>Leocastre, A. (1961) Portugal</td>
<td>q, H, Hd</td>
<td>( h_e = 0.30 ) to ( 0.40 ) (H-Hd)</td>
<td>Model tests with cubes (5 cm)</td>
<td></td>
</tr>
<tr>
<td>Davis &amp; Sorensen (1970) USA</td>
<td>H</td>
<td>( h_e = \frac{1}{3} H )</td>
<td>Observation of natural water-falls</td>
<td></td>
</tr>
<tr>
<td>USSR - USA</td>
<td>q, H</td>
<td>( p = 0.525 \times q \times 0.54 \times H^{0.225} )</td>
<td>adjusted curve for prototypes data</td>
<td></td>
</tr>
<tr>
<td>Rui Martins (1975) Portugal</td>
<td>q, H</td>
<td>( h_e = 1.5 \times q \times 0.60 \times H^{0.10} )</td>
<td>adjusted curve for prototypes data</td>
<td></td>
</tr>
<tr>
<td>Tarasimovich, I.I. (1979) USSR</td>
<td>q, H, he, K, k_1</td>
<td>( h_e = K \times q^{2/3} \times H^{0.225} \times K_1 )</td>
<td>From information of prototypes in USSR</td>
<td></td>
</tr>
<tr>
<td>Pinto, M.L.S. (1983) Brazil</td>
<td>q, H</td>
<td>( h_e = 1.2 \times q \times 0.54 \times H^{0.225} )</td>
<td>Adjusted curve for prototypes data</td>
<td></td>
</tr>
<tr>
<td>Mason, P.J. (1985) England</td>
<td>q, H, he, p, d</td>
<td>( p = 3.27 \times q \times H^{0.60} \times \frac{he-p}{0.30 \times d^{0.10}} )</td>
<td>From model and prototype informations</td>
<td></td>
</tr>
</tbody>
</table>

- q = specific discharge (m³/s/m)
- H = total head (m)
- he = tailwater depth after erosion (m)
- p = scour hole depth (m)
- Hd = head on the spillway crest (m)
- dm = mean particle size (m)
- d_{90} = size of 90% of the material (by weight)
- g = gravity acceleration (m/s²)
- k_1 = rock strength coefficient
- K = spillway geometry coefficient
a) Tarasimovich (1978) presented a more simple formula based on prototype information from Soviet Union: \( h_e = k \cdot q^{2/3} \cdot H^{0.225} \cdot K_1 \), where \( K_1 \) is a factor that considers the rock quality.

b) Yudetskii (1963), based on an extensive study in hydraulic model tests, developed a set of graphics in which the scour depth is defined once the dimension of the rock blocks is known. The main contributions of this work were: the measurement of the pressure fluctuations between rock mass joints, that were helpful for the thorough understanding of the rock erosion process in plunge pools; the relationship between the dimensions of joint openings and the drainage effect between blocks; the balance between the block's own weight and the upward pressure, etc...

However, none of the mentioned studies considered the interlocking (or cohesion between blocks) nor the geological influence. Recently, Spurr (1985) made an attempt to combine both hydraulic and geological aspects involved in plunge-pool erosion. A scour formula (alike Mason's, Veronese's etc...) that better calibrates the scour data of a known prototype (similar hydraulic and geological conditions) is used in the prediction of scour depth for a new project. A correction factor that accounts for the bedrock quality, discharges and spilling time, is applied over the first estimate. It is also proposed a classification of rock masses regarding their erosion resistance for plunge pools. Obviously this method depends on the availability of prototype data.

4. GEOMECHANICAL PROPERTIES OF ROCK MASSES REGARDING EROSION RESISTANCE

Most of these properties were discussed by several authors as explained in item 3. Spurr (1985) proposed a classification for the erosion resistance of plunge pool foundation but, in a later paper (Spurr, 1986), recognized that its implementation depends upon the publication of "more comprehensive field records of the geology, spill-time and plunge-pool surveys". So, this chapter will summarize some basic concepts attempting to provide a basis towards a future classification.

4.1 Genesis, lithology and structures

The interpretation of rock mass behaviour at hydraulic structures in operation is very important to achieve a preliminary qualification of their suitability for energy dissipation. From the data presented in Tables 3, 4 and 6 it is possible to establish the following general trends in litho-genetic terms:

1. The most common igneous rocks (granites, diorites, basalts, rhyolites, etc.) are more suitable for energy dissipators, not even due to their high mechanical resistance, but also because they are relatively homogeneous and isotropic. On the other hand, metamorphic and sedimentary rocks are potentially less adequate because they have an intrinsic discontinuity system related to its formation: schistosity and bedding, respectively. These features introduce anisotropy and heterogeneity within the mass, constituting weakness surfaces of great extent and reasonably flat.

2. Igneous intrusive rocks tend to become less fractured and more resistant with increasing depths, while this is not true for sedimentary rocks (and some volcanic).

3. In metamorphic and sedimentary rocks one of the joint systems always coincide with schistosity or bedding, thus reducing its resistance to water erosion. Moreover, sedimentary rocks have low specific gravity and low compressive strength when compared to the other two genetic types. That is, they are more liable to abrasion erosion and, for a given situation, the stable unitary rock block should be bigger than igneous/metamorphic rocks, as the weight varies with the third power of the mean radius.

4. Geologic hazardous features like faults, folds, shearing zones, geologic contacts (especially in rocks with very distinct properties), should be avoided (or treated) in any case. As a summary of the above paragraphs, a preliminary qualification of rocks is proposed below:

- **Rocks**
  1. Intrusive igneous (granite, diorite)
  2. Extrusive igneous (basalt, rhyolite) & hard metamorphic (some gneiss)
  3. Metamorphic (quartzite, schists) & hard sedimentary (silicified sandstone)
  4. Soft sedimentary (sandstone, limestone)
  5. Weathered rock

- **Structures**
  1. Joints
  2. Shear Zones
  3. Faults
### Table 3. Erosion at ski-jump spillways in USSR (from Taraimovich, 1978)

<table>
<thead>
<tr>
<th>Dam (Year)</th>
<th>River</th>
<th>Spillway Data (m)</th>
<th>Foundation Geology</th>
<th>Maximum Erosion Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dneproges</td>
<td>-</td>
<td>25 150 35 17</td>
<td>Gneiss-granite</td>
<td>50 8</td>
</tr>
<tr>
<td>Farkhad</td>
<td>-</td>
<td>4 520 14 5</td>
<td>Sandstone</td>
<td>75 18</td>
</tr>
<tr>
<td>Bukhtarma (60)</td>
<td>Irtish</td>
<td>4 520 67 14</td>
<td>Gabbro</td>
<td>no data -</td>
</tr>
<tr>
<td>Supkhum</td>
<td>-</td>
<td>20 000 87 9</td>
<td>Gneiss-granite</td>
<td>no data -</td>
</tr>
<tr>
<td>Irikla</td>
<td>-</td>
<td>7 805 23 18</td>
<td>Fractured tuffs</td>
<td>118L -</td>
</tr>
<tr>
<td>Bratsk (64)</td>
<td>Angara</td>
<td>17 500 95 12</td>
<td>Diabase</td>
<td>65L 12</td>
</tr>
<tr>
<td>Ust-Ilim (80)</td>
<td>Angara</td>
<td>14 900 80 11</td>
<td>Diabase</td>
<td>73L 12</td>
</tr>
<tr>
<td>Sayan-S. (80)</td>
<td>Yenisei</td>
<td>15 900 166 18</td>
<td>Hornfelsic schist</td>
<td>132L 41</td>
</tr>
<tr>
<td>Krasnoyarsk (72)</td>
<td>Yenisei</td>
<td>12 000 80 18</td>
<td>Granites</td>
<td>102 17</td>
</tr>
<tr>
<td>Bkhakra</td>
<td>-</td>
<td>8 200 165 -</td>
<td>Sandstones</td>
<td>- -</td>
</tr>
<tr>
<td>Inguri (85)</td>
<td>Inguri</td>
<td>2 500 225 34</td>
<td>Fractured limestones</td>
<td>124 46</td>
</tr>
<tr>
<td>Toktogol (78)</td>
<td>Naryn</td>
<td>3 400 183 20-30</td>
<td>Fractured limestones</td>
<td>no data 5-10</td>
</tr>
</tbody>
</table>

Q = design flood  H = total head for Q  L = laboratory

### Table 4. Spillways with erosion problems

<table>
<thead>
<tr>
<th>Dam River (Year)</th>
<th>Spillway Data (Design)</th>
<th>Geological Features</th>
<th>Erosion Problems</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jaguar (1970)</td>
<td>Ski-jump with shallow plunge pool.</td>
<td>Quartzites (strong schistosity, dip left bank). One main subvertical joint set.</td>
<td>Reached 37m depth and 100m length in 7 years, with a left bank trend.</td>
</tr>
<tr>
<td>Grande Brazil</td>
<td>Q = 13 600 m³/s; H = 33.5 m; q = 130 m³/s.m</td>
<td>Quartzites (schistosity dip 750 d/s). Three joint sets. Same faults. Sericitic-schists, folded, less fractured.</td>
<td>Early spilling before concrete slabs were finished caused severe damage to the rock mass with an u/s trend.</td>
</tr>
<tr>
<td>Peixoto (1956)</td>
<td>Ski-jump at right abutment.</td>
<td>Quartzites (schistosity dip 750 d/s). Three joint sets. Same faults. Sericitic-schists, folded, less fractured.</td>
<td>Early spilling before concrete slabs were finished caused severe damage to the rock mass with an u/s trend.</td>
</tr>
<tr>
<td>Grande Brazil</td>
<td>Q = 10 400 m³/s; H = 39.8 m;</td>
<td>Quartzites (schistosity dip 750 d/s). Three joint sets. Same faults. Sericitic-schists, folded, less fractured.</td>
<td>Early spilling before concrete slabs were finished caused severe damage to the rock mass with an u/s trend.</td>
</tr>
<tr>
<td>BM (1982) Brazil</td>
<td>Concrete chute protected by 10m of anchored concrete slab q = 93 m³/s.m</td>
<td>Soft sandstone</td>
<td>Scour &quot;hole&quot; 8 to 10m deep by 15m wide. Qrec = 840 m³/s during 10 days (q = 84 m³/s.m)</td>
</tr>
<tr>
<td>Picote (1957)</td>
<td>Ski-jump with 40m of water in lower pool. (narrow valley) Q = 11 000 m³/s; q = 137 m³/s.m</td>
<td>Sound granite, two main joint systems (horizontal &amp; vertical). Prismatic unit blocks</td>
<td>Reached 22m depth in 6 years (spill during floods), with an u/s trend. Stabilization of valley walls with concrete.</td>
</tr>
<tr>
<td>Douro Portugal</td>
<td>Ski-jump with 40m of water in lower pool. (narrow valley) Q = 11 000 m³/s; q = 137 m³/s.m</td>
<td>Sound granite, two main joint systems (horizontal &amp; vertical). Prismatic unit blocks</td>
<td>Reached 22m depth in 6 years (spill during floods), with an u/s trend. Stabilization of valley walls with concrete.</td>
</tr>
<tr>
<td>Kariba (1959)</td>
<td>Free-overfall with 20m of water in lower pool (narrow valley). Q = 1 600m³/s; H = 100m; q = 70 m³/s.m</td>
<td>Biotitic gneiss and quartzites, folded and faulted.</td>
<td>Scour hole 60 m deep and 119 m wide.</td>
</tr>
<tr>
<td>Zambezi Zimbabwe</td>
<td>Free-overfall with 20m of water in lower pool (narrow valley). Q = 1 600m³/s; H = 100m; q = 70 m³/s.m</td>
<td>Biotitic gneiss and quartzites, folded and faulted.</td>
<td>Scour hole 60 m deep and 119 m wide.</td>
</tr>
<tr>
<td>Tarbela (1976)</td>
<td>Ski-jump, with 18m of tailwater depth</td>
<td>Highly contorted and faulted beds of limestone and phyllite; basic dike (50m thick) 60°d/s dip, obliquely (45°) to jet direction</td>
<td>Reached 20m locally in weeks. The dike formed a ridge that deflected the jet towards the right bank. Expensive remedial works. (Qrec = 8 900 m³/s)</td>
</tr>
</tbody>
</table>

u/s = upstream  d/s = downstream  q = specific discharge downstream the piers
4.2 Fracture spacing and compressive strength

The fracture spacing defines the unitary rock block average dimension, which is a traditional parameter for the minimum eroding velocity analysis, while the compressive strength is an indirect measure of the abrasion resistance. Although simple, the concept of block dimension may be also applied to the more complicated situation where water pressure fluctuations are involved (Nascimento, 1983): if the rock block is some times greater than the whirlpool diameter, the average instantaneous pressure fluctuation should approach zero and, in the opposite case, this fluctuation will be maximum when block dimension approaches zero. The combination of these two basic properties allows an immediate definition of the rock mass main characteristics. By adapting a graphic presented by John (1962) to the data discussed on item 5, it was possible to propose a second approach to erosion resistance classification, which is presented in Figure 7.

![Figure 7: Rock Mass Classification According to Erosion Resistance](image)

### Figure 7: Rock Mass Classification According to Erosion Resistance

- OC = Uniaxial compressive strength
- e = Fracture spacing

Note: It was retained the 1.0 m spacing, which corresponds to a reference block in a rock mass where the R.Q.D. equals 100% and the 0.25 m spacing that corresponds to an average block diameter (considered as "loose" material) commonly adopted for scouring water-pool analysis.

4.3 Number of joint sets and shape of unitary rock block

The fracture spacing concept is a simplification in the description of rock masses, derived from the traditional definition of average fracture intersections along a borehole core. In the actual tridimensional case it is necessary to define the number of different joint sets, characterized by their spatial orientation and spacing (ISRM, 1978).

The number of joint sets has an important influence on the mechanical behaviour of the rock mass: the higher the number of joint sets, the higher the degree of freedom for displacements without participation of the intact rock. So, for erosion resistance, it is desired that the rock mass has a low number of joint sets. Rock masses present joint sets varying from one (e.g. dikes) to three (e.g. granites) and even four (e.g. some gneisses).

Combining the number of joint sets and the corresponding fracture spacings it is possible to define the shape, dimensions and orientation of the unitary rock blocks, provided the joints are regularly spaced with high persistence.

Müller (1965) proposed a classification of prismatic rock blocks based on the edge dimensions: d1 and d2 of the basal plane, and d3 of the vertical axis (d1=d2 in all cases). Adding this criteria to that suggested by ISRM (1978) it was defined the classification presented on Table 5.

Contrary to Vodicar (1963) conclusion, the shape of rock block is a very important property related to the mechanism of block dislodgement by pressure fluctuations. In order that the rock block is not pulled-out from the plunge-pool bottom, it is necessary that the pressure in excess of its submerged weight be supported by the interlocking among the adjacent blocks. It is difficult to measure the interlocking, although it is possible to obtain an indirect estimate by analysing scoured and unscoared water pools, in order to compare the geological and geomorphological factors that control it. Nevertheless, the pull-out force is a function of the basal area of the block, while the interlocking is directly proportional to the lateral contact area of the block. So, let us compare the relationship between the basal and lateral areas of three block types where \( d_{\text{maximum}} > d_{\text{minimum}} \).
Table 5. Classification of rock blocks

<table>
<thead>
<tr>
<th>Rock mass shape</th>
<th>Geometric Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compact</td>
<td>Few joints or widely spaced joints</td>
</tr>
<tr>
<td>Cubic*</td>
<td>Equidimensional jointed (3 sets): $d_1 \approx d_2 \approx d_3$</td>
</tr>
<tr>
<td>Columnar*</td>
<td>One dimension dominates: $d_3 &gt; d_1$ and $d_2$</td>
</tr>
<tr>
<td>Slabby*</td>
<td>Two dimensions dominate: $d_1$ and $d_2 &lt; d_3$</td>
</tr>
<tr>
<td>Irregular</td>
<td>Great variation of shape and dimensions</td>
</tr>
<tr>
<td>Fragmented</td>
<td>Hardly fractured: $d_1, d_2$ and $d_3 &lt; 0.10$ m</td>
</tr>
</tbody>
</table>

(*) Note: Müller (1963) designated these blocks as great, medium and small when $d_{\text{maximum}}$ is, respectively: 1,00m; 1,00 > $d_{\text{max}}$ > 0,10m; < 0,10m.

4.4 Other properties

There are several properties of the joints in a rock mass that are of interest to the subject. In a certain way they were all included in the interlocking concept, because it is difficult, with the present knowledge, to separate their effects. Thus, these properties will be only briefly discussed (more details can be obtained in: ISRM, 1978; Goodman, 1976 and Hoek, 1983):

1. Infilling: as already observed by Yuditski (1963) and Martens (1971) it has little influence on strength since it can be easily removed by the water flow, although it may affect the drainage conditions.
2. Aperture: open joints tend to be more quickly penetrated by the water jet than tight ones, but here it must be remembered the concept of the critical aperture value proposed by Yuditski (1963), who studied the effects of a water jet on plastic blocks separated by slots and instrumented with pressure transducers.

For the same test conditions the maximum pressures occurred for slot widths around 2,0 mm (reducing or increasing the slot widths led to pressure reductions from 10 to 25%). This means that there is a critical value fur the slot width, below which the pressure fluctuations are restricted and over which the dissipation is facilitated. In other words, pressure fluctuations vary with the drainage conditions in the joints, which depends on the aperture.

3. Naviness and roughness: these two properties effectively determine the degree of interlocking. The more roughness and locking exist between the joint walls the will be their resistance to dislocations.

In short, it can be said that tight, clean, rough and locked joints are more resistant than open, filled, smooth and plane ones.

5 CONSIDERATIONS ON DESIGNING HYDRAULIC STRUCTURES ON HARD ROCKS

5.1 Blasting effects

The effects of blasting may change considerably the rock surface subjected to water
erosion. Normally, blast design, including hole geometry and explosive distribution, does not fully account for the rock discontinuities, spacing and orientation.

In order to avoid large free blocks in the rock surface to be carried away by water flow, the blasting and drilling techniques must be well controlled. A well finished surface with a minimum disturbance in the rock mass is a must for hydro structures. Practical tests in the field as to define the blasting procedures and well designed technical specifications for the job are strongly recommended.

This approach was successfully adopted at La Grande complex in Canada, where most of the hydraulic structures were not concrete lined (spillways, tunnels). Murphy & Levai (1982) fully reported the Canadian experience pointing out that "blasting and drilling techniques were more important than geological conditions to assure excavations quality".

5.2 Unconfined walls

Very often erosion in excavated rock channels and plunge pools progresses sideways as lateral walls slide down by the flow action. At the wall surface there is little effect of interlocking (and gravity) to keep blocks stable. That is particularly critical in the case of rocks with vertical or sub-vertical jointing system. In this case some remedial measures may be taken during design and construction as:
- to enlarge the excavation laterally as to avoid the direct impact of the water jet;
- to provide berms or a gentler slope to improve the stability;
- to perform a smooth blasting at the wall surface;
- to protect the less stable zones with rock bolts, wire-mesh, shotcrete, etc...

5.3 Unlined chutes and rock cascades

Flow on an unlined rock chute induces shearing stresses and small pressure fluctuations. As explained before, whenever possible rock chutes should be used in lieu of excavated plunge pools. No general rules are applied for designing the ramp, that must be adapted topographically to the rock profile. The chute stretch near to the dam may be designed with a gentler slope as to decrease the flow velocity in this zone. Care should be taken in designing the lateral walls as the flow depth in a rough surface is higher than in concrete and incorporates more air. Also, the geotechnical protection of these walls should be carefully regarded. If the chute is partially lined the contact between concrete and rock is a weak point. To overcome this, an aeration recess is strongly commended to get the jet far from the transition as well as to incorporate air to the flow. It's well known that air-entrained flows causes less erosion.

Figure 8. Unlined spillway

Artificial rock cascade may be efficiently designed as energy dissipator downstream of spillways. The rock steps are normally exploited as quarry for the dam. Good examples are La Grande II (Canada) and Dartmouth (Australia). The protection of the vertical faces in the step is normally required at least in the first steps. Critical geological features in the chute (or steps) must be avoided or conveniently treated during the construction. Field inspection "in situ" must orientate modifications in the design. Faults, sheared zones, highly fractured and/or slaby zones, etc..., must be regarded with care.
5.4 Unlined tunnels

For a better hydraulic performance, the unlined tunnels should be excavated from the downstream end in order to have the "saw-tooth" excavation shape favourably orientated with respect to flow direction.

5.5 Geomechanical treatments

The type of geomechanical treatment to protect the rock surface subjected to flow action will depend upon: the stress acting on the rock; the rock conditions; the expected performance of the structures; the acceptability of damages that do not risk the structures; service time of the structure, etc...

In general, the following types of treatment should be considered (alone or combined):

- removal of loose materials: rock blocks, soft infillings, etc...
- application of water-blaster;
- application of dental concrete in scour holes;
- slush grout in open joints;
- shotcrete in fractured zones;
- rock-bolting of rock wedges, walls, etc...
- drainage of sub-surface discontinuities;
- grouting.

5.6 Practical examples of structures designed on hard rock

The erosion resistance of hard igneous rock has to be considered in designing hydraulic structures, in order to reach an economical and safe project. Table 6 presents a list of spillways and tunnels already built in Brazil, Canada, Australia, USA and a short report on their performance. It is remarkable the excellent behaviour of basalts in Brazil. Unlined tunnels and spillways with shallow (or without) plunge spools have shown only minor erosion even after drastic floods.
<table>
<thead>
<tr>
<th>Dam/year</th>
<th>Structure &amp; Spillway Data</th>
<th>Geological Features</th>
<th>Record of Operation</th>
<th>Erosion Problems</th>
</tr>
</thead>
<tbody>
<tr>
<td>S. Santiago</td>
<td>Chute spillway ending in flipbucket that throws the jet on rock steps (4 m drops)</td>
<td>Vesicular-amigdoidal basalt at the steps; columnar dense basalt at the lateral walls of the excavation</td>
<td>Operation under drastic flood conditions during wet seasons in the last 7 years; discharges as high as 100 m³/s/m; dissipation by direct impact of the jet on the rock steps.</td>
<td>Minor erosion in dense basalt; small pockets in breccia or vesicular basals; excellent performance.</td>
</tr>
<tr>
<td>Iguazu River</td>
<td>(1979)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Salto Osorio</td>
<td>A service chute spillway (4 gates), ending in a flipbucket that throws the jet on an unlined channel; an emergency spillway with a flipbucket discharge into a shallow plunge-pool. Qd= 27,000 or Q= 160 Hz= 53 (service); Hz= 62 m (emergency) -9</td>
<td>Dense basalt at the top of the channel walls; breccias and vesicular amigdoidal basalt at the bottom of the channel</td>
<td>Max. flood at service spillway in 1983 Q= 150 m³/s/m; both spillways released high discharges every year.</td>
<td>Minor erosion of columnar basalt at the channel wall (unconfined); excellent performance of both spillways.</td>
</tr>
<tr>
<td>Iguazu River</td>
<td>(1975)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foz do Areia</td>
<td>A chute spillway throws the jet in a shallow plunge pool excavated in dense basalt and the floor in breccia</td>
<td>Lateral walls of the</td>
<td>Operation near max. discharge in 1983 Q= 160 m³/s/m; other floods released in the last 5 years;</td>
<td>No erosion was observed in the pool; only a minor erosion in an end-sill (probably due to blasting)</td>
</tr>
<tr>
<td>(1979)</td>
<td></td>
<td>pool excavated in dense basalt and the floor in breccia</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Iguazu River</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foz do Areia</td>
<td>Two unlined diversion tunnels with horseshoe section. Qd= 12 m diam. (122 m²) and 580 m</td>
<td>Breccia and columnar</td>
<td>The tunnels operated under pressure during flood: velocities up to 15 m/s; 2.5 years operation</td>
<td>No erosion was observed after inspection</td>
</tr>
<tr>
<td>(76/78)</td>
<td></td>
<td>jointed dense basalt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Iguazu River</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Caesoeira</td>
<td>Two unlined chute spillways: a service spillway (left) Qd= 10,000; 10 tainter gates 10 x 10; an emergency spillway (right) Qd= 7,000 m³/s, 16 tainter gates 10 x 7, Hz= 18.</td>
<td>Sound dense basalt</td>
<td>Max. record discharges of 7,000 m³/s in the service spillway; velocity at the beginning of the rock channel -18 m/s</td>
<td>No erosion at all either in the concrete/rock interface or in the rock chute.</td>
</tr>
<tr>
<td>Cururupu</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sao Simao</td>
<td>A chute spillway ending in a flipbucket that throws the jet on a shallow channel; Qd= 24,100 or Q= 133; Hz= 59.8; tailwater depth: 15 m; 10 tainter gates 15 x 19</td>
<td>Vesicular amigdoidal to dense basalt (channel bottom) lying on a sedimentary breccia</td>
<td>In operation since 1978; max. observed specific discharge of 75 m³/s/m for a total head of 72.9 m</td>
<td>Scour hole -12 m depth; much less than expected by hydraulic model (probably due to blasting)</td>
</tr>
<tr>
<td>Location</td>
<td>Type</td>
<td>Material</td>
<td>Details</td>
<td>Notes</td>
</tr>
<tr>
<td>---------------</td>
<td>-------------------------------</td>
<td>-------------------</td>
<td>-------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>Água Vermella (1978)</td>
<td>Ski jump with shallow plunge pool (10 m); tail water depth</td>
<td>Dense basalt</td>
<td>Discharged a large flood in 1979 and normal floods every year.</td>
<td>Pool inspected after dewatering; no erosion at all.</td>
</tr>
<tr>
<td>Grande River</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brasil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. A. A. Old (1979)</td>
<td>Short concrete slab followed by an unlined chute; Qd = 10,000</td>
<td>Granite-gneiss fractured and weathered in some places; faults and sheared zones in the unlined chute</td>
<td>In operation since 1980; some higher discharges were released; velocity in the rock chute ~25 m/s</td>
<td>Moderate erosion holes were observed near the concrete slab; large erosion holes in the faults; successful correction with dental concrete.</td>
</tr>
<tr>
<td>S. Francisco River of the slab; 8 tainter gates</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brasil</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>La Grande II (1977)</td>
<td>Stilling basin spillway, partially lined, followed by rock steps (~10 m) in cascade, 1500 m long; Qd = 10,280; Hd = 110; 8 lift gates</td>
<td>Sound gneiss-granite with vertical main joint system and sub-horizontal joints near surface</td>
<td>It was not expected to operate very often; however discharges of 2900 m³/s were released; velocities in the lower steps ~26 m/s</td>
<td>Loose blocks removed in the unlined reach of the stilling basin; erosion observed in a zone of weathered rock, mainly in the walls, at step no2; no risk for the structures.</td>
</tr>
<tr>
<td>La Grande River Canada</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>La Grande II (1976/1978)</td>
<td>Two unlined diversion sumps</td>
<td>Sound gneiss-granite</td>
<td>Velocities as high as 14 m/s were reported during 3 years of operation</td>
<td>No erosion</td>
</tr>
<tr>
<td>Pindari River (Australia)</td>
<td>Uncontrolled and unlined chute spillway; Qd = 6,500</td>
<td>Rhyolite</td>
<td>Max. observed discharge of 1.350 m³/s</td>
<td>Acceptable erosion; dental concrete applied where necessary</td>
</tr>
<tr>
<td>Severn River (Australia)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dartmouth (1974)</td>
<td>Uncontrolled spillway with a cascade in rock steps (15 m drops) ending in a rock trap</td>
<td>Granite</td>
<td>No records of operation. The cascade is 3 times larger than the spillway</td>
<td>Some erosion at the crest of each step.</td>
</tr>
<tr>
<td>Murray River (Australia)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hell Hole (1975)</td>
<td>Uncontrolled spillway with a curved cascade in rock steps (33 m drops)</td>
<td>Granite</td>
<td>No information</td>
<td>Some erosion at the crest and some rocks at the toe of each step.</td>
</tr>
<tr>
<td>Middle Fork River (1975)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kangaroo Creek &amp; Parangana (Australia)</td>
<td>Unlined spillways</td>
<td>&quot;sound rock&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Qd - design flood (m³/s); q - specific discharge downstream the piers (m³/s.m)
Hd - total head for Qd (m); gate dimensions: width (m) x height (m).
6. MAIN CONCLUSIONS

(i) In the authors' opinion the understanding of prototype performance on equivalent rock mass is the best criteria for the design of a new structure. The engineering judgement and engineer's experience accounts more than any empirical formula or theoretical study.

(ii) Scouring formulae should be used to predict the plunge pool depth when the rock is not competent to withstand flow action or when there is no data available on rock quality. Clearly, the hydraulic parameters H and q play the most important role in such formulae.

(iii) The rock quality must be taken into account for the design of a hydraulic structure. The main aspects to consider are: weathering, fracture spacing, abrasion resistance, main discontinuities (such as schistosity, contacts and faults) and interlocking between rock blocks.

Common igneous rocks (granites and basalts) are more resistant to erosion than sedimentary or metamorphic rocks when subjected to flow action.

(iv) For sound rock (no weathering or extensive fracturing), the effect of shear stress (or velocity), as in unlined chute spillways, causes less erosion than large pressure fluctuations of a jet entering in a plunge pool.

(v) Vertical unconfined walls, as in plunge pools, are more liable to erosion due to pressure fluctuations; the same effect may be expected in narrow river valleys. Highly fractured rock worsen the situation in such cases. Thus, care should be taken in the design and protection of the walls.

(vi) The blasting technique is important in defining a well finished rock surface. An artificially broken rock surface is more prone to erosion, as loose blocks may be carried away by water flow.

(vii) Field inspection during construction is very important; rock stabilization may be necessary and the remedial solutions normally applied are slurh-grout, denta1 concrete, steel mesh, rock bolts and shotcrete.

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Geomechanical properties of rock mass of Xingó dam, Brazil

Propriétés géomécaniques du massif rocheux du barrage Xingó, Brésil

C.J.G Aragão & R.I.B. Souza, Generation Projection Department, Companhia Hidro Elétrica do São Francisco,
Recife, PE, Brazil
R.A. Abrahamo, Promon Engenharia, SP, Brazil

ABSTRACT: This paper presents the methodology used during the stage of project design for Xingó
Hydroelectric Development, as well as geomechanical parameters both of the intact rocks and the
rock mass.

Therefore, envelopes strength obtained through laboratory direct shear tests are presented, as
well as deformability modulus obtained through uniaxial compression tests in dull rock core,
dilatometric tests in boreholes and plate bearing tests on rock mass, and, at last, deformability
tests on the granular material to be used in the main dam. The results are analysed taking into
account the local geology with respect to the rock mass weathering and fracturing degree, and
are compared with results obtained in other project designs.

RÉSUMÉ: Cette communication présente la méthodologie adoptée dans la phase du projet de la
Centrale Hydroélectrique Xingó, ainsi que des paramètres géomécaniques de la roche intacte et du
massif rocheux. De cette façon on montre des courbes intrinsèques des résistances obtenues à partir
des essais de cisaillement direct. On présente les valeurs des modules de déformabilité obtenus à travers des essais de compression simple réalisés en carottes de sondage. En outre, on présente des dilatomètres en trous de sondage, des essais en plaque direct sur massif et
déformabilité du matériau granulaire qui devra être employé dans le barrage. Ces résultats sont
analysés en fonction de la géologie locale compte tenu le degré d'altération et fracturation
du massif rocheux.

Quelques comparaisons avec les résultats d'autres ouvrages sont aussi réalisées.

1. INTRODUCTION

One of the greatest difficulties in analysing the stability and deformability of structures
founded on rock masses lies in the characterization and definition of adequate geomechanical
parameters of jointing rock masses. It is known that these mass behavior under load action is not
conditioned by the intact rocks properties but by the existing discontinuities referred to
their litology, weathering and fracturing degree, and also as a function of specially important
gеological features. On the other hand, the big technological progress achieved in the field
of jointing rock mass analysis methods requires more and more the improvement of investigation
techniques and a more detailed joint network, so as to adequately provide these analysis methods
inputs. Considering the above referred, it has been decided for Xingó Hydroelectric Development
(one of the greatest in the world, in its kind) to carry out a large geomechanical investigation
program in order to define the rock mass properties and its spatial variability, emphasizing
the spatial disposition of discontinuities. Investments in this way usually lead to better
elaborated projects, and sometimes to more economical solutions.

2. GENERAL FEATURES OF XINGÓ DEVELOPMENT

Xingó Hydroelectric Development, a property of Companhia Hidro Elétrica do São Francisco, is located
in the São Francisco River, in the Northeast of Brazil. 75 km downstream Paulo Afonso
Hydroelectric Complex, in a narrow valley region, with its reservoir totally inserted in the
river canyon. The main dam is a concrete face rockfill dam, with a maximum height of 140 m and
an embankment volume of about 12,600,000 m³. The spillway, located on the left bank, has the
capacity of 33,000 m³/s. The power intake are in gravity structures, with a maximum height of
70 m. The steel penstocks, 9.5 m in diameter, will be exposed. The power house is semi-outdoor
and allows for the installation of 10 units of 500 MW each one. River deviation will be made

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through four tunnels, 16 m in diameter, excavated on the right bank, with an average length of 675 m.

3 GEOLOGY

The regional geology is characterized by a crystalline basement of the middle and lower Pre-Cambrian age, consisting of a metamorphic and igneous rock complex, which during the upper Pre-Cambrian suffered intrusion of granitic rocks. These rocks consist of migmatite in which the oldest component is represented by metabasites generally full of feldspar and rich in iron-magnesium, dispersed in the younger component represented by granitic rocks characteristically gray. Frequently these rocks appear sound and fractured, most of their discontinuities being vertical and subvertical, with a great amount of faults and dikes, which are predominantly found between N40°E and N70°E. The regional faults are extensive and preferentially between the N50°E and N65°E directions. The petrography analysis shows the presence, in decreasing occurrence order, of plagioclase, biotite, hornblende, quartz, microcline and amphibole. The soils covering is from the quaternary age and consists of residual, colluvium and alluvium soils. The average soil and weathering rock layer thickness ranges from 9 to 5 m, with a maximum of 15 m.

4 GEOMECHANICAL INVESTIGATIONS

The main purposes guiding the geomechanical investigation programs were the following: To determine the thickness and type of the rock mass covering soils; to establish the mass structural geology, so as to make possible the definition of a representative geomechanical.
model; to evaluate the rock mass permeability characteristics for seepage analysis; to evaluate the rock mass and intact rock mechanical characteristics in terms of deformability and shear strength; to evaluate the deformability of the granular material to be used in the dam.

The methodology used in the investigations distribution aimed to determine the spatial variability of mechanical properties and discontinuities, as well as detect important geological features, considering a geometrical distribution taking into account the works lay-out.

5 DRILLINGS, TRENCHES, SHAFT AND GEOPHYSICAL SURVEY

94 rotary drillings were made with diameter sampling NX and 86mm, amounting to 4650 m of boring. Still in this program, 690 water pressure tests, 141 boreholes inclination measurements and 227 drill core orientations were performed. The water pressure test shows a permeable rock mass, specially beyond 20 meters of depth. Only 9 intervals with higher than 10⁻³ cm/s permeability occurred, of which 7 in the initial intervals of the holes, varying from 10⁻³ cm/s to 8 x 10⁻³ cm/s. In the remaining ones, permeability varied from 9 x 10⁻⁴ cm/s to 3 x 10⁻⁶ cm/s.

To make possible the surface mapping and determine the soil thickness, 46 trenches in the soil were performed, amounting to 2030 m of linear excavation, with the use of D-8 tractors provided with scarifier tooth.

As a direct access to the structure foundations or the tunnels rock mass was not available, "in situ" tests in a rock excavated trench in the power house location were performed, in which plate bearing tests were done and samples for direct shear tests in discontinuities were gathered. To investigate the probability of sub-horizontal discontinuities under the power intake, a 4 m in diameter and 60 m deep shaft was done, added by 4 rotary drillings with integral sampling, amounting to nearly 270 m of samplings.

9020 m of geophysical prospecting lines through seismic refraction were performed, to determine soil horizons in the dikes foundations and borrow areas.

Finally a program of seismic reflection amounting to 24 km of prospecting lines was performed, to know the conditions of the river bottom, to determine the sediments thickness, and to detect existing boulders and the bedrock configuration.

6 UNIAXIAL COMPRESSION TESTS

18 uniaxial compression tests were performed through drill rock cores, with NX diameter, most of them samples of sound granite-gneiss rocks. The average value of the uniaxial compression strength was of 1,156 kg/cm², with a standard deviation of 177 kg/cm². It was found an average Young's modulus of 721,700 kg/cm², with a standard deviation of 125,600 kg/cm².

From the figure 03 it can be inferred that Xingó intact rock presents a medium to high strength and a high elasticity modulus.

![Figure 03 - Mere's Classifying Graph](image-url)
7 LABORATORY DIRECT SHEAR TESTS

In the rock excavated trench, at the power intake location, some sub-horizontal discontinuities, with diversified contact types, were found. Through rotary drillings with 8" in diameter, samples were collected to perform laboratory direct shear tests, three of them with rock to rock contact, and five ones with clayey infilled contact.

After duly prepared, these samples were tested in 4 load cycles, with different normal stresses. All tests were performed with submerged and embedded samples. The table below shows the normal and shearing stress values of the 8 tested samples.

<table>
<thead>
<tr>
<th>SAMPLE NO</th>
<th>IN NATURA</th>
<th>AFTER FAILURE TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st CYCLE</td>
<td>2nd CYCLE</td>
</tr>
<tr>
<td>o</td>
<td>t</td>
<td>o</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>1</td>
<td>5.4</td>
<td>1.41</td>
</tr>
<tr>
<td>2</td>
<td>7.4</td>
<td>1.53</td>
</tr>
<tr>
<td>3</td>
<td>12.1</td>
<td>2.03</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>0.39</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td>0.13</td>
</tr>
<tr>
<td>6</td>
<td>3.9</td>
<td>5.0</td>
</tr>
<tr>
<td>7</td>
<td>7.7</td>
<td>10.73</td>
</tr>
<tr>
<td>8</td>
<td>12.2</td>
<td>13.92</td>
</tr>
</tbody>
</table>

**TABLE 01:** Strength Parameters obtained in the laboratory direct shear test (kg/cm²).

Figure 04 shows the failure average envelopes for samples with rock to rock and clayey infilled contacts, both for "in natura" (1st. cycle) tests and for after failure cycles (2nd., 3rd. and 4th. cycles). As expected, the shear strength of the samples with fulfilling material appeared reduced, compared with that of the unfilled discontinuities. It was observed that tests performed "in natura" and after the failure of samples with fulfilling material, presented straight average envelopes quite close, belonging to the same statistic universe. This is justified by the observation, in the discontinuities, of previous shear evidences, including streaks on the walls, besides the mineralogical examination of the fulfilling material, which presented a high content of mixed, that in a certain way reduces the shear strength.

"In natura" tests of rock to rock contact samples presented shear strength envelopes superior to those of the after failure tests, as expected with this kind of material. It can be observed that in the after failure tests, the cohesion intercept is significantly reduced (from 1.44 to 0.63 kg/cm²), while the friction angle is only reduced from 47° to 44°.
It was also tried the determination of deformability $K_n$ and $K_t$ parameters, which can be obtained from the direct shear tests. Table 02 shows $K_n$ and $K_t$ values and normal stresses, both for fulfilled and rock to rock contact samples, either in "in natura" or in after failure tests.

<table>
<thead>
<tr>
<th>TEST NO</th>
<th>$\sigma$ (kg/cm²)</th>
<th>$K_n$ (kg/cm²/cm)</th>
<th>$K_t$ (kg/cm²/cm)</th>
<th>TEST NO</th>
<th>$\sigma$ (kg/cm²)</th>
<th>$K_n$ (kg/cm²/cm)</th>
<th>$K_t$ (kg/cm²/cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.4</td>
<td>89</td>
<td>36</td>
<td>5</td>
<td>2.5</td>
<td>18</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>9.1</td>
<td>206</td>
<td>79</td>
<td></td>
<td>1.0</td>
<td>-</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>12.1</td>
<td>600</td>
<td>111</td>
<td></td>
<td>1.5</td>
<td>-</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>16.8</td>
<td>455</td>
<td>135</td>
<td></td>
<td>2.1</td>
<td>-</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>7.4</td>
<td>71</td>
<td>18</td>
<td>6</td>
<td>3.9</td>
<td>900</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>10.7</td>
<td>213</td>
<td>34</td>
<td></td>
<td>7.6</td>
<td>1200</td>
<td>128</td>
</tr>
<tr>
<td></td>
<td>14.8</td>
<td>292</td>
<td>44</td>
<td></td>
<td>11.5</td>
<td>1300</td>
<td>156</td>
</tr>
<tr>
<td></td>
<td>6.9</td>
<td>-</td>
<td>24</td>
<td></td>
<td>15.4</td>
<td>1850</td>
<td>100</td>
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<tr>
<td>3</td>
<td>12.1</td>
<td>29</td>
<td>28</td>
<td>7</td>
<td>7.7</td>
<td>-</td>
<td>188</td>
</tr>
<tr>
<td></td>
<td>15.9</td>
<td>212</td>
<td>36</td>
<td></td>
<td>12.4</td>
<td>-</td>
<td>268</td>
</tr>
<tr>
<td></td>
<td>8.0</td>
<td>-</td>
<td>29</td>
<td></td>
<td>15.9</td>
<td>-</td>
<td>315</td>
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<td></td>
<td>4.0</td>
<td>-</td>
<td>10</td>
<td></td>
<td>4.0</td>
<td>-</td>
<td>141</td>
</tr>
<tr>
<td>4</td>
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<td>56</td>
<td>5</td>
<td>8</td>
<td>12.2</td>
<td>472</td>
<td>174</td>
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<tr>
<td></td>
<td>1.5</td>
<td>63</td>
<td>5</td>
<td></td>
<td>15.4</td>
<td>1366</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>125</td>
<td>6</td>
<td></td>
<td>5.2</td>
<td>-</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>167</td>
<td>10</td>
<td></td>
<td>8.2</td>
<td>-</td>
<td>11</td>
</tr>
</tbody>
</table>

**TABLE 02 - Parameters $K_n$ and $K_t$ obtained in Laboratory Direct Shear Tests**

**CNS.:** Tests NO 1 to 5 = No rock to rock contact
Tests NO 6 to 8 = Rock to rock contact
* Tests "in natura" (1st cycle)

8 PLATES BEARING TESTS

4 "in situ" tests were performed, three of which in vertical planes, corresponding to the trench walls, and one in the horizontal plane, corresponding to the trench bottom. The bearing tests were done over a circular plate of 90cm in diameter. The load was applied through two hydraulic jacks, reacting against two anchors consisting of 12 cordages with 1/2" in diameter, anchored from 9m to 15m deep. The rock mass inner displacements were measured through a multiple rod extensometer, anchored in varied depths. Every anchorage location was decided considering drill rock cores performed in the middle of areas to be loaded, in a 5cm, 15cm and 27cm medium depth from the surface. Loading was performed in cycles, with a maximum load of 5, 10, 15, 25, and 50 kg/cm², with a loading of 30 kg/cm²/h. The maximum load of every cycle was kept constant until stabilizing the displacements.

In every load or unload stage, readings of displacements were followed by deflectometers with up to 0.001mm readings. Before beginning tests, a reference system check with simultaneous readings of the unloaded system temperatures and deformations was performed, during nearly 24 hours. It was found that temperature deeply affects the reference system, as observed in a period of time comprised between 6:00 and 12:00 p.m., with oscillation of 270 to 300. Because of these results, tests were only performed at night, between 20:00 and 6:00 o'clock.

The deformability modulus values were estimated based on the elasticity theory, according with formula recommended by the International Society for Rock Mechanics (ISR M).

The table below presents the average values for every test, in kg/cm², with the respective deformability modulus standard deviations, as obtained after the stabilization of displacements, and measured either from the starting point of the test or the starting point of every load cycle.

<table>
<thead>
<tr>
<th>Tests Location</th>
<th>From the Starting point of test</th>
<th>From the Starting point of load cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench bottom</td>
<td>12049 ± 586</td>
<td>15575 ± 2221</td>
</tr>
<tr>
<td>Left lateral wall</td>
<td>50642 ± 18186</td>
<td>80521 ± 20325</td>
</tr>
<tr>
<td>Right lateral wall</td>
<td>28835 ± 9315</td>
<td>41896 ± 6434</td>
</tr>
<tr>
<td>Frontal wall</td>
<td>33790 ± 15357</td>
<td>50026 ± 28572</td>
</tr>
</tbody>
</table>
In the estimate of deformability modulus average values, results with an equal to or lower than 0.02mm deformity were expurgated since even between 20:00 and 6:00 o'clock the temperature change arose deformations of that order. Most of the expurgated values corresponded to the 270cm deep anchoring or the 1st load cycle, which pressure on the plate was of only 5 kg/cm². The trench bottom test considered data of 5cm deep anchoring, the reason for which this test presented the least deformability modulus value, since it corresponds only to the zone most affected by the excavations.

In an attempt of correlating the deformability modulus values with the maximum normal stress applied in the test, the secant modulus values in every stress cycle were grouped, for the four performed tests. Figure 05 presents, for every test, modulus average values of every cycle resulting from the normal stress applied. The same figure shows, as an illustration, average values of deformability modulus obtained through plate bearing tests, performed in Itaparica Dam foundations, a work which is being developed in the same river, nearly 80km upstream Xingó. It can be noticed that the average values found in the two places belong to the same magnitude order. Values obtained in test nº 1 (Itaparica) correspond to a less fractured and less altered rock mass. As expected, it can be observed that for greater normal stresses, result dispersion is lower, considering that, for greater stresses, the load application system and measure system interferences are lower, due to greater deformations.

Figure 05 - Deformability Modulus Versus Normal Stress

9 DEFORMABILITY TESTS WITH DIATROMETERS

A total of 38 dilatometric tests was performed, allotted in 11 boreholes, and in a variety of depths. The selected borings corresponded to the Power Intake and Power House, Deviation Tunnels Entrance and T-7 Trench locations, where plate loading tests were performed. This procedure aimed first to establish a correlation between deformability modulus obtained through plate bearing and dilatometer tests, and to extend that correlation to other mass zones. A D-003 dilatometer from INESC of Lisbon was the equipment used, which allowed to simultaneously obtain deformation readings in four directions, at 45° from one another. The basic methodology adopted in the tests consisted of an initial loading with increases from 2 kg/cm² up to 10 kg/cm² and, starting from this point, with stress increases of 5.0 kg/cm² and unloading stabilized in 10 kg/cm², until maximum stresses of 20, 35 and 50 kg/cm² were reached.

Reading precision of hole wall displacements is 3% of the read value ± 1 μm. When the measured displacements were of small magnitude, nearing the mentioned precision limit, values were expurgated, not suitting the respective modulus calculations.

2178
The table below presents the average values of deformability modulus in kg/cm², obtained for every loading cycle and every zone of the tested mass.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>1st cycle (10 kg/cm²)</th>
<th>2nd cycle (20 kg/cm²)</th>
<th>3rd cycle (35 kg/cm²)</th>
<th>4th cycle (50 kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-7 Trench</td>
<td>28,200</td>
<td>38,000</td>
<td>45,000</td>
<td>52,200</td>
</tr>
<tr>
<td>Tunnel Entrance</td>
<td>25,900</td>
<td>46,600</td>
<td>62,500</td>
<td>70,500</td>
</tr>
<tr>
<td>Power Intake and Power House</td>
<td>21,700</td>
<td>34,000</td>
<td>56,300</td>
<td>88,300</td>
</tr>
<tr>
<td>Global</td>
<td>25,300</td>
<td>36,800</td>
<td>50,500</td>
<td>67,000</td>
</tr>
</tbody>
</table>

In regard to the work scale, the rock mass appears as migmatitic, showing a great deal of lithologic types and occurrence forms. This characteristic makes insignificant a differential zoning of the rock deformability in terms of lithology. On the other hand, alteration zones also present a whole erratic distribution, what turns impracticable the mass zoning with different weathering degrees.

Thus, the distribution of tests did not allow a lithologic or weathering criteria, for which reason tests were inevitably addressed to the rock mass fractured zones, with inclined or subvertical fractures and with moderately altered or pratically Sound rock of these zones. This way, tests results represented an intermediate deformability range of the mass, with upper and lower deformability peaks corresponding to the non-fractured Sound rock and the extremely altered one, respectively.

Deformability modulus results obtained with dilatometers presented the same magnitude order to the ones found in plate bearing tests, as shown in figure 05.

10 GRANULAR MATERIAL DEFORMABILITY

4 loading plate tests with granular material were performed. These tests aimed to obtain deformability parameters of the materials which will form the dam massif. The tests were performed applying a load over a circular plate 26.5 cm in diameter, placed on the surface of the material to be tested, this one already duly compacted inside a steel cylinder with an 88 cm inner diameter, and 90 cm high. Settlements were measured on the plate surface and at 11 cm below it, by means of a tassometer.

Two dry tests were performed, one with a 2.00 ton/m³ density, and another with a 2.28 ton/m³, and two saturated tests, with the same already tested samples.

The grain size distribution of the material tested is shown below:

<table>
<thead>
<tr>
<th>Sieve</th>
<th>2&quot;</th>
<th>1&quot;</th>
<th>3/8&quot;</th>
<th>No 4</th>
<th>No 40</th>
<th>Retained accumulated %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>12</td>
<td>65</td>
<td>80</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

Loading was applied in 8 load stages with maximum stresses varying from 0.2 kg/cm² to 25.6 kg/cm², in a ratio 2 of geometric progression. The load application speed was of 0.2 kg/cm² up to the 3.2 kg/cm² stress, and of 0.4 kg/cm² starting from this stage up to the maximum stress of 25.6 kg/cm². All load stages were kept until the stabilization of displacements.

After reaching stabilization in the maximum stress, the test was unloaded in 3 stress stages, with 5.4, 1.6 and 0 kg/cm². The saturated tests were performed with the same sample, by carrying out a new load, identical with that of the dry sample, flooded at the stage of 0.4 kg/cm².

The determination of deformability modulus was made in accordance with Poulos and Davis', indications (1974), assuming that Poisson's ratio is 0.35. Average values of deformability modulus are shown in the table below:

<table>
<thead>
<tr>
<th>TEST</th>
<th>DENSITY (ton/m³)</th>
<th>SATURATION</th>
<th>AVERAGE DEFORMABILITY MODULUS (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.0</td>
<td>no</td>
<td>474</td>
</tr>
<tr>
<td>2</td>
<td>2.28</td>
<td>no</td>
<td>375</td>
</tr>
<tr>
<td>3</td>
<td>2.0</td>
<td>yes</td>
<td>1,865</td>
</tr>
<tr>
<td>1</td>
<td>2.0</td>
<td>no</td>
<td>469</td>
</tr>
<tr>
<td>2</td>
<td>2.26</td>
<td>no</td>
<td>600</td>
</tr>
<tr>
<td>3</td>
<td>2.0</td>
<td>yes</td>
<td>1,309</td>
</tr>
</tbody>
</table>
Results of test n° 4 are not shown, since they presented an expansion in the sample, under stresses stages of 0.8 and 1.6 kg/cm², and very small and irregular deformations under greater stresses.

II CONCLUSION

In spite of the investments spent in an intense program of geomechanical investigations, results allow to carry out the projects under safer technical bases, leading to more economical solutions, mainly in regard to the excavations slopes, spacing of deviation tunnels and stability of concrete structures. As to the mass deformability, project considerations, prior to the investigations, pointed to values far superior to those ones obtained in the tests performed.

ACKNOWLEDGEMENTS

The authors are grateful to Companhia Hidro Elétrica do São Francisco for easing the publication of this paper.

They also thank Creuza Santos de Almeida and Alfred Constantin Barza for the translation, and Rosa Amara de Andrade Mazula for typing the text.

REFERENCES


Engineering-geological evaluation of the Late Eocene clay rocks relative to civil engineering

Aspects de l’ingénierie géologique dans l’évaluation des roches argileuses de l’Eocène inférieur en relation avec le génie civil

G.I. Chokhonelidze, Georgian Polytechnical Institute, Tbilisi, USSR
E.A. Djavakhishvili & G.S. Gadelia, Sector of Hydrogeology & Engineering Geology, Academy of Sciences of the Georgian SSR, Tbilisi, USSR

ABSTRACT: The paper presents engineering-geological characteristic of fresh and weathered Upper Eocene sandy-clay rock varieties rather common within the Alpine orogen just with the example of Tbilisi, the capital of the Georgian SSR. Fresh clay rock varieties are notable for rather high strength indices, though shortly after rock stripping the latter decreases abruptly. Thus, specific coherence becomes nearly one tenth as great as its initial value. All these must be borne in mind at laying foundations within such clay rocks.

RESUME: On donne une appréciation du point de vue de la géologie technique, des variétés de roches sable-argileuses nouvelles et altérées de l’Eocène supérieur lesquelles sont très répandues dans la zone de l’orogène alpin, sur l’exemple de la capitale de la RSS de Géorgie, Tbilissi. Les variétés nouvelles de roches argileuses de l’Eocène se caractérisent par des indices de résistance très élevés, lesquels après le découpage de la fouille dans un très court délai s’abaissent d’une manière brusque. Le coefficient de cohésion s’abaisse plus que de 10 fois. Il faut tenir compte de cette circonstance lors de l’exécution des travaux de fondation.

Sandy-clay deposits of the Late Eocene are rather common within the Alpine orogen zone, composing low-mountain and piedmont relief forms, wherein planning and national-economic development of lands proved to progress on a large scale.

Here we may cite the example of Tbilisi, the capital of the Georgian SSR, the major part of which is erected upon Late Eocene rocks. The latter are represented by random interstratification of sandstones, siltstones, argillites, and subargillites, the clay varieties being the weakest ones. Band of the latters may be as thick as 5 to 6 metres with thin layers of sandstones.

Subargillites are of dark grey colour and of hydromica-montmorillonite type with thin-laminated texture and aleuropic and randomly pelitic structure. The parenchyma is composed by clay mineral flakes with uniformly dispersed rounded pyritic secretions, their D ranging from 0.002 to 0.008 mm. Dark grey, nearly black colour is due to the presence of charred plant tissue. Humus amounts to per cent as determined by Tjuxin. The clay mass contains nearly 1.6 per cent of finely dispersed gypsum. As soon as the damp sample dries out finest extracts of sulfate salts get effloresced upon its surface. Absorption capacity of subargillites amounts to 44 milligram-equivalents. Table 1 lists some physical and mechanical properties of subargillites.

The above subargillites are notable for rapid cracking. As soon as a foundation pit is excavated subargillites shortly get weathered, lose density and crack. In a few days exposure to air subargillite piles transform into sliced debris. This process is favoured by increased concentration of readily soluble sulfate salts within the pellicular water and crystallization of these salts at rapid evaporation of the bound water. Thus all the above provide for fissuring. Argillites are of dark grey colour and present a hydromica-sericite type with minor content of montmorillonite and faited, partially thinly lamellar microstructure, pelitic and aleuropic structure. The parenchyma involves rather dispersed eucryptocrystalline calcite and charred plant remnants. In the open air they split to small oblong fragments poorly getting soaked in water.

As for sandstones they are fine-, medium- and coarse-grained, inequigranular
| Bocene subar- | Rock density | Maximum | Plasticity | Swelling | Rock shift | Rock shift resistance |
| gillites and | in grams | molecular and its | capacity | capacity, | resistance | after 3 months' exposure |
| rocks of zo- | per square | moisture limits | per cent | per cent | per cent | to air |
| mules with | centi- | | | | | |
| their weathe- | metre | | | | | |
| ring crust | | | | | | |
| Unweathered | 2,3-2,4 | 29 | 20\(\frac{56}{30}\) | 12 - 0,675 | 34* | 0,3* | 12* | 0,025 |
| cryptojointy | across | | | | | | |
| and clumpy | stratification | | | | | | |
| zonules | 3 - along | | | | | | |
| | | | | | | | |
| Rock debris | 1,9-2,0 | 23 | 15\(\frac{45}{30}\) | 0,2 | 18** | 0,05** |
| - oxidized | | | | | | | |
| zonules | | | | | | | |

* Estimated in situ

** Laboratory estimations
and of graywacke and arkose type on chloritic-clay and carbonate-siliceous cement. At their weathering we come across secondary filmy cement with hydrous ferric oxide.

Crush resistance of the graywacke sandstones along stratification is about 50 MPa, while that across stratification is as great as 68 MPa. If these rocks get soaked this number nearly halves, therefore these rocks are classified as softening ones.

Ultimate crush resistance for the arkoses amounts to 44 MPa, while the softening factor is but 0.34.

The two types of sandstones are notable for their bad jointing, which is well defined within the weathering crust with clumpy weathering zonules.

As for subargillites they belong to differentiated average rocks and thus readily fail and get weathered.

Herein prevails physical weathering just with elements of chemical erosion. Weathering crust of the above rock complex is 7 to 15 metres thick.

Physical weathering results in discontinuous or discrete environment with unequally broken weathering profile involving four weathering zonules:
1. That of fine crushing involving clay-sandy rocks.
2. Broken rock zonule composed of coarse and fine subargillite debris, inter-debris hollows are commonly filled with clay-sandy mass, while surface of the debris is often covered by selvedges.
3. En bloc or clumpy zonule resembles a "knock-down" rock, composed by well piled non-uniform platy blocks. Joints in-between the blocks may be both gaping and partly filled with clay-sandy matter.
4. Just below it we come across the cryptojointy zonule on the whole similar to the bedrocks and dissected by thin hairline cracks to large blocks.

Chemical erosion proceeds within all the above zonules, but it is not so much obvious everywhere. Thus rocks of the cryptojointy and partially those of the clumpy zonules resemble non-decayed rock varieties. They are of dark-grey colour and rather compact, and still sulphate weathering proceeding within them results in hydrolysis of pyrite and generation of sulphuric acid. The latter dissolves minerals, this making for removal of readily soluble sulphate salts (such as smithrite, etc), which are partially dissolved in the pellicular water.

As for clumpy zonule herein weathering processes are somewhat more active being mainly confined to the block surfaces. The latters are notably altered, whereas the internal part of blocks is not so much affected by weathering agents.

Rock debris zonule suffers intensive intrusion of weathering agents. As a result organics and ferriferous minerals get fully oxidized, rock debris becomes coloured chocolate due to ferric hydrite content, and at last gypsum undergoes profuse crystallization.

In the zonule of fine crushing within the arid climatic zone peculiar for the environs of Tbilisi rapid evaporation favours accumulation of readily soluble minerals and increase of their concentration. Gypsum and jarosite are rather prominent in this zonule.

Thus it may be inferred that weathering of subargillites of selective oxidation-accumulation type provides formation of non-uniform discrete medium and partly alters their composition. The latter in its turn results in anisotropic medium with somewhat decreased soil strength indices. Some features of rocks from various zonules are listed in Table 1.

From the engineering-geological point of view the above clay rocks deserve special attention. Under natural mode of occurrence the above rocks are characterized by high strength indices, but when weathered they don't withstand high and steep slopes, cover with vertical and not-unidirectional joints, lose stability, subside and finally slide. These rocks at exposure of the clumpy and cryptojointy weathering zonules easily crack, monoliths shortly become jointed, disintegrate into small particles, their structure is disturbed, all these hampering accomplishment of laboratory tests. Therefore, to acquire reliable strength and deformation indices one should conduct just in situ tests. As for shift resistance it was determined by the bulging method.

Soils of the clumpy zonule were subjected to bulging. Height of scarp was as great as 40 cm because it exceeded the size of the greatest soil inclusions nearly 5 times. Width of the scarp amounted to 120 cm. Thus, scarp height-width ratio was 1/3. There was a 15-20 centimetres' wide trench on either side of the collapsing body separating the latter from the massif.

A double screw jack attached at 1/3 from the bottom of the trench served to exert force on the wall of the body. Dynamotometers were fixed at the heads of the
jack to measure the exerted pressure. Dynamometers helped to estimate P-max.,
that is pressure, causing soil bulging, and P-min. — that is pressure, resulting
in displacement of the failing mass. In this case soil coherence is zero. Dis-
placement is considered to be completed if the mass has undergone shifting by
10 per cent.
To determine the rock shift resistance the bulging mass was dissected to
prisms, then we estimated total weight of the mass along with that of the ele-
mental prisms.
Thus we should cite the example of clay rocks rather common in the environs
of Tbilisi. They are represented by thick (5 to 6 metres) band of subargillites
with subjacent argillites. As it was quoted above shearing strength of soils
was estimated inside the dug holes by bulging method with the help of a jack.
Tests indicated that the angle of internal friction of the unweathered argilli-
tes was as great as 34° with specific coherence being about 0.5 MPa. To reveal
weathering effect upon alteration of rock strength indices tests were repeated
just in three months in the same dug holes which had been left exposed to
atmosphere agents. Now angle of internal friction was nearly a third of the
initial value (12°) while specific coherence amounted to 0.025 MPa.
Now, when characterizing the rocks of subargillite weathering crust
zonules as basements for structures it should be observed that the rocks of the
cryptojoints and partly those of the clumpy zones prove to be the strongest
ones. And still one should disapprove of their bad disintegration taking place
shortly after their stripping. Thus in 3 months' exposure to air specific co-
herence of these rocks decreased 10 times and the angle of internal friction
became a third of the initial value. Then, while the rocks of the oxydized
rock debris zonule are characterized by somewhat worse strength indices (angle
of internal friction = 18°, specific coherence = 0.05 MPa), still their hygro-
scopic properties as against those of the rocks of cryptojoints and clumpy
zonules are much more favourable. It is due to the fact that their swelling
capacity does not exceed 0.2 per cent, the plasticity limits are less, and,
furthermore, when stripped they don't disintegrate. All the above allows
conclusion that at excavation works we should leave a thin layer of oxydized-
rock debris zonule, which might perform a role of protective cover for sub-
jacent easily disintegrating rocks.
So it may be inferred that the Upper Eocene clay rocks are rather suscep-
tible to exposure to the atmosphere, therefore it is quite impermissible to
leave foundation pit bare for a long time. The foundation should be laid
immediately after excavation works. To get reliable rock strength indices all
the tests should be conducted shortly after exposure of the foundation.
The creep model if the intercalated clay layers and the change of their microstructure during creep

Le modèle 'creep' (glissement) des couches d'argile intercalées, et le changement de leur microstructure pendant le procès

Xiao Shufang, Wang Xianfeng, Cheng Zufeng & Nie Lai, Changchun College of Geology, Changchun, Peoples' Republic of China

Abstract: In this paper, the behavior and the experimental equations of creep of intercalated clay layers distributed in the Permian sandstones-shale system have been studied. The natural microstructure and, in particular, the changes of microstructure of intercalated clay layers during different stages of creep have been examined by electron scanning microscope. The model which describes the whole process of creep is given. This model would make a contribution to a better understanding of the creep mechanism and predicting the long-term behavior of creep by using the short-term testing results.

Résumé: Dans cet papier, le caractère intriqué et les équations expérimentales du cheminement des teneurs argiles intercalées dans le système du strate grès-argileux ont été étudié. La microstructure naturelle et en particulier, le changement de la microstructure de tel intercalation durant courant différents du cheminement a été examiné par microscope électronique. Le modèle difinissant ce procédé entier du cheminement est rendu. Celui modèle contribuera à meilleur comprendre du mécanisme du cheminement et prédire le trait du cheminement de longue durée par employer resultates des expériences à court terme.

The intercalated clay layer is a weak intermediate clay layer with high water content, which was formed from the thin soft clayrocks crushed by bedding slip and weathered by the physical-chemical process of groundwater. The existence of intercalated clay layers brings a serious threat to the stability of the slope or the foundation of dam.

The special genesis of intercalated clay layers makes their microstructure and mechanical properties differ from natural clay. Because of the difficulty in cutting the undisturbed samples, so far the study on this subject is insufficient.

We have cut down a number of undisturbed samples with success from the sandstones-shale system by means of the cutting machine which was designed by ourselves, conducted the drained shear creep test to study the creep behavior and equation of them and, in the meantime, examined the change of the microstructure during creep by the electron scanning microscope so as to make a series of typical pictures of microstructure which are corresponding to every stages of creep.

1 THE CREEP CURVES AND EXPERIMENTAL EQUATIONS OF INTERCALATED CLAY LAYERS

1.1 The creep curves

After analyzing 10 sets of testing curves at different stress level, we can sum them up as three types (Fig. 1)

1. The stable creep curve: It is the typical curve while the stress level is low.
2. Type A curve: It consists of four section, i.e. the instantaneous strain, the primary creep, the steady state creep and the accelerated creep. The strain rate of steady state creep is the minimum which is denoted by the symbol $\gamma$, and the time at the beginning and terminal of the steady state creep are expressed respectively as $t_0$ and $t_1$. While the intercalated clay layer is quite thin, its creep curve appears to be the type A.
3. Type B curve: There is no steady state creep in the whole process of creep, the primary creep is followed directly by accelerated creep. The plot turns from concave-downward to concave-upward. The strain rate at the turning point is the minimum. It is the type of curve while the clay layer is comparatively thick.

1.2 The experimental equations

It is obvious that the total strain is the sum of the strain of each stage, i.e.
\[ \gamma = \gamma_s + \gamma_i + \gamma_a \quad \text{(for Type A)} \]
\[ \gamma = \gamma_s + \gamma_i + \gamma_a + \gamma_z \quad \text{(for Type B)} \]

Where

- \( \gamma_s \) = the instantaneous strain
- \( \gamma_i \) = the primary creep
- \( \gamma_z \) = the steady state creep
- \( \gamma_a \) = the accelerated creep

The curve fitting expressions for each stage of creep can be obtained by regression analysis.

1. The primary creep \( \gamma_i \)
   
   For Type A, the curve fitting expressions are power laws of the form
   
   \( \gamma_i = at^n \)

   Where

   - \( t \) = time
   - \( n \) = constant, less than 1
   - \( a \) = constant

   For Type B, the strain-time behavior can be approximated by the following
   
   \( \gamma_i = [1 - \exp (-bt)] \)

2. The steady state creep

   For Type A, the creep could be represented by the equation
   
   \( \gamma_z = \gamma_s + \gamma_i + \gamma_a \)

3. The accelerated creep

   So far a number of empirical equations have been developed to express the primary and steady state creep, but no simple equation has been found for accelerated creep. By applying the linearization method suggested by D. Varnez, we examined the simple relations between \( \gamma, t, \gamma, t \), and their logarithms, exponential, and powers to see if they were linear. We found that the reciprocal rate-time \((1 - \gamma/\dot{\gamma})\) plot approximate to linear in accelerated creep stage. It means that the famous Saito relation is fit, hence we can express the result as follow

   \( \dot{\gamma} = C / (t - t_f) \)

   Where

   - \( \dot{\gamma} \) = shear strain rate during accelerated creep
   - \( C \) = constant
   - \( t_f \) = the time to failure, it can be estimated by extending the line to the \( t \)-axis (Fig. 2)

---

Fig. I. The creep curve of intercalated clay layer

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Integration of these equations yields

\[ \gamma = \gamma_p + \sigma t \ln \left( \frac{t_{max}}{t} \right) \]

Where the constant of integration C could be determined from an observation that \( \gamma = \gamma_p \) at \( t = t_{max} \).

Finally the strain-time relationship of whole process of creep was expressed in the form.

For Type A

\[ \gamma = \begin{cases} \gamma_p + \sigma t \\ \gamma_p + \sigma \left( t - t_{min} \right) \\ \gamma_{min} + C \ln \left( \frac{t_{min}}{t} \right) \end{cases} \]

For Type B

\[ \gamma = \begin{cases} \gamma_p + \sigma (1 - \exp(-bt)) \\ \gamma_p + C \ln \left( \frac{t_{min}}{t} \right) \end{cases} \]

Where the parameters \( a, b, c, D \) are constant when a constant shear stress is applied for any time. They are given in Table 1.

Depending on the data in Table 1, we can make a plot of shear stress against the \( \ln t \), and express the result on the form \( \gamma = 0.0352 - 0.047 \ln t \).

In order to explain these experimental creep equation, we have observed the natural microstructure and its change at different creep stages.
Table 1. The parameters of experimental creep equation

<table>
<thead>
<tr>
<th>Sample</th>
<th>Primary creep</th>
<th>Steady state creep</th>
<th>Accelerated creep</th>
</tr>
</thead>
<tbody>
<tr>
<td>(MPa)</td>
<td>min</td>
<td>min</td>
<td>h</td>
</tr>
<tr>
<td>1</td>
<td>0.0263</td>
<td>1200</td>
<td>0.0131</td>
</tr>
<tr>
<td>2</td>
<td>0.0263</td>
<td>78</td>
<td>0.026</td>
</tr>
<tr>
<td>3</td>
<td>0.0263</td>
<td>98</td>
<td>0.026</td>
</tr>
<tr>
<td>4</td>
<td>0.0310</td>
<td>70</td>
<td>1.17</td>
</tr>
<tr>
<td>5</td>
<td>0.0310</td>
<td>15</td>
<td>1.70</td>
</tr>
</tbody>
</table>

The normal stress \(G = 0.05\) MPa.

2.1 The natural microstructure of intercalated clay layer and its change during creep

After taking the microstructure picture from the top to the bottom of the layer by means of electron scanning microscope, we found that the natural microstructure of intercalated clay layer is characterized by zonation. The whole layer could be divided into three zones:

1. The top contact zone. It is the zone which contact with the rock wall. The microstructure of this zone is unoriented and loose. The water content is high.

2. Oriented zone, this is a very thin zone (50-500\(\mu\)m). The aggregates of clay particles are arranged as scaly and imbricate.

3. Unoriented zone, it is relatively thick and the arrangement of the aggregates is unoriented. The microstructure is hexagonal structure having higher strength bond. Moreover, we found there were some cracks and partial oriented arrangement in this zone.

Some microstructure character of each zone is shown in Table 2.

Table 2. The zonation of microstructure of intercalated clay layer

<table>
<thead>
<tr>
<th>Zone</th>
<th>Thickness ((\mu)m)</th>
<th>The contact of aggregate</th>
<th>The degree of orientation</th>
<th>The degree of compaction</th>
<th>Character</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top contact zone</td>
<td>800</td>
<td>Edge-edge (major)</td>
<td>No</td>
<td>Very loose</td>
<td>Weak bond</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Edge-face (minor)</td>
<td></td>
<td>Porosity 50%</td>
<td>No microcrack</td>
</tr>
<tr>
<td>Oriented Zone</td>
<td>400</td>
<td>Face-face</td>
<td>Oriented aggregate 70%</td>
<td>Relatively compactive</td>
<td>Scaly and Imbricate</td>
</tr>
<tr>
<td>Unoriented Zone</td>
<td>5200</td>
<td>Edge-face</td>
<td>Very few</td>
<td>Compaction</td>
<td>Stronger Bond</td>
</tr>
</tbody>
</table>

The character of this zonation make it easy to shear along the top contact zone. But under the condition of the higher stress level or, the quite thick layer, the shear will extend from microcracks in the unoriented zone.

2.2 The microstructure change during creep

By comparing the microstructure pictures we found that

1. in the stage of primary creep, the shear strain rate decreased with time, it is because that the aggregated flex, close up, inlay, push and squeeze each other with the lapse of time. The aggregates close up each other can be verified by compacting phenomenon during shear creep. (Fig. 3)

The picture 1 shows the phenomenon of push and squeeze of aggregates. From this picture, we can see some aggregates are squeezed into the pores. The edge-edge and face-face contacts start to turn into face-face contact gradually.

The picture 2 shows phenomenon of inlay and interlock of aggregates. In picture 3, we can see the flex of domain of clay particles. All of these led up to strain-hardening. On the other hand, the increasing orientation of microstructure would make the thickness of the absorbed water film increasing and the strength of bond decrease, these led up to strain-softening. The strain-hardening and the strain-softening exist simultaneously in the whole process of creep. But in the stage of primary creep, the strain-hardening is major.
2 In the stage of steady state creep, the strain rate is constant and the viscosity coefficient is constant too. It means that the strain-hardening must reach equilibrium with strain softening. But it is not easy to reach such a balance unless the shearing creep progressed along the thin top contact zone, because the disturbed structure range is relatively thin. It is easier to keep the balance between strain-hardening and strain-softening for quite a long time. In such a case, the strain-time curve appears as Type A. On the other hand, in the case of higher stress level or thicker layer the shearing progressed through whole clay layer, both strain-hardening and strain-softening were too complex to reach a steady stage. The strain-rate curve appeared as Type B, i.e. no steady state creep stage.

3 The change of microstructure in accelerated creep stage was the characteristic of the increasing the degree of orientation. The strain-softening was major and, with the development of orientation the strain rate increased rapidly, at the end, to failure. The microstructure of the shearing surface arranged as scaly and imbricate.

4 The recovery of the microstructure when we put the failure sample into the water for several days and observed the change of microstructure of the shearing surface, we found that after 10 days soaked in water, most of oriented microstructure in the shearing zone was recovered.

3 THE CREEP MODEL OF INTERCALATED CLAY LAYER

In order to describe the creep behavior of intercalated clay layer, we tried to apply a complex creep model. The construction of the model was based on the following recognition.

3.1 When the shear stress is less than the long-term shearing strength, the shear strain at \( t = 0 \) is instantaneous strain \( \gamma_0 \) and when \( t \to \infty \), the shear strain tends to a constant and the sample would not fail. The creep behavior can be described by Kelvin model which gives the creep as

\[
\gamma = \frac{t}{G_1} + \left( \frac{1}{G_2} \right) \cdot \left[ 1 - \exp \left( -G_1 t / \eta \right) \right]
\]

3.2 When shear stress is greater than \( \gamma_0 \), the steady state creep occurs.
By combining a Kelvin model and a Bingham model in series, we can obtain the complex model which can represent the primary and steady state creep stage. The equation is

\[
\gamma = \frac{t}{G_1} + \left( \frac{1}{G_2} \right) \cdot \left[ 1 - \exp \left( -G_1 t / \eta \right) \right] + \left( \frac{f - \Delta \gamma}{\eta_1} \right) \cdot t / \eta_1
\]

3.3 When strain reaches to \( \gamma_0 \), the steady-state creep will change into accelerated creep, the viscosity coefficient \( \eta \) will decrease with time.
A special dashpot having two swiveling holed pistons is used to describe the change process of viscosity \( \eta \). While the holes of two piston open onto each other, the \( \eta \) is minimum and the strain rate is corresponding to maximum. Conversely, swiveling piston until the hole of two piston stagger each other, then, \( \eta = \infty \) and \( \dot{\gamma} = 0 \) Combining the special dashpot and Kelvin-Bingham model in the way shown in Fig. 4.
Fig. 6 The model of the whole process of creep of intercalated clay layer

From Fig. 6, we can see that when such a model is subjected to a constant stress which is less than \( \gamma_{\infty} \), the strain will only cover the primary creep. If \( \gamma_{\infty} \) and \( \gamma_{\infty} \) are known, the strain produced will be the sum of the strain of Kelvin and Bingham model. If \( \gamma > \gamma_{\infty} \), the strain produced will be given by

\[ \gamma = \gamma_{\infty} + t \left( 1 - e^{-\gamma_{\infty}/\eta_s} \right) \]

Here \( \eta_s \) is not a constant. It is a function of the \( t \). Having the aid of the experimental equation, we can express \( \eta_s \) in the form

\[ \eta_s = \frac{tt}{(B - A \ln(t - t_0))} \]

The whole creep process of intercalated clay layer could be represented by the equation

\[ \gamma = \frac{t}{G_s} + \left( \frac{t}{G_s} \right) \left[ \left( -\frac{G_s}{\eta_s} t \right) \right] + \left( 1 - e^{-G_s/\eta_s} \right) \frac{t}{\eta_s} \]

\[ \gamma = \gamma_{\infty} + B - A \ln(t - t_0) \]

where \( \gamma_{\infty} \) is the shear strain at \( t = t_0 \), \( G_s, \eta_s, \eta_s, B, A \) are parameters being irrelevant to stress level and time. They can be calculated from several sets of testing curves by micro-computer.

The flow diagram is given as following page.

The values of \( \eta_s, \eta_s, A, B \) are given in Table 3.

![Table 3](image)

Table 3. The values of parameters \( \eta_s, A, B \)

<table>
<thead>
<tr>
<th></th>
<th>Group I</th>
<th>Group II</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma ) (MPa)</td>
<td>0.067</td>
<td>0.070</td>
</tr>
<tr>
<td>( G ) (MPa)</td>
<td>( 1.2 \times 10^7 )</td>
<td>( 1.7 \times 10^7 )</td>
</tr>
<tr>
<td>( B ) (MPa)</td>
<td>( 0.3 \times 10^9 )</td>
<td>( 2.5 \times 10^9 )</td>
</tr>
<tr>
<td>( A ) (1/( t_0 ))</td>
<td>0.13</td>
<td>0.0212</td>
</tr>
<tr>
<td>( B )</td>
<td>0.80</td>
<td>0.312</td>
</tr>
</tbody>
</table>

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The flow diagram

**LET** \( n, m \) = \( r, \infty \)

- \( n \) is the number of the points on the \( \gamma - t \) curve
- \( m \) is the number of the points on the \( \gamma - \gamma \) curve

**INPUT** \( t, r, y \)

**COMPUTE** \( \gamma \) values of \( n \)

- \( n = n + m \times 10^6 \)

**COMPUTE** \( R(t) \) \( i = 1, 2, \ldots, n \)

- \( R(t) \) is the by entering the assuming to creep equation

**COMPUTE** \( A(i) \) \( i = 1, 2, \ldots, m \)

- \( A(i) = \frac{\gamma}{\left( R_i - R_{i-1} \right)^2} \)

**COMPUTE** \( \gamma_m, \gamma_m \) is the minimum of \( A(i) \)

- \( \gamma_m \) is the corresponding to \( \gamma_m \)

**INPUT** \( \gamma, \gamma \)

- \( \gamma \) obtained from the rate of steady state creep

**INPUT** \( U, U, \ldots, U \)

- \( U, U, \ldots, U \) are \( n \) values of \( t \) in accelerated creep section

**INPUT** \( V, V, \ldots, V \)

- \( V, V, \ldots, V \) are \( n \) values of \( \gamma \) in accelerated creep section

**COMPUTE** \( A, C \)

\[
A = \left( \sum U_i \cdot V_i \right) - \left( \sum U_i \right) \left( \sum V_i \right)
\]

\[
C = \frac{\Pi \sum (U_i - A \cdot V_i)}{n}
\]

**PRINT** \( A, C \)

**COMPUTE** \( U_m \)

**COMPUTE** \( V_m \)

**COMPUTE** \( B \)

\[
V_m = \frac{\gamma}{\gamma} + \left( \frac{\gamma}{G} \right) \left[ \frac{1}{1 - \exp \left( -G \cdot t \gamma / \eta \right)} \right] + \frac{t \cdot \gamma}{\eta}
\]

\[
B = C - V_m
\]
In conclusion, it seems to be that the model is conducive to analysing the mechanism of creep and predicting the long term behavior of creep, however, for practical purposes it is also necessary to inquire further the relationship between the creep parameters and the index of microstructure change. This is a subject that we will deal with in the near future.

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Chemical stabilisation of dispersive loessial soils, Banks Peninsula, Canterbury, New Zealand

Stabilisation chimique de loess dispersifs de la Péninsule de Banks, Canterbury, Nouvelle Zélande

D.H. Bell, P.J. Glassey & M. D. Yetton, Geology Department, University of Canterbury-Christchurch, Christchurch, New Zealand

ABSTRACT: The loessial soils of Banks Peninsula display mass movement and tunnel-gully erosion features that are controlled by gross pedological layering, and by soil properties such as clay mineral dispersion and moisture-dependent strength changes. Chemical stabilisation techniques have been developed that render these potentially erodible soils both non-dispersive and volumetrically stable to repeated wetting and drying: satisfactory stabilisers include hydrated lime (Ca(OH)$_2$), quicklime (CaO), Portland cement and orthophosphoric acid (H$_3$PO$_4$). Stabilising chemicals are commonly applied by mixing and recomposition at optimum moisture content, with typical field uses including the lining of drains and the backfilling of service trenches: slurry methods have also been developed for the filling of pre-existing erosion cavities where soil strength is not a critical factor. Selected case histories illustrate field placement techniques, and stress the need for a sound engineering geological site model as the basis for any successful loess stabilisation programme.

RESUME: Les sols loessiques de la Péninsule de Banks montrent des glissements de terrains et des ravinements souterrains contrôlés par un litage pédologique grossier et par les caractéristiques de sol telles la dispersion des argiles et les variations de la teneur en eau qui y sont liées. Des techniques de stabilisation chimique ont été développées rendant ces sols potentiellement érodables, à la fois non dispersifs et volumétriquement stables aux actions répétées de l’humidification et de l’assèchement: ces stabilisants satisfaits incluent: de la chaux hydratée (Ca(OH)$_2$), de la chaux maigre (CaO), du ciment portland et le phosphore orthophosphorique (H$_3$PO$_4$). Les stabilisants chimiques sont généralement mélangés par mélange et recomposition à état hygrométrique optimum selon des techniques de terrain caractéristiques telles le revêtement de drains et le remblayage de tranchées de service: différentes boues on aussi été expérimentées pour le remblayage de cavités d’érosion préexistantes, où la résistance du sol n’est pas un facteur critique. Une sélection d’exemples illustre les différentes techniques appliquées sur le terrain et insiste sur la nécessité, pour le géologue ingénieur, d’un solide modèle de base pour tous les programmes de stabilisation de loess couronnés de succès.

1 INTRODUCTION

Both primary airfall ("in situ") loess and secondary ("reworked") loess-colluvium deposits blanket much of the eroded volcanic centres that form Banks Peninsula, Canterbury, New Zealand (Figs 1A & B). The loessial soils reach a maximum depth of about 15m, and display a variety of mass movement and erosion features (Fig 1C) that are related to gross pedological layering and to material properties such as clay mineral dispersion and moisture-dependent strength changes. Chemical stabilisation techniques have been developed to remedy loess erodibility problems, and these typically involve either mixing and soil recomposition at optimum moisture content or some form of slurry application. Hydrated lime (Ca(OH)$_2$) is the principal stabilising chemical used to date, although quick lime (CaO), Portland cement and orthophosphoric acid have all been successfully tested (Evans & Bell 1981; Bell 1982, 1983).

Our objectives in this paper are:

1. to briefly review the nature of loess instability on Banks Peninsula, especially in relation to urban development on the Port Hills to the west of Lyttelton Harbour (Fig 1B),
2. to outline previous research into chemical stabilisation of Banks Peninsula loess, and to discuss the results of recently completed laboratory studies of lime-stabilised soils (Glasssey 1986).
3. to describe field stabilisation techniques that have been developed for Banks Peninsula loess, including the application of lime-cement slurries for erosion cavity filling (Yetton 1986).

4. to demonstrate the critical importance of engineering geological site models to successful chemical stabilisation practice in erodible loess soils.

The recent research described here results from an ongoing investigation programme being conducted at the University of Canterbury, Christchurch, into the geotechnical characterisation of and remedial measures for the loess deposits of Banks Peninsula.

2 LOESS EROSION

2.1 Geologic and Geomorphic Setting

Banks Peninsula consists of three principal eruptive centres that were active in Miocene times (from about 11 to 6 million years ago - Weaver et al. 1985) and formed as offshore volcanic cones on which radial drainage patterns had become established by the commencement of the Pleistocene. Multiple glaciation in the Southern Alps resulted in outwash aggradation that progressively infilled the seaway between the offshore volcanic complex and the uplifted Triassic basement strata to the west, and some 300m of fluviolacustrine gravels underlie the Canterbury Plains (Fig 1B). In addition, a complex interfingering relationship between glacial outwash and interglacial estuarine muddy sediments occurs near the present coastline because of Pleistocene sea level fluctuations as great as 150m.

It is generally accepted (Griffiths 1973; Ives 1973; Trangmar 1976; Bell 1978) that the quartzofeldspathic loess deposits of Banks Peninsula originated from glacial grinding, and were transported by the dominant north-westerly winds; a minor contribution from periglacial sources is also likely, and a further source may have been the sediments exposed on the continental shelf to the east of Banks Peninsula during glacial low stands of sea level (Raeside 1964). Several paleosols have been recognised in the thicker loess sequences, although no satisfactory correlation with glacial episodes has yet been established (Ives 1973; Griffiths 1973): coarser calcareous and finer non-calcereous facies have also been recognised, apparently reflecting variation in vegetation cover since the end of the...
Pleistocene (Griffiths 1973). Loess distribution reflects the topographic relief on Banks Peninsula at the time of primary airfall deposition ("in situ" loess), and any subsequent reworking by slope processes during the Quaternary to form secondary deposits of loess-colluvium and mixed loess-volcanic colluvium (Bell & Trangmar, in prep): varying proportions of intermixed, weathered volcanic fragments reflect proximity to bedrock outcrops and slope morphology (Fig 1C), as well as changes in vegetation cover, especially during cold climate episodes.

2.2 Instability Problems and Remedial Options

Five regolith types and six associated forms of erosion have been recognised on Banks Peninsula (Table 1): the relative proportions of loessial and volcanic-derived components have been used to classify the regoliths, whilst slope processes involving both particle and mass movements have been grouped (Bell & Trangmar, in prep). Tunnel-gully erosion involves initial subsurface development of "pipe-like" tunnels above or below the fragipan (Figs 2A & B), and subsequent progressive enlargement and collapse to form open gullies. Slide-flow mass movements (Fig 2B) are likewise controlled by gross pedological layering (such as fragipan development) and by soil shear strength properties (for example, moisture-dependent strength changes in loess): soil creep can be regarded as a critical factor in slide-flow initiation, whilst earth and debris falls (Varnes 1978) develop most commonly on excavated slopes. Sheet erosion involves the downslope movement of soil particles as a consequence of overland flow, and rill gullies typically develop in dispersive or erodible loessial materials: wind erosion of fine-grained sediments is an extremely common phenomenon on bare ground, but is only considered to be of "nuisance" value on Banks Peninsula.

Engineering problems affecting residential areas, rural land and transportation routes on Banks Peninsula include high intensity rainstorm-generated rapid slope movements in all regolith types, as well as the more "insidious" (and slower) forms of surface and subsurface erosion in the loessial soils. Tunnel-gully development may progressively undermine house foundations or other engineering structures, whilst scurbs along service trenches (Table 1) may not become noticeable for months or even years after installation: in addition, there are often considerable difficulties in tracing subsurface erosion cavities (for example, beneath a major road), and this is an essential prerequisite to any remedial works. Control of surface and subsurface water flows, together with adequate drainage of retained cuts or fill batters, has long been recognised as the principal remedial option in erodible and/or dispersive loess, but this clearly may not be sufficient because the soil properties have not been changed. The use of chemical stabilisation techniques to render soils non-erodible (and non-dispersive) is now generally accepted, at least within the Christchurch urban area where property values can justify such additional expenditure on site remedial works.

3 LABORATORY GEOTECHNICAL DATA

3.1 Previous Research

The use of chemical stabilisation techniques to remedy engineering problems associated with dispersive soils is relatively well documented (see, for example, Ingles & Metcalf 1973; Winterkorn 1975; Sherard et al 1977). Both hydrated lime (Ca(OH)₂) and orthophosphoric acid (H₃PO₄) render Banks Peninsula loessial soils non-dispersive and erosion resistant at stabiliser additions as low as 1% by weight, whilst other property changes include a significant increase in the unimpered unconfined compressive strength and modification of the compacted dry density/ optimum water content relationships (Evans & Bell 1981). The addition of 2% by weight of hydrated lime to Port Hills loess has also been shown to effectively eliminate shrink/swell behaviour, and at the same time to increase the coefficient of permeability by about 10⁵ m²/s in specimens recompacted at optimum water content (Evans & Bell 1981). As yet unreported laboratory trials using quick lime (CaO) and Portland cement have achieved similar property changes in Banks Peninsula loess, whilst the use of gypsum (CaSO₄.2H₂O) is presently under investigation: calcium chloride (CaCl₂) addition was also found to render Port Hills loess non-dispersive, but the lower compacted dry density and strength properties did not justify field trials (Bell 1982). The research projects completed by Glassey (1986) and Yetton (1986) have provided significant new data on the chemical stabilisation of Banks Peninsula loessial soils, as well as necessitating modifications to some of the earlier conclusions regarding laboratory property changes: the following discussion is confined to selected geotechnical parameters that are altered by the addition of hydrated lime (Ca(OH)₂).

3.2 Soil Classification Parameters

Winterkorn (1975) reports that lime stabilisation of clayey soils is accompanied by a decrease in the percentage of clay-sized particles because of the effects of flocculation, and as this property change had not previously been investigated for Banks Peninsula loess,
<table>
<thead>
<tr>
<th>EROSION PROCESS</th>
<th>REGOLITH TYPE</th>
<th>GEOTECHNICAL CONTROLS</th>
<th>ENGINEERING PROBLEMS</th>
</tr>
</thead>
</table>
| Water—Gully    | In situ loess; loose-collopluvium | 1. Low intergranular cohesion  
2. Rapid slaking and dispersion  
3. Erodibility by moving water  
4. Low permeability layers controlling subsurface flow  
5. Shear probes cracking on dry aspects lacking continuous vegetation cover  
6. Natural and/or artificial sources of water (eg. seepage or leaking services) | 1. Structural damage due to ground subsidence above active tunnels  
2. Water and/or sediment discharge into streamways etc.  
3. Sectors along service trenches backfilled with erodible soil  
4. Landslide initiation by subsurface flow concentration into fills etc. |
| Slope—Mudflows (fall; slide; flow) | Loose-collopluvium; mixed collopluvium | 1. Steep natural or cut slopes (<25°)  
2. Seasonal or diurnal moisture variation in surface layers (eg. 1m deep)  
3. Impeded internal drainage  
4. Low permeability layers  
5. Shear failure along variable shear strength planes  
6. High intensity or prolonged rainfall | 1. Retractive (upwash) movement of landslides (minor undermining)  
2. Downslope debris accumulation (property damage etc.)  
3. Disruption of stream discharge systems  
4. Structural damage (eg. retaining wall collapse) |
| Soil—Creep     | In situ loess; loose-collopluvium; mixed collopluvium | 1. Seasonal or diurnal moisture variation in surface layers (eg. 1m deep)  
2. Impeded internal drainage  
3. Plastic behaviour of loess with increasing water content  
4. Profile layering with variable shear strength planes  
5. Stock compaction  
6. Slope angles <15° | 1. Precursor to slope—fall initiation under rainstorm conditions (eg. "surface" slides)  
2. Localised structural damage (rare) |
| Volcanic—Cohlopluvium | 1. Potential movement surface at bedrock contact  
2. Moisture−dependent strength changes in collopluvium | 1 to 2 as above |
| Sheet and Fill  | All types (but especially longitudinal variations) | 1. Frangible, weakly structured topsoil  
2. Summer desiccation  
3. Dispersion if loess content high  
4. Thin low permeability cover formed by sheet wash (→ increased runoff)  
5. Discontinuous or no vegetation cover  
6. Heavy rainfall on steep slopes (≥10°) | 1. Downtown sedimentation effects  
2. Gully gully development and deep (≥1m) scour  
3. Subsurface erosion and tunnel development (→ problems as above)  
4. Localised undermining of cut faces (→ soil or debris falls) |
| Wind           | All types (but especially longitudinal variations) | 1. Condensed to bareground or depleted vegetation cover  
2. Function of wind velocity, machinery disturbance, etc | 1. "Reversion" effects for green vegetated slopes  
2. No significant engineering problem |
|                |              | 1. Reestablishment of suitable vegetation cover  
2. Use of various "dune control" techniques |

Table 1. Erosion processes and remedial options for regoliths developed on Banks Peninsula.
A. DEEP TUNNEL - GULLY DEVELOPMENT MODEL

1. Discontinuous vegetation cover 
   deep cracking through fragipan (B2)
2. Tunnel formation in loess colluvium (C) below fragipan
3. Tunnel enlargement 
   collapse of fragipan
4. Complete roof collapse 
   renewed tunnelling in floor debris

Notes: 1. Typical depth to base of fragipan 1 to 1.5 m.
2. Lateral erosion and downcutting after Stage 4 may create open gullies up to 4 m deep and 10 m wide.

B. GENERALISED SLOPE INSTABILITY MODEL

Figure 2. Engineering geological site models for loess instability (after Evans & Bell 1986; Bell & Trangmar in prep.).

Glassey (1986) tested a bulk sample of loess-colluvium from Westmoreland Subdivision, Christchurch, where serious erosion problems had occurred during residential development. The laboratory procedure involved stabiliser mixing and Standard Proctor compaction at optimum water content, and was followed by 14 days of moist-curing in sealed bags at 20°C and 99% relative humidity prior to disaggregation by hand in distilled water with a deflocculant added. the sand fraction was determined by dry sieving, and silt and clay by hydrometer analysis using New Zealand Standard methods. At 1% by weight of stabiliser addition a reduction in both clay and sand percentage is evident, while the silt content has increased by 7% (Fig 3A): at 2.5% by weight of hydrated lime the sand percentage has increased, and continues to do so steadily up to 10% by weight of stabiliser increases (Fig 3A). At 5% by weight of lime addition there is a marked drop in clay percentage and this remains effectively unchanged at 11% up to 10% by weight of stabiliser (Fig 3A): whilst it is certainly possible that the test method used has influenced the percentages of sand, silt and clay determined for the lower lime percentages, we nevertheless can confirm the earlier observation (Winterton 1975) concerning a decrease in the clay fraction due to lime addition.

Although Winterton (1975) reports that a reduction in the plasticity index occurs with lime stabilisation and is usually accompanied by an immediate increase in the plastic limit. Atterberg limit tests carried out on the same 14-day moist-cured Westmoreland Subdivision samples (Glassey 1986) in fact show an initial increase in plasticity index from 7 to 13 (at 1% by weight of hydrated lime) which is due principally to an increase in the liquid limit (from 23 to 32); at higher lime percentages the plasticity index decreases only slightly (to 10 at 7.5% by weight of stabiliser), and the liquid limit behaves similarly (Fig 3B). The plastic limit does increase consistently (from 16 to 22), and the soil cannot be rolled into a 3mm thread at 10% by weight of hydrated lime addition (Fig 3B); the soil activity is maximised at 5% by weight of hydrated lime (Fig 3B) but remains low. Our results are in fact in agreement with data reported by Herrin & Mitchell (1981), and other
A. GRAIN SIZE DISTRIBUTION

Grain size (Weight %)
- sand (60-2000 μm)
- silt (2-60 μm)
- clay (<2 μm)

B. ATTERBERG LIMITS

Water Content (%)
- Soil not plastic (no $W_p$ or 10% Ca(OH)$_2$

C. STANDARD COMPACTION

Optimum Water Content (%)
- CMC
- $\rho_{max}

Maximum Dry Density (t/m$^3$)

D. UNIAXIAL SWELLING STRAIN

$\varepsilon_s$ (%)
- Field sample (in situ density)
- 14 day moist cured (recompacted)
- 7 day moist cured then 7 day wet/dry cycles (recompacted)
- 7 day moist cured then 7 day air-dried (recompacted)

Figure 3. Soil classification parameters and lime content (from Glassey 1986).

Research (for example, Clare & Cruclley 1957; Brandl 1981) confirms that the plasticity of low activity soils is increased by the addition of hydrated lime.

An increase in optimum water content and a corresponding decrease in maximum compacted dry density is well documented for lime-stabilised soils (Hervin & Mitchell 1961; Alexander et al 1972; Winterkorn 1975), and similar trends have also been reported for Banks Peninsula loess (Evans & Bell 1981). Testing of the Westmoreland bulk sample at lime percentages up to 10% (Glassey 1986) has shown an increase in the optimum water content from 13 to 17%, and a decrease in the maximum dry density by 0.18 t/m$^3$ under Standard Proctor conditions (Fig 3c). The changes in compaction characteristics are particularly pronounced at 1% by weight of stabiliser addition, reflecting both particle size modification and a decrease in water retention capability even with very low percentages of hydrated lime: similar changes were observed by Evans & Bell (1981) for samples from other parts of the Port Hills, and Yetton (1986) also reports similar trends for loess from Charteris Bay, Banks Peninsula.
A marked reduction in the shrink/swell behaviour of lime-stabilised soils is reported by Winterkorn (1975), and Evans & Bell (1981) determined reductions in uniaxial confined swelling strain values from 15-20% in untreated loess to less than 1% in materials stabilised with 2.5% or more by weight of hydrated lime. For Westmoreland samples cured for 14 days by various methods (Fig 3D) Glassy (1986) also observed a marked reduction in uniaxial confined swelling strain, but this was not maximised until 10% by weight of stabiliser addition: the data previously reported by Evans & Bell (1981) were for 28 days air-equilibrated loess samples, however, and we believe that differences in curing history (and specifically insufficient time for complete reaction product development) may explain these differences for low lime percentages. Swelling properties of Banks Peninsula loess are due principally to soil skeleton disruption by increased pore water pressures and a reduction in soil moisture suction on void saturation, the relatively small amount of smectite clays present being considered to have only minimal influence on shrink/swell behaviour (Miller 1971; Bell 1978). The effect of hydrated lime addition is both to increase void space within the soil skeleton and to provide intergranular bonding, thereby reducing or effectively eliminating any swelling potential: this is also important in terms of slaking behaviour, and Glassy (1986) has shown that lime-stabilised loess did not disaggregate significantly during severe 24 hour wet/dry cycles although the control (0% lime) collapsed immediately upon immersion for the first slaking cycle.

3.3 Erodibility and Dispersion

The pinhole test originally developed by Sherard et al (1976a) was modified by Evans (1977) to assess loess erodibility on Banks Peninsula, and Evans & Bell (1981) detail the currently used six fold classification (D1-2; HD4-1). It is clear, however, that the test method does not necessarily identify true dispersity, which is defined as clay mineral defloculation (Emerson 1967; Holmgren & Planagan 1977; Bell 1978, 1982), because both silts and fine sands may undergo collapse at the lowest test head (50mm). Scharf and Trangmar (1981) have demonstrated that a poor correlation exists between the pinhole test and both the porewater analysis and ESC dispersions methods (Sherard et al 1976b) when applied to loessial soils from the eastern South Island, whilst Goldsmith & Smith (1985) found clayey silts associated with tunnelling soils near Auckland (Fig 1A) to be classified as dispersive using the latter tests but non-dispersive (in fact HD1) by the pinhole test.

In testing loess soils from Banks Peninsula it has become standard practice in recent years to use both the pinhole test (Evans 1977) and the Crumb test (Emerson 1967) as measures of, respectively, erodibility and dispersivity: both parameters are therefore commonly quoted, and variants such as the Modified Crumb test (Sherard et al 1976b) have also been used (Yetton 1986). A modified form of the pinhole test classification has been proposed by Yetton (1986), and further evaluated by Glassy (1986); in this system test data are plotted as discharge against elapsed time for each head (Fig 4A & B), and soils are classified as erodible at a given head if "sustained erosion" continues for 3 or more minutes. A loess soil may thus be assigned an erodibility class such as E90 or NE, or an intermediate classification such as E180-390, and a dispersion category (for example, Emerson Crumb Class 1).

The effects of hydrated lime on the erodibility of Banks Peninsula loess were demonstrated by Evans & Bell (1981), with percentages as low as 1% by weight rendering the soil non-erodible (ND1) after 7 days' curing: similar results were obtained using orthophosphoric acid which, although acting initially as a dispersant, produces insoluble aluminium phosphate hydrates by reaction with the soil (Bell 1982). Glassy (1986) has shown clearly for his Westmoreland bulk sample (Fig 4A) that hydrated lime additions of between 1 and 10% by weight change an E180-1000/Emerson Class 2 untreated soil into one that is non-erodible (NE) and non-dispersive (Emerson Crumb Class 4): Yetton (1986) obtained similar results for his Ngaiho Lane loess samples (Fig 4B). The effects of hydrated lime on soil erodibility are well documented (see, for example, Machan et al 1977), and relate to the formation of insoluble hydrated calcium aluminate and silicate gels which provide longer-term intergranular bonding, and which are produced following the "immediate" flocculation of clay minerals: Evans & Bell (1981) also demonstrated that calcium carbonate formation is an important factor in rendering non-erodible the loessial soils of Banks Peninsula by hydrated lime addition.

3.4 Strength Characteristics

Any strength gain from lime addition is dependent on the soil properties, and on the conditions under which stabilisation is carried out (Merrin & Mitchell 1961; Neubauer & Thompson 1972; Brand 1981). Winterkorn (1975) reports increases in unconfined compressive strength of as much as 60 times (compared to the untreated values), whilst Evans & Bell (1981) found that 5% by weight of hydrated lime addition to Port Hills loess maximised strength gains at about 6 times. Other researchers (for example, Clare & Cruachley 1957) have
also demonstrated that there is an "optimum" lime addition beyond which strength is lost, and Brandl (1981) attributes this to the effects on the soil fabric of unreacted or partly reacted lime. Glassy (1986) has tested the Westmoreland loess-colluvium over a range of lime percentages (0, 1, 2.5, 5, 7.5 and 10% by weight) and under three different curing conditions: laboratory specimens were mixed with the stabilising chemical at optimum water content for that lime percentage, and compacted in accordance with the New Zealand standard (Proctor) method prior to curing for 14 days.

Curing history has a considerable influence on the stress-strain behaviour of loessial soils (Figs 5A, B & C). Samples moist-cured for 14 days at 20°C and 95% relative humidity show marked differences between the untreated controls and the lime-stabilised soils (Fig 5A): the latter behave as brittle materials with strains of less than 2% at peak stress, whilst the unstabilised but recompacted soils display plastic deformation typical of Port Hills loess in the moisture range 11 to 16% (Bell & Trangmar, in prep). Samples moist-cured for 7 days and then subjected to 24-hour wetting and drying cycles show peak stresses about one-half of those moist-cured for 14 days, reflecting the effects of alternating saturation and gravity drainage on the stabilised soils as well as testing in the soaked state (Fig 5B): strains at peak stress are, however, again less than 2% for the lime-stabilised material, although the untreated controls did not survive the first wetting cycle and disaggregated completely by breaking with some dispersion. Samples that were moist-cured for 7 days and then air-dried for 7 days to a water content of 2-3% show considerably higher peak stresses and corresponding strains of 1-2% (Fig 5C): of particular note, however, is the behaviour of the untreated control which has a failure stress almost double that of the lime-stabilised specimens.

Uncorroded compressive strength data (Fig 5D) demonstrate the importance of curing history on both lime-stabilised and untreated loessial soils: the CaO control samples, for example, range from almost 4000 kPa in the air-dried state to zero in the saturated state, and reflect the importance of soil suction in maintaining stability of natural and cut faces in loessial soils. Strength data reported in this study also differ significantly from the earlier results reported by Evans & Bell (1981): not only does the air-dried control sample have a φ2 value more than double that of any lime-stabilised specimen, but minimum strength
has been obtained with 5% lime addition for both the air-dried and moist-cured materials (Fig 5D). The tangent modulus at 50% of failure stress ($E_{50}$) also shows the effect of curing history, the curves resembling those obtained for unconfined compressive strength (Fig 5E): strains at failure (Fig 5F) show little change for the brittle air-dried specimens as a function of lime percentage, but clearly reflect the change to plastic deformation in untreated loess at water contents around 15%.

In summary, the study by Glassey (1986) has provided fundamental data on the stress-strain behaviour of Banks Peninsula loess as a function of both lime percentage and curing history. Although extremely high dry strengths have been observed in situ loess, the unconfined compressive strength data obtained here confirms the critical dependence of loess strength on water content, and quantifies for local soils the benefits of lime-stabilisation in situations where soaked strength is necessary. Further investigation of varying curing times is clearly warranted, as is the evaluation of both treated and untreated soil strength as a function of water content.

4 FIELD PRACTICES

4.1 Chemical Stabilisation Techniques

There are two principal methods of chemical stabilisation practice, the first for shallow soils involving mixing and recompaction, and the second utilising slurry or grout addition at depth (Bell 1982). In the former the soil is excavated and pulverised, the stabiliser is added as a powder or as a liquid, thorough mixing is achieved at or close to optimum water content, the mixed material is spread to a predetermined depth over a suitably prepared surface, compaction in layers to a required density is carried out, and a period of curing is normally allowed for the development of the desired soil properties: field practices, including methods of soil pretreatment, mixing and compaction, are well documented (see, for example, Ingles & Metcalfe 1973; Dunlop 1977). In situ stabilisation of soils using grouts and slurries has similarly been practised for many years (see, for example, Lee et al 1983), although direct chemical addition to soil masses of low
permeability (such as clays and clayey silts) is generally ineffective because of penetration difficulties and the low solubilities of many stabilising chemicals in water (Bell 1982).

Evans & Bell (1981) describe the first successful field trials to stabilise loessial soils on Banks Peninsula using both orthophosphoric acid (H₃PO₄) and hydrated lime (Ca(OH)₂). Three different applications of orthophosphoric acid were reported, the first involving conventional mixing and recompaction for a designed fill retention structure; the second utilising a loess-fine sand-stabiliser slurry mix to backfill an erosion cavity that had developed around a leaking water supply pipeline; and the third attempting to reduce the severity of cut-batter "fretting" by spraying the diluted chemical onto exposed faces to develop a stabilised "skin". Two case studies involving hydrated lime application to loess were also reported by Evans & Bell (1981), the first to stabilise the sides and bottoms of tunnel-gully interception trenches at Westmoreland Subdivision, and the second to provide a non-erodible drainage channel (or "water table") for a pylon access track on the Port Hills. Bell (1982, 1983) has described subsequent developments of hydrated lime stabilisation techniques, which were preferred over orthophosphoric acid for reasons of safety during handling and ease of application in powdered form: lime-stabilised backfilling of service trenches (such as sewer lines) is now routinely practised in dispersive loessial soils in parts of Christchurch, whilst the construction of lime-stabilised linings for a water race was also carried out with some success. Three examples of recent field practices involving chemical stabilisation of Banks Peninsula loessial soils are briefly described in the following sections (4.2 - 4.4).

4.2 Subsurface Drainage Control

The role of both surface and subsurface water in slope movement and/or erosion problems in loessial soils has long been recognised (refer also Fig 2B), and remedial drainage works (preferably at source) clearly provide the key to successful site stabilisation (Evans & Tranquair 1977; Evans & Bell 1981; Bell 1982, 1983). However, whilst interception of surface flows and controlled discharge to either a natural watercourse or a stormwater disposal system is a desirable objective, the construction of cut-off drains in dispersive or potentially erodible loess soils cannot be recommended without careful design to ensure the long-term integrity of the system. In addition, the formation of often tortuous tunnel-gully systems may involve subsurface water sources (such as seepage discharging from bedrock) as well as surface flows, and there may be practical difficulties in tracing individual erosion cavities or even in identifying the various water sources. Evans & Bell (1981) have described the construction of open lime-stabilised cut-off channels to intercept active shallow tunnels at Westmoreland Subdivision, and Yetton (1986) provides a thorough review of remedial drainage control measures for erodible loess soils on Banks Peninsula: he discusses in particular the construction of 1-2m deep interception drains with shaped concrete or cement-stabilised loess bases, the use of PVC lining systems, design criteria for gravel filters with perforated drainage coil, and long-term surveillance requirements such as inspection shafts.

The Landsdowne Terrace case study (Fig 6) provides an example of current field drainage practice in Banks Peninsula loess, and illustrates both the requirement for a site erosion model and the application of chemical stabilisation techniques. By late 1985 the home owner had become concerned at the discharge of sediment and water into his basement garage and onto a steep driveway beside his house (Fig 6A), and professional advice was sought. Site inspection, augering and hand excavation located an active "natural" tunnel above the western corner of the house, with a complex cavity system extending beneath the footings and another carrying the sediment observed on the adjacent driveway (Fig 6A): the excavated trench across the rear of the house also showed erodible (≤64-128) losses to a depth of at least 1m below the path level, although no active tunnels were identified, and it was concluded that a major part of the problem was the fact that stormwater from the roof had been fed directly into the ground near the western corner of the house (Fig 6A).

In order to provide long-term security against further undermining of the house foundations, a series of remedial works were designed to intercept any subsurface seepage above the house, and to discharge all water thus collected into an existing stormwater system to the northeast (Fig 6A): in addition, the cavities located beneath the footings were to be backfilled with a lime-cement-sandy gravel slurry and effectively sealed against further water entry. The cut-off trench design profile (Fig 6B) utilised a 5% by weight hydrated lime-stabilised and shaped base, a suitable geotextile filter fabric to line the excavation, and perforated drain coil placed at the base of a fine gravel filter to conduct water to the driveway; during construction, impermeable black PVC sheeting was incorporated on the base and downslope side of the trench for additional protection against seepage. The active tunnel near the western corner of the house was led into the cut-off trench, and the stormwater
downpipe from the roof was also continued away from the foundations and into the improved subsurface drainage system. All construction (apart from excavation of part of the trench) was carried out by the house owner acting on professional advice, and the installed cut-off drain has performed entirely as expected during its first winter with no further sediment discharge onto the driveway or seepage into the basement garage.

4.3 Excavation and Backfilling

Excavation of erodible or dispersive loess, mixing with predetermined quantities of stabilising chemical, and recompaction at or close to optimum water content has become the most common soil stabilisation practice on the Fort Hills (Evans & Bell 1981; Bell 1983): the technique has been applied to the production of non-erodible materials in a variety of situations (such as service trench backfilling), and use can also be made of the gain in wet strength (for example, in fill retention). Yetton (1986) describes engineering geological investigations and remedial measures for the Ngaoi Lane site (Fig 7), where large collapse holes appeared beneath the road surface following the passage of a heavy vehicle. Subsurface erosion was occurring principally because runoff from the road was being fed into a leaking culvert via an unsatisfactory intake (Fig 7a); in addition, three small tunnels at a depth of 3-4m contributed to the formation of erosion cavities in the loess-coluvium soils. Laboratory testing of samples from the site showed that 1.5% by weight of hydrated lime was sufficient to render the soil non-erodible (E > 1000), and to produce a dry density close to or greater than the in situ values under New Zealand Standard (Proctor) compaction (Fig 7d).

Design of remedial measures for the Ngaoi Lane site illustrates the philosophy underlying "excavation and backfilling" as a remedial option (Fig 7b). Surface water sources outside the collapse area were intercepted and piped where feasible into the local stormwater system, and the culvert beneath the road was removed during site preparation and not replaced. Collapsed material was excavated into bedrock, which was present beneath the site at a maximum depth of 4.5m, and 100mm diameter pipes were concreted into each of the intercepted tunnels: slotted drain coils in premix filter gravel were also placed to carry any other seepage waters beneath the road and into an existing erosion cavity (Figs 7a & c).
Excavated loess mixed with 2% by weight of hydrated lime was then recompacted in 200mm layers using hand methods in the downslope cavity, and the front-end loader wheels in the main cut: lime-stabilisation was continued to 1m above the erosion tunnels, and then unstabilised loess was compacted up to alignment grade (Fig 7C). The reconstructed section of Ngaii Lane has performed satisfactorily for 24 years, and the out-flow from the diverted tunnel system is routinely checked for silt and water quantities; in retrospect, the only significant improvement would have been to use either a plate compactor or a motorised roller for recompaction, because samples tested during construction ranged from only about 70 to 90% of maximum dry density (Fig 7D).

4.4 Slurry Filling of Cavities

Evans & Bell (1981) describe the filling of erosion cavities that had developed around a leaking mains water supply pipeline using an orthophosphoric acid-loess-fine sand grout mix which was placed by slurry pump at a water content of about 25%: although erosion resistant, the grout was of low strength and new cavities developed outside the stabilised area, in part because seepage water control was not implemented as had been recommended. Yetton (1986) has conducted both laboratory and field trials to evaluate the use of lime- and cement-stabilised slurries admixed with sands and sandy gravels: loess has not been used with the slurry mix, although diffusion reactions of hydrated lime with the loessial soils forming the cavity walls has been demonstrated on a laboratory scale. Although unsatisfactory for small tunnels where adequate drainage cannot be installed, the technique offers considerable potential for larger cavities and significant cost-savings compared with excavation and back filling may be achieved: in some situations, such as tunnel erosion and subsidence under buildings, slurry filling may provide the only economic and effective solution.

At the Henderson's Culvert site (Fig 8), where subsurface erosion over many years had produced a complex pattern of cavities up to 5m below road level, a full geotechnical investigation was first carried out to identify tunnel systems and soil properties (Figs 8a & 8c). After placement of perforated drain soil beneath the fill (Fig 8a) and interception of upslope water sources, the main exit was boxed off with timber shutters and three slurry entry points were identified for the estimated 11.5m$^3$ required to fill the erosion cavity.
Mix proportions were varied, the first containing 5% hydrated lime and 3% Portland cement, the second 5% by weight of each stabiliser, and the third 3% hydrated lime and 7% Portland cement to provide greater bearing capacity beneath the pavement surface: in all cases a sandy gravel was used as the binder, and little segregation was observed during pumping. The placed slurry was allowed to set for 3 weeks before removal of the boxing, and a major advantage of the slurry method was the fact that one lane of the road could be kept open to traffic throughout the stabilising operation. Future performance of the slurry-filled cavity will be monitored periodically, but providing the control of surface and subsurface water has been effective there is no reason to anticipate renewed erosion in the surrounding loess-colluvium soils (as occurred at Rocky Point).

5 FUTURE DEVELOPMENTS

5.1 Site Investigation Practices

Successful chemical stabilisation of dispersive loessial soils on Banks Peninsula is critically dependent on the determination of site erosion models, and both field and laboratory investigations must precede any attempt at soil treatment by one or more of the methods outlined in this paper. The prime objectives must be to 1) define the soil mass characteristics, such as gross pedological layering and erosion cavity systems; and 2) quantify soil material properties, such as erodibility and dispersivity, by appropriate laboratory testing. In this regard the philosophy is no different from any other geotechnical site investigation programme (Bell & Pettinga 1984), in that an "objective-oriented" approach must be adopted from the outset, and the tentative site model refined and quantified to the stage where suitable remedial measures can be recommended. The investigation of erosion processes in dispersive loessial soils cannot, however, be regarded as "conventional" because of the considerable difficulties in identifying and tracing tunnel-gully systems, especially in built-up areas where housing or road precludes direct subsurface evaluation of the site.

We believe that an engineering geological approach, such as that recommended by Bell & Pettinga (1989) for residential subdivision of land in New Zealand, is to be preferred for
the investigation and solution of erosion problems in the loessial soils of Banks Peninsula. Yetton (1986) has reviewed the various field and laboratory methods that may be relevant to such studies, and our recommended procedure for the investigation of subsurface erosion in loess is as follows:

1. Problem definition by preliminary site inspection, air-photo studies, and discussion with interested parties.

2. Site characterisation by a. engineering geological mapping at appropriate scales (1:1000 or larger); b. subsurface data collection from test pits, auger holes and/or Scala penetrometer profiles; and c. laboratory testing of selected samples for index properties (such as pinhole erodibility and Emerson Crumb class).

3. Assessment of the nature and extent of subsurface erosion by techniques such as physical inspection of cavities (if sufficiently large), dye or smoke tracers, and geophysical methods if appropriate.

4. Identification of surface and subsurface hydrology, and incorporation into a definitive site erosion model.

5. Determination of remedial options including chemical stabilisation if appropriate, and preliminary costing as an aid in method selection.

6. Design of recommended remedial measures (if accepted by client), including laboratory evaluation of parameters such as optimum water content/compacted dry density relationships and/or slurry mix properties.

7. Verification of site erosion model during remedial works, with modifications to design if required, and close geotechnical supervision to ensure that design objectives are met by the contractor or client.

8. Routine inspection after completion to monitor effectiveness and performance of erosion control measures.

9. Additional remedial works if further subsurface erosion problems develop, or field performance is unsatisfactory.

In summary, chemical stabilisation has to be regarded as a "normal" option for remedial erosion control in dispersive loessial soils, and this has in fact become accepted in recent years on Banks Peninsula. Difficulties remain, however, persuading both potential clients and contractors that such techniques are necessary if subsurface erosion problems are to be minimised, whilst on the other hand the application of chemical stabilisation without first determining an engineering geological site model is equally of concern. Again, the often very complex nature of subsurface erosion in dispersive loessial soils is such that long-term performance of remedial works cannot be "guaranteed" in any contractual sense, and this is especially so where cavity filling by slurry methods is used for small tunnels where physical inspection is not possible. We are, nevertheless, convinced that our approach to site investigation and remedial design is appropriate to the geotechnical nature of the problem, and that control of surface and subsurface water remains the prime objective in loess stabilisation on Banks Peninsula.

5.2 Alternative Stabilising Chemicals

Bell (1982) has reviewed various stabilisation methods and alternative chemicals for the control of dispersion in engineering soils, and we believe that for Banks Peninsula loess viable options are limited to lime (either as Ca(OH)\textsubscript{2} or as CaO) and Portland cement. The feasibility of phosphoric acid (for example, H\textsubscript{3}PO\textsubscript{4}) has, of course, been demonstrated by Evans & Bell (1981), but safety and handling problems are important considerations that limit its future use; the possibility of using gypsum (CaSO\textsubscript{4}·2H\textsubscript{2}O), which has proved successful in tunnelling soils in eastern Australia (Rosewell 1977), is currently under investigation for loess stabilisation. A major future development is the design of various lime-cement slurry mixes for both cavity filling and cut-off or retaining wall construction; the use of loess as part or all of the binder remains to be investigated in such applications. In situ soil stabilisation by chemical grout penetration, either under pressure or by gravity drainage, remains to be evaluated, and "frettling" of cut-batter leading to gradual face retreat by topples and falls is an engineering situation where some form of chemical stabilisation should be applicable. The principal future developments that we envisage are, however, the continuation of laboratory characterisation of the property changes that are partly documented in this review, and the integration of field and laboratory techniques into a "code of practice" for chemical stabilisation in dispersive loessial soils.

6 CONCLUSIONS

1. The loessial soils of Banks Peninsula display mass movement and tunnel-colly erosion features that are controlled by gross pedological layering, and by soil properties such as moisture-dependent strength changes and clay mineral dispersion; subsurface erosion commonly results in sediment and water discharge, whilst subsidence due to cavity formation is potentially serious for house or road foundations.
2. Chemical stabilisation techniques have been developed that render potentially or actively eroding loessial soils both non-dispersive and stable to wetting and drying cycles: satisfactory stabilisers include hydrated lime (Ca(OH)\textsubscript{2}), quick lime (CaO), Portland cement, and orthophosphoric acid (H\textsubscript{3}PO\textsubscript{4}).

3. Hydrated lime has been most widely used for soil stabilisation on Banks Peninsula, with field application by loess pulverisation, stabiliser mixing in quantities between 2 and 5% by weight, and recompaction at or close to optimum water content: complementary laboratory studies have demonstrated the importance of curing history for strength gain, and have documented changes in various soil classification parameters over the range 1 to 10% by weight of lime addition.

4. Slurry methods have been developed for backfilling of erosion cavities, with loess-fine sand-orthophosphoric acid, sand-lime and sandy gravel-lime-cement mixes being field tested: the slurry technique may offer significant cost advantages over excavation and recompaction, and in certain cases is the only feasible option for site remedial measures.

5. All stabilisation methods in Banks Peninsula loess require the formulation of a site erosion model which incorporates both field engineering geological and laboratory geotechnical data: chemical stabilisation is a "normal" remedial option for dispersive loessial soils, but control of surface and/or subsurface water flows remains the prime design objective.

7 ACKNOWLEDGEMENTS

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Engineering geological aspects of foundations in swelling soils
Fondation sur sols gonflants: Aspects du géologie de l'ingénieur

J.L. Justo, University of Seville, Spain

ABSTRACT: The damage produced annually by expansive soils to man-made structures has been evaluated in 1979 in seven thousand million dollars, and exceeds the combined effect of earthquakes, tornados and landslides.

Swelling soils are greatly influenced by environmental conditions.
Total suction is a fundamental parameter for the study of swelling soils.

A review has been made of laboratory testing methods, design methods for foundation on swelling soils and mapping of these soils.

RESUME: Les dégâts produits chaque année par les sols gonflants aux structures construites par l'homme ont été évalués en sept mille millions de dollars. Ces dégâts surpassent ceux produits par l'effet combiné de tremblements, ouragans et glissements de terre.

L'environnement a beaucoup d'influence dans les sols gonflants.
Une révision a été faite des essais de laboratoire, des méthodes de projet et de la cartographie géotechnique des sols expansifs.

1. INTRODUCTION

Swelling soils, as collapsing soils, are greatly influenced by environmental conditions, such as climate and the presence of water or vegetation, due to the relation between environmental changes and suction.

Krohn and Slosson (1980) have calculated the damage produced annually by expansive soils to man-made structures in more than seven thousand million dollars (referred to 1979), and they say: "although not as sudden or traumatic as an earthquake, tornado or landslide, expansive soils cause more damage to these structures than the combined effect of the other aforementioned hazards".

As indicated by Professor Salas (1980), an expansive soil is not really neither a kind of soil nor a group of soils. It is actually a phenomenon produced by the conjunction of a clayey soil and favourable ambient conditions.

It may be very dangerous to use general identification criteria for expansive soils. A soil which may be non-expansive as the foundation of a heavy building may cause distress due to volume increase in a concrete canal lining. For this reason, in the first part of this paper we shall put the emphasis in design rather than identification or classification. Notwithstanding, at the end of this report we shall talk about the role of the geologist in the mapping of expansive soils.

2. SUCTION

Total soil suction is defined as "the negative gauge pressure relative to the external gas pressure on the soil water to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (i.e. permeable to water molecules only) membrane with the soil water" (Review Panel, 1965).

Soil suction is a measure of the soil affinity for pure water.

The most important step towards a rational design of foundations on partly saturated soils was the acknowledgement that suction changes produce changes in moisture content and changes in moisture content produce changes in stress and volume of the soil.

Total suction has two components: collute and matrix suction.

Matrix suction is defined as "the negative gauge pressure relative to the external gas pressure on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water."

Osmotic or solute suction is "the negative gauge pressure to which a pool of pure water must be
subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water. The proof that solute suction is not only of academic interest is given by the 10 cm heave of buildings founded below the water table in Adelaide, as a result of the seepage of salt-free water into the soil. A nominal decrease in matrix suction and an important decrease in solute suction was checked into the soil (fig. 1). On the other hand the effect of both components on soils may be quite different (Aitchison, 1965; Aitchison and Martin, 1973; Richards et al., 1984).

Laboratory methods employed in design to measure suction in unloaded samples are shown in Table 1.

![Table 1](image)

**Table 1**

<table>
<thead>
<tr>
<th>Method</th>
<th>pF range</th>
<th>Suction measured</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suction plate</td>
<td>0-2.9</td>
<td>matrix</td>
<td>Aitchison &amp; Richards, 1965.</td>
</tr>
<tr>
<td>Pressure membrane</td>
<td>2-5.1</td>
<td>total</td>
<td>&quot;</td>
</tr>
<tr>
<td>Common</td>
<td>2-6.18</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>special</td>
<td></td>
<td></td>
<td>Richards, 1969.</td>
</tr>
<tr>
<td>Vacuum desiccator</td>
<td>4.6-7</td>
<td>total</td>
<td>Pile &amp; McInnes, 1984.</td>
</tr>
<tr>
<td>Thermocouple</td>
<td>3.2-4.8</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>psychrometer</td>
<td></td>
<td></td>
<td>Richards, 1965a.</td>
</tr>
<tr>
<td>Thermistor</td>
<td>4.0-5.5</td>
<td>&quot;</td>
<td>McKean, 1980; Zry, 1984.</td>
</tr>
<tr>
<td>Filter paper</td>
<td>0-7</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
</tbody>
</table>

The suction plate, the pressure membrane apparatus and the vacuum desiccator "control suction." The samples are quickly weighted before and after the test. The percentage change in weight is plotted against the control suction. The suction for zero change in weight is interpolated graphically and accepted as the original suction (Aitchison and Richards, 1965).

On the other hand, the psychrometer and the filter paper "measure" the suction. Both methods and the vacuum desiccator find the total suction through the following relationship between total suction and relative humidity:

\[
pF = 6.5 + \log_{10}(2 - \log_{10}H)
\]  

Using distilled water without flushing below the membrane, the pressure membrane apparatus will ultimately measure the matrix suction, as the membrane is, in the long run, pervious to solutes - (v. Richards, 1980).

In non-saline soils, the suction measured by many techniques coincide (fig. 2). This figure - and the statements of other authors (Aitchison and Richards, 1965) show that in most soils solute suction is negligible, as techniques measuring total and matrix suction lead to the same result. (Coleman and Marsh, 1961; Brackley, 1973; Hamberg and Nelson, 1984; Johnson, 1980).

![Fig. 2](image)

Fig. 2. Suction/moisture content relationship for Clay loam (Crosey & Coleman, 1960)
In saline soils total suction may be 10 times larger than matrix suction (v. Aitchison and Richards, 1965). In saline expansive clays the definition and exact measurement of solute suction may be difficult owing to double layer phenomena (which cations belong to the soil water and which to the clay?) as shown by Richards (1980).

For these reasons many authors feel that the important variable in most situations is total suction (Richards, 1979 and 1980). Total suction is directly related to the relative humidity of the air and hence to the surface drying of soil profiles and also it is a measure of the ability of the trees to take up moisture (Pile and McInnes, 1984).

The filter paper has the advantage of the wide range of suction that it can measure. Once the paper is calibrated it is equilibrated in a closed container (a moisture cell) with a soil sample for seven days. The water content is determined by weighing to four decimal figures, and the suction is inferred from the calibration data.

Equation 1 shows that the most common range of suction for semi-arid soils, say $p_f$ 3.5 to 4.5, is associated with relative humidities above 96.5%. Thus the problem is to measure relative humidity very accurately over a small range in order to determine total suction over a large range, which calls for careful testing. Both psychrometers should be used in a constant temperature room, and, in addition, require frequent calibration checks and a dedicated operator, who should be acquainted with the basic principles of soil psychrometers. In particular, measurements at the lower end of the range for the thermocouple psychrometer require great care in calibration, equilibrium time, and to obtain a clean and dry sample chamber (Pile and McInnes, 1984; MacFarlane and Gray, 1979; Malone, 1960).

3. THE INFLUENCE OF STRESS PATH IN THE COLLAPSE-SWELLING OF SOILS AT THE LABORATORY

A partly saturated soil may suffer volume changes due to pressure or suction changes. It has been observed that the stress-path has an important influence in the final void ratio reached.

3.1. Tests in normal odometers

Figure 3 (Justo et al., 1984) shows the "natural moisture content" curve, obtained loading the soil without changes in moisture content.

![Diagram showing natural moisture content curve](image)

The same figure shows the curve obtained - soaking the sample under loading.

"Free swelling" is defined as the swelling of a soil under a small "nominal" pressure that according to the authors may range from 1 to 10 kPa. This definition implies that the value of the nominal pressure is not important as far as it is small. Figure 3 shows that this is untrue. So, the term free swelling alone should never be used, and the nominal pressure should be clearly specified.

As seen in figure 3 there are many possible definitions of the "swelling pressure.

From a practical point of view, the most important one is the "swelling pressure-3" of figure 3 corresponding to the crossing of the natural moisture content and the "soaking under loading curves". When we apply the "swelling pressure-3" to the sample, soaking does not produce any deformation on it. Below this pressure soaking causes swelling, and above collapse.

So, in general, a soil will swell or collapse after being flooded, depending upon whether the external pressure is smaller or larger than the swelling pressure (v. Justo and Sætersdal, 1979).

Clayey sands, gypseous silts and loess, when wetted under low pressures may swell, while under high pressures may show a tendency to collapse. Low pressures are applied by many engineering structures such as floors and foundations beams, and that must be taken into account in design. Even clays may collapse under moderate loads (fig. 4).

This shows that instead of distinguishing between expansive and collapsing soils, we should better talk about "soils that usually be have as expansive or soils that usually be have as collaps-
The "swelling pressure-2" is the one obtained in a swelling pressure test.

The "loading after soaking" curves stay above the "soaking under loading" curve in the swelling zone. This result has been confirmed by many authors (Justo and Saetersdal, 1979).

The "swelling pressure-4" corresponds to the crossing of any of the "loading after soaking" curves with the null deformation line. It is independent of the soaking pressure for the practical range of this soaking pressure (Justo et al., 1984).

The relation between the four definitions of the swelling pressure have been studied by Delgado (1986). Swelling pressure-1 is by definition smaller than swelling pressure-3, and swelling pressure-4 is the largest (v. Ali and Elturabi, 1984).

Tissot et al. (1986) at this conference attribute the difference between swelling pressures 2 and 4 to friction at the oedometer. According to the detailed study of Delgado (1986) the difference is due to stress-path.

We see that to predict the stresses and volume change produced by swelling soils, not only the final values of pressure and suction are important, but also stress-path.

If the sample at the oedometer is left to shrink after loading, the suction of the soil will come finally to an equilibrium with the relative humidity of the air at the room in which the oedometer is placed. In the tests carried out by Delgado (1986) this relative humidity was around 50% and the final suction was in pF = 6. Under these conditions the shrinking under loading curve is a straight line in a natural scale. According to several authors, volume change ceases at pF values between 5.5 and 6.0 (v. Hamberg and Nelson, 1984; Mitchell and Avalle, 1984).

Figure 5 shows, in a same graph, the "natural moisture content", the "shrinkage under loading" and two "soaking under loading" curves, obtained from the natural moisture content and the shrinkage under loading curves respectively.

3.2. Test in suction-controlled oedometers

Actually, the pressure-deformation relationship should be measured simulating suction changes under field conditions. Oedometers allowing suction control are compared in table 2.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Type of suction controlled</th>
<th>Range, kPa</th>
<th>Principle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Escario (1967)</td>
<td>Matrix</td>
<td>&lt;100</td>
<td>Direct measurement</td>
</tr>
<tr>
<td>Escario (1967, 1969b)</td>
<td>Matrix or total</td>
<td>&lt;12,000</td>
<td>Pressure membrane</td>
</tr>
<tr>
<td>Kassif &amp; Bui-Shalom (1971)</td>
<td>Matrix</td>
<td>0-2,500</td>
<td>Osmotic</td>
</tr>
<tr>
<td>Aitchison &amp; Martin (1973)</td>
<td>Matrix or solute</td>
<td>10-1,000</td>
<td>Pressure membrane</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0-38,000</td>
<td>Osmotic</td>
</tr>
</tbody>
</table>

The advantage of Escario's apparatus (fig. 4) is its greatest range of suction. The deformations of the semi-permeable membrane, which are very important, may be taken care of by working at constant vertical pressure during each test (Escario and Saux, 1973).
Swelling followed by collapse, as suction is decreased, is often measured in some soils (Fig. 7). In these cases flooding of the sample at the oedometer could lead not only to an error in the amount of movement, but also in the sign of it. The peak exhibited in the swelling pressure-2-time curve for dry samples (Brackley, 1973) measured at the oedometer is probably related to this fact, and shows the need for automatic measurement of swelling pressure-2.

Figure 8 shows several tests carried out in Escario's apparatus. The samples are first loaded and then suction is brought up to zero step by step.

When a clay swells around a pier foundation, the pier friction induces stresses in the soil to counteract the tendency to swelling. So, the suction decrease is coupled with an increase in total stress (above the base of the pier) or a decrease in total stress (below the base of the pier). When suction is decreased as the sample is loaded or unloaded, the final swelling reached is little dependent on stress-path, and depends mainly on the final suction and total stress (Fig. 8), reaching finally the "seeking under loading curve".

In the practical range 5-200 KPa all the stress-strain curves for constant suction may be assimilated to a straight line in a semilog scale without important loss of accuracy.

A method for calculating vertical movement, based on the oedometer of Aitchison and Pater (1973) has been proposed by Aitchison et al. (1973). The method (Table 3) is based on a separate calculation of strains produced by changes in total stress, matrix suction and solute suction. But the parameters $C_m$, $I$, and $I'$ are so strongly interrelated (see Fig. 5 and 8; v. Richards et al., 1984) that one wonders whether this would not be better to follow the corresponding stress-path in the oedometer.

Richards (1980) has shown the difficulty of defining and measuring solute suction, and of applying solute suction gradients to a saline soil. Owing to that the terms $C_m$ and $E_m$ should be joined in one related to total suction, and the environmental conditions should be followed as exactly as possible, including the type of solute (v. Richards et al., 1984).

Hemoglobin and Nelson (1964) use the equation:

$$ r = C_0 \left( \log p_2 - \log p_1 \right) + \frac{\Delta p}{\Delta t} $$

They take $C_0$ as the slope of an unloading branch in an oedometer test (v. fig. 3). This is not the right stress path.

They find $I$, using a clog test procedure similar to the coefficient of linear extensibility ($CL_E$).

They claim good agreement between measured and calculated heaves in a test plot. Suction was cal...
Table 3
Method proposed by Alcherson et al. (1973) for calculating movements in expansive soils

1. Calculation of strains
(a) Unit strain produced by a change in total stress:
\[ \varepsilon = C_p (\log \frac{p_2}{p_1}) \]
(b) Unit strain produced by a change in matrix suction:
\[ \varepsilon_m = I_m (\log \frac{\psi_{m1}}{\psi_{m2}}) \]
(c) Unit strain produced by a change in solute suction:
\[ \varepsilon_s = I_s (\log \frac{\psi_{s1}}{\psi_{s2}}) \]
where \( p \) = total stress, \( \psi_m \) = matrix suction, \( \psi_s \) = solute suction

Parameters:
- \( C_p \) = compression index at specified values of \( \psi_m \) and \( \psi_s \)
- \( I_m \) = Instability index (matrix) at specified values of \( p \) and \( \psi_m \)
- \( I_s \) = Instability index (solute) at specified values of \( p \) and \( \psi_s \)

2. Vertical movement
\[ \Delta h = \int_0^L (\varepsilon_m + \varepsilon_s) \, dz \]

...calculated through the suction-moisture content relationship from measured initial and final moisture contents. In this case the strain due to the pressure was negligible compared with the strain produced by the suction change...

4. EVOLUTION OF DESIGN METHODS

The principles governing the behaviour of expansive clays have been understood for some time (Wooltorton, 1936). Early designs were necessarily based upon simple parameters.

In Burma design was based on identification parameters (e.g. aspect and sodium content), minimum bearing pressure deduced from a survey of behaviour of different buildings, and depth of the active layer deduced from a study of moisture content with depth and time at the site.

Loading tests were employed so as to check the maximum pressure that could be applied under maximum moisture conditions so as not to exceed the allowable settlement.

On the other hand, in Texas the main parameter used was the depth of the active layer, so as to find the safe foundation depth (Simpson, 1936). That was probably due to the belief that the heaving of buildings was practically independent of their weight (Terzaghi and Peck, 1948), a statement not tenable today (v. Justo and Saetersdal, 1979). Notwithstanding, shrinking may prevail after construction (Fig. 9), and in that case settlement will increase with foundation pressure. So, to take the foundation below the active layer is, generally speaking, a better decision than increasing the pressures.

The drying effect of the transpiration of trees, mainly fast growing ones, and its shrinkage action was reported already by Cooling and Ward (1948). The height of the mature tree was one of the parameters chosen to estimate the reach of the drying effect. Buildings with shallow foundation should be kept away from trees a distance equal to the height of the tree. In the case of dense rows of trees with paved areas adjoining the building, the drying effect can exceed 1 1/2 times the tree's height (Justo and Saetersdal, 1979).

Modern design methods are based upon a calculation of the movement of the foundation and a comparison with the so-called "allowable movements".

In the following paragraphs we shall review the main methods for calculating foundation movement...
5. METHODS BASED UPON FLOODING THE SAMPLE IN THE OEDOMETER

In hydraulic structures the presence of water can eventually saturate the foundation soils. To calculate heave in these cases, a series of tests were devised in which the sample was flooded at the oedometer.

Later on, this was forgotten, consciously or unconsciously, and they were employed in cases in which suction remained far from zero.

We shall review some methods of movement of the foundation based upon tests in which the sample is soaked through at the oedometer.

5.1. McDowell's method

It is based upon a given relationship between volumetric swell and pressure for every free swell. As this relationship is not constant at all, the results obtained may be far from the reality (v. Table 6).

5.2. Double and simple oedometer tests

They have been commented by Justo and Saetersdal (1979), and Justo et al. (1984).

They use a wrong stress-path, corresponding to the "loading after soaking curves" in figure 3, and they overestimate the settlement.

5.3. Direct method

Usually moisture change occurs after construction, and for that reason the "direct method" uses the "soaking under loading curve".

As indicated above, in the practical range (5-200 kPa) this curve may be approximated by a straight line (Belgado, 1986; Justo and Saetersdal, 1979). In that case it is strictly enough to test two samples so as to obtain the curve. Notwithstanding, due to possible differences between the initial state of samples, it is better to test at least three.

The direct method was the basis of the Army Corps of Engineers method for calculating swelling of foundation (fig. 10).

![Diagram showing swelling vs depth]

Procedure for estimating total swell:

1. On basis of boring log profile select samples at intervals for swell tests.
2. Load specimen in consolidometer to overburden pressure plus weight of structure, add water and observe swell.
3. Compute swell in terms of percent of original specimen height and plot v. depth.
4. Compute total swell which equal to area encompassed by percent swell v. depth curve, for example, using curves shown above.

Total swell = 1/2 × (250-30) × 2.8/100 = 3 cm.

Fig. 10. Method of Army Corps of Engineers (1965) for calculating swelling of foundation

6. METHODS BASED UPON THE CHANGE OF SUCTION OR MOISTURE CONTENT

These methods are based upon a measurement of initial suction or moisture content, and an estimate of the corresponding equilibrium value.

6.1. Estimate of equilibrium suction

If the soil is covered by an impervious surface, a dynamic moisture stability is finally reached below the center of the surface (v. Justo and Saetersdal, 1979).

If there is a water table near the surface (15 m deep in heavy clay, 6 m in common clay, 3 m in sandy clays or silts) water pressure will come to an equilibrium with the water table (v. Justo and Saetersdal, 1979).
When there is a perched water table within 3 m of the surface, the pore water pressure may approach zero (saturation) above the water table (Johnson, 1980). This supports the use of tests in which the sample is flooded at the oedometer in these cases. In South African soils the final suction is in some cases, close to zero (Brackley, 1975).

We see that the depth of the water table is an important parameter for design.

Russam and Coleman (1961) introduced a climatic parameter, the Thornthwaite moisture index, in the study of partly saturated soils. A soil moisture balance is made throughout the year. When soil moisture deficit always exists (desert or semi-desert conditions) very small seasonal changes are produced. As a result, sealing of the surface produces little effect on suction (De Bruijn, 1973).

For intermediate climates and when the water table is deeper than indicated above, an estimate of equilibrium suction can be made from soil type and Thornthwaite moisture index (Aitchison and Richards, 1965).

This relationship is not far from linear between the two points:

\[
\begin{align*}
I_t &= 40 \\
I_t &= 20
\end{align*}
\]

\[
\begin{align*}
p' &= 5 \\
p' &= 3
\end{align*}
\]

where:

\( I_t \) = Thornthwaite moisture index.

A better method for estimating the equilibrium suction is proposed by Aitchison and Richards (1965): if a suction measurement is taken beneath the zone of seasonal influence, the pore water pressure at another depth, under sealed conditions, may be computed as if both points were in static equilibrium.

Table 4 shows the relationship between in situ suction, unloaded laboratory suction and overburden pressure. In the tests carried out by Kassif and Ben Shalom (1971), above a certain degree of saturation (65 to 80% depending on density) \( \alpha \) becomes unity. The usual degree of saturation for expansive clays is not frequently less than 65% (v. Juste & Sætersdal, 1979; Delgado, 1986). Frequently the magnitude of the overburden pressure is insignificant, and in such cases is ignored.

\[
\psi = \frac{\alpha}{\alpha_0} \psi_0
\]

\( \alpha \) is similar to the pore pressure coefficient \( K_p \), and so \( \alpha = 0 \) for an incompressible soil and \( \alpha = 1 \) for a saturated compressible soil.

In expansive clays frequently the magnitude of \( \psi_0 \) is insignificant and, in such cases, is ignored.

It has been shown that very often the equilibrium suction is far from negligible. In the Mediterranean climate of Adelaida, suctions ranging from p' 4 to p' 5.5 have been measured beneath long established major buildings (Aitchison and Woodburn, 1969). Johnson (1980) speaking about the heave accumulated below an impervious surface, states: “Seasonal heave can be substantial, even exceeding the long-term heave, for sections in semi-arid climates; seasonal heave is not substantial in the wet and humid climate”. In any case, in the borders of a sealed surface there is a situation of dynamic equilibrium and seasonal variations may be important.

In this respect, it is interesting to notice that in a study carried out by Mathewson et al. (1975) about the damage suffered by 136 houses in Texas, the dominant mechanism of failure was one of edge movement.

In all these cases, it seems that the calculation of ground movement may, in principle, be different to the one obtained flooding the sample at the oedometer.

6.2. Richard's method

Richards' method is indicated in Table 5.

It is based upon the assumption that, during volume change, the change in volume of water equals the change in total volume, or, in other words, that the volume of air voids remains constant. We see in Figure 11 that this is approximately true in an important central part of the swelling process. At the end of swelling the soil becomes saturated without volume change.

Richards assumes that, owing to the presence of shrinkage cracks, the vertical strain is only one third of the total volume change.

McKeen (1980), on the other hand, has found that this coefficient may range from 0.5 to 0.8. Formerly the method used the initial moisture content (measured) and the final moisture content (estimated).

The equilibrium moisture content may be estimated as the equilibrium suction (v. Delgado, 1986). In China it seems that shrinkage movements after construction are dominant (Xingfang and Lin, 1984; Chen, 1984; Xingfang and Lin (1984) have studied the ratio between minimum moisture content and plastic limit.

Sokolov and Amir (1973) have developed an ingenious method, checked with field measurements, to
Table 5
Richards' method for the prediction of the foundation movement in expansive clays

Volumetric strain:
\[ \frac{AV}{V_0} = \frac{w_1 - w_2}{100 + w_1 G} \]

where \( w_1 \) = the initial water content in percentage, \( w_2 \) = the final water content in percentage, and \( G \) = specific gravity of soil particles.

Vertical linear strain:
\[ e = \frac{1}{3} \frac{AV}{V_0} \]

---

Fig. 11. Relationship between total specific volume and specific water volume for a natural clay from Israel (Lytton and Watt, 1970)

Fig. 12. Suction versus moisture content - curves under (a) zero load and constant density, (b) swell pressure testing conditions, (c) free swelling conditions (Kassif and Ben Shalom, 1971)

find the moisture distribution in homogeneous soils under pavements. The parameters needed are the depth of the active layer, the amplitude of moisture changes on the uncovered area and the moisture content below the active layer.

The inconvenience of this approach is that the difference between the moisture contents of two neighbouring samples may be larger than the change in moisture content produced by suction changes (v. Johnson, 1980).

Beal (1984) has developed a method by which the relationship vertical strain - moisture content may be obtained directly. The soil may be loaded with a small surcharge, as existing on soils under slabs-on-ground supporting low-rise lightweight structures and with small thickness of the active layer. Under these circumstances, Richards' relationship holds well in the central part of the shrinkage process, but tends to overestimate swelling, specially when the surcharge increases (from 5 to 25 kPa). Beal recommends better the use of his apparatus.

The modern approach is to measure the initial total suction, to estimate the final total suction and to apply Richard's equation - through the suction-moisture content relationship. To avoid hysteresis effects observed in figure 2, the samples should be wetted or dried from natural conditions (Snethen, 1980). But the relation suction-moisture content is strongly stress-dependent (fig. 12). For this reason, in situ it may change with depth.

For all these reasons Richards' method may only be applied, as an approximate method to the calculation of the movement of slabs-on-ground for low-rise lightweight structures or pavements, when the depth of the active layer is small, as it happens in Australia (v. Holland and Lawrence, 1980). For these conditions Holland and Lawrence have found an excellent agreement between the seasonal heaves measured and calculated by Richards' method.

Finally Snethen uses Richards' equation, but measuring the initial moisture content and estimating the final suction. He does not apply the factor 1/\( \alpha \), but uses a "compressibility factor", \( \alpha \), to take into account the fact that in less plastic soils the volume change may be less than the change in the volume of water. Roughly:

\[ T = \begin{cases} \frac{5}{3} \alpha = 0 & \beta > 45 \\ \frac{1}{3} \alpha = 1 & \beta < 45 \end{cases} \]

\[ S = \begin{cases} \alpha = 0.0275 \beta, & \beta = 0.125 \end{cases} \]

6.3. The Instability, Index method

For the calculations of slabs-on-grade in Australia it is usually assumed that moisture changes occur under constant total stress.
Under such circumstances, the equations of Table 3 may be reduced to:

\[ c_z = I_{pt} \times \Delta pF \]

where:

- \( c_z \) = unit vertical strain
- \( \Delta pF \) = change in total suction
- \( I_{pt} \) = a "constant" called by Aitchison et al. (1973) the Instability Index
- \( I_{pt} \) may be measured during a shrinkage process in the apparatus of Figure 13.

The pF is increased up to 4.5 in a vacuum desiccator. Contrarily to what is said by the authors the apparatus should apply to the sample not only the overburden pressure, but overburden plus pressure due to the construction.

According to File and Mcinnes (1984) the values of \( I_{pt} \) obtained are remarkably consistent. The authors give also a method to measure \( I_{pt} \) during swelling.

Mitchell and Lavalle (1984) recommend calculating the Instability Index through the relationship:

\[ I_{pt} = \frac{c_z}{\Delta pF} = \frac{c_z}{\Delta w} \times \frac{\Delta w}{\Delta pF} \]

\( c_z/\Delta w \) may be calculated measuring the length of soil cores during a shrinkage process, weighting the sample each time the length is measured, and finally oven drying to find the water contents. In Adelaide the ratio \( c_z/\Delta w \) is linear during the first two days of core shrinkage (fig. 14).

The ratio \( \Delta w/\Delta pF \) is measured also in a shrinkage test. According to Mitchell and Lavalle (1984) for \( w = 0 \) \( pF = 0.8 \).

For every pedological type of soil, an approximate relationship between Instability Index and plasticity index has been presented by Mitchell and Lavalle (1984) for Australian soils.

Mitchell and Lavalle (1984) have found an excellent relationship between the measured seasonal heave and the prediction from laboratory measured - soil suction profiles and Instability Indexes measured in shrinkage tests (fig. 15).

Richards et al. (1964) have shown that the Instability Index is not a constant, but varies substantially with stress (fig. 16 and 17), may also vary with soil suction (fig. 17), and changes with solute type, concentration and previous history.

Fig. 13. Cell for measurement of Instability Index - (File & McInnes, 1984)

Fig. 14. Linear strain-moisture content relationship (Mitchell & Lavalle, 1984)

Fig. 15. Comparison between predicted and observed seasonal heave (Mitchell & Lavalle, 1984)

Fig. 16. Suction-controlled oedometer tests on compacted samples after Kassiff et al., 1975 (Richards, 1979)
As indicated in figure 17, at very low pressures the Instability Index is little dependent on stress, and this may explain the success of Mitchell and Avalle prediction (fig. 15).

Figure 18 shows the influence of confinement in the vertical strain.

Different authors have found different relationships between the Instability Indexes in swelling and shrinkage (v. Pile and McInnes, 1984; Richards et al., 1984).

The Instability Index may be 6 to 11% for clays containing expanding lattice minerals and less than 4% for clays which do not contain significant amounts (Richards et al., 1984).

Finally, Richards et al. have found from in situ measurements of soil suction and ground movement that the Instability Index in situ may be much larger than found at the laboratory (Fig. 19).

The Instability Index can be found with a pressure membrane apparatus (v. Richards et al., 1984).

McKeen (1980) has found good agreement between the heaves measured and calculated from suction profiles.

7. ELASTIC METHODS

The use of elastic methods in expansive-collapsing soils have been summarized by Justo and Saetersdal (1981) and Justo et al. (1984b).

A finite element method has been applied by Richards (1979) to the calculation of the stresses and deflections of a slab-on-grade subject to a given change in the suction profile.

Justo et al. (1984) have developed a method applicable to any type of foundation on swelling-collapsing soil. As an example it has been applied to the calculation of a pier foundation.

The stress-path followed is indicated in figure 20.

We are only interested in the strains produced by the net load on the pier and the wetting of the soil. So, the zero state of strains corresponds to the overburden pressure $p_{0}$ (point 0). From this point on, the stress-strain path in situ is 01, and the calculation stress-strain path 03.

To this end, we apply to each soil element, within the active layer, the swelling 03, which corresponds to its overburden pressure and to the "wetting-under-loading" curve corresponding to its final suction.

The modulus of deformation of the soil element corres-
ponds to the straight line IF. As this deformation modulus depends upon final pressure, unknown at the beginning, an iterative procedure is followed.

As an example we have applied the method to a building, four-storeys high, damaged during a shrinkage period.

Figure 21 shows the actual plan of the foundation.

Figure 22 shows the plant idealization, the discretization of the foundation and the vertical deflection of the surface layer (ground and foundation).

Figure 23 shows the vertical stresses in a section through a pier.

Table 6 shows the comparison between measured heave and the heave calculated by the finite element method and by several well known methods.

Movements of the building and ground movements at several depths and times have been measured (Fig. 24).

The calculations have been made for the building and for the free soil profile.

The prediction made by McDowell's method is too low. This method should not be used today.

The double oedometer test gives too high heaves, because, as indicated earlier in this paper, it uses a wrong stress-path.

Two calculations have been made with the direct method, using the true soaking under loading curve (general method) or substituting it by a straight line (simplified method). Both calculations lead nearly to the same result. Although the calculated heave of the free soil profile is too high, because the method assumes saturation of the soil, the calculated movement of the foundation is only a little too high compared with the absolute maximum measured at the building.

Fig. 20. Stress-strain paths around a pier foundation (Justo et al., 1985a)

Fig. 21. Foundation plan and layout of levelling marks of building at El Arakal (Justo et al., 1985a)
The use of the finite element method with the soaking-under-loading curve improves this result. The best result is obtained if, in the finite element method, the measured movement of the free soil profile is used as an input.

This finite element method has also been applied to the calculation of the stresses and deflections of the 10 cm-thick concrete lining of a canal in the province of Cordoba (Spain). The canal has a parabolic section, a length of 20 km, a discharge of 50 m³/s, and lies on expansive clay.

Depending on soil type it was decided to substitute, in some sections, 1-2 m of thickness of expansive clay by a non-expansive silty sand.

Figure 25 shows a detail of the grading machine. Figure 26 shows the concreting machine, and figure 27 a detail of it. Figure 28 shows the filling of the joints with bitumen. Figure 29 shows the canal finished. Up to the moment not a single fissure has appeared.

8. GEOTECHNICAL MAPPING OF EXPANSIVE SOILS

It may be said that the trials to establish maps of expansive soils are in a completely preliminary stage, due to the complexity of the theme.

The following criteria should be followed to establish the risk degrees: climatic, mineralogical, soil profile and depth of water table.

Table 7 shows some trials to establish these maps.

Neither Tourtelot nor Kron & Blossom have considered climatic criteria, which is a defect in his maps, otherwise very useful.

On the other hand, Kron & Blossom indicate a correspondence between his maps and the climatic map of the United States. Something similar is said in Australia.
Fig. 23. Vertical stresses in a vertical section for a single pier. El Aralal (Delgado, 1986)

Table 6

Measured and calculated heaves by methods based upon flooding the sample in the oedometer and by the finite element method (FEM)

<table>
<thead>
<tr>
<th>Method</th>
<th>Pier heave cm</th>
<th>Heave of free soil profile cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>McDowell's</td>
<td>0.0</td>
<td>2.2</td>
</tr>
<tr>
<td>Double oedometer</td>
<td>19.2</td>
<td>31.0</td>
</tr>
<tr>
<td>Direct (general)</td>
<td>7.0</td>
<td>16.0</td>
</tr>
<tr>
<td>Direct (simplified)</td>
<td>6.9</td>
<td>17.0</td>
</tr>
<tr>
<td>FEM with unit heave from direct general method</td>
<td>5.7-6.0</td>
<td>15.7-16.0</td>
</tr>
<tr>
<td>FEM with unit heave measured during period Nov.81-Apr.84</td>
<td>3.4-4.9</td>
<td>7.0</td>
</tr>
<tr>
<td>Measured in situ (one-year maximum)</td>
<td>3.6</td>
<td>6.1</td>
</tr>
<tr>
<td>Measured in situ (absolute maximum)</td>
<td>5.5</td>
<td>(9.3)</td>
</tr>
</tbody>
</table>
Fig. 24. Monthly rainfall and variation of levels with time. El Aranal - (Justo et al., 1985b)

Fig. 25. Canal Genil-Cabra. Grading machine (Courtesy of the Confederación Hidrográfica del Guadalquivir)

Fig. 26. Canal Genil-Cabra. Concrete machine (Courtesy of the Confederación Hidrográfica del Guadalquivir)
Fig. 27. Canal Genil-Cabra. Concrete
ing machine (Courtesy of the Conference Hidrográfica del Guadalquivir)

Fig. 28. Canal Genil-Cabra. Filling of joints with bitumen (Courtesy of the Conference Hidrográfica del Guadalquivir)

Fig. 29. Canal Genil-Cabra finished (Courtesy of the Conference Hidrográfica del Guadalquivir)
Table 7. SOME TRIALS FOR ESTABLISHING GENERAL GEOTECHNICAL MAPS OF EXPANSIVE SOILS

- Touraiot (1973)
  Map of montmorillonite outcrops at the United States
- Krlion and Slosson (1980)
  Map of expansive soils of the United States based upon "Coale" and mapped montmorillonite
- Cedex (Spain)
  Map of expansive clays of Spain (being printed)
- Climatic maps

At this conference

- Canuti et al. (1986)
  Map of expansive clays of Addis-Ababa (Ethiopia)
  Pedological classification and thickness
- Peng Da-Tien (1986a & b)
  China & Guanzhi (China)
  Mainly genetic classification. Somewhat age & mineral type. Not mapped in his papers

Figure 30 shows a climatic map of Spain based on Thornthwaite Index. Expansive clay problems occur in dry-subhumid and semi-arid areas.
A climatic map of Australia has been produced by Gentilli in 1948 (Aitchison and Richards, 1965). Most expansive clay problems occur in sub-humid and semi-arid areas.
Figure 31 shows a sector of the Addis-Ababa map presented by Canuti et al. at this conference.

Fig. 30. Climatic map of Spain according to Thornthwaite Index (Rodríguez Ortiz, 1975)
Fig. 31. Extract from the map of potentially swelling soils in the Addis Ababa metropolitan area. Potential swelling hazard classes (Canuti et al., 1986)

9. REFERENCES


Deformation parameters of macroporous loess soils
Les paramètres de déformation des loess macroporousses

D. Milović, Yugoslavia

ABSTRACT: Foundation loess soil is in some cases very sensitive to the additional settlement. Generally, this settlement is caused by the penetration of water into the loess soil and can be very dangerous for the structure. In this study, the results of the laboratory investigations of the undisturbed loess samples are presented. All examined samples were cut from blocks in pits. On the basis of the results obtained on a relatively large number of samples (about 450), a correlation between the undisturbed compression strength and the modulus of linear deformation has been shown. The results of the one-dimensional consolidation tests show that the modulus of linear compression (oedometer modulus) depends on the natural water content and on the dry density. On the basis of the results obtained, a correlation between the oedometer modulus and the initial dry density has been shown. In order to determine the sensitivity of loess to the additional settlement due to wetting or saturation, a certain number of the consolidation tests has been performed. After consolidation under a vertical stress $\sigma_v$, the sample was saturated. In such a way it was possible to determine the value of the specific coefficient of settlement in function on the initial water content and on the applied vertical stress $\sigma_v$. Using the results of the investigations mentioned above, it is possible to calculate the settlement for natural water content and also the potential settlement which could occur due to additional saturation of loess subsoil.

RESUME: Le sol loessique dans certains cas est très sensible au tassement additionnel. Généralement, ce tassement est provoqué par la pénétration de l'eau dans la couche de loess et habituellement est très dangereux pour la structure. Dans cette étude, on a montré des résultats des essais de laboratoire effectués sur des échantillons qui ont été prélevés dans des blocs dans les puits. À la base des résultats obtenus sur un nombre d'échantillons assez élevé (environ 450), la corrélation entre plusieurs paramètres a été établie. Grâce aux résultats, nombreux des essais de compression simple, on a trouvé une corrélation reliant les valeurs de résistance à la compression simple et des modules de déformation linéaire. Les résultats des essais de consolidation unidimensionnel montrent que le module oedémétrique dépend de la teneur en eau et de la densité initiale. Pour déterminer la sensibilité du loess au tassement additionnel du à la saturation, un certain nombre d'essais a été fait. Dans cette série d'essais, les échantillons ont été saturés après la consolidation sous la charge verticale $\sigma_v$. Dans cette manière il était possible de déterminer les valeurs de coefficient de tassement en fonction de la teneur en eau initiale et de la charge verticale appliquée. En utilisant les résultats des essais mentionnés ci-dessus, il est possible de calculer le tassement qui correspond à l'humidité naturelle et à la fois le tassement qui pourrait se produire due à la saturation additionnelle du sol loessique.

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1 INTRODUCTION

Loess is a wind deposit sediment transported from the flood plains of glacial rivers. The natural undisturbed landloess is a loose open structured macroporous soil composed of silt particles separated by clay coatings or aggregates of clay particles (Gibbs and Holland 1960; Larionov 1965).

Large areas of the earth’s surface are covered with loess. This kind of soil is considered to be unstable as a foundation material because of potential for large settlement. In some cases loess soil goes through a great decrease of volume upon wetting and the collapse of internal structure occurs when the stresses between particles exceed the bond strength provided by the clay coating (Holtz and Gibbs 1951).

The comprehensive state of the art report on collapsing soils are by Northey (1969) and review of geotechnical investigations of loess is by Lutgenegger et al (1979).

Loess deposits cover many parts of Yugoslavia. Difficulties with building on loess indicate that the behaviour of loess under load depends to a very large extent on the initial water content and the initial dry density. Experience gained during past decades shows that loess undergoes structural collapse and subsidence due to saturation if the initial dry density is low. In these cases, especially if the change of water content is not homogeneous, the differential settlement occurs and structures can be severely damaged.

In order to correlate some important parameters of loess which govern the shear and deformation behaviour, numerous laboratory tests were carried out.

2 DESCRIPTION OF LOESS

Loess in Yugoslavia is composed of fine sand silt and clay particles. Typical grain size distribution curves are shown in Fig. 1. Atterberg limits tests revealed that the examined loess would be classified as silt or clay of low to medium plasticity (CL-CL).

![Grain size distribution curves and Atterberg limits of loess soil](image)

Figure 1. Grain size distribution curves and Atterberg limits of loess soil.

Differential thermal analyses and x-ray diffraction show that the main constituents of Yugoslav loess are quartz, feldspar and calcite. Montmorillonite is the major cementing constituent in the most of loess deposits.

3 UNCONFINED COMPRESSION TEST RESULTS

On the undisturbed loess samples with various values of the initial dry density numerous laborat
ory unconfined compression tests have been carried out. Several studies concerning the effect of mechanical disturbance on the shear and deformation properties of loess (Milovic 1971) indicate that the undisturbed loess obtained from hard carved blocks removed from the test pits can provide samples of good quality. For that reason, all tests have been performed on the samples cut from blocks.

The unconfined compression tests were conducted on groups with the same water content. Some typical results are shown in Figs. 2 and 3. So, in Fig. 2 is shown the relationship between the unconfined compression strength $q_u$ and the water content $w$, for the undisturbed loess samples of dry density varying between $\gamma_d = 14.6-14.8 \text{kN/m}^3$.

![Graph showing unconfined compression strength vs. water content](image)

Figure 2. Relationship between the unconfined compression strength $q_u$ and water content $w$

As shown in Fig. 2, the unconfined compression strength decreases with the increase of water content. In Fig. 3 is shown the relationship between the unconfined compression strength $q_u$ and the dry density $\gamma_d$, for samples with the water content $w = 22-24\%$.

The curve shown in Fig. 3 indicates the increase of the unconfined compression strength with the increase of dry density.

The values of the modulus of deformation $E$, deduced from the unconfined compression tests, are shown in Fig. 4.

On the basis of numerous test results the following simple relationship between the modulus of deformation $E$ and the unconfined compression strength $q_u$ may be suggested:

$$E = 100q_u$$  \hspace{1cm} (1)

A certain number of the unconfined compression tests has been performed on the undisturbed samples cut both in vertical and in horizontal direction. The degree of transversal anisotropy $n = \frac{E_v}{E_h}$, where $E_v$ and $E_h$ are moduli in vertical and horizontal direction, respectively, ranged between $n = 1.30$ and $n = 1.62$. 

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4 CONSOLIDATION TEST RESULTS

In the study of foundation problems on loess it is important to take into consideration the collapse potential and to determine the amount of collapse that may occur. The collapse potential, suggested by Knight (1963) is defined as:

$$CP = \frac{A_1 E_c}{\gamma E_s}$$

............... (2)
where \( H_0 \) is the height upon wetting and \( H o \) is the initial height.

The change of the void ratio during the process of subsidence can be measured by one dimensional vertical compression of the undisturbed loess samples which are laterally restrained. As a measure of the degree of subsidence the following expression can be used:

\[
i_m = \frac{e_n - e'_n}{1 + e_n} \times \frac{\Delta e_n}{1 + e_n}
\]

where \( e_n \) is the void ratio before flooding at the vertical stress \( \sigma_n \) and \( e'_n \) is the void ratio at the end of subsidence under the same vertical stress \( \sigma'_n \), as shown in Fig.5.

Figure 5. Typical subsidence curve for loess

The values of the coefficient of subsidence \( i_m \) have been evaluated on the undisturbed loess samples with various initial dry densities. During the consolidation tests samples were saturated under the vertical stress \( \sigma = 100 \text{ kN/m}^2 \), \( \sigma = 200 \text{ kN/m}^2 \) and \( \sigma = 300 \text{ kN/m}^2 \). The first group consisted of samples with the same initial dry density \( \gamma_d = 12.5-13.0 \text{ kN/m}^3 \) but with various values of water content. Consolidation test results and values of the coefficients \( i_m \) for the first group of samples are shown in Figs. 5 and 7.

The another groups consisted of samples with dry density \( \gamma_d = 13.5-14.0 \text{ kN/m}^3 \), \( \gamma_d = 14.0-14.5 \text{ kN/m}^3 \) and \( \gamma_d = 15.0-15.5 \text{ kN/m}^3 \) and with various water contents. Some typical results of the consolidation tests and the values of the coefficients \( i_m \) are shown in Figs. 8 and 9.

As shown in Figures 6-9, the values of modulus of compressibility \( E_{oe} \), determined by one dimensional consolidation tests, depend to a large extent on the initial dry density and initial water content. The values of the coefficient of subsidence \( i_m \), which expresses the sensitivity of loess to the additional settlement due to saturation, depends also on the initial dry density and initial water content.

The magnitude of settlement of loess foundation soil is a function of these two parameters, and can be calculated by the following expression:

\[
\delta = \sum_i \frac{\Delta \sigma_z}{E_{oe}} H_i + \sum_i i_m H_i
\]

where \( H_i \) is the thickness of a layer and \( \sigma_z \) is the vertical stress acting in the middle of a layer.
Figure 6. Consolidation test results
1. $w = 4.1\%$
2. $w = 9.4\%$
3. $w = 13.2\%$
4. $w = 16.5\%$
5. $w = 19.6\%$
6. $w = 22.6\%$
7. $w = 28.0\%$
8. saturated

Figure 7. Coefficients $i_m$
1. $w = 13.2\%$
2. $w = 16.5\%$
3. $w = 19.6\%$
4. $w = 22.6\%$
5. saturated
Figure 8. Consolidation test results
1  \( w = 5.6\% \)   2  \( w = 10.6\% \)   3  \( w = 13.6\% \)
4  \( w = 16.2\% \)   5  \( w = 22.0\% \)   6  saturated

Figure 9. Coefficients \( i_m \)
1  \( w = 13.8\% \)   2  \( w = 16.1\% \)   3  \( w = 20.0\% \)
4  \( w = 22.0\% \)   5  saturated

5 CONCLUSIONS

On the basis of numerous laboratory test results one may conclude:
- deformation and shear parameters of Ioannos soils can be evaluated with high precision if the samples are taken only from blocks in pits;
- deformation parameters \( E_{oe} \) and \( i_m \) which govern the value of total settlement, depend to a large extent on the initial dry density and initial water content;
- Loess deposits exhibit anisotropic properties, what will be taken into account in the calculation of stress distribution.

REFERENCES


Residual soils as construction materials for earth dams
L’utilisation des sols residuels comme material de construction pour barrages en terre

R. Genevois & A. Prestininzi, Earth Science Department, University of Rome

ABSTRACT: The paper reports the results of compaction, permeability and shear strength tests obtained on residual soils samples drawn in the area of Souani (Algeria). The light-red coloured samples belong to some layers intercalated among a red soil deposit to be used for the construction of an earth dam. Early laboratory tests results strengthened the field estimate that the materials were of poor quality, since high plasticity indexes and low dry densities turned out. Nevertheless both "red" and "light-red soils" showed fairly like values of undrained and drained shear strengths. "Light-red soils" permeability values resulted higher than those of "red soils", clashing then with the higher values of plasticity indexes. The mineralogical composition, showing the presence of large amounts of calcite, and the poor grading of "light-red soils" seem to justify the observed behaviour.

RESUME: Cet article rapporte les résultats obtenus avec des essais de compaction de perméabilité et de résistance au cisaillement sur des échantillons de terres résiduelles provenant de Souani (Algérie) et prélevés dans des niveaux d’un dépôt de "terres rouges", dont l’exploitation était prévue pour la construction d’un barrage en terre. A première vue les caractéristiques physico-mécaniques étaient médiocres et cela était confirmé par les essais Proctor à différentes énergies de compaction, dont les résultats donnaient hautes valeurs de plasticité et basses valeurs de densité sèche. Pourtant les résistances au cisaillement, drainées et non drainées, étaient du même ordre de grandeur de celles obtenues pour les "terres rouges". Les valeurs de perméabilité finalement étaient plus hautes en comparaison des "terres rouges" et cela en contraste avec les limites de Atterberg. La composition minéralogique, qui a montré la présence de grandes quantités de calcite, et les granulométries, qui ont donné des coefficients d’uniformité très bas, semblent justifier le comportement observé.

1 INTRODUCTION

Some of the geotechnical results, obtained on the occasion of the feasibility study of an earth dam, are reported in this paper with the aim to show the weight of mineralogical composition on soil behaviour.

The studied reservoir is located near the village of Souani (Fig.1) at the western edge of the Taman Mountain Chain (Western Algeria).

In the preliminary stage of the design, the soil exploration pointed out, besides the existence of rather difficult conditions from both geological and hydrogeological point of view (Genevois & Prestininzi, 1980), the presence, in the only soils suitable as construction materials, of some intercalations of soils with seemingly poor physical and mechanical characteristics. By means of many core boings, these soils resulted to be present in beds and lenses with a general sub-horizon-
Fig. 1. GEOLOGICAL SKETCH

a- Continental formations (Quaternary); b- Old terraces and their edges (Pliocene-Quaternary); c- massive dark-grey limestones (Jurassic); d- green marls and marly limestones (Jurassic); e- schists and quartzites (Paleozoic); f- faults; g- probable faults; h- dip and strike of beds; i- section lines; l- dams sites.
Fig. 2. LITHOLOGICAL SECTIONS

a- Silty clayey continental formations; b- gravelly continental formations; c- silts and clays lenses; d- old talus debris; e- tuffs and basalt flows; f- clays with gypsum; g- massive lime stones; h- halite and carnallite; i- probable faults; j- boreholes.
tual attitude: this fact and the mining conditions did not warrant a convenient selection of the materials regarded suitable for the earth dam.

A number of laboratory tests have been then scheduled with the aim of a better knowledge of their geotechnical characteristics related, particularly, to the change of the water content and the compaction degree.

2 GEOLOGICAL OUTLINE

The regional bed-rock is formed by paleozoic metamorphic rocks overlaid by a transgressive sequence continuous from Trias to Upper Cretaceous. Metamorphic rocks outcrop south of the reservoir site, but they have not been found in the executed drillings.

Outcropping rocks consist of jurassic marls, marly limestones and limestones, sometimes dolomitic, dark-grey coloured and with some paleokarstic marks. The overburden consists of recent continental formations, quite complex and differentiated from both lithological and granulometric point of view.

Rotary drillings and geoelectric survey allowed to know the soils sequence up to a depth of about 150 m.

Geological sections point out, below the continental formations, the presence of a triassic salt dome, made by halite and carnallite, that has created several hydrogeological problems and whose ascent has been conditioned by N-S and NE-SW fault system (Genevois & Prestininzi, 1980). Figure 2.

3 CONSTRUCTION MATERIALS

The construction materials consist, essentially, of residual soils, red-brown coloured, characterized by quite variable grain size distributions.

"Light-red soils" consist of the same residual soils with high percentages of generally pulverulent calcareous encrustations. They were found as intercalations with a maximum thickness of about one meter at a depth of 6 m. The X-ray diffraction analyses, executed both on the whole sample and on the percentage finer than 2 μ, pointed out decreasing amounts of quartz, calcite and, subordinately, clay minerals such as montmorillonite, illite and chlorite.

Consequently, geotechnical tests have been programmed to evaluate:
1. their behaviour related to different compaction energies;
2. their undrained strength at the different saturation degrees obtained in the compaction tests;
3. their drained strength once compacted at optimum proctor values and saturated by a 200 kPa back-pressure, keeping constant the value of the dry density;
4. the permeability coefficient of samples compacted at optimum proctor values.

Mean values of physical and compaction characteristics of the considered soils are in Table 1, while their grain size distribution curves are showed in Fig.3.

<table>
<thead>
<tr>
<th>TABLE 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOIL TYPE</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>RED SOIL</td>
</tr>
<tr>
<td>LIGHT-RED SOIL</td>
</tr>
</tbody>
</table>
4 COMPACTION TESTS

Three series of tests have been executed in a small mould (sizes: 10cm x 20 cm²) with different compaction energies, obtained changing the number of blows for each layer and keeping constant the drop height of the rammer. The same specimen, obtained for the determination of the compaction characteristics, have been subsequently used for shear strength and permeability tests.

The results of compaction tests are showed in Fig.4 and the data of the relative applied energies are in Table 2, considering that the energy corresponding to test B is that of a Modified A.A.S.H.O. one.

The obtained results show that, increasing the compaction energy, the maximum dry density increases while the optimum water content decreases, but they never reach the values obtained with a Modified A.A.S.H.O. test on "red soils" (Tab.1).

Both "red soils" and "light-red soils" showed moderate swelling values after compaction, being respectively as low as 2.0-2.5% and 3.0-3.5%.

5 SHEAR STRENGTH

The mechanical behaviour of "light-red soils" has been analyzed both singling out the undrained strength change with the different densities of proctor specimen, corresponding then to different saturation conditions, and measuring the drained strength of samples compacted at optimum water content and then saturated.

The unconfined strength values ($\sigma_f$) as a function of the saturation degree (Fig.5) show that:

Fig. 3. Grain size distribution curves. R-S: red soils fuse. LR-S: light-red soils.

<table>
<thead>
<tr>
<th>TEST</th>
<th>LAYER</th>
<th>BLOWS</th>
<th>$\gamma_{max}$ (KN/m³)</th>
<th>W opt. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5</td>
<td>20</td>
<td>16.76</td>
<td>19.5</td>
</tr>
<tr>
<td>B</td>
<td>5</td>
<td>33</td>
<td>16.96</td>
<td>19.0</td>
</tr>
<tr>
<td>C</td>
<td>5</td>
<td>50</td>
<td>17.25</td>
<td>18.0</td>
</tr>
</tbody>
</table>
1. the undrained strength values increase, due to the increase of the compaction energy, are quite slight for saturation degrees near to 100%;
2. the maximum strength values are obtained with water content and dry densities lower than those corresponding to the optimum proctor values.

This latter observation is probably linked with the high plasticity of "light-red soils", realizing that maximum shear strengths are always found with water contents lower than the plastic limit of the material.

The CU triaxial tests (one example is shown in Fig. 6) have been carried out on...

Fig. 6. Diagrams of a CU triaxial test on "light-red soil" compacted to optimum modified proctor values. Not saturated specimen.
samples compacted to the optimum values of Modified A.A.S.H.O. The specimen were then not saturated, resulting the saturation degree in the range of 84-91%.

The results of such tests are consistent with those of unconfined tests, the shear strength values being anyhow quite high.

The shear strength of fully saturated samples has been measured on samples compacted at optimum proctor (Modified A.A.S.H.O.) and then saturated by a back-pressure of 200 kPa, applied for a convenient period of time. The results of one of these tests are shown in Fig. 7.

![Diagram of a CD triaxial test on "light-red soil" compacted to optimum Modified A.A.S.H.O. values and saturated.](image)

**Fig. 7.** Diagrams of a CD triaxial test on "light-red soil" compacted to optimum Modified A.A.S.H.O. values and saturated.

![Diagram of permeability tests on "red soils" and "light-red soils" (K curve), whose values are on the right y-axis.](image)

**Fig. 8.** Permeability tests on "red soils" and "light-red soils" (K curve), whose values are on the right y-axis.

6 PERMEABILITY

Permeability tests have been carried out in triaxial cells on cylindrical samples (sizes: 6 cm x 20 cm²) compacted to optimum proctor (Modified A.A.S.H.O.) and then saturated by a back-pressure of 200 kPa. The results of some representative tests are shown in Fig. 8. The permeability coefficients of the "light-red soils" turned out 10-20 times those of "red-soils", but this result is not strengthened by the higher plasticity indexes of the former ones.

This behavior seems to be related
both to the mineralogical composition, that is to the presence of a great amount of calcite, and to the grain size distribution, that is to the small values of the coefficient of uniformity.

7 CONCLUSION

The results of laboratory tests, carried out on samples of a residual soil, pointed out a particular behaviour of some light-red intercalations present in the altogether red soil deposit.

The presence of large amount of calcite produces relatively high values of plasticity but low swellings, as it was to be expected. The low sensitivity of these materials to the variations of the compaction energy is to be linked, besides the presence of calcite, to their poor grading. As a consequence, the maximum dry densities in compaction tests result lower than those obtained with the same tests on "red soils".

Nevertheless shear strength parameters of the two types of soils don't turn out significantly different: the undrained strengths provided, on the average, high values, partly sensitive to compaction energy variations only for saturation degrees lower than 80%.

The shear strength parameters for drained conditions and whole saturation of specimen are lower than the previous one, but they don't differ essentially from those of "red soils".

The only coefficient of permeability is significantly different: the "light-red soils" come out more permeable than the "red soils", but they still remain in the range of soils considered impervious.

The final issue of the tests carried out is that no substantial differences are to be expected in the mechanical behaviour of the soils dug from the suggested quarry and used for the dam construction.

REFERENCES


Shallow foundations in lateritic soils
Fondations peu profondes sur sols lateritiques

J. Militsky & R. Davison Dias, UFRGS, Brasil

ABSTRACT: The aim of this paper is to present aspects of the lateritic soils such as shear strength, methods of analysis, design and construction that are peculiar to this soils.

To solve a problem of a shallow foundation, if analytical methods are chosen, it is necessary to obtain compressibility and shear strength properties. Typical values of lateritic soils from South Brazil are presented, with the effect of soaking on these values.

For three soils from Paraná State it is presented a comparison between values of bearing capacity obtained using 'general shear' and 'local shear' approach using test specimens under natural moisture content and soaked, with the virtual pre-consolidations pressures.

Different aspects of foundation practice is presented, considering the special behaviour of lateritic soils.

RESUME: L'objectif de cette publication est de présenter des caractéristiques de la reconnaissance, propriétés du sol, méthode d'analyse, calcul et construction des foundations superficielles qui sont typiques des sols lateritiques.

Pour résoudre un problème de fondation, si on veut employer des méthodes analytiques, il est nécessaire d'obtenir les paramètres de compressibilité et résistance au cisaillement. Valeurs typiques de sols lateritiques au sud du Brésil et des effets de les inondations en ces valeurs, sont présentées.

Pour trois sols lateritiques de l'Etat du Paraná, il est présenté une comparaison entre les valeurs de la capacité portante obtenue en utilisant paramètres de résistance ou cisaillement de sol intact, sous des conditions naturelles et inondées.

Des différentes aspects de la construction des fondations superficielles originés par la conduite spéciale des sols lateritiques sont présentés.

1 ANALYSIS

Analysis of a shallow foundation consists on the study of two problems: deformation under load and failure. To tackle the problem of deformation (settlement analysis) and failure, the available analytical procedures are used, and soil properties must be obtained.

There are some aspects of shear strength and compressibility of lateritic soils that are different if a comparison is made with the sedimentary ones.

- Shear strength: Shear strength of lateritic soils is related to soil structure, chemical and mineralogical composition and degree of saturation. For sedimentary materials, key factors are percentage of fines, pre-consolidation, relative density, etc.

Data on typical values of shear strength of natural lateritic soils are scarce. Tables 1 and 2 presents some values of soils from Rio Grande do Sul State (South Brazil).

On the current applications, data on shear strength are obtained from laboratory test usually direct shear or triaxial on samples taken from blocks or special samples. Special care must be taken with moisture variation during the process of transport.

Variation on the degree of saturation affects shear strength (see Tables 1 and 2), and Fig. 1 to 3, specially the "cohesion intercept" (Dias and Oshling, 1983, Militsky, 1985). Recently (Fredlund et al., 1978, He and Fredlund, 1983) the suction has been included in a general formula of shear strength of partially saturated soils.

- Compressibility: Determination of soil compressibility can be made using: a) laboratory tests such as confined compression (pneometer) and triaxial, and b) field tests: plate tests, pressuremeter (special).

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### Table 1. Strength parameters, virtual overconsolidation pressure and allowable soil pressure of Paraná States Soils.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Bedrock origin</th>
<th>Test condition</th>
<th>C (kN/m²)</th>
<th>Virtual overconsolidation pressure qₜ (kN/m²)</th>
<th>Bearing capacity/3 (General shear) Qc (kN/m²)</th>
<th>Bearing capacity/3 (Local shear) Qc' (kN/m²)</th>
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</thead>
<tbody>
<tr>
<td>Maringá</td>
<td>Sandstone</td>
<td>Natural</td>
<td>8.5</td>
<td>32</td>
<td>50</td>
<td>291</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soaked</td>
<td>4.5</td>
<td>26</td>
<td>40</td>
<td>153</td>
</tr>
<tr>
<td>Goiove</td>
<td>Sandstone</td>
<td>Natural</td>
<td>17.0</td>
<td>24</td>
<td>80</td>
<td>186</td>
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<tr>
<td></td>
<td></td>
<td>Soaked</td>
<td>5.8</td>
<td>25</td>
<td>75</td>
<td>125</td>
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<tr>
<td>Mandaguari</td>
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<td>Natural</td>
<td>12</td>
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<td></td>
<td></td>
<td>Soaked</td>
<td>5</td>
<td>26</td>
<td>35</td>
<td>114</td>
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### Table 2. Strength parameters, virtual overconsolidation pressure and allowable soil pressure of Rio Grande do Sul State Soils.

<table>
<thead>
<tr>
<th>Other lateritic soils</th>
<th>Bedrock origin</th>
<th>Test consolidation</th>
<th>C (kN/m²)</th>
<th>Virtual overconsolidation pressure qₜ (kN/m²)</th>
<th>Bearing capacity/3 (General shear) Qc (kN/m²)</th>
<th>Bearing capacity/3 (Local shear) Qc' (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravataí - RS</td>
<td>Oenozoic Sediments</td>
<td>Natural</td>
<td>24</td>
<td>33</td>
<td>150</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soaked</td>
<td>22</td>
<td>20</td>
<td>130</td>
<td>210</td>
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<tr>
<td>Cachoeirinha - RS</td>
<td>Oenozoic Sediments</td>
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<td>38</td>
<td>12</td>
<td>180</td>
<td>954</td>
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<td>314</td>
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<td>Soaked</td>
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<tr>
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<td>Soaked</td>
<td>23</td>
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<td>Independência - POA/RS</td>
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<td>30</td>
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<td>110</td>
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</table>

a. Factors affecting compressibility: For sedimentary materials there are relations between Atterberg limits and compressibility (e₀). For lateritic soils such relations showed large dispersion, being useless for practical applications.

Apparently the natural void ratio (e₀) present a better correlation with compressibility (Lacerda, 1985), for applications such correlation must be established for materials with the same composition and origin and will have validity for these materials.

Regional studies (Gehling, 1981, 1982, 1983 and Dias et al. 1982, 1983) in South Brazil suggest that soil structure has considerably influence on compressibility.
Colapsibility: Collapsible soils are defined as "material that present a radical change in structure and large variation (reduction) in volume due to saturation, with or without external loading". Collapse occurs due to a particle re-arrangement, with volume variation, due to the increase in the saturation degree, being dependent on:
- soil structure partially saturated
- tensions existing able to develop collapse
- failure of cementing boundaries
Vargas (1974) studied the problem for lateritic soils, using test specimens consolidated under natural moisture for different pressures. When deformation stopped, test specimens where inundated. Additional deformation appeared due to soil saturation, bigger for small pressures and reducing as the value of external load increases. From a certain value of pressure collapse disappears. According to the author possibly there is a certain pressure that destroys weak bounds, test specimens under larger pressures are unaffected. The effect of saturation is not related to cement dissolution or capillary tension forces on these materials.
Superficial porous soils are still affected by collapsibility, even after being under heavy rain for large periods. As they are permeable, water from rain does not saturate the material, just when that occurs the behaviour is affected.

1.1 Methods to predict settlement and bearing capacity

On current applications, settlement can be estimated using the unidimensional model (consolidation). Willner et al., 1982, compared predicted and measured values, obtaining variations between 30 and 115% (predicted/real). Other references on settlement prediction can be found in Williams, 75, Barksdale et al., 75, Sowers and Glen, 65, Martin, 1977.
It has been recognized that the use of the virtual pre-consolidation pressure as the contact pressure of shallow foundations leads to small and allowable settlement (Vargas, 1953, 1979).
To compare the values of bearing capacity obtained using the classical Terzaghi approach for "general shear" and "local shear" (with a factor of safety equal to 3 to failure) and the virtual pre-consolidation pressure in oedometer tests performed in block samples, three soils from Paraná State (South Brazil) were chosen.
Typical SPT values are equal or lower than 3. Soil properties are presented in Table 3 and 4. Special features of these soils are: the material identified as Mandaguari presents a high void ratio and a low degree of saturation. Grain size distribution was different, but with similar macro-structure. Shear strength obtained in direct shear tests showed different values for test specimens at natural moisture content. When soaked there was a reduction in shear strength and a similar value was obtained.
Values of bearing capacity (with a factor of safety of 3 to failure) obtained under condition of "general shear" and shear strength parameters at natural moisture content where higher than the general over consolidation pressure, leading probably to unacceptable settlement. When "local shear" was considered, with values of shear strength at soaked conditions the same order of magnitude of the virtual overconsolidation pressure was found.
The same exercise was made for soils from Rio Grande do Sul State (Table 1) with known undisturbed properties. With a similar structure of Paraná soils, the only material of the list is referred as Carazinho (massive macro structure). The other soils present moderate to strongly developed macro structure. Typical SPT values of these soils are on the range of 7 to 10 (Carazinho presents SPT = 3).

---

### Table 3. Index characteristics of Paraná States Soils

<table>
<thead>
<tr>
<th>Grain size distribution</th>
<th>W_L</th>
<th>W_p</th>
<th>I_p</th>
<th>θ</th>
<th>W_h</th>
<th>S</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without dispersing solutions</td>
<td>Gravel</td>
<td>Sand</td>
<td>Coarse Silt</td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
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<tr>
<td>Maringá</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>77</td>
<td>14</td>
<td>19</td>
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<tr>
<td>Goioere</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>63</td>
<td>7</td>
<td>29</td>
</tr>
<tr>
<td>Mandaguari</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>18</td>
<td>24</td>
<td>30</td>
</tr>
</tbody>
</table>

*S Mean values*
Table 4. Unit weights and void ratio of Paraná State soils

<table>
<thead>
<tr>
<th></th>
<th>γ (KN/m³)</th>
<th>γd (KN/m²)</th>
<th>e</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Mean values</td>
<td>Range</td>
<td>Mean Values</td>
</tr>
<tr>
<td>Maringá</td>
<td>15.0</td>
<td>14.6-15.3</td>
<td>13.7</td>
</tr>
<tr>
<td>Goioere</td>
<td>16.5</td>
<td>15.2-18.2</td>
<td>14.0</td>
</tr>
<tr>
<td>Mandaguari</td>
<td>12.8</td>
<td>12.4-13.1</td>
<td>9.5</td>
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</table>

**Fig. 1.** Properties of three lateritic soils from Paraná State, South Brazil - Shear strength obtained with direct shear apparatus and the soaking effect.
Fig. 2. Properties of three lateritic soils from Paraná State, South Brazil - Oedometer test at natural condition and soaking effect.

Fig. 3. A typical collapsibility test.
All values obtained with general shear were higher than the virtual overconsolidation pressure ($q_c$). Local shear must be considered always. Shear strength at natural moisture content leads to values of bearing capacity obtained with local shear closer to $q_c$. On average, "soaked shear strength" presents safer values of bearing capacity.

2 FOUNDATION PRACTICE

Some aspects of the practice of shallow foundations on lateritic soils will follow:

- **Minimum depth**: there is a considerable number of cases of problems in current practice due to the constructions of foundations at depths where the foundation soil can be affected by seasonal changes in moisture. Practice indicates 1.00 m as a reasonable value of minimum depth.

- **Remoulding**: in some cases the practice of excavation of all foundation area, and the exposure of the foundation soil to long periods when rain can occur or internal traffic can damage soil structure it is not unusual remoulding of the superficial area. According to the Brazilian Code of Practice (NBR-6122/1984). A 10 cm layer of concrete must be poured immediately after excavation covering all the area.

- **Pre-loading**: due to the conditions of high permeability and partial saturation, settlement occurs immediately to load application. A common practice to reduce settlement in large areas is the use of pre-loading (De Mello, 1980; Cenpolina, Mello and Oliveira, 1979; Cenpolina and Ruppolo, 1982).

- **Colapsibility**: this problem is present in cases of pipe failure, leakage of reservoirs, long buildings without superficial drainage or pavement close to the walls, like silos and other industrial buildings. According to Berberian (1982), in Brasilia, the variation of the water table due to the Paraná Dam construction is responsible for the problems 10 years after the construction of buildings. Another case reported in the literature is the one by Mello, 1985, when settlement of both shallow and small diameter bored pile due to collapse are presented.

- **Compaction of soil below foundations**: lateritic soils when compacted present higher shear strength, lower compressibility and higher stability to the water action. In industrial buildings, such as deposits, silos, where the use of continuous foundations along the walls is an adequate solution, excavation and compaction can be used successfully. In some cases a small amount of cement is used as a stabilizer agent.

Brasilian practice of foundation construction on residual soils can be found in Golombek, 1985.

3 CONCLUSIONS

- Under laboratory conditions, soaking of a lateritic soils can reduce shear strength, but compressibility may remain unaffected.

- Soaking effect on shear strength of partially saturated soils can be explained by the reduction in suction forces.

- Some lateritic soils present colapsibility, as a result of an unstable structure.

- Comparison between allowable bearing pressure and virtual overconsolidation pressures for the lateritic soils studied showed that:
  a. the allowable bearing pressure obtained considering "general shear" is considerably higher than the virtual over consolidation pressure;
  b. "local shear" and soaked test specimen represents a better approach for soils with unstable structure, being the allowable bearing pressure of the same order of magnitude of the virtual overconsolidation pressure.

- Field tests and case records showed the effect of colapsibility on shallow foundations and small diameter bored piles.

- Lateritic clays present settlement immediately after load application (laboratory or field conditions).

**Recommendations**:

a) Laboratory
  - odometer tests give a realistic picture of soil behaviour under load. According to practice, foundation settlement for pressures up to the virtual overconsolidation pressure is small (and allowable for current structures);
- direct shear tests are simple to perform, the results can be used to study bearing capacity. Analysis must take into account unstable soil structures. Such materials must be analyzed using soaked samples and "local shear" condition.

b) Field
- the Standard Penetration Test (SPT) can be used an indication of soil structure stability

\[
\text{SPT} \leq 3 \quad \text{are typical of low } \phi_c
\]

\[
\text{SPT} = 6 \text{ to } 10 \quad \text{indicates stable structures}
\]

- as the SPT is not a measure of soil compressibility, empirical relationship between SPT results and allowable bearing capacity can be misleading;

- plate and pile loading tests are recommended as a way to improve the knowledge of soil behaviour under foundations loads.

REFERENCES


Foundation problems in nonconsolidated fill
Problèmes de fondations dans les remblais non consolidés

D. Milović, T. Todorović & M. Pavlović, Kosovoprojekt, Belgrade, Yugoslavia

ABSTRACT: In this Paper are presented the results of the laboratory and field investigations performed in the nonconsolidated fill of height approximately 30 m. In order to study the possibility of applying shallow foundations, several field load tests with square foundation of width \( B = 1.0 \text{ m} \) have been carried out. In the first group field load tests have been performed on the natural soil. In the second group of tests the soil was completely saturated. In the third group of field load tests the existing soil has been replaced by compacted gravel and in the last one the gravel layer has been reinforced by steel bars. On the basis of the registered values of settlement during field load tests, the ultimate bearing capacity has been determined by Van der Veen's and Mazurkiewicz's procedures. In such a way it was possible to compare the obtained values of the ultimate bearing capacity of soil and to confirm the favorable effect of gravel layer overlying soft soil.

RESUME: Dans cet article on a présenté des résultats des essais de laboratoire et des essais sur place, effectués dans un sol non consolidé d'épaisseur de 30 m environ. Pour examiner la possibilité d'appliquer les fondations peu profondes, plusieurs essais sur place avec une fondation carrée de coté \( B = 1.0 \text{ m} \) ont été effectués. Dans la première série les essais de chargement sur place ont été faits, le sol de fondation ayant la teneur en eau naturelle. Dans la deuxième série le sol de fondation était saturé. Dans la troisième série des essais de chargement le sol existant était excavé et remplacé par le gravier compacté et dans la dernière série la couche de gravier était renforcée avec des barres en acier. A la base des valeurs des tassements, enregistrées pendant les essais de chargement, on a appliqué la méthode de Van der Veen et celle de Mazurkiewicz pour déterminer la charge de rupture du sol de fondation. Dans cette manière il est possible de comparer les valeurs obtenues de la charge de rupture du sol et de confirmer l'effet favorable de couche graveleuse surmontant le sol mou.

1 INTRODUCTION

The ultimate bearing capacity and settlement are the most important problems when structures are founded in nonconsolidated fill. Such soils exhibit unfavorable shear and deformation parameters, thus creating very difficult foundation conditions.

In order to define the failure conditions produced by the load acting on the limited surface of the nonconsolidated soil and to study the improvement possibility of shear and deformation parameters, several field load test have been performed.
2 SOIL CHARACTERISTICS

The soil in nonconsolidated fill of height 30 m, according the liquid limit and plasticity index, can be classified as C1 clay of low plasticity. Grain size distribution curves show that there is 30% or more of clay and silt particles. The natural water content is about \( w = 20\% \), reaching the values of \( w = 35\% \) when saturated. The values of dry density are low, ranging between \( \rho_d = 12,9-13,5 \text{ kN/m}^2 \). The cone penetration tests, carried out in the field, show that the cone resistance until 20 m of depth is practically of constant value \( R_p = 1000 \text{ kN/m}^2 \), thus indicating very unfavorable shear and deformation characteristics.

3 FIELD LOAD TEST RESULTS

In the same test field all load tests have been carried out with the concrete square foundation of side \( B = 1,0 \text{ m} \), and at the depth foundation \( D_f = 1,0 \text{ m} \). By means of the hydraulic jack a controled pressure on the foundation was applied. The settlement of the soil under the foundation was registered by precise level instrument.

In the first group of field load tests (A) the soil was with natural water content. The relationship between the applied vertical load and settlement is shown in Fig. 1. Curve A is the average of three tests.

![Figure 1. Load-settlement curve; natural water content of soil](image)

The ultimate bearing capacity, determined by Mazurkiewicz's method is equal to \( q_f = 260 \text{ kN/m}^2 \) and by Van der Veen's procedure \( q_f = 250 \text{ kN/m}^2 \).

In the second group of field load tests (B) the foundation soil was saturated during several days. The average load settlement curve is shown in Fig. 2.

The ultimate bearing capacity, determined by two above mentioned methods, is equal to \( q_f = 270 \text{ kN/m}^2 \).

In order to improve the shear and deformation parameters of the foundation soil, the existing soil was replaced by the compacted gravel 1,0 m thick. The results of the field load tests (C) in two layer system are shown in Fig. 3.
The ultimate bearing capacity, obtained for this group of tests is equal to $q_u = 300 \text{ kN/m}^2$.

The effect of saturation of two layer system, where the upper layer was made of compacted gravel and the lower layer was natural soil, has been examined in the fourth group of tests (D). The average curve which represents load-deformation relationship is shown in Fig. 4.
The ultimate bearing capacity for this group of tests was about $q_f = 280 \text{ kN/m}^2$.

The last group of field load tests includes the two-layer system, where the upper layer was made by compacted gravel with steel bars reinforcement (E). Typical results of such tests are shown by load-settlement curve in Fig. 5.
The ultimate bearing capacity, obtained by these tests, is \( q_f = 350 \, \text{kN/m}^2 \).

4 ANALYSIS OF THE RESULTS

On the basis of the field load test results the values of the ultimate bearing capacity for each examined case were determined. For natural soil with natural water content the critical value of the load which can produce the failure is \( q_r = 250 \, \text{kN/m}^2 \). If the foundation soil is saturated the critical failure load is lower, having a value \( q_r = 210 \, \text{kN/m}^2 \). The value of the failure load increases if the nonconsolidated soil is replaced to some depth by compacted gravel. In this case the ultimate load reached the value \( q_r = 300 \, \text{kN/m}^2 \), and \( q_r = 280 \, \text{kN/m}^2 \) when the foundation soil was saturated. The reinforcement of the gravel layer has a very favorable effect, increasing the ultimate bearing capacity up to \( q_r = 350 \, \text{kN/m}^2 \).

For comparison, in Fig.6 are shown load-settlement curves for all examined cases.

![Diagram](image)

**Figure 6.** Load settlement curves

The curves shown in this figure clearly indicate beneficial effect of gravel layer beneath the foundation and particularly its reinforcement. It is of interest to note that the registered settlement under the load \( q = 150 \, \text{kN/m}^2 \) for saturated nonconsolidated soil is \( \delta = 10.5 \, \text{cm} \) whereas in the case with compacted gravel layer 1 m thick, the settlement was 2.8 cm only.

The deformation characteristics can also be determined on the basis of the results obtained from field load tests, using the theoretical solution for stresses and displacements produced by square foundation.
The determination of stresses and displacements, produced by a rectangular flexible foundation, was treated by several researchers (Marguerre, 1931; Poulos, 1967; Milović, 1974).

In this study the double trigonometric series have been used. The displacement functions for \( u \), \( v \) and \( w \) have the following form:

\[
\begin{align*}
   u &= \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} U_{mn} \sin \alpha x \cos \beta y \\
   v &= \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} V_{mn} \cos \alpha x \sin \beta y \\
   w &= \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} W_{mn} \cos \alpha x \cos \beta y
\end{align*}
\]

\[
\begin{align*}
   \epsilon_x &= A_{11} \delta_x + A_{12} \delta_y + A_{13} \delta_z \\
   \epsilon_y &= A_{12} \delta_x + A_{22} \delta_y + A_{23} \delta_z \\
   \epsilon_z &= A_{13} \delta_x + A_{23} \delta_y + A_{33} \delta_z \\
   \gamma_{xy} &= A_{56} \delta_x + A_{65} \delta_y \\
   \gamma_{xz} &= A_{55} \delta_x + A_{55} \delta_y \\
   \gamma_{xy} &= A_{55} \delta_x + A_{55} \delta_y
\end{align*}
\]

where:

\[
A_{11} = \frac{E (1-\mu)}{(1+\mu)(1-2\mu)} \quad A_{12} = \frac{D_{ij}}{(1+\mu)(1-2\mu)} \quad A_{55} = \frac{E}{2(1+\mu)}
\]

According to the obtained solutions for componental stresses and displacements, the dimensionless coefficient for the vertical displacement is equal to \( I_w = 0.72 \) (for \( \mu = 0.45 \) and for rigid foundation) and \( I_w = 0.31 \) (for \( \mu = 0.30 \) and for rigid square foundation).

Using this theoretical solution and measured values of settlement during field load tests, the values of the deformation modulus \( E \) between 3600 kN/m² and 32,000 kN/m² have been obtained.

5 CONCLUSIONS

On the basis of the results of field load tests one may conclude that:
- the compacted gravel layer over nonconsolidated soil increases the value of the ultimate bearing capacity and increases the deformation modulus of the foundation soil;
- the reinforcement of gravel layer by steel bars leads to the increasing of the ultimate bearing capacity and increases the deformation modulus of the foundation soil;
- the reinforcement of gravel layer by steel bars leads to the increasing of the ultimate bearing capacity and of the modulus of deformation of the foundation soil.

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A contribution to the solution of the problem of reclamation of buildings founded on alluvial soil with low bearing capacity

Une contribution à la solution du problème de la récupération d'édifices fondés sur des sols alluvionaux de basse capacité portante

M. Djordjević & T. Todorović, KosovoProjekt, Belgrade, Yugoslavia

ABSTRACT: The problem of shallow foundations on highly compressible soil with low bearing capacity often presents a complex and responsible task for engineers. Foundation on alluvial deposits is specific because of stratification, heterogeneity in composition and anisotropy of physical and mechanical properties in the vertical and horizontal directions. Underground water destroys connections between particles and activates porepressure, and in this way increases deformability and decreases strength, while oscillation of underground water level causes changes in strain conditions. All of this results in increasing total and differential subsidence and imperils the stability and function of the structure.

By using the correct method of geotechnical investigation and analysing spacious strain conditions in ground loaded by the structure, these problems can be noticed in good time and appropriate measures taken to avoid collapse of soil and differential subsidence on a large scale, which could jeopardise the stability and function of the structure.

This paper concerns the solution of the problem of reclaiming a twelve-storey building which inclined immediately after construction due to increased differential subsidence so that its function and safety were put into question. On the basis of a detailed analysis of strain conditions a special kind of reclamation work was used under the part of the building which had not settled too much and the structure has subsequently been in use for a long time.

Theoretical and practical solutions given in this paper could be useful in the solution of similar problems in construction and reclamation work on structures founded on low bearing capacity alluvial soil.

RESUME: Le problème de fondation directe des immeubles sur sol compressible est en pratique souvent un devoir complexe et responsable. La fondation sur dépôt alluvial est spécifique par ses détails structuraux, l'hétérogénéité de sa composition et l'anisotropie de ses caractéristiques physico-mécaniques en direction horizontale et verticale.

L'eau souterraine détruit le sol par compression interstitielle et diminue sa résistance par une augmentation de compressibilité.

Utilisant une méthode correcte d'examen géotechnique il est possible de prendre des mesures pour résoudre ces problèmes et éviter l'effondrement du sol et l'augmentation de l'affaissement différentiel.

Cet article présente le problème d'assainissement d'un immeuble à douze étages, avec une inclinaison qui s'est produite immédiatement après la construction du bâtiment.

Les solutions théoriques et pratiques données par l'article suivant peuvent servir résoudre des cas semblables.
1 SOLUTION OF THE FOUNDATION PROBLEM

In practice, the foundation of buildings on low bearing-capacity soil with considerable deformability represents a complex and responsible task. If the soil is, as in this case, alluvial, heterogeneous by composition and anisotropic by its physical and mechanical characteristics in both horizontal and vertical directions, then the solution of the problem becomes even more complicated. Underground water then destroys the soil by activating pore pressure, reducing resistance and increasing soil deformability. Oscillations of underground water-levels cause changes in strain, all of which results in an increased total and differential subsidence of the building’s foundations, jeopardising the structure’s stability and function.

With the application of correct methods of geotechnical investigation and analysis of spurious conditions of strain in loaded soil, these problems can be noticed in good time and appropriate measures taken to ensure the stability of the building, which might otherwise be imperilled. In such a case, reclamation measures are more complex and delicate, depending on the type of foundation and soil characteristics.

This paper presents the solution of the problem of reclaiming a twelve-storey building which inclined immediately after construction as a consequence of differential subsidence, putting its function and security into question. The building was built on alluvial deposits in the toe of a hill slope.

Figure 1. The position of the building before and after reclamation
The geotechnical investigation of the location before the construction was incorrectly performed, and shallow foundation was used in this case.

The building is a frame structure with a height of 38.5 meters. Foundation was shallow, 2.1 meters under the surface, on a 20.0 x 20.0 m slab of reinforced concrete.

Load on the contact with soil was 190 kN/m². A geological cross-section of the ground with correspondent characteristics of natural layers and the distribution of strain under the building is given in fig. 2.

\[ \frac{1}{2}B = b \quad \frac{1}{4}B = \frac{1}{2}b \quad \frac{1}{2}b = t + x \]

\[ V = \frac{z}{b} \quad P_s = 190 \text{ kN/m}^2 \]

\[ \tan \theta_z = \frac{2q_z}{(z \cdot tg \frac{b}{b} - \frac{b z}{z^2 + b^2})} \]

\[ M = \frac{1}{1 - \beta} \quad x = \left( \frac{P_z}{q_s} \cdot V \right) \]

**Figure 2.** Ground cross-section with the distribution of strain and corresponding soil characteristics
Construction work began in December 1975. The auscultation of subsidence was started immediately after construction of a foundation slab on four fixed points (one on each corner of the slab). The building settled uniformly until May 1977. Between May 1977 and February 1978, the side of the building close to the hill was raised and the one towards the river subsided increasingly. It is assumed that this resulted not only from the homogeneity of the ground but also from the sliding on the hillside on the foot of which the building was located. We should also that the distance from the building to the river is approximately 200 meters. Between February 1978 and the time when reclamation work was undertaken, the side of the building facing the hill subsided only slightly, while subsidence in the other side had further increased. The top of the building was leaning towards the river by 29.6 cm. There was a tendency of further increased subsidence towards the river and it was necessary to intervene. Additional geotechnical investigation was performed: four drillholes were made on the corners of the building, while four standard penetration tests and large-scale laboratory tests were also carried out. These analyses provided all the necessary data for analysing the subsidence of the building and designing a reclamation programme.

By analysing strain conditions, or more precisely shifts in strain-points, both vertically and horizontally, it was concluded that the further rotation of the building could be avoided by building a diaphragm wall with a drainage system for the purpose of preventing rotation caused by sliding. This, in turn, would make it possible to straighten out the structure by increasing subsidence around the R_2 and R_3 fixed points, where there had been little subsidence until that time. Using the final element method for analysing strain conditions in the given circumstances of the technical solution it was concluded that ground sliding could be avoided and strain conditions redistributed in the zone subject to load from the building. In this way it would be possible to increase subsidence in the R_2 and R_3 fixed points by 14 cm and straighten up the building.

Here is the technology used in reclaiming this building:

The first possibility to be examined involved supporting that part of the building in which subsidence was greater by a mega pile system. This was abandoned, however, due to the great length of these piles, the possibility of their bending and the depth of layers with good bearing capacity (over 20 meters).

The solution finally chosen involved weakening the soil under that part of the building where there was little subsidence. The purpose was to provoke increased deformation in the weakened zone and thus straighten up the building. Only sand and mud discovered at a depth of over 5.5 meters under the surface (below a layer of clay) would be extracted from under a part of the foundation slab. This would produce cavities in the ground, to be filled by surrounding material. In this way the upper layers of clay would be made to subside. Eleven inclined boreholes were made along the edge of the building towards the river. Their position can be seen from Figs. 1 and 3.

The boreholes were inclined by 50 and 60 degrees in relation to the horizontal plane, which enabled the uniform extraction of material from under one-third of the building's foundation slab. The boreholes were lined during construction work with tubes, until the appearance of sandy mud. Further boring was performed without casing to fill out holes continually during boring and reduce the density of sand layers. A total of eleven boreholes was made, with a length of 171.70 m.

The extraction of material from the inclined boreholes began on November 14th 1978 and lasted until the 22nd. Sand and mud from the boreholes were extracted by the "air-lift" method. Air under pressure was pumped through the boreholes, and the material was taken out through separate pipes. The placing of the boreholes and quantity of extracted material were decided in advance.
and during the actual performing of the work they were checked in special tanks. After the sedimentation of mud and sand in the tanks, pure water was drained and the quantities of extracted material were calculated in dry weights (in kN or m³). A total of 80.76 kN or 5.38 m³ was extracted. After the reclamation work was over, cased boreholes with protective covers were left on the site in case the necessity for re-extraction arose.

Figure 3. The position of the inclined boreholes

TIME OF SETTLEMENTS

KEY:

SETTLEMENTS FIXED POINTS $R_1 - R_4$
CALCULATED SETTLEMENTS IN TIME OF IMPROVEMENT
TIME OF IMPROVEMENT
FIXED POINT

Figure 4. Time diagram of the building’s subsidence
Before, during and after the reclamation work, permanent surveys were carried out of fixed points to determine the verticality of the building. Fig. 4 shows the layout of these fixed points as well as the results of the calculations and measurements of the building's subsidence (on a time diagram).

A rapid increase in subsidence at the $R_2$ and $R_3$ fixed points during reclamation work is evident as compared to other fixed points. The reaction of the building to the extraction of material was apparent only six hours after the "air lifting" began. Observation of fixed points and the building's behaviour in the course of the seven years after the reclamation work was completed shows that subsidence has been uniform in all fixed points and that soil consolidation is now in its final stages. Reclamation has been carried out successfully, the building has been straightened up and is being normally used.

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Stabilization of soil with gravel piles
La stabilisation du sol au moyen des pieux de gravier

T.Todorović, Kosovoprojekt, Belgrade, Yugoslavia

ABSTRACT: Stabilization of soil with gravel piles represents an up-to-date procedure, whereby by
ramming gravel piles into the deformable soil its deformable and resistance qualities are im-
proved, and the time of its consolidation is shortened.

Former methods of using gravel piles did not enable us to determine either a reliable level of
soil improvement, or quantitative indexes which make possible planning and constructing of piles
according to already known conditions and allowed deformations.

On the basis of numerous experimental researches, observations of realized structures, and a
series of trial loadings on the soil improved by gravel piles, together with the experience the
author has previously acquired in his research work lasting for several years, a new solution is
given with appropriate formulas and nomographs, where the characteristics as well as the level of
the soil improved by the gravel piles, as per the specific conditions, can be determined in ad-
vance. Consequently, all earlier dilemmas as well as the performance of these operations according
to intuition, without expert and scientific basis have thus been overcome.

Application of gravel piles has proved on many building sites as a technically correct and a
more rational solution when compared with the other solutions to deep foundations or soil repla-
ce, by construction of gravel sub-base (cushion) in the conditions of a high level of underground
waters etc.

All previous results of the author, as well as the most up-to-date knowledge based upon trial
loadings of gravel piles with different mutual disposition have been synthesized, which repre-
sents a totality connected to these problems.

RESUME: La stabilisation du sol au moyen des pieux de gravier représente un procédé moderne consi-
stant à enfoncer les pieux de gravier dans le sol instable, ce qui améliore ses qualités en le re-
ndant resistant à la déformation. En plus, le temps de sa consolidation est réduit.

Les méthodes appliquées jusqu'à présent et reposant sur les pieux de gravier ne permettaient
pas de déterminer avec certitude le degré d'amélioration du sol, les indicateurs quantitatifs né-
cesaires à l'élaboration de projets et le mode d'exécution des pieux pour les conditions fixées
à l'avance et les déformations admissibles, non plus.

Grâce aux nombreuses expériences, aux observations faites sur les ouvrages réalisés et à une sé-
rie de charges d'essai par lesquelles on a agi sur le sol stabilisé au moyen des pieux de gravi-
er, ainsi qu'aux expériences acquises dans les recherches que l'auteur avait faites pendant plusi-
eurs années, le présent article offre une nouvelle solution accompagnée de formules et de nomog-
rames correspondants, où les caractéristiques, le degré d'amélioration et l'importance de la dé-
STABILISATION OF SOIL BY GRAVEL PILES

An up-to-date procedure of gravel piles performance gives possibility to very deformable and very low bearing soil, in conditions of high level of ground water, to improve, up to degree, which is need, from aspect of acceptance and transmission of stress, from depth with pressure of foundation object construction, or embankment during the construction of highways on deformable soil. It means, that effect of soil improvement by gravel piles is directly consequenced to increase of bearing, decrement of settlement and to term of consolidation in exchanged conditions of drainage.

Experiences of author, up-to-date, about applience of gravel piles as the facts of stabilisation, very often of deformable muddy soil, are based on results of settlements observations of industrial and other objects, founded on gravel piles. In additions to this analyse of resistance of ramming on over 8000 piles is done. These datas are correlated with texts "in-situ" and with laboratory tests.

Previously applied facts, mostly gave satisfactory results, but their applience about schedule and depths, as well as the expected results, very often givesome experienced results or works had been done according to the instructions of investigator-designer.

In order to find theoretical and rational correct solution, of soil stability by gravel piles, this problem has been aproached more seriously in recent five years.

In the papers of M.Vlahović and T.Todorović (lit. 2,3,4 and 5) is treat the complex problem of applience of gravel piles as factors of deformable soil stabilisation, and reached theoretical and practical solutions, have been checking in practise.

On the base of previously geomechanyc terrain investigation, and especially on the base of static and dynamic penetrations tests, and after performing of trial gravel piles with greater precious is possible to determine the effect of soil improvement.

In order to solve this problem the new equitant of compressibibility is introduced, which, according T.Todorović (1983) is as follows.

\[ C_D = \frac{6.6 \cdot \mu}{g} \quad \text{or} \quad C_D = \frac{Mg \cdot h}{\varphi \cdot e/N} \cdot \frac{1}{z} \cdot \mu \]  

\[ \ldots (1) \]

where is:

\[ C_D - \text{dynamic constant} \]
\( \sigma' - \text{stress} \)
\[ \sigma' = \frac{Mg}{F} \]  
\( M - \text{mass of blow} \)
\( g - \text{acceleration of gravity} \)
\( F = \pi r^2 h' - \text{surface of cassing base (r-semi-diameter of linned cassing)} \)
\[ \zeta = \frac{h}{5} \]  
\( \zeta - \text{high of catt flow} \)
\( \bar{s} - \text{averag settlement by one blow} \)
\[ S = \frac{\bar{s}^2}{N} \]  
\( \bar{s} - \text{interval of cassing ramming} \)  
(usually is \( \bar{s} = 0.5 \text{ m} \))
\( N - \text{number of blows at penetration cassing for interval } \bar{s} \)
\( P - \text{overburden weight} \)
\( \delta' - \text{bulk density of soil} \)
\( z - \text{depth from surface of terrain to the middle of lamel (layer) i.e. analysed interval} \)
\( \mu - \text{Pausson's coefficient (for low bearing soils = 0.3-0.5)} \)

Influence of ground watter is involved overburden weight, friction between of weak, resistance muddy soil and geologic walls of cassings in dynaminic conditions are neglected in accordance to liquefaction and tyxotropis in areas of cassing in course of work.

![Figure 1. Summary diagram of empty cassing battered during the gravel pile performance](image)

On the base of (trial) tests results of gravel piles, it been defined the correaltive connection between the dynamic compressibility constant \( (C_D) \) and compressibility coefficient \( (C_s) \) reached by static penetration test - T.Todorović (1983)

\[ C_s = 0.5 \] \[ C_D \]

\[ C_s = 1.5 \frac{C_{kd}}{G'} \] \[ \delta' \]  

\[ C_s = 0.75 \] \[ C_D \]

\[ C_s = 1.5 \frac{C_{kd}}{G'} \] \[ \delta' \]  

(5)
where is:

\( C_s \) - compressibility coefficient, determined by static penetration test

\( Q_kd \) - penetration resistance of cone of static penetrometer

\( F_g \) - overburden weight on the depth \( z \).

Previously detailed geotechnical investigations of terrain are defined by its natural construction and authoritative parameters of physical mechanical properties of investigated medias. For engineer-geological model of terrain, prepared in such way, and for defined foundation construction and loadings, analyse of stress schedule is to be done and settlement sizes (without improvement) of soil. Decrement of settlement sizes can be reached by suitable schedule, by depth and by energy of gravel piles compressibility. Degree of soil improvement (T) can be defined and by ratio of dynamic constant (\( C_D \)) reached by battered (end-topped) piles and dynamic constant of improved soil (\( C_D' \)) given by battered casing in the group of piles in a distance (I).

![Diagram](image)

Figure 2. Schematic review of depth and schedule of gravel piles in dependence from soil characteristics schedule and stress size

Dynamic component of improved soil (\( C_D' \)) is defined by the following equation.

\[
C_D' = \frac{Q_kd \cdot k}{F_g} \cdot \frac{m + R}{I - R} \quad \ldots \quad (7)
\]

\[
C_D' = \frac{Q_kd \cdot k}{F_g} \cdot \frac{I}{m} \quad \ldots \quad (8)
\]

\[
C_D' = \frac{Q_kd \cdot k}{F_g} \cdot y \quad \ldots \quad (9)
\]

\[
y = \frac{I}{m} \quad \ldots \quad (10)
\]

\[
I - \text{distance between adjacent piles centres (I-m+R)} \quad \ldots \quad (11)
\]

\[
m - \text{distance between periphery of adjacent piles, if the piles have the same diameters m = I-R} \quad \ldots \quad (12)
\]

\[
R - \text{diameter of gravel pile}
\]

Coefficient of soil improvement (T) can be defined by the following ratio:

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\[ T = \frac{C_D'}{C_D} \quad \ldots \ldots \quad (13) \]

From here is

\[ C'_S = 0.75 \frac{C_D'}{C_D} \quad \text{or} \quad \ldots \ldots \quad (14) \]

\[ C'_S = C_S \cdot y \quad \ldots \ldots \quad (15) \]

where is

\[ C'_S \] - compressibility coefficient of soil improvement

consequently

\[ T = \frac{C'_S}{C_S} = \frac{0.75 C_D'}{C_S} \quad \ldots \ldots \quad (16) \]

Performance of gravel piles, is recommended from the periphery of to the middle part of foundation. In such case soil of density is increases, and at the same time improvement affects can be observed. In this case, every battered pile gives securicity data for control and correction or improvement degree (T) or correction of piles schedule in order to make works rational as much as possible.

Optimal schedule is in a way of chess-board position, but their mutual distance according T. Todorović (1983) is given by following relation.

\[ I = 0.1 R \cdot C_s + R \quad \text{i.e.} \quad \ldots \ldots \quad (17) \]

\[ I = 0.1 R \cdot 0.75 C_D + R \quad \text{or} \quad \ldots \ldots \quad (18) \]

\[ I = 0.075 R \cdot C_D + R \quad \ldots \ldots \quad (19) \]

In the range of \( I = 2 \) \( R \) to \( I = 5 \) \( R \) maximum effects can be reached. Distances, smaller from \( I = 2 \) \( R \) are unration, because, in this case, practically complete change of soil is made by gravel. Over \( I = 5 \) \( R \) has neglibly influence to improvement of degree and time of consolidation.

Respectively for materials with seepage coefficient \( K = 10^{-4} - 10^{-5} \) cm/sec what is the most frequent case. Afterward, such schedule couldn't satisfy geometrical conditions of schedule piles under foundations, except in cases of bigger foundation slabs.

Optimal diameter of piles should be in the following limits \( R = 20-30 \) cm max \( R = 50 \) cm.

Degree of improvement, on this way, can be foreseen and controlled during the piles construction, on the battered casings data of controll piles, between the group of adjacent piles.

Correlation which is done and reached technical solutions had been controllt in practise. One of numerous examples are results of effects of battered gravel piles for conditions of industrial hall founded on heterogeneous, muddy-silty soil saturated with water. Computed settlements were between 8 and 15 cm. Because of equipment sensitivity should be reached the condition to make settlement motion in limits to 3 cm but differential settlements shouldn't be larger from 1 cm.

Applicience of gravel piles, done according to defined conditions in advance on 52 foundation of hall, realises settlements in the period of 3 years from 0.5 cm to 0.8 cm Fig.3. To decreement of settlements exact effects of piles, was tampon layer of gravel 0.5 m.

Calculation reached that the of settlement will be 0.7-1.2 cm. Difference between by calculation reached and measured settlement is consequence of same criterion in relation to energy of gravel pile compact from desired conditions. Nevertheless it should be said that checking in practice, confirm given solutions.

In the paper of M. Vlahović and T. Todorović (1984) on the base of 29 test results by dynamic and static penetrations made in silt of sand had been reached correlation between compressibility coefficient from static penetration test \( C_S \) and compressibility constant \( C_D \) performed by test SPT as follows:
Figure 3. Diagram of observed settlements of object founded on gravel piles

\[
C_D = 10 \ C_s \\
C_s = 0.1 \ C_D
\]  \hspace{1cm} (20)

\[
C_s = \frac{1}{3} \ C_D 
\]  \hspace{1cm} (21)

This correlation can't be applied in whole to discussion of gravel piles problem as well; it should be used equant in No. 5.

The same authors tried (lit. 5) to make correlation between compressibility constant \(C_D\) reached by consolidometer test in laboratory and dynamic constant of compressibility \(C_D\), where is given the following mathematical relation.

\[
C_s = \frac{1}{3} \ C_D 
\]  \hspace{1cm} (22)

This correlation can't be accepted in whole, beacause of great deviation from 70% and 130% even 217% in one case at laboratory tests of clayey soil it is about elastoplastic deformed, and in the second case the point is, failure of soil, causes by dynamic load to soil. It can be usefull information for further investigators, and as realize of correlative connection.

In order to check up-to-date investigation results on solving the problem of reclamation of soil with gravel piles, a series of trial loading was performed over foundations, dimensions 1m x 1,0 m placed on battered gravel piles loading (diameter length \(R = 200\) mm) with difference schedule i.e. mutual distance (I) between centres of piles. At each of tested place performed test of dynamic and static penetration were and after performance of gravel piles one static penetration in the field between piles as well in order to controll degree of soil improvement. Trial loading performed on natural soil (without improvement) is refferent data (curve) in relation to results, reached by trial loading of gravel piles with diameter \(R = 20\) cm, battered in mutual distance between centres \(I = 1.2R\) \(I = 1.6R\); \(I = 2.5R\) and \(I = 5R\). Diameter of piles (casing) was \(R = 200\) mm. But it should take into consederation that real diameter is slightly greater because of lateral indentation of gravel into deformable soil.

Schedule of gravel piles as well as the results of static penetration and battered casings into deformable consolidated, silty sandy soil, are given on the fig. No. 4, No. 5, No. 6 and No. 7, and diagram of trial loading given on the fig. No. 8.
Figure 4

Figure 5
Figure 8. Diagram of trial loadings

I - natural soil (without piles)

curves of trial loading II, III, IV and V with different distance between piles

On the base of trial loading tests as well as penetration test before and after derivation piles, can be concluded that mutual distance between piles has essential influence to the degree of soil improvement (T).

Referential modulus of natural soil deformation, reached by trial load for $6 = 100$ kN/m² was $I = 3600$ kN/m² but at the distance between piles centres $I = 1.2R$, $E = 670 000$ kN/m² what is 18.6 times greater form $E_{ref}$ or for $I = 1.6R$ $E = 40 500$ kN/m² what is 11 times greater form $E_{ref}$ at $I = 2.5R$ $E = 23 800$ kN/m² what is 6.6 times greater from $E_{ref}$ and for $I = 5R$ $E = 11600$ kN/m² for 3.2 times greater than $E_{ref} = 3600$ kN/m².

Penetration test performed after piles performance in ratio to previously performed from the aspect of degree soil improvement are in complete agreement with results of trial loading test and conclusion, established before.

On the base of complete fond of data previously and additional investigations considering already established correlations and on the base objects obscultation, it was establish functional connection between the degree of improvement (T) distances between piles (I) diameter (R) with compressibility coefficient, determined by static penetration test ($C_{op}$) Fig. 9.
Dependences of sizes T and R are defined by geometrical regression by methods of the smallest squares, geometrical function

\[ Y = A \cdot X^B \]  

or in the concrete case

\[ R = A \cdot T^B \]  

Values of coefficients A and B are determined for already known numerical values \( R_i \) and \( T \) by following equation.

\[ B = \frac{\frac{1}{n} \sum (\ln T_i \ln R_i) - (\frac{1}{n} \sum \ln T_i)(\frac{1}{n} \sum \ln R_i)}{\sum (\ln T_i) - \left(\frac{1}{n} \sum \ln T_i\right)^2} \]  

\[ A = \exp \left[ \frac{1}{n} \left( \sum \ln R_i - \left( \frac{1}{n} \sum \ln T_i \right) B \right) \right] \]  

Coefficient of correlation is determined from the following equation:

\[ r = \frac{\sum (\ln T_i \ln R_i) - (\frac{1}{n} \sum \ln T_i)(\frac{1}{n} \sum \ln R_i)}{\sqrt{\sum (\ln T_i)^2 - (\frac{1}{n} \sum \ln T_i)^2} \sum (\ln R_i)^2 - (\frac{1}{n} \sum \ln R_i)^2} \]  

\[ R = A \cdot T^B \]  

Dependences of coefficients A and B from \( C_s \) are determined by linear regression through method of the smallest squares when the following equations are performed.
\[ A = P + Q \cdot C_s \] \hspace{2cm} (29)

\[ B = P_1 + Q_1 \cdot C_s \] \hspace{2cm} (30)

Values of coefficients \( A \) and \( B \) can determined as well as by diagrams, shown on the Fig. 10 and 11.

Coefficients \( P \) and \( Q = P_1 \) i.e. \( Q_1 \) are determined from following equations.

\[ P = \frac{n \sum_i \frac{C_{si}}{A_i} - \left( \frac{\sum C_{si}}{n} \right) \left( \sum A_i \right)}{ \sum_i \frac{C_{si}}{A_i} - \left( \frac{\sum C_{si}}{n} \right)^2 } \] \hspace{2cm} (31)

\[ Q = \frac{\sum_i A_i}{n} - P \frac{\sum_i C_{si}}{n} \] \hspace{2cm} (32)

Correlation coefficient in this case is shown by the following equation.

\[ r = \frac{n \sum_i \frac{C_{si}}{A_i} \left( \frac{C_{si}}{C_{si}^2} \right) A_i - \left( \frac{\sum C_{si}}{n} \right) \left( \sum A_i \right)} {\sqrt{\left[ n \sum_i \frac{C_{si}^2}{A_i^2} - \left( \frac{\sum C_{si}}{n} \right)^2 \right] \left[ n \sum_i A_i^2 - \left( \sum A_i \right)^2 \right]}} \] \hspace{2cm} (33)

CONCLUSION

On the base of data of over a few thousand of battered gravel piles, on the base of static and dynamic penetration tests surveying osculatation of settlement objects founded on gravel piles and performed trail loads the following conclusions can be made:

Aluvial muddy and silty-sandy deformable soils water saturated as well as nonconsolidated soils, with great success can make stabilisation by gravel piles.

Investigation shown in this paper give the following possibilities.

Introducing of the new notion dynamic compressibility constant \( C_D \) in accordance to the equation (1). Correlation between compressibility coefficient \( C_s \) by static penetration test and dynamic compressibility constant \( C_D \) can be established by applience of equation \( \text{No} \text{ 5} \).
Dynamic constant of improved soil $C_D$ defines deformation properties of improved soil, on the base of data of battered casing in the field between performed gravel piles. Degree of soil improvement ($T$) is given by equation $N^O 15$.

Optimal schedule is chass-mat position but mutual distances between centres of piles dependence from the beginner's and from the required, coefficient of improved soil. Optimal values of distances between the centres of piles, are in the majority of cases $I = 2-5R$, and they can be diameter $R = 20-30$ cm, max 50 cm ant to, take care about geometry foundation conditions. At very expressive deformable soil with $C_S = 10$, diameter of piles can be increased even to 50% because of lateral ramming of gravel at blows.

Gravel piles, performed on specially chosen test places with different mutual distances and shedule as well as well as static penetration tests and test of trial loadings fig. $N^O 4$, $N^O 5$ $N^O 6$, $N^O 7$ and $N^O 8$ confirm previously results and make possibility for determination of degrees of soil improvement ($T$) fig. 8. This problem is treated in the equations $N^O 23$ to 33 by appliance of geometry regression and by method of the smallest squares.

It means, that further investigators and designers not laaming on some previously experience and inituition for determined paycnial-mechanical properties of soil and sort of foundation construction, can chose in advance, schedule, depth and mutual distances between centres of piles, which make them possibility for reliable coefficient of degree of soil improvement. In order to reach bigger degree of objects safety from damage settlements and accelerate the time of soil consolidations, it is fayoumane to make, over gravel piles, tampon layer from gravel of special depth, which, beside good deformable properties, improve the conditions of drainage and the time of consolidation make shorter.

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Engineering geological aspects of roading in disturbed schist terrain
La géologie de l'ingénieur au service de la construction des routes dans terrains de schistes disturbés

J.M. Bryant, Ministry of Works and Development, Alexandra, New Zealand

ABSTRACT: As part of a comprehensive hydro-electric development scheme in Central Otago, New Zealand, some fifty kilometres of new roading is required. Twenty kilometres of this length will traverse the steep-sided slopes of a gorge cut through a block-tilted schist range. Very large fossil landslides affect about half of the alignment corridor through the gorge.

This paper describes the geology of the area and some of the problems associated with the investigations, design and construction of the new highway. Of particular concern has been assessing the influence of road construction on the stability of the fossil landslides and the possibility of reservoir induced movements affecting the highway. A design philosophy of leaving the existing factor of safety unchanged or slightly improved was adopted.


Ce rapport décrit la géologie de cette région et certains des problèmes soulevés par les recherches de design et la construction de cette nouvelle grande-route. L'influence de la construction d'une route sur la stabilité des glissements de terrain fossil, et la possibilité des mouvements provoqués du réservoir ont été particulièrement étudiées.

1 INTRODUCTION

Three hydro-electric dams are proposed as part of the Clutha Valley Development Project in the Central Otago region of New Zealand's South Island. The first of these, the 62 m high Clyde dam, will necessitate fifty kilometres of new road when the existing highway is inundated by the proposed reservoir. The route forms a major transportation link with the interior and its important tourist locations. No alternative route is available because of topographic constraints. The location of the new road in relation to the hydro-electric development is shown on Figure one.

2 GENERAL GEOLOGY

2.1 Background

Central Otago is a region of block faulted tilted mountain ranges separated by broad troughs partially infilled with Tertiary and Quaternary sediments. The Clutha River flows in a south-westerly direction along one such trough until it joins the Kawarau River at Cromwell. Here, the Clutha veers south-east through an Antecedent gorge between the Dunstan and Caicnu Ranges (Figure one). The river is incised in an inner rock gorge leaving alluvial terrace remnants on either side. It is through the Cromwell Gorge that the most difficult construction conditions have been encountered and it is on this area that this paper concentrates.

2.2 Basement Geology

Basement rock in the Cromwell Gorge area is schist of intermediate metamorphic rank (textural zone IV). The schist is dominantly a grey quartzofeldspathic variety with minor pelitic schist and a few thin bands of greenschist (Wood 1982). Schistosity surfaces are usually poorly developed and are generally sub-parallel to segregation laminae. The schistosity mainly follows the macroscopic folding but may be locally contorted.
2.3 Geological Structure

The bedrock structure is dominated by faults which have produced the elevated schist blocks and in places controlled sections of the Clutha River and some of its tributaries. Most of the deformation occurred during late Tertiary - Quaternary time and in the Cromwell Gorge was concentrated along the Dunstan Fault; a major reverse fault on the south-east flank of the Dunstan Range (figure one). The fault zone is known to have active fault traces only a few kilometres away from the new highway alignment. Because of their proximity to the dam site, the Dunstan and other faults were subject to a seismotectonic hazard investigation (Officers of the NZGS 1983). The active traces of the Dunstan Fault were estimated to have moved, on average once every 13,500 years with the timing of the last fault movement on the nearest trace indicated as at least 2000 years. However, the geological structures closer to the dam site show no signs of movement in the last 150,000 years. Other faults within the gorge follow a rough north-west to south-east trend (Thomson 1975) with occasional deflections trending normal to this.

A broad antiform, which has its axis trending roughly east - west along the northern slopes of the upper gorge, is significant in that it forms dip slopes on the road side of the valley for a 2.5 km section between Sonora Creek and Italian Creek (see figures one and two). Flexural slip, resulting from regional folding, has formed foliation shear within the schist on the south limb of the antiform. The shear is common in the gorge and forms a zone about one metre thick. Foliation shears are persistent over large distances and recur repeatedly with spacings of a few tens of metres.

2.4. Alluvial Deposits

Several phases of Pleistocene fluvio-glacial aggradation have formed a sequence of outwash gravel terraces in the Cromwell Gorge. The youngest terrace lies about 40 m above the present river level and most of the existing road is founded on it. The new reservoir will inundate most of this terrace. The older terraces are discontinuous and small to be of significance to the roadway.

Only the younger terrace, which is preserved as a semi-continuous surface, can be readily correlated with well defined terraces at either end of the gorge. An age of 15,000-30,000 years has been assigned to this terrace (Officers of the NZGS 1983). Older alluvial terraces are preserved only as minor, higher level remnants. Their surfaces have been modified by degradation and burial by accumulation of slope deposits such that correlation with known formations is uncertain.

Alluvial deposits consist of greywacke, schist and schist-derived quartz gravels. Thin
boulder layers are rare but sand, either stratified or forming a matrix, is common.

2.5 Superficial Deposits

Widespread fans and coalesced sheets of colluvium and fan alluvium form footslope deposits partially overlying the lower terrace. Both materials are derived either directly from the schist bedrock or from the overlying mass movement deposits. Because of the very low rainfall in the area, deposition has been sporadic and layers of loess are often present within the surficial debris units.

Loess characteristically forms a one metre thick layer blanketing terraces and lower slopes. Local depressions such as old river channels and the trough formed between alluvial fan deposits and the hillside have invariably been infilled with loess and erosion debris and depths of up to 12 m have been encountered. Elsewhere, local thickening is believed to be related to turbulence caused by topographical disruption of the silt laden wind. The loess is predominantly a mica-rich silt and fine sand derived under periglacial conditions from braided river flats upstream of the gorge during or following the last glacial advance.

3 LANDSLIDES

3.1 Landslide Characteristics

Of most significance from an engineering viewpoint are the widespread landslides which affect the roadcutting corridor for about half of its length through the gorge (see figure one). The landslides are characteristically large ($10^6 - 10^7$ m$^3$) and extend up to a height of 1100 m. Depths up to 140 m have been produced by drifting and greater depths are inferred from a consideration of surface and subsurface morphology. A typical plan and cross-section is shown in figures two and three.

In places the landslides abut against each other to form broad zones of instability up to 3.75 km wide. Neither the size nor the frequency of landslides are unique to the Cromwell Gorge. Similar landforms are widespread in other valleys in schist terrain in the Central Otago region.

The failure mode appears to be predominantly translational. Mapping and drilling have revealed some curvature of the slip surface at lower levels which in most slide areas is part of a fossil river channel. Lateral river erosion was probably the major initiating factor in slide development. The failure plane, commonly dipping at around 20° - 25°, is almost certainly structurally controlled although its exact nature is sometimes problematic. Where the schistosity forms dip slopes in the central part of the gorge, the failure plane is believed to coincide with foliation shears or even low strength politic layers. Elsewhere, low angle faults have been mapped in areas adjacent to sliding which could be related to slip surfaces.

The presence of landslides on both sides of the Clutha River (and other valleys in the region) and at a wide range of orientations relative to foliation dip suggests another mechanism of failure may also have been operating. Deep down-cutting by river erosion would have given rise to considerable stress relief and some form of sheet jointing may well have formed in sympathy. Matheson and Thomson (1973) describe anticlinal warping (the assumed axis parallel to the valley trend) with accompanying flexural slip in response to erosional stress relief. The flexuring, however, is almost normal to slip surfaces and was prominent for materials of much lower modulus than schist. Rebound caused by erosional processes is known from igneous and metamorphic terrains (Nicholls 1980), particularly those with high horizontal to vertical stress ratios. More specifically, Holzhausen (1978) concludes that sheet fractures are limited to massive, homogeneous rocks and cites examples which are neither of the magnitude nor depth necessary to produce the large scale landslides described in this paper. Nevertheless, some sort of stress relief phenomena is considered by this author to be the most plausible to account, at least in part, for landslide development.

3.2 Mass Movement Deposits

The bulk of the mass movement deposits consist of chaotic schist debris which is typically blocky with a crushed and shattered schist matrix. The blocks sometimes reach spectacular size, up to 100 m long and tens of metres thick. Many have moved without any significant disintegration or change in orientation leading to difficulties in interpretation. During site investigations, a drilling criteria of 15 m of sound rock to prove in situ led to anomalies. In retrospect, a length of 50 m would have been more definitive.

Some landslides appear to have a layered structure with a layer of very coarse blocky material above the basal gouge and an upper layer of finer chaotic debris. Multiple slip surfaces, particularly on the steeper, lower portion, and reworking of the surficial layer by colluvial processes add to the vertical variation. With the larger landslides, the influence of mass...
movement is thought to extend below the basal gouge where a zone of disturbed schist has been recognised. The schistosity has been dragged and oversteepened in a downslope direction at the lower levels where the slip surface dip is assumed to be near horizontal.

3.3 Groundwater

The regional groundwater table is generally deep in accordance with the low annual precipitation. Seepages, where present, only occur low down on landslides, usually just above terrace level. Drillholes and piezometers have revealed perched water tables on the basal gouge surface and sometimes at intermediate gouge surfaces within the landslide. Excavated exposures show a complex spatial distribution of groundwater seepage which reflects the lateral complexity of gouge zones.

Figure 2. Detail of landslides on dip slopes in central part of Cromwell Gorge.

Figure 3. Cross section through Number Five Creek landslide showing subsurface geology and proposed earthworks.
3.4 Age and Activity

The toes of most landslides extend down to or close to the present river level although the exact level is often obscured by alluvium. There are several exceptions where landslides only extend down to some 30 m to 80 m above river level which suggests an older age corresponding to failure at a higher river base level. However, mass movement may not necessarily have been contemporaneous with fluvial degradation and lateral undercutting daylighting the predetermined failure plane.

The age and activity of the landslides has been the subject of considerable interest in relation to both the roading and reservoir impoundment. Some notable features regarding the age of the majority of the landslides are as follows:

(i) The size of the slide masses and presumed angle of sliding is inconsistent with the type of landslide considered able to develop under the present day, semi-arid climate.

(ii) There are no well defined unvegetated head scarsps, lateral scarsps or tension cracks at their crest which would indicate recent movement.

(iii) The surface morphology has been greatly modified by erosion and stream incision, indicating that a significant period of time has elapsed since significant movement occurred.

(iv) The base of many of the landslides is buried by alluvial gravels which is tentatively dated at 15000 to 20000 years.

(v) There are no well defined terrace remnants with recognisable trends which can be correlated with older outwash surfaces. This implies that either mass movement has destroyed the older terrace remnants or that erosion has removed them.

(vi) At one site, excavation revealed a large 'nest' which appeared to have been buried to a depth of 14 m vertical during mass movement. The bird, animal and vegetative fragments comprising the nest were of a type that is either extinct or found in a rain forest environment. A Radiocarbon dating of the plant fragments revealed an age of 8900±105 years, B.P. (based on half life).

It is inferred that the main phase of landsliding occurred during the penultimate interglacial or possibly during an interstadial period of the last glaciation when the rainfall was probably many times greater than that experienced today. A secondary phase of movement, involving the reactivated lower portions of some of the landslides, persists to the present. Lateral widening of the valley by fluvial erosion at the time of deposition of the younger terrace gravels is thought to be responsible for this initiation. Fresh tension cracks and other signs of recent movement are clearly evident. Apart from reactivated movement along an intermediate slip surface the mobility of the slide mass above the basal slip surface was initially considered to be low to non-existent, based on morphological evidence. However, monitoring of certain landslides was initiated because of concern over their position or anticipated behaviour following reservoir filling.

Table one presents some information on the mobility of these critical landslides. It can be seen from table one that of the landslides cited, at least part, if not the main bulk of the slide mass, is currently active. Much of this movement appears to be in the form of steady creep, but at least two of the landslides appear to have been reactivated by a rainstorm in 1980. In the nearby Kawerau Gorge, rates of 10-20 mm/year have been recorded from similar deep-seated landslides with significantly greater rates on lower, clearly active segments (Salt, 1983).

In summary, it is clear that some of the slides in the Cromwell Gorge are known to be active, with failure occurring either along basal or intermediate slip surfaces; and part or all of the remainder are undergoing creep movement. Factors of safety can be expected to be close to or at unity.

4 ROADING DESIGN

4.1 General

The new road is to be a dual carriageway with a sealed width of 8.54 m. A debris berm of 3.5 m has been incorporated in some places although this has been dispensed with in certain circumstances. Formation level is typically 5-20 m above the proposed lake level.

Construction is by conventional cut and fill techniques with cuts up to 145 m and sidings fills up to 54 m in height. The higher fills not extend down to the present day river level but will also be inundated by the proposed lake for much of their height.

Landscaping objectives have been incorporated with geotechnical design wherever possible. The main features are a 'rounding off' of battens so that they merge with the natural country around the perimeter and a restriction of the use of benches on cut battens.

4.2 Design in Areas of Rock

The most significant defects in schist bedrock are foliation shears and fault zones. Where the alignment traverses foliation dip slopes, cuts are designed to lie parallel to schistosity.
TABLE 1 - MOBILITY OF CRITICAL LANDSLIDES:

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Significance</th>
<th>Movement</th>
<th>Duration of Monitoring (b.p.)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clyde Slide</td>
<td>Rises above right abutment of Clyde Dam</td>
<td>3-5 mm/yr</td>
<td>6 years</td>
<td>Slide complex is actually a series of separate failures. Movements measured from above abutment.</td>
</tr>
<tr>
<td>Number Five Creek</td>
<td>Oversteepened lower slopes are currently active</td>
<td>6 mm/yr (horiz)</td>
<td>121 years</td>
<td>Control survey of trig near edge of landslide - Inclinometer movements within active portion slide which developed during 1981</td>
</tr>
<tr>
<td>Slide</td>
<td></td>
<td>29 mm/yr (horiz)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7 mm/yr (horiz)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 mm/yr (horiz)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nine Mile Creek</td>
<td>Very large complex with several active portions</td>
<td>5 mm/yr (horiz)</td>
<td>2 years</td>
<td>Inclinometer movement within oversteepened portion</td>
</tr>
<tr>
<td>Slide Complex</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cairnmuir Slide</td>
<td>Slide toe daylights above lake level; well defined tension crack at head</td>
<td>28 mm/yr (downslope)</td>
<td>1 year</td>
<td>Precise surveying of deformation pillars</td>
</tr>
<tr>
<td>Cromwell Slide</td>
<td>Slide opposite Cromwell town; could flood terrace if displacement is sudden</td>
<td>33 mm/yr (downslope)</td>
<td>3 years</td>
<td>Precise surveying on lower slopes. Precise surveying on upper slopes. Inclinometer movements.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14 mm/yr (downslope)</td>
<td>7 years</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-6 mm/yr</td>
<td>3 years</td>
<td></td>
</tr>
</tbody>
</table>

NB: The inclinometer measurements tend to underestimate the actual surface movement.

(1.6 to 1.5:1). Where the alignment is oblique to the strike of schistosity, steeper cuts (0.25 to 1:1) have been used. Occasional fault zones have given rise to essentially planar-type failures even at the flatter slopes.

4.3 Design in Areas of Landslides

The frequency and size of landslides together with the problems encountered during the earlier stages of highway construction have necessitated a high degree of geotechnical input during the investigation, analysis and design stages. It is not within the scope of this section to give a detailed treatment of these aspects.

The overall philosophy adopted is one of avoiding or minimising cuts in landslide areas by shifting the alignment cut from the slope onto buttressing fills. The proximity of the existing road and the river place limitations on the amount of shift that can be tolerated. With smaller landslides it is possible to design the cut batter such that unloading of the landslide head is achieved. However, the scale of the majority of the landslides is such that the proposed earthworks are diminutive by comparison (see figure three for example).

There are several factors which have to be considered when analysing the stability of landslide slopes. The most important are the existing stability of the entire landslide, its stability when the cuts and fills have been constructed and similarly the effects of lake filling and toe inundation on the overall stability. Of secondary importance is the stability of cut batter in the chaotic landslide debris.

A non-circular analysis was used to check stability where preformed slip surfaces are known to exist. In the toe region the landslides are often buttressed by alluvial gravels so that the most critical failure paths involve breaking away from the gouge through the chaotic debris. For an existing slope, the failure path usually daylights at the narrowest point where the slip surface is closest to the ground. With earthworks completed, the toe of cut or inside edge of a filling fill is usually the most critical case. The influence of elevated groundwater tables
due to lake filling is not expected to extend into the cut batters therefore only the dry case was considered in the analyses.

For landslides of the size and complexity that occur in the region it was not considered possible to accurately model a landslide for analysis. In particular, the geometry of the piezometric surface or surfaces, before and after lake raising, was considered to be the most difficult to determine. A relative approach, is comparing post-construction factors of safety with existing factors of safety, is more meaningful than an absolute approach. Thus although the existing hillside may have a derived factor of safety of close to, or less than unity, the design was considered adequate if a similar or greater factor of safety was obtained for the same model but with earthworks completed and the lake level raised.

For cut batters, a circular analysis and a search routine were used to find the most critical surface. This method ignores the presence of intermediate slip surfaces which are generally thought to be ubiquitous from cut batter exposures. The thin gouge seams had a poor recovery rate in drill holes and intermediate slip surface locations were consequently unable to be determined with any certainty. Although the presence of a pre-existing gouge surface was ignored in the analysis this under-estimation was offset by the blocky nature of the debris (some of the blocks are as large as the batters) which introduces an element of conservatism.

4.4 Design of Fill Slopes

Fill slopes are formed of granular material derived from either excavated rock, landslide debris or alluvial gravel. The outside of the fill is zoned with coarser material for wave protection.

A standard batter slope of 1.75:1 is used, with modifications where additional buttressing is required or where topographical constraints necessitate a steeper slope. Circular analysis was used to check the fill stability, and a minimum target factor of safety of 1.25 was adopted.

Particular attention is paid to under-cutting all loess and ensuring that the toe of each fill slope is adequately keyed into the landslide debris or colluvium which comprise the majority of foot slopes.

4.5 Seismic Design Considerations

From historic and instrument records the Cromwell Gorge area lies outside the main regions of seismic activity in New Zealand. Nevertheless, a major fault with active traces is known to exist only a few kilometres away from the southern end of the Gorge. A recurrence interval of 8000 years has been derived for a maximum credible earthquake centred close to the Clyde dam site (Hancox, 1984).

For fill slopes, the seismic design philosophy allows for small displacements for small earthquake events (150 mm for an event with a 40 year return period) and larger displacements for stronger events (1-2 m for an event with a 100 year return period). The 40 year period is based on 150% of the pavement design life.

For cut slopes in landslide areas, the concept of relating design life to pavement life is inappropriate. Accordingly, a precedent assessment has been used to adjudicate the susceptibility to earthquake hazard. Based on the data outlined in section 3.4, it is concluded that the majority of landslides appear to have resisted en masse movement since the time of their initial development (reactivation of oversteepened portions and long term creep are not considered to be manifestations of seismic activity). In the intervening period of quiescence the landslides would have been subject to at least one major earthquake event (less than 8000 years ago) and are likely to have withstood more than one event. This reasoning has been applied in more detail for the Clyde landslide where the hazard is assessed to be very low to extremely low, provided that appropriate local stabilising work is carried out (Officers of the NZGS, 1983).

The threat of direct tectonic displacement either along the Dairy Creek segment of the Dunstan - Cairnmuir faults or secondary displacement along adjacent faults has not been specifically considered. In accordance with the assessment of the local seismicity, the hazard to the road is correspondingly very low and displacement magnitudes are likely to be less than those due to earthquake accelerations.

5 CONCLUSIONS

Approximately half of the 50 km of new road under construction has encountered difficult conditions through the Cromwell Gorge. Large fossil landslides developed on schist terrain are responsible for many of the construction problems. Some of these landslides are currently active, some have been reactivated by road construction and most appear to exhibit long term creeping behaviour.

Although the scale of earthworks is generally insignificant when compared with that of the landslides, a policy of avoiding or minimizing cuts has been adopted. Of more concern is the effects of inundation of the landslide toe on the groundwater table. However, analyses show that when taken in combination with the earthworks only small changes relative to the existing
stability result. The adopted design and construction philosophies are considered to have enabled the realignment of the highway in difficult terrain that will be stable in the long term.

6 ACKNOWLEDGEMENTS

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Zoning of the Black Sea shelf and littoral according to their morphogenetical character
Les zones du littoral de la mer Noire d'après leurs caractères morfogénétiques

J.M. Buachidze, K.I. Djangava & I.Ph. Khachapuridze, Academy of Sciences of the Georgian SSR, Tbilisi, USSR

ABSTRACT: The shelf and littoral areas of the Black Sea are subdivided into 11 distinct morphogenetic zones. These zones are a consequence of (1) eustatic sea-level changes, (2) tectonic movements, and (3) river discharges. Interaction of these three factors determines the shelf-formation processes and littoral-zone dynamics.

RESUME: Les terrasses continentales et la zone littorale de la Mer Noire se subdivisent en 11 zones morphogénétiques nettement tracées. Ces zones se sont formées et résultent des facteurs suivants: (1) des oscillations eustatiques du niveau de mer, (2) des mouvements tectoniques et (3) par la décharge des rivières. L'interaction de ces trois facteurs détermine les processus de la formation des terrasses continentales et la dynamique de la zone littorale.

INTRODUCTION

The Black Sea basin is a young geosyncline. Its geologic structure and morphology, as well as the character of the diagenetic processes at work on the shelf and in the littoral zone, were determined during the Quaternary by the geologic development of the Caucasus, the Crimea, the Pontian synclinoriums, and by the Scythian plate.

In the southeast and south, the Black Sea basin is bounded by the mountain structures of the Adzhari-Trialety folded system and by the Pontian synclinoriums. These structures are composed mainly of Mesozoic-Cenozoic volcanic and sedimentary rocks. In the Bosporus area, the Black Sea truncates the Istranca anticlinorium. The Black Sea is bordered on the west by the Mesiy platform and the Dobroja massif of crystalline rocks. The Mesiy platform forms the plain south of the Danube and is covered by a thick mantle of alluvial-marine deposits. In the north, the Black Sea cuts into the Scythian plate. In the Crimean area, the sea cuts the sedimentary deposits of the Crimea mountains, and in the northeast it is delimited by the linear folds of the south slope of the Great Caucasus. In the east, the western submerged zone of the Georgian block adjoins the Black Sea.

CLASSIFICATION

Bathymerically, the Black Sea basin is subdivided into three parts: (1) the shelf zone (to 100–200 m), composing 28 percent of the total area; (2) the steep continental slope (from 100–200 to 2,000 m), occupying almost 30 percent of the total area; and (3) the abyssal, flat-bottomed deep (from 2,000 to 2,250 m), composing 42 percent of the total area.

The shelf is not distributed uniformly around the Black Sea. The shelf in the north and west parts, adjoining the Scythian and Mesiy platforms, is broad. In the Bosporus and Crimean areas, the shelf is much narrower, whereas little shelf is developed along the Caucasian and Turkish coasts.

The terrigenous detritus in the Black Sea basin is nonuniform. Mountain rivers of the Pontian and Caucasus folded structures carry principally coarse detritus (pebbles, sand), and the rivers of the plateau regions of the Scythian
Fig. 1 - Black Sea shelf districts: I, Bosphorus; II, Zonguldak; III, Sinop-Samsun; IV, Trabzon-Batumi; Y, Kolkhida; II, Sudauta; VII, Gagra; VIII, Sochi-Novorossiysk; IX, Azov-Kerch'; X, Gorno-Crimean; XI, Dniepr-Danube. Legend: 1-Quaternary - marine deposits; 2-Neogene - clay, sandstone; 3-Paleogene sedimentary deposits - sandstone, flysch; 4-Paleogene volcanic-sedimentary deposits - basalt, tuff, marl; 5-Cretaceous - Limestone, marl; 6-Jurassic - shale, limestone, tuff; 7-Paleozoic - limestone, quartzite; 8-Archean-Paleozoic - crystalline schist, granites; 9-Mesozoic-Cenozoic - granite, diorite, syenite; 10-diorite, diabase; 11-disjunctive folds; 12 - shelf area; 13 - shelf-district boundaries.
and Meziy platforms and the Georgian block carry suspended detritus and fine-grained sand. The nature of the shelf is a consequence of (1) eustatic sea-level changes, (2) tectonic movements, and (3) river discharge. Interaction of these three factors determines the shelf-formation process and littoral-zone dynamics. Accordingly, the Black Sea shelf can be subdivided into distinct districts or zones (Fig. 1).

**BOSPORUS**

This district (Fig. 1, I) comprises the south-west area of the Black Sea basin on either side of the Bosphorus between Edine (Bulgaria) and Baba (Turkey) Capes. The total length of the shoreline is 450 km; the width of the shelf ranges between 5 and 19 km. The widest shelf is north of the Bosphorus and the narrowest shelf is southeast of it.

The north boundary is the south edge of the Meziy platform, and the eastern boundary is the west end of the West Pontian synclinorium. Most of the coast and shelf is in the eugeosynclinal zone of the Stara Planina mega-anticlinorium, the Istranca anticlinorium, and the West Pontian synclinorium. This zone is composed mainly of Palaeozoic metamorphic rocks, but scattered outcrops of young folded sedimentary-rock complexes are present. In the Burgas Bay area (Bulgaria), the sea cuts into ancient metamorphic complexes of a eugeosynclinal zone. Morphologically, the shelf of the Bosphorus district can be subdivided into two subdistricts separated by the Bosphorus strait.

The west subdistrict has a length of 250 km and an average width of 40 km; the total area is 10,000 square kilometres. The shelf can be recognized down to the 100-m isobath, and it is characterized by a comparatively smooth surface with a slope of about 0.002.

The geologic development of different structural zones is reflected in the shore and submarine slope structure. In areas where the sea shears the Palaeozoic metamorphic complexes of the eugeosynclinal zone, the coast is mainly retrograding and is marked by distinct cliffs. Conversely, the miogeosynclinal areas are characterized by easily eroded Mesozoic-Cenozoic sedimentary-rock complexes. The latter supply great amounts of sand that is redistributed in the littoral zone by the longshore currents. For example, the dunes of Karaburun (west of the Bosphorus), which are formed by the drifts of the Istranca river, are present for a distance of 50 km toward the strait.

In the east subdistrict, the shelf has an average width of 19 km. Along the continental slope there are well-defined submarine canyons.

**ZONGULDAK**

This district is between Baba and Kerempe Capes; it has a total length of 155 km, an average width of 7 km, a pronounced steepness (0.05), and a comparatively small area of 1,300 square kilometres (Fig. 1, II). The district is confined to the eugeosynclinal zone of the West Pontian synclinorium, which is composed mainly of Cretaceous-Tertiary flysch and volcanic rocks. The Filips and Koca Rivers contribute material which is deposited along the steep submarine slope and numerous canyons.

The coastline of the district is retrograding - i.e., most of the eroded material is deposited at considerable depth, and some is transported eastward by longshore currents.

**SINOP-SAMSON**

This district is in the central part of the south Black Sea coast between Kerempe and Yasun Capes (Fig. 1, III). The shoreline is 450 km in length. The average width of the shelf is about 6 km and the slope is less than 0.01. The district includes parts of both the West and East Pontian synclinoriums. The shelf apparently joins the miogeosynclinal zone, which is composed of sandy-marilaceous and conglomeratic formations of Neogene age. The eugeosynclinal zone is composed of Palaeogene-Cretaceous flysch and volcanic formations.
In addition to tectonic forces, several rivers (e.g. Aksu-Irak, Yeşil, Bolaman, Cür) play as essential role in the formation of the broad shelf and the delta; these rivers contribute great quantities of detritus composed principally of Tertiary volcanic and granitic rocks and Palaeogene metamorphic material. The solid load is distributed on either side of the outfall of the rivers and forms a continuous beachline that extends for several hundred kilometres.

The large peninsulas have a Neogene base upon which deltaic marine sediments of Quaternary age have been deposited. The recent regression of the sea level behind many lakes, some of which periodically are connected with the sea. The absence of submarine canyons can best be explained by recent periods of tectonic uplift that coincided with the intensive sedimentation on the shelf. These forces resulted in strong denudation processes in the nearby source area.

TRABSON-BATUMI

This district is between Yasun and Taşkıhsâzı Cages (Fig. 1, IX). The shelf here is the narrowest in the Black Sea, averaging 2.5 km in width, and it has a steep slope of 0.075. The district is limited to the eugeosynclinal zone of the north Pontian synclinorium and to the west end of the Adızâraz-Trialety folded system. The district is characterized by linear folds that extend along the basin and rise to 3,000 m in the mountainous East Pontian and Adızâraz-Imereti ridges. These ridges are composed of Palæogene-Cretaceous volcanic rocks.

The coastline is mainly retrograding; the narrow shelf band is further reduced in width between Trabzon and Hopsa, 30 km south of Batumi. In the Çoruh River outfall near Batumi, the shelf widens only insignificantly.

The narrow, steep shelf is principally a result of the varied neotectonic movements which formed the littoral zone of the East Pontian structures. Local widening of the shelf resulted from sedimentation of the material contributed by small rivers along the Turkish coasts. The shelf structure reflects the morphogenetic forms of the adjacent basement characterized alternately by abrupt cliffs, middle Eocene volcanic rocks, and sandy-pebbly beaches.

In the eastern part, the littoral-zone dynamics are governed by the Çoruh River. According to Mandych (1967), the river contributes 1.1 million tons of gravel and 2 million tons of sand per year. These deposits are derived from the Adızâraz-Trialety folded system and the eastern Kuzy Anatolou Daglari.

The Çoruh River flows in a deep channel. In the outfall section, where the thickness of alluvial-marine formations is greater than 60 m, the river has formed the vast Kackar lowland and the Tabyı Cape at Batumi. In the Çoruh River outfall, the shelf is well defined by the 100-m isobath, and its maximum width is 5 km. On the submarine slope in the Çoruh outfall, a branched canyon splits the entire shelf.

Sand transported by the Çoruh River moves in two directions: some is deposited in the submarine canyon, and the rest is distributed by longshore currents to the north, where it accretes onto Tabyı Cape, bounds the port of Batumi, and is deposited on the shelf. A substantial part of the longshore current is interrupted by the port of Batumi. The littoral zone is characterized by a pebbly beach. On the submarine slope, three zones can be recognized: (1) shingle gravel, up to 5 m water depth; (2) sand, from 7 to 15 m; and (3) ooze, developed below 15 m.

In the areas where volcanic rocks are exposed (Turkish coasts of the district; Ealender and Taşkıhsâzı Cages), large blocks of eroded material are formed. The maximum washout, near Adila, amounts to several meters per year.

KOLKHAIDA

This district is between Taşkıhsâzı Cape (25 km north of Batumi) and the Kumașta River outfall (near Sukhumi), and has a total length of 190 km (Fig. 1, Y). The shelf is divided morphologically into three subdistricts: (1) Poti (between Taşkıhsâzı Cape and the Inguri outfall), characterized by a broad shelf, where canyons of the Supsa, Rioni, and Inguri Rivers are developed; (2) Ochamchire (between the Inguri and Kodori Rivers), characterized by a broad shelf (up to 15 km) and a gently sloping (0.01); and (3) Sukhumi (between outfalls of the Kodori and Gumișta Rivers), characterized by a narrow (2,5 km) and steep (0.05) shelf.

The Kolkhida district is confined to the western submerged zone of the Geor-
gian block, which, during the Quaternary, underwent maximum subsidence of more than 500 m in the central part (at the latitude of Poti). It is apparent, therefore, that Kolkida was a shelf zone during the different stages of the Quaternary transgression, and that the present shelf was formed by intensive sedimentation during the general subsidence of the district.

The New Black Sea terrace in the Kobuleti-Poti dune strand represents the most recent regression in the Kolkida district. Carbon-14 dates of peat deposits in the New Black Sea terrace yielded an age of 2,650 years B.P.

Shoreline elevations of the Holocene regression phases are delineated by peat deposits in some locations, as on the right bank of the Inguri River.

Lack of a directionally well-defined flow system is typical in the littoral zone of the Kolkida district. In the south, a south-to-north flow direction is present. The Kintrishi and Natanebi Rivers contribute sandy-pebbly material (about 8,000 tons/year), but the quantities are insufficient for maintaining a dynamic equilibrium under the conditions existing on the steep, abyssal submarine slope; thus, the intensive washout near Kobuleti has developed.

The Supsa River contributes exclusively sand, at a rate of 20,000-25,000 tons/year; part of this sand is deposited in the Supsa submarine valley, and the rest is moved southward, where quasi-equilibrium conditions exist in the Supsa-Natanebi interfluence. The Supsa River sands are high in magnetic content. The greatest quantity of detritus comes from the Rioni River, which annually contributes more than 500,000 tons of sandy material consisting of granitic debris, Paleozoic metamorphic rocks, and Mesozoic volcanic and sedimentary rocks of the Great Caucasus mega-anticlinorium. The Wave direction in the central part of the Kolkida district has interfered with the formation of unidirectional flow of detritus. As a consequence, the Rioni River sands are spread both south and north of the river outfall. Construction of the port of Poti and the override of the Rioni River sediment have caused an intensive accumulation in the new outfall area, as well as a pronounced washout south of the port.

The coasts in the Gumista-Inguri interfluence are characterized by a variable dynamic regime, which can be explained by the heterogenous structure of the shelf; the different thicknesses of the recharge sources at each section; and the influence of the Rioni, Inguri, Kodori, and Gumista submarine valleys. These valleys all collect large amounts of detritus.

Intensive coastal washout, near the town of Ochamchire, is due to engineering projects that limit the supply of the Rioni River detritus moving southward.

GUDAUTA

This district is between the Gumista River and Pitsunda Cape; it has a total length of 75 km and an average width between 25 and 30 km (Fig 1, YI). The shelf is a bank area of 2,000 square kilometres and has gentle slope (0.04) up to the 150-m isobath.

The broad shelf of the Gudauta bank was formed by two processes: (1) the deposition of Meotian-Pontian conglomerates in the coastal part of the sea on the shelf foundation by the Bay, which carried large amounts of coarse material, and (2) the former intensive uplift of the Gudauta flexure.

Geological data indicate that the thickness of the Quaternary deposits on the shelf is between 15 and 20 m; below these deposits, Meotian-Pontian conglomerates are consolidated.

The coasts of the district are mainly retrograding. The local character of the detrital deposits is due to their being derived from erosion of the Musseno hill, and from material carried by a few small rivers that contribute total solids of 15,000 tons/year.

An insufficient supply of material to the littoral zone is responsible for an acute deficit of beach-forming material and the instability of the whole littoral zone.

In addition, landslides, widespread in the Maycop Series and in the Meotian-Pontian formations, play an important role in the formation of the littoral zone. The lowest level of these slides is below the contemporaneous sea level. The Cherno-Ambarisky, Petropavlovsky, New Aphoni, and Sukhumi landslides exert particular influence on the periodic deformation of the shoreline. Although these landslides supply substantial amounts of material to the littoral zone, this material does not contribute to formation of the beach, because it is mainly clay.
GAGRA

This district lies between Pitsunda Bay and the Uzynya River (Fig. 1, YII). The shoreline extends for 60 km. The most notable characteristic of the district is the narrow, steep shelf that is present everywhere except along the shore north of the Bazb' River and the northwest of the Gagra resort. In the latter area, the outer brink of the shelf is limited by the 100-m isobath, situated 300-700 m from the shore.

The formation of the shelf projection apparently is associated with subsidence of the marginal zone of the Black Sea "deep". According to Milianovski (1968), this zone of subsidence is joined with the uplifts of the Great Caucasus by Gagra and Gudauta regional structures.

In the Bazb' River outlet area, the accumulation off Pitsunda Cape projects far out to sea and to considerable depths. According to seismic data, valleys on the submarine slope cut several dozen meters into Miocene-Pliocene conglomerates.

According to data collected from seismic sections and core wells drilled in the Bazb' delta, Quaternary deposits are 40 m thick on land and 5-35 m thick on the submarine slope.

In the Gagra resort area, the thickness of recent sediments on the submarine slope is minimal; in a few areas, recent deposits are absent and Cretaceous limestone crops out at the bottom of the slope. The seashore in this area is subjected to periodic washout.

Significant factors in present development of the littoral zone are drifts of the major rivers (Uzynya, Psou, Bazb') that form longshore deposits. The combined load of the Uzynya and Psou River drifts is 200,000 tons per year.

According to Mandych (1967), the Bazb' River carries 170,000 tons of coarse material per year. In accordance with prevailing wave direction, the deposits from the Bazb' are spread from northwest to southeast. Most of this material moves along the Bazb' river submarine valleys to considerable depths; the rest is distributed in the Pitsunda Cape littoral zone. The distal part of the cape, which is currently well developed, is characterized by alternate periods of washout and accretion of the shelf, depending on storm direction.

SOCHI-NOVOROSSIYSK

This district lies between the Uzynya River and the Anapa Cape, a distance of 300 km (Fig. 1, XIII). The shelf is characterized by an average width of 6 km and a relatively steep (to 0.06) slope. The district is confined to the Novorossiysk synclinorium, composed of Cretaceous and Paleogene flysch.

Ostrovsdky et al. (1971) indicated that late Pleistocene and Holocene stages of transgression and regression were important in the formation of the shelf zone. The shelf is outlined clearly at the 100-m isobath. Ostrovskiy et al. further indicated that overdeepening of the Tsone, Pahada, Ashe, and Shakhe valleys was influential in the development of the littoral zone and the adjacent shelf during the various stages of regression. Old erosional valleys, which now are below sea level, represent the zones of intensive sedimentation and are common in the district. The most distinctly outlined is the Shakhe canyon; its head cuts into the upper part of the shelf. The coasts of the district are subjected to intensive abrasion, and landslides occur at the Sochi-Thapsee intersection. The tongue of the landslide commonly subsides into the sea. The tongue of the landslide commonly subsides into the sea. The few existent drifts in the littoral zone are supplied mainly by small rivers in the district.

AZOV-KERCH

This district (Fig. 1, D), between the Anapa and Magonon Capes, represents a shallow shelf with a gentle slope (0.003). The shelf, located in the zone of the Indo-Kuban trough, was formed tectonically.

The coasts of the district are composed of intensely eroded sedimentary formations of Tertiary and Quaternary ages. The shallow Sea of Azov, which now lies entirely within shelf limits, is an area of intensive sedimentation from two major rivers - the Don and the Kuban'. The thickness of the silty accumulation in the central part of the Sea of Azov, according to Shmolyov et al. (1971), is more than 30 km; below are the Neoeuxinian layers.

Some of the material entering the Sea of Azov is carried by submarine flow.
through the Kerch' Strait and is deposited on the broad shelf within the Black Sea.

GORNO-CRIMEAN

This district comprises the southeast Crimean seashore, and is characterized by a comparatively broad shelf in the south part (up to 50 km) that narrows considerably (to 5-15 km) along the southeast coast of the Crimea (Fig. 7, X). The coast is oriented parallel with the Crimean Mega-anticlinorium, which is composed of sandy-shaly Triassic and Jurassic strata and Upper Jurassic carbonate rocks.

According to Zhivago (1958), the coasts of this district are undergoing weak uplift. The bottom of the sea in this stretch obviously is subjected to uplift, and considerable narrowing and steepening of the shelf results.

The coasts of the district are mainly retrograding, as a result of (1) the lack of sediment supplied to the littoral zone by rivers, and (2) recent tectonic movements.

DNEPR-DANUBE

This district is between Cape Khersonesskiy (western Crimea) and Cape Emine (Bulgaria) and has a total length along the coastline of 1,035 km (Fig. 1, XI). The district is characterized by the widest shelf in the Black Sea, having an average width of 150 km. The shelf is well outlined by the 100-m isobath, and its slope is insignificant (0.001).

Geostucturally, the shelf of the district is confined to the south end of the Scythian platform (and its outcrop), to the Hercynian Dobruja massif, and to the Mezy platform.

The Mezy platform represents an area of subsidence and accumulation of thick Quaternary sediments (up to 400 m). On the Dobruja massif, Paleozoic magmatic rocks are exposed; they are overlapped in the shelf zone by Quaternary deltaic-submarine deposits. In contrast, the Scythian platform in the shelf and littoral zone area is overlain by a Miocene-Pliocene and Quaternary sedimentary mantle of platform type.

The broad shelf of the district is due to its confinement to the platform areas, which undergo differential neotectonic movements. Some areas (Dobruja massif) are subjected to uplift, and others (e.g., the Mezy platform and the Gulf of Karkinitskiy, where Quaternary deposits crop out on the present seafloor) to deepening.

Major rivers (the Danube, Dnepr, Dnestr, Bug, etc.) play an important role in the intensive development of recent sedimentation processes on the shelf and in the littoral zone by supplying tens of millions of cubic meters of sandy-silty debris. For the most part, this debris is deposited on barrier beaches and spits in the littoral zone, commonly causing the formation of limans.

The most retrograding coasts are typical of the Odessa area, where littoralzone deformation and a marked drift deficit are caused by vast landslides. At their lowest level, these landslides commonly terminate on the submarine slope.

CONCLUSIONS

On the basis of the foregoing, some preliminary conclusions can be made.

The shelf can be traced along the entire Black Sea perimeter (4,000 km) except for small areas where the submarine-canyon heads extend nearer to the coast, where there are steep rocky beaches, and where the shelf is buried under young deltaic drifts.

Where the shelf is well defined, it is outlined mainly by the 100-m isobath. However, in anomalous areas, it is limited mainly to the deltaic caps. Where the shelf is present on separate blocks which were elevated intensively during the neotectonic stage, the edge of the shelf is bounded by the 200-m isobath.

This relation can be seen in Dobruja, the Crimea, the Gedaha batl, the Sinop-Samsun district.

A relation between the form of the shelf and the geotectonic situation is apparent. The widest shelves are characterized by ancient platforms and continental plateaus; a somewhat narrower shelf corresponds to miogeosynclinal zones; a narrow shelf is confined to eugeosynclinal zones.
Morphometric data of the Black Sea shelf indicate that a gentle slope (0.001-0.002) is common on broad shelf and a steep slope (0.03-0.07) characterizes narrow shelves. Vast canyon development is characteristic of narrow shelves, whereas canyons are absent on wide shelves.

The main factors involved in shelf formation in the Black Sea area: (1) upward neotectonic movements of the seabed, (2) intensive sedimentation processes in the long river-outfall zones, (3) coast submersion, and (4) eustatic sea-level variations. A quantitative evaluation of the effect of each factor remains to be determined.

RECOMMENDATIONS

Present studies of coast dynamics do not permit detailed conclusions to be drawn about the dynamic processes along the entire Black Sea basin perimeter. However, detailed studies in selected districts permit determination of certain characteristics of the dynamic processes.

A study of the distribution of the three standard dynamic coast types ("washed out", stable, and accumulative coasts) produces the following approximate values: washed out, 37 percent; stable, 48 percent; accumulative, 25 percent.

The three geotectonic types of the shelf (miogeosynclinal zones, eugeosynclinal zones, and submerged blocks) in the Caucasus can be used to infer information on dynamics for other coasts along the Black Sea. It is concluded that the state of the coast is determined not only by its geologic structure (sediment load from rivers and wave regime), but also, to a considerable extent, by the structure beneath the shelf. The steep shelves and their coastal slopes are influenced intensely by wave force, but the gently sloping shelves are not as strongly affected.

Results from present studies of the Black Sea shelf indicate a need for further systematic investigations in this area. The practical significance of this problem is great because, without a geologic survey of the shelf, especially at its upper part, it is impossible to institute effective coastal fortification or to make long-term plans for national-economic exploitation of the seashore. In addition, the shelf contains substantial reserves of useful minerals such as sand for building, magnetic sand, manganese, rare metals, oil, etc. The shelf should be considered for its coastal underground water reserves as well, because in many areas intensive submarine discharge of underground water takes place.

Further study of the shelf zone should concentrate on a detailed analysis of the geologic conditions. Engineering investigations must be done in conjunction with geologic research, using modern quantitative and qualitative methods to evaluate geologic processes. Because of the consanguinity of the shelf with contiguous geotectonic coast elements, the geologic-engineering study must encompass the coast as well as the shelf zone.

Geologic and engineering surveys of the shelf should include: a bathymetric survey, a sampling of bottom beds with core barrels, geophysical investigations, and drilling of stratigraphic-test wells. Methods to be employed would include comprehensive echo-sounder profiling; geo-acoustic emplacement, which has been used successfully in some shelf areas; corehole drilling on the submarine slope from mobile platforms; and use of the rotary method of drilling on land along the coast, especially in the overdeepened valleys and in the zones of submarine-canyon influence.

Special attention should be paid to the study of submarine valleys (canyons). Determination of their genetic types is of great theoretical interest and has practical significance for the evaluation of littoral-zone stability.

Wide use should be made of isotope methods and absolute geochronometry (especially the radiocarbon method) on the shelf deposits, in order to determine the common age correlates and to interpret and make a prognosis of differential neotectonic movements of the littoral zone.

Experimental sand exploitation on the submarine slope in the Don River outfall area shows the possibility of sand extraction for engineering purposes. Moreover, attention should be paid to the effect of submarine exploitation on the littoral-zone dynamics in each specific case. This knowledge is important in considering the possible submarine output of useful minerals, specifically of magnetic sand from the shelf in the Kotulets-Supsa area.

Long-term observations should be made of shoreline dynamics and of the sedimentation process on the shelf, and attention should be directed first to the coastal zone. It will be necessary to organize precision geodetic observations,
which will allow a quantitative evaluation to be made of the role of modern tectonic processes in littoral-zone dynamics.

Special attention should be paid to the estimation of the solid load of the rivers and its interaction with littoral-zone dynamics. Moreover, constructors of hydrotechnical structures along the rivers should be obliged to cover, with inert materials, those parts of the coast which will undergo deposit deficit caused by regulating the river runoff.

In the case of littoral-zone washout in valuable exploited territories (e.g., in the Pitsunda Cape), it is first necessary to restore the profile of the littoral zone by filling in with adequate amounts of material. Later, efforts should be made to stabilize that part of the coast.

REFERENCES


Stability and evolution of Las Caneras Beach, Canary Islands, Spain
Stabilité et évolution de la plage ‘Las Caneras’ aux Canaries, Espagne

J.M. Rodríguez Ortiz, Polytechnical University, Madrid, Spain
C. Prieto Alcolea, Equipo de Asistencia Técnica, Madrid, Spain

ABSTRACT: Las Caneras Beach is the west side of a sandy isthmus connecting the north corner of the Island of Great Canary with the volcanic cone of the Isleta. In front of the beach a stepped sequence of cemented sand reefs has developed in the Holocene. The uppermost reef appears as a dissected barrier island ap by. 60 m wide and 1.500 m long. It emerges in mean and low tides and creates a quiet lagoon protecting the beach from the Atlantic waves.

In the last 50 years the beach has gained considerable sand volumes and consequently the water depths in the lagoon have decreased. This seems associated with breaches open in the bar and the progressive destruction of the same, scoured seaside by breaking waves. Interruption of eolian transport by a screen of buildings is also responsible of increased sand deposition.

In 1978 a first model of sediment transport across the bar end inside the lagoon was prepared. This model has been checked in 1984 with satisfactory results. A series of protective measures have been developed following these studies.

RESUME: Le plage de La Caneras est à l’ouest d’un isthme sableuse que connecte l’île de Gran Canaria avec les cônes volcaniques de l’Isleta. A l’extérieur de cette plage se trouve un banc préhîtorial de sable cimenté, développé après le dernier période glacial. Cet banc fait la fonction d’une île barrière à marée haute - les houles arrêtent à la plage avec une certe amortigation.

Depuis les 50 derniers années la plage a augmentée considerably leur volume de sable et, en conséquence, la profondeur de la lagune est plus petite. D’ou la part le banc cimenté présente une dégradation progressive avec plusieurs coupes.

L’interruption de transport éolien par la construction de hautes bâtiments à l’isthme on a révélé responsable de l’accumulation de sable à la plage.

A 1978 on a préparé le premier modèle dynamique pour la lagune, le banc et la plage; cet modèle a été confirmé par un second étude à 1984. Quelques mesures de protection ont été recommandées pour recouvrir la situation initiales de la plage.

1. INTRODUCTION

Las Caneras Beach is located at the Northern point of the Great Canary Island, facing the Conifital Bay and it defines the Western limit of the town of Las Palmas.

This beach was part of the original sandy isthmus connecting the island to an extinct volcano called the Isleta. Nowadays this isthmus is entirely covered by the urban development of the town.

The bay facing the beach is open to NW winds but the beach is protected against wave action by a bar or reef of weak cemented sandstone, emerging in low tides; the mean tidal range is 2.30 m.
In recent years increasing sand accretion on the beach berm and in the enclosed basin has been observed; water depths have dramatically decreased favouring the growth of subaquatic vegetation as well as the modification of the current field, making difficult the natural renovation of sea water.

On the other hand the bar has shown some breaches and erosive features, arousing some concern as refers to its long-term stability.

In 1978 a first survey was carried out about the dynamics of the bay and the condition of the bar. A simple model of sediment transport was proposed in order to explain the processes involved. That work has been revised in 1984 as a check of the original hypotheses. As a result only minor changes should be required to adjust the observed behaviour to the measured evolution.
2. THE ORIGINAL STUDY OF 1978

2.1 Working method

The geological features of the area were investigated first as the origin of the bar was uncertain in absence of similar bars or barrier islands in the Spanish coasts. It became apparent that this and other submerged bars found offshore corresponded to temporary detentions in the last transgressive process (fig. 2).

A parallel research was devoted to the establishment of wind and wave (sea and swell) climates, currents and tides. Local data were scarce and, outside some direct measurements, most of the computations were based on inferred values from the nearby La Luz Harbour and some published data ("Ocean Wave Statistics" and "Routing Charts").

Contours of the sea bed, from the beach to the ocean side of the bar were also determined and compared with profiles derived from existing nautical charts and early surveys since 1860.

The original morphology of the bar and the isthmus was also historically traced back.

2.2 Physical factors

From available models of elolian sediment transport it was deduced that only winds in excess of 15 km/h were effective to put sand grains into motion on the beach. Frequencies of these winds were: N 25.5%; NE 2%; E 1.8%; S 0.3%; W 0.4%; NW 20.1%.

The onshore transport under wave action was also estimated through models of wave and sand overtopping of submerged breakwaters, a situation very close to that created by the natural bar; see fig. 3.
Measurement of currents showed very small velocities with little effect on bed transport. However currents in excess of 1 knot were measured around the main bar inlets.

The profiles of the beach and the soundings of the 1978 campaign are shown in the fig. 4 (where a first estimate of changes with respect to previous bathymetrical conditions is also included).

The evolution of the encroachment of the isthmus is shown in fig. 5.

The inventory of the bar showed in the inner side a rounded profile, with erosion channels 1-2 m wide and 0.2-0.3 m depth. The central part is severely pitted, with holes 0.2-0.6 m in diameter and 0.2-0.3 m depth. The outer part has a gentle slope with frequent erosion rills draining the holes above.

The inner front of the bar has been modelled by overtopping waves and its toe is paved by blocks and slabs detached from its original position following existing joints and fractures.

The ocean front is severely scoured by wave runup loosening the poorly cemented gravels and boulders supporting the top sandstone layer. When the unsupported length of the sandstone slabs exceed 2 or 3 m the cantilevers collapse paring the ocean bottom.

2.3 Conclusions of the 1978 study

- The annual accumulation of sand on the beach was evaluated in 15 m³/m. Only in front of the bar inlets a negative output of sand was measured, due to longshore transport under penetrating waves and current action.

- The two big inlets created by bar destruction are posterior to 1948, because they are lacking in aerial photographs of that time.

- Sediment movement in the basin is favouring the building up of hemi-tombolos connecting the bar with the beach.

- The bar is being destroyed by scouring and falling down of the sandstone cantilevers formed seaside. A rough estimate of this process would be 0.40 m³/m per year, i.e. only a 10-15 percent of sediments come from bar erosion. This leads to a sediment transport over the bar of about 3.5 m³/m a year, which compares quite well with the computed values.

- The urban occupation of the isthmus by high-rise buildings has created a stagnation of wind currents over the beach, thus stopping the natural "cleaning" of the beach, transporting the sand over the isthmus. Original sand volumes moved by winds of over 13.5 m³/m per year have been computed, whereas the present ones are under 7.6 m³/m and only where the wall limiting the beach can be overtopped.
FIG. 4.

BASIN CHANGES

ACCRETION

SEDIMENT LOSES

BAR CHANGES

INCREASED FRACTURING

CURRENTS OVER THE BAR
The following net budget of sand was finally estimated in 1974:

<table>
<thead>
<tr>
<th>Area</th>
<th>Accumulated in the basin</th>
<th>Accumulated on the beach</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northern area (bar protected)</td>
<td>4730 m$^3$/year</td>
<td>6145 m$^3$/year</td>
</tr>
<tr>
<td>Central area (half-protected)</td>
<td>2150 m$^3$/year</td>
<td>1770 m$^3$/year</td>
</tr>
<tr>
<td>Southern area (open)</td>
<td>2900 m$^3$/year</td>
<td>107 m$^3$/year</td>
</tr>
</tbody>
</table>

Thus, the overall sand accumulation can be evaluated in about 9500 m$^3$/year.

The study concluded that an accelerated erosion of the bar was highly improbable but some protective works were necessary in order to prevent further destruction of the bar and to keep the basin water depths within acceptable limits for recreational use.

3. THE STUDY OF 1984

This was a consequence of the recommendations of the former study due to the necessity of checking out the progression of the problems detected.

As previously suggested, the new study concentrated itself on the following aspects:

- Beach and basin bathymetry
- Evolution of the sediments and vegetation in the basin
- Inventory of the bar

The main conclusions were as follows:

a) The erosion of the bar has been not significant but always concentrated in the predicted areas (inlets and highly degraded breaches).

b) The beach widths have increased 2 to 30 m in different parts of the northern section.
The maximal elevation of the sand in this area has increased between 0 and 1 m. The central beach section has suffered the greatest changes. In the southern section width growths between 10 and 15 m and elevation rises between 0.5 and 1 m have been measured.

c) The measured volume of sand deposited on the beach amounted to 9200 m$^3$/year (i.e. 300 m$^3$/m). This has resulted in increasing elevations in the berm emerging at high water and a steeper foreshore slope.

d) A similar pattern has been observed in the different sections of the basin. The sand volume deposited amounted 17,220 m$^3$/year, i.e. a mean increase of 3.8 cm/year of bottom levels of the 500,000 m$^2$ surface.

e) A comparison between measured and computed values shows that sand deposits on the beach in the northern section are half the estimateious, whereas the basin deposits are 2.5 times those expected. This can be explained by the onshore transported sand not being able to overcome the steeper strand. In the central section, sand deposits vary from 500 m$^3$/year (+ 1 cm/year) to -100 m$^3$/year (- 2 cm/year) in the beach. Corresponding values in the basin are -500 to +1705 m$^3$/year (- 0.8 to + 10.5 cm/year), this is probably due to the enlargement of the bar inlets, favouring sand movement under waves and ebb currents.

The southern section has shown sand accumulations of 4735 and 4535 m$^3$/year on the beach and in the basin respectively (1 to 5 cm/year). Although seasonal variations may be larger due to the open condition of the coast a steady increase of sand deposits is confirmed. However, most of the sand accumulates on the strand.

4. CONCLUSIONS

As a conclusion the net sand deposits amounted about 25,500 m$^3$/year in the period considered. This figure is 35% larger than the 1978 estimate, probably by a reduced offshore sand transport under ebb-currents. However, the quality of the prediction seems good enough taking into account the scarcity of original data and the complexity of the phenomena involved.

It has become apparent that successive adjustments of the model, as that one performed in 1984, may allow to establish predictions very close to reality.

Following the 1984 inventory the main proposals focused on protective measures of the bar and moving about 40,000 m$^3$ of sand from the northern section to the central one, where erosion is more intense.

The dredging of the basin has been disregarded until checking the effects of the sand elimination from the upper beach.
Procès littoraux dans la plage de Vinaroz
Littoral evolution along the Vinaroz Beach

J. Serra Peris, Puertos e Ingeniería Oceanográfica, Universidad Politécnica, Valencia, España
J. I. Diez Gonzalez, Catedrático de Ingeniería Oceanográfica, Universidad Politécnica, Madrid, España

ABSTRACT: The growing demand of increasingly scarce beaches has stressed the study of littoral processes in all the coastal areas with the intention of increasing the knowledge of the basis for evaluating impacts derived from future acts.

This paper show the principal ideas and conclusions of the study on Vinaroz beach, in the Northern Castellon Coast. This beach has been generated supported on the Vinaroz Harbour but has shown a clear instability.

The historical cartography has been analysed pointing out the influence of the harbour constructions, the Ebro Delta and small rivers in the area.

The littoral dynamics and sedimentary conditions have receive special attention.

RESUME: La croissante demande des chaque fois plus faibles plages, a augmenté l'étude des procès littoraux dans toutes les zones côtières, au but de développer la connaissance des bases qui peuvent être utiles pour l'évaluation des impacts dérivées des futures actuations.

Nous présentons le développement et la conclusion de l'étude réalisé à la plage de Vinaroz, à l'extrême Nord de la Côte de Castellon. Cette plage a été engendrée appuyée sur le port de Vinaroz, présentant une claire instabilité.

On analysé la cartographie historique, l'influence des ouvrages du port, du delta de l'Ebro et d'autres fleuvers plus petites.

La dynamique littorale et la sedimentologie mérite aussi notre attention.

1 INTRODUCTION

La ville de Vinaroz se trouve à l'extrême Nord de la Communauté Valencienne, dans le terme de la province de Castellon avec Tarragona. Le réseau fluvial est dense puis que sont quatre les fleuves qui débouchent dans les 10 kilomètres de littoral avec qui compte. Ceux sont (De Nord a Sud): le fleuve Cenia, le fleuve de Barbigera, le fleuve cervol et le ravin D'Agua Oliva, (figure 1).

La côte est une falaise moyenne formé par des conglomerats constituées par cailloux de différents grandeur, cémentés avec des matériaux plus fins, inclus argiles, dans la quelle l'action de la mer conforme des petites côtes, avec des plages de cailloux et de quelqu'une sable, alterné avec cônes de déjection plus ou moins fossile.

Une description très claire nous pouvons la trouver écrite par Cavanilles, qui dit: "En estas Costas las olas por lo comun baten contra un terreno duro, pocas veces de piedra, y muchas de un hormigón endurecido, compuesto de chinas, cantos y marga arcillosa roxa con algunas arenas. A fuerza de los choques de las olas y de renovarse la humedad, se ablanda la base que sostiene el cortejon y se des
cerna: presenta al principio hacia el mar excavaciones y cuevas; cayendo después al agua por su propio peso masas considerables. Mientras permanecen alii caídas, sirven de parapeto a las furias de las olas, y defienden por algún tiempo la por
ción con la cual estuvieron unidas; pero cediendo en fin quedan expuestas a igual suerte las que en otros siglos existieron sin riesgos, y de este modo va haciendo el mar lentas conquistas en las costas." (Cavanilles, 1795).

Dès cette description, réalisée en 1795, la côte a changé peu. Elle continue présentant des blocs détachés de la falaise et que peu à peu sont dispersées et pourprent une étroite plage au pied d'elle formée pour de cailloux et du sable. Plage qu'on fait extenese dans les cales que s'étendent tout au long de la côte.
Dans ce rapport, nous analysons la plage de Vinaroz, connaissant que le nom la
qui est située entre le port de Vinaroz et la débouchure du fleuve Cervol. C'est
une plage relativement étroite, les matériaux sont sédimentaires et surtout
en gravier dans toute la zone. Existent aussi des formations sableuses mais ces
sont de petite importance.
Aujourd'hui, on a construit un épi en "M" au sud de la débouchure du fleuve
Cervol avec la finalité de favoriser la formation d'une plage entre l'épi et le
digue du port et de répondre de cette façon à la demande d'une plage qui existe
à la ville, offrant des alicients au tourisme.
Nous allons analyser diverses aspects de la plage de Vinaroz et nous obten-
drons ainsi une série de conclusions sur sa évolution et ses caractéristiques,
(Figure 2).

2 L'EVOLUTION HISTORIQUE A LA PLAGE DE VINA ROZ

L'information historique qu'existe de la plage de Vinaroz n'est pas très complète.
La référence aux XVIII et XIX siècles n'est pas d'utilité par manque de référence
topographique. De cette façon cette application manquerait du suffisant rigueur
scientifique. Pourtant, nous optons par analyser l'information qui existe dans
notre siècle, avec le profil topographique de la ligne de la côte pendant les
années 1936, 1975 et 1.985 et qu'on peuvent voir à la Figure 3.
A l'heure d'étudier les variations de la ligne de côte que sont minimales les éros
sions son centrées dans le secteur Nord et Central et sont rares les croissances
dans la zone proche à la digue du port de Vinaroz. Est étonnant la forte croi
ssance au Sud d'épi en "M", mais peut être prévu en fonction des caractéristiques
et de la construction de l'épi. D'autre part on peut observer que les plus gran
des variations survenirent entre 1936 et 1975, mais ne doivent pas être considé
rés comme générés par un période fortement érosif, mais comme l'accumulation des
progressives érosions dont le résultat est une réponse à un intervalle de 39 ans.
On peut dire ainsi, que la ligne de côte est légèrement régulière, avec tendan
ce au basculement vers le dique du port sans augmentation, apparentem, d'ampleur
près de celui-ci.
Conjointement au topographie de la plage de Vinaroz en détermine bathymétrie,
dont les profil se représentent à la Figure 4. Quoique la bathymétrie a été rea
lisées immédiatement après de la saison hivier, on n'aperçoit pas la barre de bri
sants dans la zone centrale, ce qui dit de la situation précaire de celle-ci et
permet attacher au volume de la plupart des matériaux, la permanence relative de
sa forme en plan. Un augmentation important des matériaux sableux dans le stran
emprunterait vers un basculement plus rapide. Dans la zone centrale la barre appa
rait légèrement en 1975 à une profondeur de trois mètres.
A mesure que nous avançons vers le Sud, la barre paraît se situer à deux mètres de
profondeur, en 1985, pendant que en 1975 on détecte aussi à trois mètres. La
faute de données sur la date dans la qu'on réalise la bathymétrie de 1975, empê
che plus grandes conclusions à l'égard de Vinaroz.
À la fin signalons que la rare profondeur de la dernière barre peut être en
relation avec la faible dynamique littoral dans le dernier hiver.

3 LE CLIMAT MARITIME

On a réalisé un étude sur le climat maritime dont l'objet était évaluer la capa
cité et le sens du transport solide parallèle à la côte.
Du à l'absence de données réels sur l'houle, on a recours à la détermination
d'un régime d'houle, en appliquant la Méthode Intégrée de P.S.Bores et partant
d'un régime de vents déterminé en fonction des Cartes météorologiques de Admirant
lly Britannique, et qu'on peut considérer valable pour unesi premières approchemen
au problème ( J.J.Biez, 1982).
Pour déterminer le volume et le sens du transport on a utilisé la formulation
du C.E.R.C., puisqu'il faut faire de certaines observations sur les résultats ob
tenus, principalement aux qu'on référent au valeur de le volume, ct celles sont
telles comme que le calcul proportionne une délimitation supérieure du transport
réel et que non doit pas analyser avec exclusivité la composante horizontale, mais les
valeurs qui correspondent au transport dans un ou autre sens du littoral.
Les valeurs déduits pour la plage de Vinaroz ont été les suivants: ( le signe
positif correspond à un sens Nord-Sud, ou de gauche à droite apposés à la mer,
pendant que le négatif c'est le contraire).
Le volume net, moyenne annuelle on chiffré en 515.000 m³/an. avec une relation
ne très haute entre un sens et un autre du transport, si l'on compare avec des
résultats obtenus dans autres secteurs du littoral plus au Sud de Vinaroz. Le sens net est Nord-Sud, puisque dans cette époque de l'année domine le contrai-
re, fondamentalement dans l'automne.

L'affirmation de que la valeur du transport à travers d'une formulation mathé-
matique, comme celle du C.E.R.C. ici employée, c'est la détermination d'une dél-
imination supérieure du volume réelle, nous la donne la valeur du transport défini-
nie par comparaison des bathymétries entre 1975 et 1985, que nous proportionne
une perte des matériaux solides de 38.000 m³/an.

Aucune des valeurs doivent être considérés comme réelles, mais comme délimita-
tions supérieures et inférieures, les données réellement à considérer sont: la
rélation entre l'un et l'autre sens et le sens du transport. Pour nos cas existe
une certaine compensation entre tous les deux volumes, étant baissé la net con-
trastant avec valeurs déduits pour secteurs du littoral plus au sud; le sens net
est Nord-Sud qui domine dans certaines époques de l'année et que doit être tenu
en considération si l'on prétend réaliser un étude des proces littoraux dans la
zone de Vinaroz.

4 SEDIMENTOLOGIE

L'analyse sédimentologique est centré dans l'étude de la granulométrie et miné-
ralogie des dépôts côtiers qui confirment la plage de Vinaroz. La granulométrie
a été déterminée en tamisant les montres avec les tamis de la série A.S.T.M.
et déterminant une série de paramètres granulométriques; s'est déterminée aussi
le contenu en carbonates des diverses fractions et de même montre complète pour à
la fin réaliser un analyse minéralogique sur une fraction complète ne carbona-
tée.

A partir de l'analyse granulométrique on comprend que les sédiments de la pla-
ège sont surtout mauvais et modérément sélectionnés, pouvant se classifier comme de
grain moyen avec bimodalité, une la fraction gravier et autre dans le sable; on
remarque clairement l'influence des apports locaux des fleuves Cenia, Cervol, le
ravin de Berviguet et Agua Oliva, avec plus poids spécifique pour les trois pre-
mières.

Le contenu en carbonates est élevé, supérieur au 70% et il est dû au grand con-
tenu dans les sédiments des éléments carbonatés suminisstrés par les rivières prochaines o comme le matériel disgregué qui provient des blocs détachées des
escarpes situés au Nord de la plage de Vinaroz.

L'analyse minéralogique nous donne un important contenu en quartz blanc dans
les sédiments de la plage de Vinaroz, entre el 40% et le 50%. D'autres espèces mi-
néralogiques trouvées sont le quartz rouge, bleu et noir, ainsi comme des grains de
tourmaline, de muscovite, de biotite et des minéraux du phye des opales. De l'ensemble des analyses granulométriques et minéralogiques on note l'
importance des apports locaux dans la formation de la plage de Vinaroz, apports
que, dans la actualité sont minimes dû à la construction des barrages dans les
courses des fleuves qu'ont réduit d'une manière importante l'arrivée des matériaux
sols à la dynamique littorale. D'autre part on deduit, en fonction de la miné-
ralogie, l'importance du fleuve Ebro, de ses apports, dans la formation des dé-
pôts côtiers au détecter la présence de quartz noir, bleu et fondamentalement de
biotite et de muscovite.

Dans la figure 5 et 6 on reprend les résultats de analyses granulométriques et
minéralogiques d'une des montres analysées, correspondant à la plage sèche en su-
perficie.

5 CONCLUSIONS

Sur i'analyse de l'évolution de la ligne de côté de la plage de Vinaroz, sur sa
dynamique littorale, ainsi comme de la granulométrie et sa minéralogie on peut
déduire les suivantes conclusions:

1. La plage de Vinaroz est légèrement régressive avec une tendance au bascula-
ment vers le port de Vinaroz.

2. La dynamique littorale nette est Nord-Sud, bien que existe un composant domi-
Figure 1

Figure 2 Plage de Vinaroz
nant contraire au sens contraire.

3. En fonction de la granulométrie existent une apport importance de matériel biseaux seulement produirait un plus grand basculement vers le port et créerait des importants épouvantes dans lui-même.

4. On denote clairement l'influence des apports locaux dans l'origen et dans la formation de la plage de Vinaroz.

5. La minéralogie fait marquer la grande importance du fleuve Ebro dans la formation des dépôts à la plage de Vinaroz.

6. À la fin la diminution des apports solides par les lits aux fleuves autant locaux comme du fleuve Ebro dans la dynamique générale.

Figure 3. Variations de la ligne de côte

- 1.985
- 1.975
- 1.936

2307
Fig. 4 (cont.)

Figure 5. Analyses granulométriques

Figura 6. Analyses minéralogique
REFERENCES


Engineering-geological and geotechnical investigation for slope stability evaluation of deep opencast mines in complex geological conditions

La géologie de l’ingénieur et la géotechnique dans l’analyse de la stabilité des talus des tranchées profondes dans des conditions géologiques complexes

Jiří Heršus, Stavební geologie, Prague, Czechoslovakia

ABSTRACT: The stability of opencast-mine slopes, several hundreds of meters high, has been studied by means of geological investigations and the measurements of geostatic stress, shearing resistance and deformational properties of the rock massif. On the basis of analysis made by the finite element method the development of slope disturbance has been predicted and monitoring system has been proposed. The data obtained using this system are employed for refining the prognosis of the slope behaviour during proceeding mining work.

RÉSUMÉ: Pour trouver la solution de la stabilité des pentes de l’hauteur de quelques cent de mètres, des mines à ciel ouvertes les valeurs de la contrainte géostatique, de la résistance au cisaillement et les propriétés de déformation de massif rocheux ont été mesurées et généralisées sauf des données géologiques. D’après une analyse de la méthode des éléments finis le procédé de la mouvement des pentes a été prédit et le système d’auscultation a été proposé. Les données obtenues à l’aide de ce système sont utilisées pour préciser le pronostic de la mouvement de pente auprès de l’avancement de l’extraction dans des mines.

1 INTRODUCTION

Along the northwestern border of Bohemia a big brown-coal basin of Tertiary age extends along the Krušné hory Fault, which is major lineament of European scale. Over a length of more than 50 km it stretches close to the foot of Krušné hory Mts. In this zone about one third of brown-coal reserves is concentrated and their working in several open-cast mines may threaten the stability of high rock slopes.

The schematic geological profile of the brown-coal basin /Figure 1/ clearly shows the conditions of coal exploitation. The Tertiary overlying claystones locally enclose sand and silt lenses, often with confined ground water. A relatively thin layer of Quaternary gravel at foot of the mountain range passes into debris with occasional blocks up to 100 m³ in size.
Figure 1. Schematic geological section of a part of the brown-coal basin. 1-gravel, rock debris near the foot of mountain slopes, 2-claystone, 3-brown coal, 4-sandstone, siltstone, 5-tuff, 6-gneiss, A= 500 - 800 m, B= 110 - 450 m.

The height of mountain slopes ranges from 500 to 800 m. They are built up of gneissses which are strongly tectonically affected. The thickness of the basin filling varies between 110 and 450 m.

The coal mining in the southeastern and central parts (Figure 1) of the basin does not present any difficulties, in contrast to the working in the northwestern part, i.e. at the foot of the mountains. There the inclination of the rock slopes is about 30° above the original surface of the basin filling, attaining up to 70° below this level.

At several places at the foot of slopes relics of rockfalls are evident. The largest one was the rockfall of Pleistocene age involving 20 million m³ material. The thickness of accumulated rocks was 60 m. This example indicates that a catastrophic slope failure caused by coal working at the foot of the mountain range cannot be ruled out.

For a detailed engineering geological and geomechanical study several localities have been selected that represent typical geological situation of the basin margin. The investigation procedure will be demonstrated with one of these localities (Heršťus 1983/).

At first, a tectonic study was carried out, complemented by using a number of geophysical methods aimed at ascertaining the position of faults and large continuous fractures. Their situation in space (i.e. completed by determination of dip) was defined more exactly by mean of boreholes and a trial gallery.
As the in situ stress of a rock massif is a very important factor for the analysis of slope stability, the measurement and evaluation of it were made with particular care. To begin with, a simplified parametric study /Aldorf 1983/ was performed of the influence of horizontal tectonic stresses using the finite element method /Figure 2/.

Figure 2. Detailed geological section and the plot showing the results of computation and measurement of geostatic stresses at the level of trial gallery. 1-solid gneiss, 2-altered gneiss, 3-dislocation breccia, 4-claystone, 5-slope debris, 6-brown coal, 7-continuous fractures in the gneiss massif, 8-trial gallery, 9-horizontal borehole, 10-vertical stress component corresponding to the weight of overburden, 11-vertical and horizontal stress components calculated for homogeneous massif /gravity forces only/, 12-vertical and horizontal stress components calculated for homogeneous massif exposed to lateral tectonic pressures of 1.2 φH, 13-vertical and horizontal stress components computed using FEM on the assumption that only gravity forces are active, 14-results of geostatic stress measurements in the gallery.

The measurement results have shown /Figure 2/ that within the slope extend the interpretation of the in situ stress of the massif in terms of mere gravity effects would be more conformable to reality. Measurements of in situ stresses in the rock massif were made using convergence method in the gallery and the stress relieving method by overcoring, with the use of a soft triaxial cell.

It is well known fact that the measurement of in situ stress of rock massif is one of the most difficult tasks of geomechanics and that the results obtained are often erroneous. For this reason the measured stresses was compared /Figure 3/ with the peak shearing resistance of the massif /envelope a/, with the residual shearing resistance /envelope b/, and with shearing resistance along the fracture surfaces /envelope c/. Many circles defining the in situ stress are evidently below the envelope which represents the weakest part of the massif. The stress determined by measurement is thus obviously in good agreement with the present stable state of the massif /Herátny 1984/.
The comparison of the orientation of the major, intermediate and minor principal stresses with orientation of trends of major fractures and fault lines shows a good accord /Figure 4/.

Taking into account the relatively small stress intensity of the rock massif within the compass of the slope, we have to assume a danger of stress concentrating in the rock basement of the basin. In order to check this supposition the horizontal stress components were measured in the basin area. /Figure 5/.

Measurements in clayey and sandy sediments were conducted using a selfboring pressuremeter, hydraulic fracturing proved competent in gneisses met in deep boreholes. It is obvious that whereas in basinal sediments the horizontal stress components are smaller than the vertical, the values of both horizontal components obtained in the underlying gneisses surpass the corresponding vertical stress component.
3 SHEAR STRENGTH AND DEFORMATIONAL PROPERTIES OF THE ROCK MASSIF

In predicting the behaviour of rock or soil slopes being unloaded during coal extraction preference is given to the finite element method. As input data for an analysis we need the records of the shearing resistance and deformation characteristics of both the massif and the fractures. In case the fracturing of the massif is intensive, the field tests give good results.

Very good correlation has been established between the modulus of deformation of the rock massif and RMR based on Bieniawski's /1979/ classification. The correlation between the modulus of deformation and the speed of longitudinal seismic wave propagation is closer /Figure 6/.

Figure 6. Relation between the modulus of deformation and RMR /on the left/ or the rate of longitudinal seismic wave propagation /on the right/

In case the degree of massif fracturing is not very high, the reliable measurement of its characteristics would require an inacceptable volume of test blocks and, therefore, the finite element method has been employed. The modulus of deformation, residual modulus of deformation, Poisson number, the shearing resistance of rock, and the shearing resistance along fracture with its normal and tangential stiffness have been used as input data into computation /Figure 7/. These values have been determined in laboratory. The computation by means of the finite
element method /Doležalová and Leitner 1985/ enabled us to determine the shear strength and the modulus of deformation of the block massif model.

The method has been tested on a strongly fractured massif, where the field tests still yield reliable results./Herštus et al.1983/. A very good agreement of shear strength values obtained by direct measurement with those given by FEM is seen from Figure 8.

Figure 7. Model used for computing the deforming properties and shearing resistance of rock massif

Figure 8. Comparison of predicted and measured values of the shearing resistance of the massif

The computation and measurement of the modulus of deformation also provided according results, which implies that this procedure is applicable in practice.

4 RESULTS OF STABILITY ANALYSE AND MONITORING SYSTEM

The presumed character of the disturbance of the slope examined has been assessed on the basis of analyses using the finite element method and of geological and engineering judgement /Figure 9/.

In soils and weak rocks of the basin filling the occurrence of landslide with compound non-circular sliding surface is presumed. As concerns the rock slopes, their near-surface rocks are expected to be disturbed by tensile stress, which may result in rockfalls /Herštus 1982, Rybář 1985/.

The analysis has shown that the preservation of relatively small pillar in the basin filling, which would imply only economically permissible loss of coal substance, can change a catastrophic slope failure into a disturbance that would not endanger the mining works.
Figure 9. Presumed stages of slope failure

With respect to the character of the inferred slope disturbance the monitoring system has been proposed and set up. It consists (Figure 10) of piezometers, probe inclinometers, strip failure locators, multiple linear extensometers and portable tiltmeters. Surface deformations are measured using an electrooptical telemeter.

Figure 10. Monitoring system and comparison of predicted and measured displacements
5 CONCLUSIONS

The results of measurements of deformations caused by coal extraction carried out until October 1985 (upper part of Figure 10) have demonstrated a relatively good agreement between the predicted and real displacements near beneath the surface, but rather great difference between the forecast and the reality in deeper parts of the slope.

With regards to that the non-linear analysis by finite element method essentially depends on the form of constitutive laws, iterative calculations have been proposed as a next step to coordinate the computed and measured values. The knowledge of constitutive relations thus improved will make it possible to predict the slope movements during further mining works with greater accuracy.

REFERENCES


Application of S.I. anchorages of soil traction
L’application des ancrages du type S.I. aux sols tractionés

F. Dante Seta, Seta Hidroviario Arquitectura y Tecnologia Estructural S.R.L., Argentina

ABSTRACT: This paper consists of the application of S.I. ANCHORAGES OF SOIL TRACTION used in the following structures: Footbridges over Pergamino Stream, Province of Buenos Aires (Sedimentary soil-mud - clay - sand)-Bridge over "El Soberbio" Stream, Province of Misiones (rocky soil)-Bottom structures-Pumping Station (D.I.P.O.S. Provincial Office of Sanitary Works-Providence of Santa Fe (sedimentary soil-sand).


TRACTIVE FOUNDATION INTO SEDIMENTARY SOIL
FOOTBRIDGES OVER PERGAMINO STREAM (Province of Buenos Aires)

Here are included design criteria and super and substructure construction technology applied to great span footbridges built up in Pergamino City.
The 81 m effective span has been resolved with rational lines and forms directly introduced into the sites, studying the convenient modulation of a structure suitable for the peculiarities of the river bed. The strip or bond between the end supports possible to be constructed in the banks of the stream has been structured on the basis of two cantilevers of 22.5 m span and 22 m central isostatic span supported on the ends of each cantilever.
The end cantilever spans, articulated into a pile header which receives the auto-weight vertical reactions and the footbridge overload, are stabilized by means of active anchorages of soil traction on a die concreted in situ, with a proper lever arm suitable for achieving the balance of the overturning moment stressed in the embedding of these floor bands.

CONSTRUCTIVE PROCESS:
The isostatic and bracket span were precast in plant; the foundation structures of both banks were constructed in situ; assembly pile header-turnbuckle anchorage die, piles and S.I. ANCHORAGES OF SOIL TRACTION. In the mass of anchorages, bond and embedding turnbuckles were forecast in the precast cantilever spans that must be jointed to the soil anchored mass to transmit the loads according to the planned structural diagram. In the first place, cantilever spans of both banks were positioned and gathered with the help of cranes of appropriate capacity; the ends were temporarily supported in an auxiliary steel strut; the seal was concreted and tightened with the mass of foundation anchorages; so that the momentary stresses correspond with the temporary supported diagram.

After the cantilever spans were lined, the mounting of the central isostatic span was carried out going forward from one of the bank; one end being supported in the traction unit and the other, hanging from a telescopic crane working in a service earthfill.
The stressing of the definitive turnbuckles was simultaneously carried out with the recovery of the temporary turnbuckles.

CONSTRUCTION TECHNOLOGY OF S.I. ANCHORAGES:
The turnbuckles of S.I. ANCHORAGES were prepared in plant with high-resistance steel cords to be latter placed, and sheathed in steel jacket or P.V.C. Externally a steel mesh of Ø 4.2 mm of 15 cm is arranged. The steel sheath diameter is function of the turnbuckle power. To the mentioned frame spacers are bonded to center the sheath inside the steel mesh and the assembly in regard to the drilling. The turnbuckles prepared in this way, were rolled in ø 2.2 m diameter to be transported.

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The drilling of the soil was carried out with a rotary drill that caused inverse circulation of bentonite mud within an approximately 25 cm diameter.

After getting the anchorage elevation, the tools were changed and an expander was adopted to drill a bigger diameter until obtaining twice the dimension of the shaft. Thus, the anchorage capsules were shaped with a length experimentally determined as convenient and verified by calculus.

Then the anchorage was inserted despite the tixotropic liquid medium, paying attention to center it in the drill socket.

The turnbuckle head shaping the anchorage remains naked while the rest of the raw shaft takes the place in the capsule or anchorage bulb protected by the steel sheath.

A down pipe reaching the bottom of the drilling is used to inject the cement paste that dislodges the bentonite mud and slits up the volume corresponding to the tubular drill bit formed between the drilling section and the turnbuckle protection sheath all along the raw shaft.

The cement injection must emerge at the surface. The construction technology must be carried out very carefully, paying special attention to the cast in situ of the big-diameter piles, i.e., the mouth of the feeder pipe should not leave the cement stratum during the process. If the process is interrupted, the bentonite and the cement are mixed.

Accomplishing this technology, the first amount of injected material mixed with bentonite is recovered at the surface. The injection should continue until the cement paste penetrates and emerges with the same density and consistency.

The continuous and surrounding penetration of the paste from the bottom elevation produces by abrasion the cleaning of the turnbuckle active elements which remain placed, imbedded and confined to the anchorage capsule. In this way it is possible the bond of cords and bulwark traction efforts are developed and transmitted to the surrounding ground. The soil is considered as an effort generating nucleus or epicenter transmitting to the upper strats conoid volume the balance condition required for the anchorage.

When the nucleus is hardened, the turnbuckle is stressed from the surface through the support plate and the post-tightening and the fastening of the plate into the ground are inserted; the superstructure would be bonded to this plate.

Before the turnbuckle is tightened, the injected shaft must be isolated from the upper support plate to avoid the prestressing; in this way the shaft has a free distortion and the plate has always compatible with the resultant tension of the ground.

This tightening operation means to stress all the elements of the turnbuckle, its anchorage included, to the allowable working stress with a safety factor determined by the working load. Unlike the passive anchorage, and considering the advantages of no sliding or overturning distortions of the superstructure to which are bonded, the active anchorage are a guarantee of operation because the effort is inserted before the working stress and this is a load test in itself.

The turnbuckle protection guarantee is achieved against the corrosion agents because before attaching the steel cords they should pass through a concrete strong barrier, a steel sheath and the nucleus injection.

The last nucleus injection is carried out in the interior of the sheath sealing it and the upper discontinuous span of the shaft, just beneath the plate, serving up all the existing spaces.
SPAN OF CANTILEVER BEAM
FILE HEADER
NATURAL SOIL LEVEL
PRESTRESSED BLOCK SOIL ANCHORAGE
PRESTRESSED CONCRETE CORE WALL
BEAM-HEADER BONDING BY PRESTRESSED

FREYSSINET JOINT

COMPRESSION FOUNDATION
WITH PILES
ANCHORAGE SHAFT
BULB OR ANCHORAGE
CAPSULE

FOUNDATION ANCHORAGE INTO
SEDIMENTARY SOIL

DRILLING
Ø 250 mm
STEEL SHEATH
CEMENT INJECTION
PRESTRESSED CORD

DRILLING OF THE SOIL
Ø 450 mm
INJECTION MORTAR
REINFORCEMENT TO
AVOID FISSURES
# Ø 6 mm / 15 cm
PRECAST COWL

DETAIL OF S.I. PASSIVE ANCHORAGE INTO
SEDIMENTARY SOIL
1. GEOMETRIC CHARACTERISTICS

Total span: 202.42 m, formed by 4 static spans of which 5 have 29 m and 2 have 28.71 m. Each span is made up of 4 "I" prestressed beams, the upper flanges shaping the floor and structurally bonded by transverse beams. The transverse continuity of the floor is achieved with a sealing concrete and bonding frames.

The main beams are precast: they are supported on the corresponding piles and abutments by means of a transverse beam bonded to each pillar and forming in the abutment as well as in the intermediate pile a tubular hollow-section shaft of precast concrete. This assembly is bonded to the foundation rock by a concrete "node" built up in the place of each foundation, embedded to the basalt rocky soil through the drilling of some active turnbuckles. These active turnbuckles prestressed by efforts of about 1000 t are shaping the deep embedding taking advantage of the excellent conditions offered by the surface rocky soil that acts like a natural element of foundation. This circumstance makes possible the construction of the bridge using the modern technique of prefabrication in plant and later transportation and mounting in situ. This would be also applied to the structures of piles and floor beams minimizing the preparation of the concrete in situ. Only the concrete of the seal that gives transverse continuity to the slab and to the stiffening beams is prepared in situ.

In the design of the abutments or false abutments was applied the same structural criterion of intermediate pile concerning to the absorption of stresses coming from the superstructure and the vertical and horizontal reactions. Considering the soil push effects or hydrostatic pressures of the water tension generated from the action of Uruguay River Dam and following the general idea of using natural material easily obtained, the ends of the earthfill were designed with a rockfill that reaches the floor beam elevation till the half of its head and allows the free support of the structure. The rockfill, formed by blasting rubble stone of good performance in its treatment, weight and internal friction, is, if adequately arranged, a real structure acting like a breakwater or cofferdam able to resist perfectly not only the horizontal pressure of the earthfill soil but also the hydrostatic and hydrodynamic actions and protecting from the erosive action of the flood water in the head, even in the hypothetical case of any sharp descent of Uruguay River Dam.

This granular structure, like dams or breakwaters, has an excellent performance against the horizontal pushes and also presents the peculiarities of a granular mantle. An optimum drainage is guaranteed. From the functional and structural point of view, as this material is almost incompressible and without slippings, there would not be the typical slumps and rotations, as frequently observed in the access to the bridges, and subsequent sinkage of the approximation slab.

2. DESCRIPTION OF THE STRUCTURE: FOUNDATIONS - CONSTRUCTION TECHNOLOGY

Coming from the actions of the superstructure and piles (water and wind powers) the resultant of
the vertical, horizontal and moment forces should be transmitted into the foundation soil-rock, in this case to cause the corresponding edge stresses and reactions that would be balanced with the resulting tension.

In this design the balance is achieved considering the tubular hollow-section pile, precast and prestressed concrete, as embedded into the foundation soil. The embedding is undertaken constructing a nodular element that constitutes the bond device between the rock and the pile elevation. To construct the connecting nodules over the sound rock it would be necessary to blast the enough amount of rock to get the rock elevation where the nodule would be concreted. In this reinforced concrete nodule, properly bonded and with sufficient steel sheets inside the concrete mass radially arranged due to the circular section of this element, 4° 8 drillings would be carried out coinciding with each sheet until reaching the elevation, in depth, that guarantees the embedding according to the calculus.

This anchorage depth determines a circular section approximately 5 m in diameter; from this section and 10 m depth it is generated a truncated cone, the minor base downwards and 5 m in diameter, the major base, coinciding with the surface of the rock, approximately 25 m in diameter. If it is adopted a 45° angle as the starting failure angle, this failure would come about after the cut parameters of the surface casing of the truncated cone are surpassed, this signifies that the anchoring capacity should be given by the weight of the rock mass defined by the mentioned truncated cone.

The anchoring capacity, assuming a 10 m anchorage depth, the volume of the resultant and casing rock of the stress bulb and its equivalent load is about 800 to 1000 tn.

Placed, tightened and injected, the anchorage turnbuckles, the rock mantle and the concrete nodule are precompressed in such a way that for the maximum horizontal and flexure stresses coming from the piles, the edge stresses would remain in compression values.

The rock truncated cone is prestressed starting from the surface nodule with a maximum value previously compensating, with the security coefficient, the action of the real stresses, meaning as mechanism that compression stresses are introduced opposite to the traction stress. Thus the stress of the structure is introduced in a way similar to the prestressed structures in general and also means a real load test that guarantees the good performance of the natural implanted elements, i.e. the rock and the steel ribs. As mechanism also means that vertical and horizontal elevations should not appear because what is produced is the decompression of the rock and not the elongation of the anchorage turnbuckles, already happened when introducing the corresponding efforts.

Therefore, the feasible rock fissures are “made a seam” avoiding the possible corrosion of the turnbuckles.

Each turnbuckle is made of high resistance steel cords, with a failure 190 kg/mm², inserted in radial drillings and countersunk into steel sheaths to protect them.

In the connecting nodule are arranged the anchorage of the assembly turnbuckles of the pile elevation structures. The nodule of the water piles are built up with the help of a concrete jacket precast in plant.

This casing, truncated cone shape, is formed by three staves of approximately 120°, once prefabricated the casing is transported with its total height. In situ, the staves are assembled bonding them to the scaling concrete and bonding reinforcement; sheaths are arranged in these seals to thread the temporary turnbuckles that should anchor the jacket avoiding the subpressure effect of the river water tension.

After the topographical survey and the cleaning of the rocky river bed are done, the mentioned casing is presented materializing a contact seal of forging material adapted to the irregulari-
ties of the rocky river bed. The tightening of the bonding turnbuckles is undergone from the upper edge of the jacket. This effort introduces compression stresses into the contact-seal section between the rock and the concrete jacket and generates stability forces enough to balance the subpressure effect due to the unwatering of the internal enclosure that brings about water tightness and makes possible the digging of the foundation elevation. Being completed this operation, the bottom and walls of the digging are strongly cleaned; the natural roughness of the surface is exposed. Frames, sheaths for the prestressed anchorage, frames for the S.I. TURNBUCKLES ends, etc., are arranged, the enclosure is concreted and the S.I. ANCHORAGES are tightened.

CONSTRUCTION TECHNOLOGY OF S.I. ANCHORAGES

In relation to the turnbuckle power, the drilling diameter and the length are determined. The drilling is accomplished with drifter and pneumatic rotary drill. In relation to this disposal, devices and material are designed sized to resist the corrosion. After the location, the anchorage is injected, the turnbuckle is tightened, the seal is injected silting up the vacuum and the last nucleus is also injected. The construction technique is similar to that of sedimentary soil anchorage construction but is different from the anchorage capsule shoping described for the footbridges of large span.

TRACTIVE FOUNDATION IN SAND

BOTTOM STRUCTURE - PUMPING PLANT - D.I.P.O.S. (Provincial Office of Sanitary Work) - SANTA FE CITY

STRUCTURE CHARACTERISTICS

Submerged slab of prestressed concrete formed by precast elements tied to soil with traction active anchorage and joint watertight-seal, designed to resist the hydrostatic subpressure action of 12 tn/m².

OPERATING CONDITIONS

The structure briefly described was constructed for the enclosure corresponding to the pumping chamber of the Pumping Plant of the main sewer system of residual effluents of Santa Fe City. The chamber is buried 12 m below the natural ground level and is subjected to hydrostatic pressure in walls and bottom (subpressure effect) with a fluctuating load head resulting from variations in the river elevations. The chamber enclosure, rectangular, 30 m length x 12 m width, has been shaped as a continuous perimeter wall of reinforced concrete, dug and concreted up to 9 m below the lower elevation of the bottom slab. At first, the chamber was designed of reinforced concrete because it has been planned a dry construction after digging the interior enclosure and lowering the napa; the enclosure would act as a core wall of the unwatering system. During the unwatering (of the enclosure), soil failures we
re produced by siphoning with important underground washouts and sinkage of the grounds nearby the siting of the works.
The process was stopped and it was necessary to design and calculate the bottom structure relying on technologies able to work 12 m under water and able to resist the subpressure and watertight to allow the unwatering of the enclosure.

DESIGN CRITERIA AND HYPOTHESIS OF STRUCTURAL PERFORMANCE

To facilitate its descent and siting in the enclosure, the bottom structure was resolved using precast elements of "U" section prestressed concrete, modulated width and variable length, compatible with the support strut of the existing perimetral wall. These ribbed elements unload the subpressure efforts into precast transverse beams of prestressed concrete and solid section, with continuous lateral brackets placed over the ribs of the "U" elements to which the precompression resulting from stressing the S.I. ACTIVE ANCHORAGES is transmitted.
The flexure span transmitting the compensation precompression of subpressure of the bottom ribbed-elements, is defined by the length between the lateral projecting ribs of the beams. The beams act like isostatic elements elastically dislocated from the S.I. ANCHORAGES.
The anchorages are symmetrically arranged to materialized an over-hanging spans that balance the span bending moments.
The ACTIVE ANCHORAGES OF SOIL TRACTION guarantee the tie of the structural assembly and transmit the stresses into the soil natural structure prestressing the high-resistance steel turnbuckles and introducing precompression efforts bigger than the tractions resulting from the bottom subpressure.
The subpressure generated by the action of the external water tension when the interior of the enclosure is emptied, activates the structural mechanism transmitting the reactions to the soil through the decompression without increasing the previous stresses of the S.I. ANCHORAGE TURNBUCKLES; the turnbuckles remain with a compression reserve during the whole service stages of the structure.
The watertight of this precast bottom structure that made possible the next works, is resulting from the summary of different favourable conditions: permanent compression of the "U" elements points and beams; active precompression of the structural assembly in the bottom of the digging; uniform layer lower disposal of granular frame, tilted up by pressure injection of cement paste impermeable membrane incorporated; latter placing of a rock upper mantle interconnected through foreshed drillings in the beams that made possible the concreting of the leveling and terminal slabs controlled by unwatering, pumping little infiltrated flows.

GEOMETRY, DIMENSIONING HYPOTHESIS AND STRUCTURAL MATERIALS

Hypothetical subpressure obtained by calculus: 12 tn/m², function of the maximum head of the external water tension.
All the structural elements were precast with stone concrete, 250 kg/m², characteristic resistance with cement (A.R.S.)
The precast "U" section elements, structuring the bottom slab, are modulated in 86 cm width x 2.5 to 4 m length.
The transverse beams adopted a uniform geometric design conditioned by the bigger stresses and having on 1.1 m width rectangular section.
For the threading and anchorages of the S.I. ANCHORAGE TURNBUCKLES, oval drillings were made in the beams, compatible allowance to the mounting tolerance; coinciding with the drillings the transverse section of the beams was increased to avoid reducing the resisting capacity.
The described structural assembly is balanced by transfer of its tractions into the soil through S.I. ACTIVE ANCHORAGES. The ANCHORAGES were dimensioned in relation to the muddy sand characteristics, considering an initial stress P = 180 tn, a bulb length or anchorage capsule of 5 m necessary for the bond transmission under compression and a shaft length or raw shaft of 20 m. Therefore, the soil resists due to the summary of auto-weight of the theoretical failure cone and to the development of the active out-resistance.
Each S.I. ANCHORAGE TURNBUCKLE is formed by 14 cords of high resistance prestressed steel, 1/2" in diameter, furnished with helicoidal frames of anchorage bulb, protection sheaths and injections pipeline.
The construction technology of the S.I. ANCHORAGES has similar characteristics to that of the Pedestrian Bridges over Ferganinho Stream.
INTERNAL BEAMS AND STRUTS TO STIFFEN THE WALLS

REINFORCED CONCRETE PERIMETRIC WALL BUILT UP IN THE 1st. STAGE

REINFORCED COATING 2nd. STAGE

BOTTOM STRUCTURE

HYDROSTATIC SUBPRESSURE 12 tn/m²

ACTIVE ANCHORAGE = 180 tn

BOTTOM STRUCTURE - CROSS SECTION - SC: 1:100

SUPPORT GRANULAR SKELTON

precast "U" elements Drilling by injection

Lower layer of lower layer of
rocky material impermeable membrane

Steel tube-drilling guide

DETAIL OF S.1. ANCHORAGE = BOTTOM STRUCTURE - SC: 1:20
1. Internal digging of the enclosure until reaching the bottom elevation of the structure.

2. Cleaning and leveling of the digging bottom.

3. Simultaneously, the bottom structure resisting elements are precast in fixed plants: transverse beams and "U" elements. Then, they are transported and stowed in situ to be submerged and positioned at the bottom of the chamber enclosure.

4. Picking and cutting of the enclosure lateral walls to wedge it over the modules of the bottom structure.

5. Placing of the guide tubes to carry out the drillings for S.I. Active Anchorages of soil traction. The drillings are made up by unskilled workers with the help of driving guide jacket from the surface.

6. Anchorage drilling construction, with bulb widening, using rotary rib and circulation of bentonic mud.

7. Cleaning of the enclosure walls in the area of contact with the bottom structure to facilitate the injection paste bond of joint and vacuum.

8. Placing of the power layer of rocky material to compact the bottom.

9. Placing of the bottom impermeable membrane and latter placing and leveling of the granular frame regulator to form the layer to be injected over the membrane.


11. Underwater descent and sitting of the precast elements.

12. Descent and sitting of adjustment modules.

13. Pressure cement injection of the granular frame over the impermeable membrane to slit up the joints, vacuum and external lateral areas of wedging.

14. Underwater tightening of the S.I. Anchorages Turnbuckles of soil traction, being the soil precompressed by stresses to neutralize the subpressure effect.

15. Unwatering of the enclosure.


17. Placing of the upper mantle of granular frame and construction in concrete of the upper leveling and terminal slab.
Engineering geology and land-use planning in New Zealand

La géologie de l’ingénieur et le planning de l’amenagement du territoire en Nouvelle-Zélande

D.H. Bell & J.P. Pettinga, Geology Department, University of Canterbury-Christchurch, Christchurch, New Zealand

ABSTRACT: We have developed an approach to land-use planning that involves air-photo interpretation, engineering geological mapping at suitable scales, limited subsurface investigation with material characterisation, and data presentation in an appropriate format. The technique has been particularly applied to the residential subdivision of land for urban purposes, and specific case studies are reviewed to highlight both data collection and presentation methods: the legal and professional implications of such an approach to urban land-use planning are also briefly examined. Extension of the methodology into other areas of environmental planning is outlined, with selected examples relating to farm management practices and soil conservation matters, and we believe that the engineering geological approach is in fact a cost-effective and rational basis for environmental and land resource management in New Zealand.

RESUME: Nous avons mis au point une technique de planification de l'occupation des sols à plusieurs volets faisant intervenir: l'interprétation de photos aériennes; la détermination partielle de propriétés physiques des matériaux à faible profondeur; et enfin, la présentation des données sous un format bien adapté. Cette approche a été appliquée tout particulièrement à la répartition, en milieu urbain, des terrains à usage résidentiel et quelques cas types sont passés en revue afin de mettre en évidence les méthodes de collecte et de présentation des données. Les implications légales et professionnelles d'une telle approche de la planification de l'occupation du territoire sont brièvement exposées. L'élargissement possible à d'autres branches de cette méthodologie est démontrée par des exemples pris dans les domaines de la planification agricole et de la conservation des sols. Nous en concluons que les méthodes du génie civil se révèlent d'un très bon rapport qualité/prix et forment une base rationnelle à l'organisation de l'utilisation des ressources et la protection de l'environnement en Nouvelle Zélande.

1 INTRODUCTION

Engineering geology data input is an accepted requirement for the design and construction of major engineering projects in New Zealand, and its importance in assessing project feasibility is increasingly being recognised (Bell 1984; Bell & Pettinga 1984). In the fields of environmental planning and urban development, however, the application of engineering geology has been less than satisfactory, despite its obvious relevance to hazard evaluation and foundation or site assessment (Bell, in press). The subdivision of land for residential purposes clearly requires a staged geotechnical approach (Taylor et al 1983), and engineering geological data input provides a cost-effective means of rational land-use planning by the early identification of site limitations (Bell & Pettinga 1985). Likewise, in determining optimum land-use for rural or forested areas an engineering geological approach can rapidly identify geotechnical constraints and available development options, as well as assisting in the establishment of appropriate resource inventories (Pettinga & Bell 1984).

Our objectives in this paper are to outline an engineering geological approach to land-use planning, and to briefly review its applications within the current legislative framework for environmental management in New Zealand. Specific case histories deal with urban development practices and investigations for rural land-use, whilst our methods of data presentation are illustrated by the reproduction of selected consultancy figure originals. This paper is not intended, however, as an overview of land-use planning practices in New Zealand, but rather to demonstrate the relevance of engineering geology principles and methods in a variety of engineering and environmental management situations.
2 GEOLOGICAL SETTING

New Zealand is located at the boundary of the Pacific and Australia-India Plates (Fig 1); it is dominated by a dextral transcurrent fault regime in the South Island, and by a subduction zone in the North Island with which is associated both andesitic volcanism and gravity tectonics (Paterson 1982). The Southern Alps, which have formed by uplift along the Alpine Fault Zone in the west of the South Island, reach to more than 3000m above sea level and contain both permanent snowfields and small glaciers; these are remnants of larger glaciers which formed much of the modern landscape during the cold-climate episodes of the Quaternary Period, and which were responsible for extensive down-valley aggradation and loess accumulation. Large tracts of the central North Island show the influence of Late Quaternary volcanic activity, and eruptions continue episodically from several vents (Fig 1): the city of Auckland, which is the largest in the country, is partly built on basaltic rocks for which there is evidence of localised eruptions within the last 10,000 years (Dibble & Neall 1984; Larter 1985).

The oldest rocks in New Zealand are late Precambrian gneisses, and these outcrop, together with fossiliferous lower Paleozoic strata and younger granitic rocks, to the west of the Alpine Fault. To the east there is extensive development of upper Paleozoic and Mesozoic schists and their "parent" greywacke strata, which also form local basement over much of the North Island. Widespread marine transgression followed fluviatile sedimentation and coal measure deposition onto basement in the late Cretaceous or early Tertiary, and the Oligocene was characterised by both limestone formation and volcanic activity: a regressive sequence was deposited in the Miocene and Pliocene, and tectonic disruption of the depositional basins is marked both by an increasing coarse clastic component in late Pliocene-early Pleistocene sediments, and by associated folding and/or faulting. It is, however, the extensive Late Quaternary sedimentation and ongoing tectonism (Fig 1) that characterises the "youthful" New Zealand landscape, with many areas also subject to episodic flooding and in steeper terrain a variety of slope movement types: New Zealand is thus subject to virtually all of the geological hazards that influence land-use planning, and these can be grouped as settlement, landslip, flooding, erosion, seismicity, volcanicity, and loss of resources (Taylor et al 1983; Bell in press).
3 INVESTIGATION METHODOLOGY

We have adapted conventional site investigation techniques (Bell & Pettinga 1984) to meet various land-use planning requirements, the prime objective being to develop an engineering geological site model from which to identify those geotechnical factors affecting a specific proposal. The methodology employed (Bell & Pettinga 1985; Pettinga & Bell 1984) involves:-

1. air-photo interpretation, preferably using stereo-pairs flown at different ground scales and times to permit identification of specific features.

2. engineering geological mapping to show at an appropriate scale both bedrock and surficial geology, as well as relevant geomorphological and hydrogeological features.

3. subsurface data collection by logging natural or artificial exposures (such as test pits), from disturbed sampling of hand-auger holes, or from geophysical techniques (for example, seismic refraction) if available or appropriate.

4. geotechnical testing to quantify material properties, either using a limited array of laboratory methods for characterisation of rock and soil materials, or from "indirect" field techniques such as the Scala penetrometer.

The project objectives and the terrain characteristics determine the emphasis given to the various investigation methods, although the approach aims specifically to: 1) identify geological hazards affecting the land in question by reconstructing a history of landscape (or site) development; and 2) evaluate existing or potential geotechnical problems in terms of engineering geology. In many respects the investigation methodology is that of feasibility studies for civil engineering projects (Bell & Pettinga 1984), and also differs very little from so-called terrain evaluation (Geological Society of London 1982) except that we do not proceed to a comprehensive system of terrain classification (such as that developed by Grant & Finlayson 1978). Whilst acknowledging the value of weighted multi-parameter zoning systems as a means of land resource data compilation, we remain skeptical of their "blanket" application to planning matters without the incorporation of adequate data on geological hazards and material properties. As an example, Molineaux (1983) carried out engineering geological mapping at 1:1000 for Moeraki Township (North Otago, New Zealand) in order to assess the landslide hazards affecting residential land and properties, and to provide site-specific input into the formulation of revised land-use criteria: any smaller scale of mapping would have obscured the extremely complex nature of rotational slides and earth flows that are developed on the metasepites-rich mudrocks which underlie much of the Township.

4 REGIONAL AND DISTRICT PLANNING

Urban planning in New Zealand is administered principally under the Town and Country Planning Act 1977 and the Local Government Act 1974: the former Act is concerned with various aspects of regional and district planning, and requires the establishment of operative Regional and District Schemes by territorial authorities. Regional planning is the function of every regional or united council, and its general purposes include the wise use and management of the region's resources as well as the direction and control of its development: included amongst the Regional Scheme provisions is the exclusion from future urban development of land subject to hazards such as flooding. The District Scheme provides for effective planning and control of land-use in the area administered by a local authority, and the Act specifies a number of natural hazards which are to be avoided or whose effects are to be minimised: these include earthquakes, volcanic activity, flooding, erosion, landslip and subsidence. The Soil Conservation and Rivers Control Act 1941 established "local" catchment authorities whose functions include river control and various drainage works, as well as soil conservation and erosion control measures: the Water and Soil Conservation Act 1967, whilst primarily concerned with the allocation and quality of water resources, also established the National Water and Soil Conservation Organisation (NWASCO) which has important research functions in areas of erosion assessment and land-use planning. A number of other Acts relate to such areas as forestry, whilst a complete reorganisation of environmental planning and management is presently being undertaken in New Zealand, with a proposed separation of conservation and development functions.

A Land Resource Inventory Series (NWASCO 1979), in which data are compiled at 1:50000 and 1:25000 and combined into some appropriate zoning classification, have also gained acceptance for land-use planning. To date there has been, however, only minimal development of urban geology mapping techniques (Hancox 1983) for land-use planning in New Zealand, and the one map produced (Johnston 1979) is more a geological map of an urban area than a document identifying specific geotechnical limitations to land-use. We have reservations about the ability of the "inventory-zoning" approach to adequately identify hazard and material limitations to land use.
Figure 2. Engineering geology data presentation for possible residential subdivision of land at Queenstown.
or development, and we believe that our engineering geological approach is a necessary input for regional or district planning: Bell (in press) has outlined such an approach within the existing legislative framework, and several studies are currently in progress to verify its suitability.

5 RESIDENTIAL SUBDIVISION OF LAND

Under the Local Government Act 1974 territorial authorities have responsibility for the residential subdivision of land, and are required to enforce certain developmental practices as well as to provide services (such as roading and water supply). Widely ranging powers also exist to refuse permission to subdivide where specified geotechnical problems or hazards exist. Although no uniform approach has been adopted, their legal liability for land subdivision has ensured that most local councils identify hazardous areas on their District Schemes, and require specific investigations (for example, some form of stability certification) prior to development approval. In the case of larger subdivisions (of more than 50 building lots) statutory provision exists for requiring a Concept Plan of the proposed development prior to submission of a Scheme Plan showing layout and services, which is the legal device by which approval to subdivide is given: no specific legislative requirement exists for engineering geological or geotechnical investigations (Bell & Pettinga 1985).

We have had professional involvement with residential subdivision of land in several parts of the South Island, principally in Queenstown and Christchurch (Fig 1), and practices clearly vary depending on the terrain and the local authority requirements. As an example, Lake County Council (Queenstown) requires the submission of an engineering geology plan and report for land designated R4 in their District Scheme (Bell & Pettinga 1985): this has now been operative for some 5 years, and one of us (DHB) has reported on more than 10 subdivision proposals for developments each involving 10-100 (+) building lots. From these investigations a methodology has evolved whereby engineering geological mapping is carried out on a scale of 1:500 or 1:1000, test pits are excavated by backhoe to assess the generally coarse-grained glacial tills and associated lake margin deposits, and limited geotechnical testing may be performed if judged to be necessary. A standardised format for graphic data presentation has also been adopted (Fig 2), involving engineering geology plans, sections and summary logs of excavations, and the written report comments on both the suitability of the land for residential subdivision and may recommend layout modifications or specific foundation design on certain lots. The engineering geological (rather than geotechnical) approach adopted has proved in our view to be entirely satisfactory given its purpose as one of pre-development (i.e. feasibility) evaluation, and it should be seen as complementary to the more detailed civil engineering design and/or construction that follows. At one site, for example, nicotineous clayey (lake-bottom) silts were identified beneath several lots and their liquefaction potential was assessed using the simple criteria given by Seed (1983): although no problems were anticipated during earthquake shaking, specific design of any house foundations in these relatively soft sediments was nevertheless strongly advised as a condition of subdivision approval.

6 ASSESSMENT OF INDIVIDUAL BUILDING LOTS

Given adequate identification of geological hazards during staged urban planning, engineering geological data input should not normally be necessary at this advanced stage of residential development except as part of specific foundation design, or for remedial works in an older built-up area that had not been investigated in such a detailed manner. By comparison with subdivision planning approval studies, we are rarely asked for professional advice on site stability or suitability for an individual building lot within an urban area, although there certainly have been occasions when engineering geological data input has been requested for the purpose(s) of remedial design, certification or litigation. Again, the development of residential building sites on the Port Hills, Christchurch, where dispersive loessial soils cause both subsurface erosion and slope movement problems, has necessitated engineering geological investigations as part of remedial design (Bell 1983): in fact, chemical stabilisation techniques have been developed and applied to these situations as a direct result of our involvement with such projects (Evans & Bell 1981; Bell et al in press).

Two examples of individual building lot assessment are presented here, and both resulted from specialist engineering geological advice being sought by civil engineering consultants on behalf of their clients. In the first (Fig 3) part of an old volcanic bedrock quarry site had been subdivided and the building lots sold: at the building permit stage a local consulting engineer was approached to certify the boulder retaining structure for some 2-3m of fill that had been constructed by the subdivider, and he became concerned at the stability of its foundations. An engineering geological investigation, involving a stadia survey, engineering geological mapping at 1:200 and a seismic refraction profile to locate bedrock, was therefore undertaken, and this identified original quarry floor fill up to 2m thick beneath the "new" fill (Fig 3). It was not feasible, however, to determine shear strength parameters for the "old" fill without
Figure 3. Engineering geology plan and sections for building lot in Christchurch

removing part of the boulder wall for access, and it was suggested that reconstruction of the wall to design standards might be a cheaper alternative than a difficult and potentially expensive geotechnical investigation. The site has still not been developed because of no stability certification has been forthcoming, and the boulder wall remains in place; one of us (DDB) has since become involved for another client in assessing part of the same quarry site, and advice has been given there that subdivision should not proceed until stabilisation is effected of loose debris on the original quarry face.

In the second example (Fig 4) a small parcel of land involving effectively a single building lot in the township of Hamner, North Canterbury (Fig 1), had been zoned as part of the "Hamner Springs Protection Zone" because of its proximity to the active Hamner Fault and the presence of large seepage flows during winter. To assist in obtaining permission for dwelling construction on the site, we were engaged to carry out an engineering geological assessment of the property, and we excavated two 2m (+) deep trenches normal to the fault so as to evaluate both the seis-motectonic hazard and the perceived drainage problem. Although trench logging (Fig 4) revealed some evidence for ground deformation beneath the site, we concluded that the fault trace was located outside the property boundary and that it was no more at risk from ground rupture or shaking than adjoining land where houses had been permitted. We therefore recommended, subject to the satisfactory operation of recently installed drainage works during the 1986 winter, that land-use zoning be changed to residential; we understand that our advice was followed, and that dwelling construction is to be approved.

7 FARM MANAGEMENT PRACTICES

We believe that engineering geology can contribute significantly and cost-effectively to farm management practices in rural areas by identifying both geological and geomorphological features of relevance, and we envisage an important role for the engineering geologist as part of catchment authority advisory "teams". Once again we consider that the investigation approach outlined in this paper is appropriate to rural land-use planning, for example for access track and/or farm dam location, and we note the importance of landscape development processes to the effectiveness of soil conservation measures (Pettinga & Bell 1984). Our first case study (Fig 5) relates,
however, to a proposed abattoir site near Akaroa on Banks Peninsula (Fig 1), where one of us (DBB) was asked to investigate the site because spray irrigation of meatworks effluent was proposed and slope stability was of concern: engineering geological investigation was limited to mapping onto an air-photo enlargement at a scale of approximately 1:3000 (Fig 5), and to auger drilling and exposure logging as part of overall site reconnaissance. This delineated the extent of seepage-controlled landsliding already affecting the property, and also enabled a preferred meatworks site to be identified (locality 2 on Fig 5) which was subjected to more detailed surface and subsurface investigation: further field trials and laboratory testing was recommended if the proposal proceeded, but in fact an alternative site was chosen on economic grounds before these studies could proceed.

Our second example concerns soft rock terrain within the East Coast Deformed Belt of the North Island (Fig 6), where Pettings (1980) recognised a two-stage landscape generated by continued stream incision accompanying Late Quaternary uplift. He recognised an older elevated or "relict" terrain which is largely stable, and an actively eroding or "rejuvenated" terrain incised into the former: in addition, three landslide associations were recognised which reflect mass movement control by bedrock lithology, rock mass structure and climatic extremes (Pettings & Bell 1984). It is clear that conventional soil conservation measures such as tree planting are inappropriate for deep-seated block and wedge slides (Figs 6A-C), or for creeping earthflows developed on complexly deformed Upper Cretaceous - Lower Tertiary mudstone- and sandstone-dominated lithologies: likewise, shallow regolith failures (Figs 6D-P) are climatically controlled and here increased evaporation through forestry could minimise slope movements. The identification of relict and rejuvenating terrain, and their associated dormant (inactive) and active landslides, is in our view fundamental to sound farm management practices because both the type of land-use and the details of farm layout must take account of landscape development processes and their current activity.

8 PROFESSIONAL IMPLICATIONS

In New Zealand practising engineers and surveyors are registered under Acta of Parliament, and there are clearly defined codes of professional practice: this situation does not exist for
Figure 5. Preliminary engineering geology plan of proposed abattoir site, Akaroa.

engineering geologists, and obviously limits the recognition that may be given to their professional advice. Nevertheless, our experience has been that engineering geological opinions are respected in the field of land-use planning, and that competent advice will frequently be sought by other professionals (such as surveyors and town planners). We are firmly of the opinion that our engineering geological approach (which is really the first stage of a full geotechnical appraisal) is highly relevant to both urban and rural land-use, and we have elsewhere (Bell & Pettinga 1985) advocated its statutory requirement for at least the residential subdivision of land in New Zealand. In the meantime, however, the questions of professional recognition and legal liability remain: allied to this are the basic training requirements that constitute a qualification in engineering geology, and even the definition of "engineering geology" itself. Some resolution of these matters will no doubt occur in the long-term, but in the short-term we see only an increasing demand for professional engineering geological services in the field of land-use planning without any corresponding formal recognition by way of certification.

9 CONCLUSIONS

1. Engineering geology data input is regarded as essential for sound land-use planning practices because it enables the prior identification of geological hazards and/or site characteristics that may provide geotechnical limitations to development.
2. We have developed an approach to land-use planning that involves air-photo interpretation, engineering geological mapping at appropriate scales, subsurface data collection using shallow excavation and/or geophysical techniques, and limited geotechnical testing of materials to develop an engineering geological site model.
3. For urban planning purposes a tiered investigation format is necessary, and hazardous areas or difficult ground conditions should be zoned accordingly at an early stage of development: our engineering geological approach to the residential subdivision of land has proved effective in different parts of New Zealand, and should in our view minimise the need for other than routine geotechnical investigation of individual building lots.
4. Our engineering geological approach to land-use planning has relevance for the management of rural and forested land, and we present examples of appropriate data input for land-use planning: the employment of suitably qualified engineering geologists by catchment authorities in New Zealand is also strongly advocated.
5. The absence of any form of registration or certification of practising engineering geologists in New Zealand creates difficulties in terms of professional recognition, and in the longer-term we believe that this must be addressed both by the profession and by central Government.

REFERENCES


La carte géotechnique d’Abidjan (Côte d’Ivoire): Méthodologie et doctrine graphique

Geotechnical map of Abidjan (Ivory Coast): Methodology and graphic system

G. Cougny, Laboratoire du Bâtiment et des Travaux Publics (LBTP), Abidjan, Côte d’Ivoire

ABSTRACT: The aim of this paper is to bring a modest contribution to the expansion and improvement of geotechnical (or engineering geological) mapping doctrine. The realization of the geotechnical map of Abidjan brought the opportunity for a general reflection about methodology and graphics of engineering geological mapping. Without claiming to solve all the encountered problems, the paper recalls the different stages of working out the map and, especially, the reflection about graphic problems and the adopted choices in this field. Developing Countries are confronted with serious problems of city-planning. They might use the suggested methodology and the inexpensive accepted graphic solutions.

RESUME: Le but de cet article est d’apporter une modeste contribution au développement et au perfectionnement de la doctrine de cartographie géotechnique (ou cartographie de géologie de l’ingénieur). La réalisation de la carte géotechnique d’Abidjan a été l’occasion d’une réflexion méthodologique et graphique sur la cartographie géotechnique en général. Sans prétendre régler tous les problèmes rencontrés, l’article évoque les différentes étapes de l’élaboration de la carte et surtout les réflexions sur les problèmes graphiques et les choix adoptés dans ce domaine.
Les Pays en Développement confrontés à de graves problèmes d’aménagement urbain pourront s’inspirer de la méthodologie proposée et des solutions graphiques peu coûteuses retenues.

INTRODUCTION

L’un des problèmes majeurs rencontrés dans l’aménagement des zones urbaines dans les Pays en Développement (P.E.D.) est la rareté des données sur l’environnement physique et en particulier sur le sous-sol.

Si l’on compare les sites des métropoles des P.E.D. à ceux des grandes villes des Pays industrialisés, on observe que les premiers sont nettement moins bien "connus" que les seconds. Cette différence est due essentiellement à trois raisons inhérentes au contexte des P.E.D.:

- début tardif des études techniques;
- faible nombre d’ingénieurs et de chercheurs se consacrant à des investigations géotechniques et plus faible nombre encore se consacrant spécifiquement à des travaux de géologie de l’ingénieur;
- conservation plus ou moins bonne des archives scientifiques et techniques imputable à de fréquents changements dans les structures technico-administratives et/ou difficultés rencontrées pour accéder à ces archives, surtout lorsqu’elles sont conservées à l’étranger, ce qui est malheureusement souvent le cas.

Les volontés des gouvernements des P.E.D. de planifier leur développement urbain sont contrariées par ce manque de données de base et c’est d’autant plus grave que les métropoles des P.E.D. enregistrent un accroissement démographique spectaculaire. Abidjan, par exemple est passée de 140 000 habitants en 1950 à 1 700 000 habitants en 1985. La ville en est à son 6e schéma directeur d’aménagement et d’urbanisme. Il est à noter que ces schémas directeurs successifs n’ont pas tenu compte explicitement des conditions du sous-sol, celles-ci n’ont été prises en considération que de façon implicite, selon l’expérience que les urbanistes avaient du terrain.

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Or, le site d’Abidjan est très hétérogène au plan géologique et donc très contrasté au plan géotechnique. On y rencontre à peu de distance des sols résiduels sur socle ancien, des formations détritiques continentales et marines, anciennes et récentes, et des formations de sols "mous" d’origine fluvioglaciaire qui posent de nombreux problèmes géotechniques (fig.1).

Dans ces contextes, la disponibilité de documents de base tels que les cartes géotechniques (ou cartes de géologie de l’ingénieur) apparaît utile, voire indispensable. Encore faut-il que ces documents atteignent leurs buts. C’est à dire qu’ils doivent être lisibles et exploitables par les aménageurs généralement non spécialisés en géologie ou en géologie de l’ingénieur.

En ce qui concerne ce dernier aspect, il faut reconnaître qu’il est très difficile pour les P.E.D. de s’inspirer des exemples des Pays Développés car, à l’évidence, la seule caractéristique commune des cartes géotechniques réalisées à travers le Monde est d’être toutes différentes: Quel modèle adopter pour atteindre les buts fixés?

A partir de ces différentes constatations, le Laboratoire du Bâtiment et des Travaux Publics de Côte d’Ivoire (LBTP), avec l’assistance technique du Centre Expérimental de Recherches et d’Études du Bâtiment et des Travaux Publics (CEBTP, Paris) a proposé au début de l’année 1983 au gouvernement ivoirien la réalisation d’une carte géotechnique de la région d’Abidjan à l’échelle du 1:50 000 (échelle du dernier schéma directeur d’aménagement). La réalisation de cette carte visait plusieurs buts:

1°) fournir aux aménageurs et urbanistes un document de synthèse, facilement exploitable, mettant clairement en évidence les contraintes d’aménagement liées aux facteurs géologiques;

2°) fournir aux concepteurs des ouvrages et aux responsables de la reconnaissance des sites d’ouvrages les renseignements utiles: - pour le choix des sites et pour les études d’avant-projets, - pour l’orientation de la phase exploratoire de la reconnaissance et pour la sélection des moyens d’investigations; - pour l’interprétation des résultats de la reconnaissance;

3°) servir de "support" pour la formation ou la spécialisation d’ingénieurs nationaux aux techniques de cartographie géotechnique;

4°) servir de "modèle" pour la réalisation d’autres cartes du même type dans d’autres zones urbaines en Côte d’Ivoire ou dans d’autres pays confrontés à des problèmes analogues;

5°) et enfin contribuer au plan international à la mise au point d’un corps de doctrine dans le domaine de la cartographie géotechnique.
PROGRAMMATION DES ÉTUDES

Les buts énumérés ci-dessus étaient ambitieux, d'autant plus qu'ils s'accompagnaient de contraintes sur les délais et sur les moyens à employer. En effet, il avait été décidé dès l'origine du projet :

- de n'utiliser que la documentation existante complétée par des travaux de surface et de télédétection, à l'exclusion de tout sondage complémentaire, ceux-ci étant jugés trop coûteux ;

- de réaliser intégralement la carte en Côte d'Ivoire, y compris la rédaction graphique et l'impression alors qu'il est fréquent que les P.E.D. s'adressent pour ce genre de travaux à des organismes étrangers spécialisés ce qui augmente considérablement le coût d'édition ;

- de limiter la durée du projet à 3 ans (délai entre le début des travaux et l'édition des documents définitifs). En fait, ce délai a été dépassé de 6 mois pour la carte proprement dite et de 1 an pour les documents d'accompagnement (notice explicative et atlas documentaire) ;

- de mener les travaux de la carte en parallèle avec d'autres travaux de recherche en géotechnique jugés prioritaires par les autorités de tutelle.

La réalisation de la carte s'est déroulée en 5 phases se chevauchant partiellement dans le temps.

Phase 1 : cahier des charges

(Identification des besoins des utilisateurs potentiels de la carte et définition des renseignements devant y figurer)

Cette phase s'est achevée en juin 1983 par la soutenance d'un mémoire de fin d'études d'un élève ingénieur de l'Ecole Nationale Supérieure des Travaux Publics de Yamoussoukro (KACOH B. 1983).

Le travail de l'élève-ingenieur a consisté à analyser (et à critiquer) un échantillon assez large de cartes géotechniques étrangères que nous avions à notre disposition. Cette analyse s'est appuyée sur le guide établi par l'AIGI pour l'UNESCO (1976) et nous a conduit à décider de la réalisation d'une carte synthétique, à moyenne échelle (1:50 000) et à usages multiples. Cette première phase a comporté aussi une enquête auprès des principaux services de l'Administration ivoirienne, destinataires présumés de la carte.

Cette phase est la phase cruciale du projet car elle permet de prévoir précisément les difficultés à surmonter pour atteindre les objectifs fixés. Il a ainsi été mis en évidence que le principal problème à résoudre était celui de la lisibilité par des non-géologues (et même par des géologues lorsque les cartes veulent superposer trop de types de facteurs).

Il a été très instructif de recueillir les premières impressions visuelles des personnes à qui l'on montrait diverses cartes géotechniques et à qui l'on demandait un commentaire spontané sur ce qu'elles voyaient. A titre d'exemple sur les résultats de cette enquête (exemple un peu caricatural mais résumant bien la situation) plusieurs cartes géotechniques utilisent un code de couleur dérivé des feux de signalisation routière (rouge, orange, vert) avec les connotations suivantes :

- vert = terrains présentant de bonnes aptitudes à l'aménagement.
- rouge = terrains à problèmes
- orange = terrains intermédiaires

Certaines des personnes interroguées ont vu spontanément les zones vertes comme des zones de végétation (1). Ce genre de problème graphique est développé plus loin.

Phase 2 : Fond topographique

Cette phase qui semblait à priori sans problème en a posé de nombreux ; la raison principale étant le décalage important entre les mises à jour des cartes topographiques et le développement urbain d'Abidjan (on rencontre ce problème dans la plupart des métropoles des P.E.D.).

Faute de carte régulière récente il a été décidé d'utiliser le fond topographique établi par l'Agence d'Urbanisme d'Abidjan en 1985 pour le schéma d'urbanisation à long terme. Ce fond a l'avantage de l'actualité mais, en contrepartie, il présente quelques distorsions par rapport au fond régulier car il a été réalisé (à l'étranger) à partir d'un assemblage de photographies aériennes non redressées.
Compte tenu de l'échelle de la carte géotechnique et de ses buts avant tout urbanistiques, ces quelques défauts géométriques ont été jugés moins génants que l'utilisation d'une topographie périnée sur laquelle il est très difficile de se repérer bien qu'elle soit géométriquement juste.

Phase 3 : Fond géologique

Les seuls fonds géologiques disponibles au début du projet étaient à 1:100 000 et avaient été réalisés dans le cadre de l'étude sédimentologique et structurale du littoral (TASTET 1979). Il a été nécessaire de les réviser et de les compléter entièrement pour distinguer les unités de terrain à terre au nord des lagunes.

Cette phase a été réalisée à partir de la documentation existante complétée par:
- des levés de terrain mais ceux-ci sont peu productifs dans la région en raison de la rareté des affleurements due à la prédominance de terrains meubles et à la densité de la couverture végétale;
- des études géomorphologiques à vue, sur carte et sur photographies aériennes. La photo-interprétation était pratiquement la seule méthode de télédétection à notre disposition. En effet, les images LANDSAT sont de peu d'utilité pour ce genre de projet en raison de leur échelle mais aussi et surtout en raison de la couverture nuageuse omniprésente sur la région. Quant aux images SPOT, elles n'étaient pas encore disponibles. Leur efficacité dans le domaine de la cartographie géotechnique à moyenne échelle reste à étudier.
Nous disposons aussi d'une image radar (SLAR) mais à l'échelle du 1/250 000. Même à cette échelle, les formations fluviolagunaires sont nettement discernables et cette technique semble très prometteuse pour les études de géologie de l'ingénieur dans les régions littorales du Golfe de Guinée et plus généralement dans la zone intertropicale, à condition de disposer d'images à 1:50 000.

Phase 4 : Étude géotechnique

Cette phase classique consiste à recenser, dépouiller, analyser et synthétiser les données d'archives géotechniques. Bien que la tendance actuelle soit à l'utilisation de l'informatique pour ce genre de travail, nous avons écarté très rapidement cette méthode car les données dépouillées s'y prêteraient assez mal pour plusieurs raisons:

- absence de repérage en XYZ des sondages ou des sites d'études;
- hétérogénéité de présentation des données qui exige une analyse faisant largement appel à l'appréciation et au bon sens et donc à une démarche plus pragmatique que systématique;
- relative rareté des données comme on l'a signalé plus haut: 570 dossiers ont été dépouillés ce qui est peu comparé aux milliers de dossiers existant sur les métropoles développées.

L'étude géotechnique visait d'une part à caractériser les différentes formations rencontrées dans la région et à les regrouper en un nombre limité d'unités cartographiées à 1:50 000 et d'autre part à identifier et évaluer les problèmes géotechniques posés pour chaque unité cartographiée par différents types d'aménagements: bâtiments, ouvrages d'art, assainissement-drainage, travaux routiers, etc.

La synthèse des phases 3 et 4 est contenue, en résumé, dans la légende de la carte (cf. figure 1 et 2, tableaux 1, 2 et 3).

Phase 5 : conception, rédaction graphique, rédaction et édition des documents de synthèse:

- atlas documentaire,
- notice explicative,
- carte proprement dite.

L'atlas documentaire recense sur 26 planches à 1:20 000 et sur une carte à 1:50 000 la localisation des dossiers exploités et leur contenu (finalité, types de sondages, essais in-situ et essais de laboratoire pratiqués). Cet atlas n'est pas publié: il est destiné aux responsables des reconnaissances de sites à qui il fournira un état des reconnaissances antérieures en vue de faciliter les reconnaissances ultérieures (COUGNY G., KONATE L. 1986).

La notice explicative est le complément indissociable de la carte. Elle précise, complète et développe les indications contenues dans la légende. Cette notice s'inspire, dans la mesure du possible, des recommandations contenues dans le guide AIGI établi pour l'UNESCO (1976).

Pour des raisons de limitation de format du matériel OFFSET disponible, la carte a été imprimée en 3 feuilles juxtaposables. Le nombre de couleurs a été limité à 4 pour deux raisons:

1°) il semble qu'au delà d'un certain nombre de couleurs, leur augmentation n'améliore plus la lisibilité (cet aspect de doctrine graphique est développé ci-après);
2°) le coût d'édition est proportionnel au nombre de couleurs et celui-ci pose des problèmes techniques de calage des planches difficiles à résoudre par des imprimeries non spécialisées en cartographie, ce qui était notre cas.

Le code des couleurs retenu est indiqué sur la figure 2.

Il serait possible de réduire le nombre de couleurs à 3 en fusionnant les planches "topographie" et "symbolisme géotechnique", la topographie apparaissant toujours en gris par tramage. Cet artifice exige une très haute qualité graphique du fond topographique; c'est pourquoi nous ne l'avons pas utilisé pour cette première édition mais nous comptons l'utiliser pour une réédition ou pour des cartes géotechniques ultérieures.

REFLEXIONS SUR LA DOCTRINE GRAPHIQUE

Les cartes géotechniques sont généralement des cartes polythématisées, les auteurs visent à faire figurer sur une seule feuille ou sur un nombre limité de feuilles, à la fois des facteurs
### Tableau 1

<table>
<thead>
<tr>
<th>Unité</th>
<th>Nature</th>
<th>Classification</th>
<th>Unité de type 1</th>
<th>Unité de type 2</th>
<th>Uptérature</th>
<th>Puissance</th>
<th>Tension</th>
<th>Puissance de l'unité</th>
<th>Puissance</th>
<th>Puissance de l'unité</th>
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<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
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<td>13</td>
<td>14</td>
<td>15</td>
<td>16</td>
<td>17</td>
</tr>
</tbody>
</table>

### Tableau 2

<table>
<thead>
<tr>
<th>Identifications</th>
<th>Géotechniques</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analyses</td>
<td>Granulométriques</td>
</tr>
</tbody>
</table>

### Tableau 3

<table>
<thead>
<tr>
<th>Description de l'aptitude à l'aménagement</th>
<th>Aptitudes de la construction</th>
<th>Amenagement</th>
<th>Vibrations</th>
<th>Aptitudes aux terrassements</th>
<th>Mise en matière</th>
<th>Exemples</th>
<th>Poids</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>sablages gravillonnés ou graviers dimensionnels</td>
<td>zones de fonds de vallées par exemple</td>
<td>médiocre</td>
<td>bonne</td>
<td>bonne</td>
<td>bois</td>
<td>moyen</td>
</tr>
<tr>
<td>1.2</td>
<td>épaves, ancien, lutte visées, arables</td>
<td>ZONE POUR PEINTURES CONSTRUCTION</td>
<td>médiocre</td>
<td>bonne</td>
<td>bonne</td>
<td>bois</td>
<td>moyen</td>
</tr>
<tr>
<td>1.3</td>
<td>sablages gravillonnés</td>
<td>gravéliers blancs pour le terrassement</td>
<td>médiocre</td>
<td>bonne</td>
<td>bonne</td>
<td>bois</td>
<td>moyen</td>
</tr>
<tr>
<td>1.4</td>
<td>épaves, ancien, lutte visées, arables</td>
<td>ZONE POUR PEINTURES CONSTRUCTION</td>
<td>médiocre</td>
<td>bonne</td>
<td>bonne</td>
<td>bois</td>
<td>moyen</td>
</tr>
</tbody>
</table>
"objectifs" (topographie, géologie...) et des facteurs plus ou moins "subjectifs" liés à des notions de qualité de terrains vis à vis de leurs aptitudes aux travaux de l'ingénieur.

Le problème graphique des cartes polythémétiques n'est pas nouveau et plus généralement les problèmes de sémioscopie graphique font l'objet de réflexions depuis les débuts de la cartographie moderne. Toute conception cartographique devrait s'appuyer sur les connaissances que l'on a des caractéristiques et les limites de la perception visuelle et de leurs relations avec les techniques de dessin et d'impression.

Les quelques considérations qui suivent ne prétendent pas régler définitivement ces questions difficiles mais apporter quelques éléments de choix aux concepteurs des cartes géotechniques. Elles s'inspirent en l'adaptant de l'excellent article de J. BERTIN (1980) auquel les citations sont empruntées.

**Carte à voir et carte à lire**

"On ne regarde pas une carte comme on regarde une œuvre d'art. On lui pose des questions et tout lecteur est en droit de poser deux types de questions devant une carte:

- à tel endroit qu'y a-t-il?
- tel caractère, quelle est sa géographie?"

Une carte géotechnique "minimale" répondrait à la première question. Elle donne en tout point la nature des unités de terrains complétée le cas échéant par une appréciation contenue dans la légende ou dans la notice explicative. Pour dresser ce genre de carte il est suffisant en théorie d'utiliser un graphisme monochromatique en rappelant chaque unité par un symbole. C'est le cas des documents de travail sur calque.

On voit tout de suite que la carte "minimale" ne peut répondre à la deuxième question qu'au prix d'un patient labeur ou en distinguant les unités par des trames monochromes. L'utilisation de telles trames est très vite limitée par la surcharge graphique. C'est un problème physiologique:

"accroître le nombre de caractères [...] a une limite: celle de la perception visuelle".

"Chaque caractère est une image. On peut-on superposer plusieurs images et par exemple plusieurs photographies sur une même pellicule et cependant séparer chaque image?".

La réponse est évidemment non.

La carte "minimale" est facile à déchiffrer par ses concepteurs. Elle peut être lue par des spécialistes pour des besoins ponctuels (par exemple étude d'ouvrages). Mais lorsqu'on vise des buts urbanistiques (étendre ou ordonner l'espace urbain) il est indispensable que la carte géotechnique donne au lecteur une impression immédiate et globale (carte à voir).

"La perception visuelle est toujours instantanée. Si elle ne l'était pas on ne conduirait jamais une automobile. Ce qui importe donc c'est la signification de l'image instantanée".

C'est pour ce deuxième but que la couleur est utile voire indispensable. Encore faut-il que le code de couleurs utilisé respecte quelques règles physiologiques élémentaires.

**Ordre et non-ordre visuel**

L'exemple cité de l'assimilation "couleur verte = végétation" par des personnes peu habituées à la lecture de cartes géologiques, et encore moins à la lecture de cartes géotechniques, amène à se poser des questions sur la bonne utilisation des couleurs en cartographie géotechnique.

L'utilisation de la couleur peut avoir trois buts distincts:

1°) identifier;
2°) différencier;
3°) apprécier.

Les deux premiers buts sont ceux des cartes géologiques classiques. Chaque terrain est identifié par une couleur et celle-ci permet de le distinguer des terrains voisins affectés d'autres couleurs. Il n'y a pas d'ordre visuel: un terrain "bleu" n'est ni "meilleur" ni "pire" qu'un terrain "rouge", il est "différent".
Le troisième but est tout autre. Il est spécifique des cartes géotechniques. Il s'agit de donner une appréciation forcement subjective sur l'aptitude à l'aménagement des terrains cartographiés. Cette appréciation utilise une échelle de "qualité" à l'intérieur de laquelle le terrain doit se situer.

"L'ordre des données se transcrit par l'ordre visuel, et les deux ordres doivent se correspondre".

La séquence déjà citée "rouge-orange-vert" par exemple, ne correspond pas à un ordre visuel (l'orange n'est pas une couleur intermédiaire entre le rouge et le vert). De plus, elle fait appel à des conventions "non-naturelles" d'où les erreurs d'interprétation signalées. Par contre, la séquence "rouge-orange-jaune" est immédiatement perçue par l'oeil comme une gradation. Il en serait de même d'une séquence "noir-bleu foncé-bleu moyen-bleu clair-blanc" ou de toute autre séquence visuelle. Il y a donc une large gamme de choix autorisés.

Sur la carte géotechnique d'Abidjan, nous avons choisi deux couleurs d'appréciation: le rouge et le jaune avec la connotation indiquée sur la figure 2:

- jaune = facteurs favorables;
- rouge = facteurs défavorables.

Ce premier classement est évidemment trop simpliste car il y a plusieurs terrains dans chaque catégorie (il faut donc les identifier et les différencier) et parce que l'appréciation portée peut être motivée par diverses raisons. Un terrain peut être rouge parce que ses propriétés géotechniques sont très médiocres ou bien parce qu'il renferme des ressources en matériaux à protéger ou pour les deux raisons jouant simultanément. Faute de disposer d'une échelle chiffrée des aptitudes à l'aménagement, pouvant se traduire par une échelle d'intensité de jaune ou de rouge, nous avons simplifié à l'extrême.

Les terrains à problèmes de la région d'Abidjan (formations fluviolagunaires exondées) apparaissent ainsi en à plat rouge. Les "meilleurs" terrains (sables argileux des hauts plateaux) apparaissent en à plat jaune. Entre ces deux extrêmes, un jeu de trames jaunes ou rouges permet de distinguer dix autres unités à terre (figure 2).

Les terrains qui ne font pas l'objet d'appréciation apparaissent en trames noir sur blanc (fonds lagunaires ou marins lorsque leur nature est connue); les zones non cartographiées apparaissent en blanc (lagunes et parc national du Banco).

L'impression visuelle générale donnée par la carte géotechnique d'Abidjan (carte à voir) permet d'informer clairement les urbanistes sur les directions prioritaires à donner au développement urbain en évitant dans la mesure du possible les zones "rouges". Un examen plus attentif (carte à lire) donne aux spécialistes de la reconnaissance et des études d'ouvrages les informations nécessaires au stade des avant-projets.

CONCLUSIONS

Les quelques réflexions contenues dans cet article peuvent être utiles aux auteurs de cartes géotechniques pour éviter certaines erreurs graphiques, et plus particulièrement aux futurs auteurs de cartes géotechniques dans les P.E.D. confrontés à des problèmes analogues à ceux que nous avons rencontrés.

Contrairement à ce qu'on pourrait croire, les principales erreurs en cartographie géotechnique ne sont pas des erreurs de positionnement, ni des erreurs géologiques ou géotechniques, ce sont les fautes graphiques qui empêchent l'utilisateur de voir ce qu'on a voulu lui montrer ou qui lui font voir parfois exactement le contraire.

Lorsque la carte géotechnique est jugée nécessaire, ce qui est le cas dans les zones urbaines de géologie très contrastée et peu reconnues comme à Abidjan, les grands principes à respecter ou les recommandations à donner pour atteindre les buts fixés peuvent se résumer de la manière suivante:

1°) une carte géotechnique n'est pas un document académique mais un outil d'aide à la décision en matière d'urbanisme ou d'aménagement du territoire. Elle doit être conçue comme telle;

2°) une carte géotechnique est généralement élaborée par des spécialistes de géologie de l'ingénieur mais elle est destinée à des non-spécialistes. Elle n'atteindra pas ses buts si on néglige cet aspect;

3°) la recherche de la lisibilité exige d'éviter toute surcharge graphique;
4°) une carte géotechnique ne doit pas se limiter à identifier et à différencier les unités de terrain cartographiées, elle doit aussi porter une appréciation qualitative sur les unités;

5°) l'utilisation de la couleur (en sus des couleurs du fond topographique et du symbolisme géotechnique) doit se limiter aux couleurs strictement nécessaires à l'appréciation de la qualité des terrains. Au delà le coût d'impression augmente et la lisibilité n'est pas améliorée pour autant;

6°) Enfin, la carte doit être accompagnée d'une légende très complète pour éviter au lecteur de se référer en permanence à la notice.

Pour conclure, on pourrait dire que la carte géotechnique ordonne dans les deux sens du terme. Elle ordonne l'espace en situant chaque unité de terrain, en l'identifiant, en la distinguant des unités voisines. Elle donne des ordres aux aménageurs en portant une appréciation qualitative sur les aptitudes des sites, et en leur indiquant comment orienter l'aménagement en respectant les sujétions dues aux facteurs géologiques.

REMERCIEMENTS

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Cuban experience on engineering geology: Engineering geological mapping of Santiago de Cuba city
La géologie de l’ingénieur au Cuba: La carte géologique de la ville de Santiago de Cuba

Rafael Guardado Lacaba, Mining and Metallurgy Higher Institute, Faculty of Geology, Moa, Hoguín, Cuba

SUMMARY

Engineering geological works a special activity within the field of Engineering Geology, started in Cuba in 1970.

The generalization of the engineering geological experience in the country made possible carrying out this research, which set to the task of:

- Analyzing the regularities of the geological make-up of the territory of the Santiago de Cuba city and generalizing the physico-mechanical properties of the rocks, and their peculiarities of the rocks, and their peculiarities.
- Studying the geological processes and phenomena, focusing special attention on gravitational processes.
- Preparing the engineering geological map at the scale of 1:25 000 and undertaking the engineering geological regionalization of this territory, characterizing four engineering geological regions.
- Plotting the seismic microregionalization map, in relation with the engineering geological regionalization.
- Evaluation of soils and their degrees of feasibility for being considered as a basis for future engineering projects.

RESUMEE

Les travaux géologiques comme activités différenciées dans le domaine de l’ingéniorat, ont commencé à Cuba en 1970.

La génération de l'expérience géologique dans le pays a possibilité la réalisation de cet investigation, qui s'était tracé comme les devoirs fondamentaux:

- L'analyse de regularités de la constitution géologique du territoire de la ville de Santiago de Cuba et la généralisation de propriétés physico-mécaniques de roches et ses particularités.
- L'étude de processus et de phénomènes géologiques en prélevant spécifiquement l'attention aux processus gravitationnels.
- A confection de la carte géologique en escale de 1:25 000 et la réalisation de la ré-
gionalisation syneique relationnée avec la régionalisation géologique.

L'évaluation de sols et de degrés de favorabilité de ces derniers pour les futures
œuvres ingénieriles.

Tout les aspects mentionné conformé le présent travail.

GENERAL CHARACTERISTIC

Santiago de Cuba city, one of the oldest cities in Cuba, was founded by Diego Velazquez
1544 and it is the second most important city of this country. The Santiago de Cuba ba-
sin is bounded in the north by Puerto Pedado and Puerto Boniato mountain ranges, in the
south by Carribbean Sea, in the east by La Gran Piedra mountain block, and in the west by
the eastern offsets of the Turquino mountain block, in the Sierra Maestra mountain range.

The engineering geological survey of this area is aimed at solving the main tasks set by
the Major Guiding Plan of the city which was conceived in two parts, the first until 1990
and the second until 2000 (Fig. 1). Such plan is the main document for urbanisation and
all urban projects that are to be carried out, such as new buildings, districts, roads,
harbours, green areas, as well as, the best use of territory within the city’s projected
limits, must be based on it.

The area of the city by the year 2000 will be made up of 7620 Ha, with a population of mo-
re than 350 000 in habitants. This plan has four main development variants: 1. Northern,

This work gives the engineering geological characteristics for assessing these four va-
riants.

In the past, no attempt had been made to produce engineering geological maps at the
scale of 1:25 000 and 1:10 000 in a more appropriate manner for planners and engineers’
use. We have tried to supply, those users with a limited geological and engineering geolo-
gical experience, with the data necessary for their work presenting them in a not so, com-
plex or detailed form.

Engineering geological research works began in 1974 with the use of non published works
from the Geotechnics laboratories of Santiago de Cuba city and data from research works on
different construction sites. The maps has been welcome by planners and engineers it can
be said that it satisfies the search for favorable areas according to the four variants
established. It has been state that the most rational variant for development of this
city is the northeastern one, which from all points of view preserve the environment and
has the most favorable engineering geological conditions for construction.

This paper is summary of regional geotechnic and engineering geological surveys undertaken
in Santiago de Cuba city in order to apply geological criteria to urban development plan-
ing, particularly in Santiago de Cuba city.

The major objectives set for this work were: (1) To study the peculiarities of the geo...
Fig. 1 General Plan of Santiago de Cuba City.

The map illustrates the layout of the city with various zones and areas marked.

Legend:
- Micro-industrial zone
- Industrial zone
- Park
- Harbour
- Educational Institution
- Downtown area
- Purified works
- Center of new housing district
- Variant for city development
- Tourist zone
- Limits of inhabited area
- New housing district for industrial development
- Non-exploited quarries
- Antonio Maceo Airport

The paper discusses the following aspects of the city:
1. Climatic conditions of this city.
2. Soil and rocks properties and behavior.
3. Intensity and distribution of geological processes and phenomena in this territory.
4. Engineering geological research works for evaluating this area.

This paper also supplies quite an amount of information on the seismic characteristics of the Santiago de Cuba basin, and a seismic microregionalization scheme, in accordance with engineering geological conditions, is presented as well.
In general, the relief of the Santiago de Cuba basin and contiguous areas ranges from mountainous to wavy, with origin closely related to tectonic and erosive, and accumulative, as well as, with the geological features of this region.

The hydrographic network of this region is not highly developed. Among the most important rivers are San Juan, Sardinera, Rio Seco and Guanã which flow to the eastern part of the city, to the west are Coscon, Catamases and Panadas rivers. Generally, these rivers flow from North to South, with a very gentle slope (magnitude equivalent to 0.006), therefore their erosive force is quite limited.

The Santiago de Cuba basin has special climatic conditions, since it is one of the driest regions, and where there are the highest temperatures in the country.

ENGINEERING GEOLOGICAL CONDITIONS OF SANTIAGO DE CUBA.

The engineering geological survey of the Santiago de Cuba territory represents a need for the solution of tasks set for the national economy.

In the general development plan for Santiago de Cuba city the perspective development of the city for the next 20 years is presented. According to present theoretical levels, it is necessary to provide the reasoning of projects for construction and exploitation of building and other works in Santiago de Cuba.

Thus, the question of the engineering geological survey of the city can be divided into two major aspects:

The first aspect comprises forecasting the characteristics of tectonic changes of the geological environment and elaborating of measures for and practical solutions to existing engineering geological problems. In order to tackle these problems a group of research works was carried out; e.g:

- Engineering geological characterization of rocks.
- Engineering geological characterization of relief.
- Engineering geodynamic conditions.
- Engineering geological regionalization.

The engineering geological survey of the Santiago de Cuba territory is concerned with geotectonic, stratigraphic and geolitohological complexes, as well as, with the engineering geological peculiarities of the composition, state and properties of rocks.

Geotectonic features of region: the area comprising the Santiago de Cuba basin is located in the Sierra Maestra anticline and represents a tectonic depression originating as a consequence of movements which affected this region, probably early in the Neogene, which had an influence on deposition of the rocks making up the La Cruz Formation. This region is tectonically active and structural floors can be observed:

a. Lower floor represented by rocks making up El Cobre Formation (Paleocene-middle Eocene).

b. Higher floor formed by rocks making up La Cruz Formation (Middle Miocene-Pliocene).
Both floors are separated by an angular discordance, differing not only in the lithological composition, but in the structural development as well.

Stratigraphy of this area can be considered as complex and varied. There are three formations in the area: El Cobre (P₁₋₁), rocks from La Cruz Formation (N₁₋₂) and the coast limes; from the Cudamar Formation (Q₁₋₃). There also Quaternary depositions to the north west of the bay and in the eastern part of the city.

El Cobre Formation has been divided into three members: Caney (P₁₋₁), Hongolosongo (P₁₋₂).

The rocks from El Cobre Formation (P₁₋₂) make up a complex of sedimentary volcanogenic rocks with a varied structure and lithology, being the oldest rocks of this region.

La Cruz Formation (Middle Miocene Pliocene). These rocks are discordantly overlying the ones under them. They represent a set of interstratifications which crop out in the southern and central parts of this area. The city is practically resting on these rocks. This formation is made up of conglomerates, sandstones, clays, marls, and different types of limestones. This terrigenous-carbonate formation comprise two members:

1. Quintero (N₁₋₁).
2. Versalles (N₁₋₂)

The southern part of the area is occupied by the Cudamar Formation (Pleistocene-Quaternary), which is lithologically composed of massive organogenous and sandy limestones, which in turn are expanded and developed more to the east of the bay.

Quaternary deposits: they are located to the north and northwest of the bay and in area of the San Juan river valley, in the eastern part of the city. They are represented by sands, gravels, clays and transitions. In them we can find alluvial, deluvial-prelluvial and efluvial deposits.

From the petrographic and stratigraphic study, a geological map can be prepared and analyzed, at the same time, according to physical properties of rocks and soils an engineering geological classification of rocks can be established (Fig. 2).

Engineering geological characteristics of relief are based on genetic aspects of tectonics and erosion of the territory. Geomorphological characteristics of the area under study are complex and varied. The relief of this area is predominantly mountainous and wavy, which is closely related to the geological structure. A distinctive feature in the case of the bay of Santiago, the characteristics which allows for assuming its tectonic genesis. In this area different forms of relief can observed: such as, hills, plateaus, gorges, terraces, valleys, etc.

In the area of Santiago de Cuba city two zones can be distinguished:

1. The northern region where the young elevations of Sierra Maestra mountain range are developing.
2. A region characterized by its heterogeneity and scarce elevations and geological phenomena.

Hydrogeological conditions of Santiago de Cuba city were studied taking into considera -
tion the characteristics of the Gascón, Parada and San Juan river basins. According to hydrogeological works undertaken in this zone there are three aquiferous complexes:

1. Aquiferous complex of sedimentary volcanogenic rock.
2. Aquiferous complex of earthy-carbonate rocks.
Aquiferous complex of quaternary sediments.

Among the technogenic processes some can be mentioned: water leakage from the water-supply and sewage network, which brings about softening of building and streets foundations, pollution of water supply sources, an indiscriminated use of underground water, thus causing saline intrusion troubles, etc. Therefore, the study of hydrogeological conditions of Santiago de Cuba's territory is closely related to man's engineering activity.

Engineering geodynamic researches take two courses:
- Regional study of paragenetic complexes of geological processes and phenomena (natural and artificial).
- Local observation and prognosis of exogenic and endogenic processes taking place in the city (Fig. 3).

The distribution or intensity of occurrence of the different geological processes and the phenomena, as well as, their influence on the engineering geological conditions for construction and exploitation of engineering structures.

Geological processes and phenomena were generalized as follows (Fig. 3):
1. Phenomena relating to weathering crust formation.
3. Phenomena relating to the action of surface and underground water: karst, slips.
5. Phenomena relating to seismic activity in the region.
6. Technogenic phenomena, settling, slips, water contamination, saline intrusion, etc.

Therefore, especial attention has been drawn on gravitational processes, like landslides, rock loosening, avalanches, etc. The gravitational processes classification comprises the various geological formations. A qualitative study was conducted on erosive processes and abrasion, and the conditions and peculiarities of the carbonate karst in the quaternary limestones from the coast were analyzed, as well.

It was concluded that particularly earthquakes represent a serious danger for construction.

The territory of the city is related to the most seismically active zones of the country, medium intensity being 7. Earthquake epicenters are located between 6 and 10 km deep into the Caribbean Sea. Periodicity of strong and weak earthquakes was studied, and their intensity is dependent on the tectonic composition of the city. The history of seismic phenomena over the last centuries was also made.

Mon-environment interaction and its reflection on geological processes and phenomena, e.g.: saline intrusion in coast areas resulting from irrational use underground water.

The engineering geological regionalization of Santiago de Cuba city is the generalized summation of the engineering geological conditions of that territory and it serves as a basis for design and construction of new engineering structures.

The main objective of the engineering geological regionalization of Santiago de Cuba city
is the engineering petrological study of the rocks that allow to characterize the most relevant features of the engineering geological division of the city.

Taking into consideration all these elements an engineering geological schematic map, as well as, an engineering geological scheme for regionalization of Santiago de Cuba's territory were prepared at scale of 1:25 000 (Fig. 4 and 5).

The engineering geological regionalization of Santiago de Cuba is based on the study and analysis of the peculiarities the geological composition, relief, physical mechanic properties of rocks, geological processes and phenomena: karst, slips, erosion, earthquakes and others.

According to the indexes of these aspects, the engineering geological regionalization scheme shows the following types of regions:

1. Region where the volcanogenic-sedimentary El Cobre Formation is developing, which is, as it was previously pointed out, divided into two subregions: 1. El Caney member subregion and $I_2$ Mongolocongo member subregion.
FIG. 4 ENGINEERING GEOLOGICAL MAP OF SANTIAGO DE CUBA. SCALE 1:25 000

II. Region where the clayey-carbonate rocks from La Cruz Formation are developing, which is divided into two subregions: II₁, Quintero and II₂, Versalles.

III. Region where the carbonate limestone semi-hard organogenic rocks from the Cudamar Formation are developing.

IV. Region where strata of clayey-sandy sediments from quaternary complexes of various genesis are developing.

The engineering geological type-classification was done with the objective of giving a sum and or total of the existing engineering geological conditions.

For preparing the type-classification map, the author made use of the engineering geological regionalization table and map. The different zones were determined according to the degree of suitability of the territory.

The engineering geological type-classification map provides a quantitative evaluation of the engineering geological conditions for design, construction and exploitation of the works projected in the Guiding Plan of the City.
FIG. 5. ENGINEERING GEOLOGICAL REGIONALIZATION MAP OF SANTIAGO DE CUBA.

1: 25,000,

I. REGION. ROCKS FROM EL COBRE
II. REGION. ROCKS FROM LA CRUZ
III. REGION. ROCKS FROM CIUDAMAR
IV. REGION. FRIABLE COHESIVE AND NON-COHESIVE ROCKS.

ENGINEERING GEOLOGICAL COMPLEX, SCARLETT FAVORABLE AND UNFAVORABLE,
HIGH GROUNDWATER AND SISMIC INCREASE (+1 TO +3).

A seismic microregionalization scheme was prepared for the territory of the city, which is related to the engineering geological regionalization.

The highest value of intensity increase ($t_2, t_3$) was assigned to the IV region of the city, which involves weak and saturated days, and sands which have a high level of underground water.

The most unfavorable zone, as far as seismic conditions are concerned, coincides with the northern and northwester parts of the bay, where there should not be constructions.
The observation of the repetition of seismic strikes influence on the development of gravitational phenomena, in the first place the rock loosening and at the same time the deformation of slopes, particularly in those zones where there are plastic clays and in their place there are rocks with a high aquiferous capacity.

The analysis of destructions caused by earthquakes shows that the zones where greater deformations buildings can be observed are those lying over the strike of the subsouthern faults, that is, those lying over the main tectonic lines.

CONCLUSIONS

From the analysis of the materials mentioned in this work, and according to the group of researches undertaken by the author, the following conclusions, about Santiago de Cuba city, can be arrived at:

1. The engineering geological conditions of Santiago de Cuba city are characterized by a high complexity, which is determined by the variety and strong dismemberment of its present relief, with elevations that in some cases are of 800 m high, and with sharp grades and slopes. It also has a heterogeneous geological profile, wide distribution of different lithological types of rocks, and the occurrence of huge faults where exogenic and endogenic processes occur.

2. This work analyses for the first time, the regularities of the geological make-up of the Santiago de Cuba region, pointing out that there all five groups of rocks in the profile, in accord with the engineering geological classification of rocks.

3. Based on the analysis of the peculiarities of the geological make-up, the physico-mechanical properties and the geological processes, an engineering geological schematic map at the scale of 1: 25 000 and its regionalization was made, according to the stability index of rocks, as natural basis for foundations of building in the city.

Four regions were characterized within the limits of the territory of the city, using the engineering geological regionalization scheme as basis.

Taking into account the engineering geological regionalization and schematic map, as well as, the seismic microregionalization map and taking into consideration the Guiding Plan of the city, it can be concluded that the most favorable zone for the city construction is that represented by the southeastern variant which embraces the II engineering geological region, characterized by a gentle relief, without too many place, nor strong erosion, where the rocks lie horizontally or almost horizontally, with a ground-water level above 10 m, and where rocks are as stable as foundations for buildings, even for high buildings. Here geological phenomena and processes exhibit a low intensity.

5. The analysis of materials referring to construction experience in Santiago de Cuba city, since the Sixteenth Century to the present allows us to stand out the bases of the most rational construction of buildings and types of foundations under certain engineering geological conditions, calculating the seismic manifestations in each zone.
The data compiled by engineerin geology in Santiago de Cuba make possible the elaboration of both, a methodology and a program for engineering geological research, and the determination of the degree of complexity of this territory by the year 2000.
Methodology of the expansive clays hazard preventive map of Spain, 1:1.000.000
Methodologie de la carte de risques dans les argiles expansives en Espagne a escale 1:1.000.000

Carlos S. Oteo & José L. Salinas, Laboratorio de Carreteras y Geotecnia, Madrid, Spain
Mercedes Ferrer, Instituto Geológico y Minero de España, Div. de Geología Aplicada, Madrid, Spain

ABSTRACT: The increasing problematic that expansive clays represent in foundations and civil engineering works in Spain, has push to carry out a Expansive Clays Hazard Preventive Map of Spain (escale 1:1.000.000) to obtain a first orientation of the swelling potential of the clay formations. In this paper, a methodology for the realization of this map is presented, as well as the criteria used to represent the distribution and potential expansive hazard of clay formations. Special emphasis is carried out in the correlation between the identification characteristics (mineralogy, moisture, granulometry, Atterberg's limits) and the swelling pressure and Lamsbe potential volume change.

RESUME: La présence de terrains avec d'argile expansive a donné lieu à des problèmes importants pour les travaux publics et les cimentations. C'est pour sa qu'on a besoin de réaliser une carte pour la prévision des risques associés à la présence d'argiles expansives. La finalité de ce travail c'est d'avoir une première approximation de l'expansivité potentielle des formations géologiques. Dans ce travail, nous présentons la méthodologie générale qu'on a suivi pour la réalisation de la carte d'Espagne 1:1.000.000. On présente, aussi, les critères les plus performants pour représenter la distribution des sols expansifs et de son potentiel de gonflement. Nous avons étudié spécialement la correlation entre les caracteristiques de identification (mineralogy, granulométrie, humidité, limites d'Atterberg) et la pression de gonflement et le changement de volume potentiel de Lamsbe.

1. INTRODUCTION

The swelling capacity of clay soils is conditioned by a great number of variables. If this capacity depends on intrinsic soil characteristics (mineralogy, fabric, etc) the natural conditions in which these soils are and, more specifically, the modifications that will be introduced into their natural state (by the changes in the zone climate, p.e.), will determine whether this capacity can develop or not. Therefore, these alternatives must be taken into account when the possibility to swell of the clay is analysed.

With a general purpose to the facility a first grade of information to the construction technicians, a scale 1:1.000.000 map of the expansivite clay hazards in Spain has been developed. In their realization, two official organizations ("Instituto Geológico y Minero de España" I.G.N.E., and "Centro de Estudios y Experimentación de Obras Públicas", CEDEX) had been collaborated. This work, described herein, is a first stage in a serie of regional studies on the theme.

The classification criteria of expansive hazard of clay formations in Spain tries to synthesize the relative importance of the different variables implied in the phenomenon. For the valuation of swelling capacity direct quantitative values obtained with the help of expansivity tests are take into account. These values are completed with other indirect values derived from the litho-geotechnical soil characteristics, grouped together by chronological formations and compared with climatic index. Finally, we come to areas of the same expansivity potential which are classified in four different groups and shown cartographically.

2. METHODOLOGY

Fig. 1 summarizes the criteria followed in order to classify the expansive clays in Spain. These criteria comprise some concurrent lines of action.

- Definition of chronolithological units (only refered to the clay formations).
Regional definitions of the climate conditions, through the Thornthwaite index, based in the previous studies of Justo and Cuéllar (1972) and the modifications by Rodríguez Ortiz (1979).

Recollection of information on the laboratory test on clay strata, by a national pool referred to the main official and private enterprises.

Sectorial examination of expansive tests, which are checked with eventual problems derived from expansivity and assignement to the different chronological units.

![Diagram showing the methodology of the expansive clays hazard preventive map of Spain]

Fig. 1.- Methodology of the expansive clays hazard preventive map of Spain

The first proceeding guaranteed a soil classification of comparable mineralogical, textural and structural characteristics (equivalent facies and geological history). This implies a specific expansive capacity of the sedimentary formation that, globally, can be related to other similar formations near of the studied. This specific swelling potential was checked with the climatic index of the corresponding zone.

At the same time, the adscription to these chronolithological units of expansive tests, which were eventually completed with plasticity tests -the most abundant data- in order to obtain more information, made it possible to value the expansive alternatives inherent in the sedimentary formation and, consequently, to evaluate swelling potential. Table I summarizes the evaluation index -almost the climate index of Thornthwaite- used. In the reported field constructive problems due to a specific soil type and climate conditions, the formation was considered as a soil of high expansive characteristics.

<table>
<thead>
<tr>
<th>Degree of expansivity</th>
<th>Average Liquid Limit</th>
<th>Extreme Liquid Limit</th>
<th>P.V.C. Lambe test</th>
<th>Probable linear expansion (%)</th>
<th>Swell pressure (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null to low</td>
<td>&lt; 35</td>
<td>&lt; 20/≤ 50</td>
<td>&lt; 2</td>
<td>&lt; 1</td>
<td>&lt; 25</td>
</tr>
<tr>
<td>Low to moderate</td>
<td>35 - 50</td>
<td>20-30/50-70</td>
<td>2 - 4</td>
<td>1 - 4</td>
<td>25 - 125</td>
</tr>
<tr>
<td>Moderate to high</td>
<td>50 - 65</td>
<td>30-40/70-90</td>
<td>4 - 6</td>
<td>4 - 10</td>
<td>125 - 300</td>
</tr>
<tr>
<td>High to very high</td>
<td>&gt; 65</td>
<td>&gt; 40/&gt; 90</td>
<td>&gt; 6</td>
<td>&gt; 10</td>
<td>&gt; 300</td>
</tr>
</tbody>
</table>

Firstly, the correlation between plasticity and swelling of soils was established by analyzing the liquid limit and the Lambe swell index reached in each case. As it can be seen in figure 2, the superposition of plasticities for the same degree of expansion is very frequent. Nevertheless, its average and extreme values are increasing globally with swelling and therefore the references which appear in Table I are more frequent statistically.

The incidence of natural water content on the expansive capacity of soils was evaluated. For
example, the preconsolidated miocene clay formation existing around of Madrid (city with swelling problems in the south zone) were classified in four different groups, obtaining the following data:

<table>
<thead>
<tr>
<th>Average natural water content (%)</th>
<th>Probable expansion (change of volume, %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>&lt;1</td>
</tr>
<tr>
<td>19</td>
<td>1 - 4</td>
</tr>
<tr>
<td>16</td>
<td>4 - 10</td>
</tr>
<tr>
<td>13</td>
<td>&gt;10</td>
</tr>
</tbody>
</table>

The graphic of figure 3 was obtained by relating the values of liquid limit and the results obtained in the swelling pressure test and introducing the natural water content of the sample tested. Approximately 90% of the values of the moisture were comprised in the interval ± 5% of the indicated representative natural water contents.

With a collateral result, a swelling criteria for spanish soils has been developed by Gteo (1986). This criteria taken into account the relation of the natural water content and the liquid limit, the swell pressure and the free swell of the clays (similar to the Vijaveriya and Ghazzaly (1973) and Guéllar (1978) criteria, but most representative of the expansivity hazard).

These criteria simplifies the subsequent integration of all the available data, not only those concerning the geological nature of the soils, but also those concerning the expansivity, avoiding as far as possible correlation errors to lithological formation which no geotechnical available data.

Finally, it should be pointed out that in order to facilitate a first data integration, it was considered useful to refer to areas with similar characteristics. For this reason the area under study was classified into zones of similar geological, morphological and climatic generalities (peculiarities which are similar to hydrographic basins). In total seven physiographic areas were classified, principally related with big river valleys, and different climates (humid, semi-arid, arid, etc).

Some of the before mentioned criteria are similar to those used for Patrick y Snethen (1976) for delimiting the expansive materials in the United States by physiographic areas.
3. PROCEDURE CRITERIA

Since the degree of expansiveness is generally classified into four different classes by several authors (Lambe, 1960; Holtz, 1969; Rodriguez Ortiz, 1975, etc), it is logical that the evaluation of the hazard due to expansiveness should also be divided into four groups. In this way, and to a certain degree, the total expansive capacity of the same kind of soils, evaluated by means of tests, can be related to that of the original lithological formation.

The established degree of expansiveness were as follows:

- Grade I = Expansive potentiality of soil: from nulle to low
- Grade II = Expansive potentiality of soil: from low to moderate
- Grade III = Expansive potentiality of soil: from moderate to high
- Grade IV = Expansive potentiality of soil: from high to very high

The estimation of the degree of expansiveness of soils of the same origin was carried out upon the basis of the highest expansive capacity that has been registered, the frequency of occurrence and its alternatives.

Complimentarily, and due to the fact that this capacity is related with the soil characteristics, which are defined through the composition, texture and structure of the soil, the degree of expansiveness can be grouped for the same chronolithologic units.

The chronolithologic units have been established according to the geological age and to the lithology of clay formations, which are the following:

Geological age:  
I. Neogene
II. Palaeogene
III. Mesozoic
IV. Palaeozoic
Lithology:  
a. Massive clays  
b. Clays with silt and sand materials, occasionally gravels  
c. Clays with carbonates  
d. Clays with sulphates, with or without carbonates  
e. Clays with sands and carbonates and/or sulphates  
f. Sands, occasionally gravels, with subordinate silt and clay contents

Finally, in the case that the chronolithological formation was situated in a climatic zone with a seasonal deficit of humidity, its expansive potential could develop more easily that without this deficit. The expansive hazard should also consider the climatic alternatives. These alternatives have been classified by means -as we previously said- of the Thornwaite Index, which decide whether a zone has a deficit in humidity from the climatological point of view.

The conjunction of all these criteria has led to the following classification of the expansive capacity of clay soils, according to the four categories which had been fixed before:

I. Non-expansive clays or dispersed in a non-clayey matrix  
II. Low frequency of occurrence of swelling phenomena or clays located in climatic areas without an annual deficit of moisture  
III. Locally high frequency of occurrence of swelling phenomena or clays located in climatic areas with annual deficit of moisture  
IV. High frequency of occurrence of swelling phenomena or clays located in areas with problems related to expansiveness.

4. DATA BASIS

- Two data basis has been developed: a) With geological criteria, b) With geotechnical criteria. The definition and mapping of the chronolithologic units on scale 1:1,000,000 (Geological Basis) was carried out starting from the following documents:

  - Basic chartography of the National Map of Geological Synthesis on scale 1:200,000 of the I.G.M.E., by reduction to the 1:400,000 scale.

  - Confrontation with the General Geotechnical Map on scale 1:200,000 of the I.G.M.E.

  - Checking with the National Map of Industrial Rocks on scale 1:200,000 of the I.G.M.E., Geotechnical Maps of Urban and Regional Planning on scale 1:100,000 and 1:25,000 of the main spanish city, carried out by I.G.M.E., the Lithological Map of Peninsular and Insular Spain on scale 1:500,000 of Macau et alia and the guide for elaborating physical medium studies (CEUTMA, 1984).

The data referring to mineralogy, texture and structure of clay soils have been obtained mostly from the information offered in the Memories of said maps. Accurate data from different sources have been also used. Some of them have been investigated specifically for this study.

In order to establish the Geotechnical Basis, several documents has been used:

- Memories of General Geotechnical Maps, Geotechnical Maps of Urban and Regional Planning of the I.G.M.E. (Scale 1:25,000 to 1:200,000)

- Previous studies of several areas for the Construction of Highways and motorways, carried out by the Ministry of Public Works and Urbanism (MOPU), Sectorial geotechnical studies proceeding from specific geotechnical reports.

- Research works or referring to specific performances.

- Plasticity and expansivity tests resulting from reports made by public and private organizations, as well as other tests made for this study.

The comparative revision and analysis of the conclusions contained in the research studies has enabled, on very with particular occasions the approach to expansivity problems linked to specific lithological zones and formations.

The degree of expansiveness of the different chronolithological units has been defined starting from about 1400 expansivity tests and from about 2000 plasticity tests. The distribution of the available tests was very variable. This dispersion is due to the constructive pressure in each
geographic surrounding but also to the ancient expansivity hazard that probably exists in the zone. For this reason the available number of tests was increased, principally, for the lithological formations with a high expansive character which was verified through experience.

5. SIGNIFICANCE OF THE CLASSIFICATION CARRIED OUT

The assignment of a degree of expansiveness to a certain lithological formation does not mean, logically, that this degree is also valid for all the clay materials existing in the formation. There are some alternatives because the geological materials are not usually continuous in space and time.

As an example of these alternatives, figure 4 presents the expansive capacity which was found for the miocene clay formations of the soil of Madrid. These soils have originated problems in building construction due to their expansivity. The graphics show the frequency, obtained in accumulated intervals of 10%, of the results that exceed the defined value. This representation has been made by relating the values obtained in the swell pressure and free swelling tests to the Lambe index.

![Graph showing expansive capacity of clay materials](image)

**Fig. 4.- Expansivity tests results distribution into Madrid clayey soils**
It can be observed that the percentage of soils with a value of expansivity which can be classified as high or very high, is important, but very minoritary. For this reason it is possible to say that into a formation considered ordinary "expansive" there is an important proportion of non-swelling materials.

Nevertheless, considering that the aim was to elaborate a map for the hazard prevention due to expansive clays, it was necessary to point out those lithological formations which had a significant proportion of that kind of clays and which, due to the climatic characteristics, could come into action.

In the final definition of the chronolithological units hazard degree an agreement was established between what could be denominated the "average hazard" and the "maximum hazard" of swelling of the formation, in order to avoid too high or little conservative valuations. The "average hazard" was adopted for expansive clay units subordinated in the formation, but the expression "maximum hazard" was accepted in those cases where these clays became predominant and, naturally, problems were reported due to expansive materials. At the same time this valuation inherent to the chronolithological units was checked with climatic factors.

6. RESULTS OF THE RESEARCH WORKS

The principal alternatives that characterises the different degrees of potential expansiveness of the chronolithological units in Spain can be summarized as follows:

I. Expansive potentiality of the soils from null to low:

- Lithological and/or climatic characteristics not linked to expansivity problems. Defining parameters:
  - Mineralogy with no or little montmorillonitic clay (chronological units IV and, partially, III).
  - Clays of disseminated texture (lithological units f and, partially, e).
  - Perhumid to subhumid climatology.
  - Low plasticity and expansivity tests with low or null values.

II. Low to moderate expansive potentiality of the soils:

- Lithological and/or climatic characteristics where normally no expansivity problems should appear. Defining parameters:
  - Mineralogy containing some montmorillonitic clay (chronological units III and, partially, II).
  - Principally disseminated clays (lithological units e and, partially, b).
  - Humid-subhumid climatology.
  - Maxima plasticities of medium type and expansivity tests with maximum values within the intermediate category.

III. Moderate to high expansive potentiality of the soils:

- Lithological and climatic characteristics which can produce expansivity problems, but no referenced. Defining parameters:
  - Mineralogy principally montmorillonitic (chronological units I and, partally, II).
  - Massive clays (lithological units c and d).
  - Subhumid-arid climatology.
  - Maxima plasticities of high type and expansivity tests with maximum values.

IV. High to very high expansive potentiality of the soils: Lithological and climatic characteristics determining expansivity problems. Defining parameters:

- Mineralogy principally montmorillonitic (chronological units I).
  - Massive clays with a high content in montmorillonite (lithological units a, c and d).
  - Dry-semiarid climatology.
  - Plasticities which can be very high and expansivity tests with high values.

The conjunction of these alternatives was specified in a Hazard Preventive Map on scale 1:1000.000. Figure 5 presents a synthesis of this map. For the explicitum formate of this paper, the scale of this figure is, approximately, 1:5.000.000. In the reduction great number of details had been disappeared.

Finally, it must be pointed out that the expansive hazard of clayed substrates in Spain increases, in general, in the Central-South of the country due to the lack of humidity and to clayed lithotypes which are geologically recent and in many cases very montmorillonitic.
Fig. 5.—Simplified version of the expansivity hazard preventive map of Spain
The approximately distribution of hazards due to expansivity of clayey substrata (35% of national territory), expressed as percentage of the clayey Spanish territorial surface, is as follows:

<table>
<thead>
<tr>
<th>Expansivity hazard</th>
<th>Surface distribution (%) (clayey substrata)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null to low</td>
<td>7.4</td>
</tr>
<tr>
<td>Low to moderate</td>
<td>64.2</td>
</tr>
<tr>
<td>Moderate to high</td>
<td>21.3</td>
</tr>
<tr>
<td>High to very high</td>
<td>7.1</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

The main conclusions obtained in this work can be summarized as follows.
- In the mapping of the natural and artificial risk hazard a previously defined methodology has been necessary.
- In the expansive clays case is necessary taken into account, as a minimum, the chronolithological characteristics of each formation, one climatic Index and the available laboratory swelling test results.
- Also, the general data on natural water content has been taken into account.
- Finally, four grades of expansivity hazard has defined and a Map, scale 1:1,000,000, has been obtained, for general and primary classification use.

8. ACKNOWLEDGEMENTS

The authors wish to express their profound gratitude to all peoples who had been collaborate in this work, with local data, with their commentaries, etc. Specially to the I.G.M.E. (with reference to Francisco Ayala, Min. Eng.) and CIDEX for the support and J. Plaza e J. Ferrero for their collaboration in the treatment of the data.

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Aggregate impact value and aggregate crushing value tests on seven lithologies from the Carboniferous Limestone of the Bristol/Mendip area, England

Essaies d’impacte et de concassage sur des aggregates de sept lithologies du calcaire carbonifère de la région de Bristol, Mendip en Angleterre

S.H. Al-Jassar, Construction Department, University of Technology, Baghdad, Iraq
A.B. Hawkins, Department of Geology, University of Bristol, Bristol, UK

ABSTRACT: Aggregate determination tests, including aggregate impact and aggregate crushing, have been carried out on samples from the Carboniferous Limestone of the Bristol/Mendip area, and the results plotted on a modified triangular diagram. It has been shown that for both aggregate impact and aggregate crushing results these triangular diagrams clearly demonstrate the behaviour of the aggregate under both impact and static loading. Logarithmic/arithmetic diagrams have been shown to give a clear correlation between the different types of aggregates, i.e. the different lithologies.

The relationship between the aggregate impact value (AIV) and the aggregate crushing value (ACV) for different rock types of the Carboniferous Limestone of the Bristol Mendip region is discussed, together with their dependence on other measured properties such as uniaxial unconfined compressive strength and the Schmidt hammer number.

RESUME: Les essais à déterminer le granulat, y compris les essais avec impacts et les essais avec compression, ont été fait sur des prélèvements du "Calcaire Carbonifère" de la région Bristol/Mendip, et les résultats étaient tracés sur un graphique triangulaire modifié. Il a été montré que pour les résultats des deux essais, le granulat-impact et le granulat-compression, ces graphiques triangulaires démontrent claire le performance du granulat sous l'essai avec impacts et l'essai de compression simple. Les graphiques logarithmiques et arithmétiques ont été montrés à être en corrélation claire entre les genres différents de granulat, c'est-à-dire les lithologies différentes.

Le rapport entre les résultats des essais avec impacts et avec compression pour des roches de genres différents de la "Calcaire Carbonifère" de la région Bristol/Mendip est discuté, avec leur dépendance d'autres attributs mesurés tels que l'essai mécanique de compression contrôlée et l'essai avec le marteau Schmidt.

INTRODUCTION

Each year about 15 million tonnes of rock are quarried from the Carboniferous Limestone in the Bristol/Mendip area, accounting for about 20% of the United Kingdom quarried rock.

The dominantly marine Carboniferous Limestone succession is underlain in the Bristol region by the continental Devonian (Old Red Sandstone) and overlain by the Gaultian and fresh water sequences of the 'Millstone Grit' and Coal Measures.

The main rock types in the Carboniferous Limestone formation consist of different carbonate lithologies, siliceous sandstone and heavily over-consolidated mudstone (shale). The calcareous rocks can be divided into six distinct lithologies, namely, shelly, crinoidal, impure, oolithic and micritic limestone, and dolomite. The geotechnical properties of the different limestone lithologies and the siliceous sandstone have been discussed by Al-Jassar and Hawkins (1977 and 1978). Discussion of the mudstone sequences (shale) will not be included in this paper as on weathering they decompose to soil, hence they are of no potential value as an aggregate.
PETROGRAPHY

The petrography of the different limestone lithologies and the siliceous sandstone within the Carboniferous Limestone has been examined using acetate peels and thin sections. The study revealed that:

a) The shelly limestone is bioclastic with a sparry cement;
b) Frequently the included shells show an orientation parallel to the bedding;
c) The crinoidal limestone is micritic in texture with some bioclasts. Some of the micrite has recrystallised to neomorphic sparite;
d) The oolitic limestone consists of ooliths cemented by neomorphic sparry calcite;
e) The micritic limestone is mainly micritic in texture but in some places the micrite has recrystallised to neomorphic sparite;
f) The dolomite, whilst micritic in texture, contains recrystallised dolomite rhombs;
g) The siliceous sandstone is a poorly graded sandstone which is strongly bound by a siliceous cement.

For further, more detailed descriptions, see Al-Jassar and Hawkins (1977, 1979).

Uniaxial Unconfined Compressive Strength

The uniaxial unconfined compressive strengths of the seven different lithologies of the Carboniferous Limestone were tested in three different directions, parallel, vertical and oblique to the bedding. Table I indicates that these different lithologies exhibit strengths, measured on 38 mm diameter 76 mm length cores, of between 105 MN/m² (very strong) and 206 MN/m² (extremely strong) and that the siliceous sandstone has the highest unconfined compressive strength. This latter is attributed to the composition and the texture, with both the grains and the cementing material being composed of silica such that grain growth has taken place, producing an extremely strong rock compared to one where the interstitial filling is of different chemistry. The results also show that the main micritic textured rocks, the micritic limestone and the dolomite show a higher compressive strength than the spartic textured rocks, the shelly and impure limestone.

A further description of the factors affecting the strengths of the different lithologies of the Carboniferous Limestone has been discussed by Al-Jassar and Hawkins (1977, 1979).

<table>
<thead>
<tr>
<th></th>
<th>Shell Limestone</th>
<th>Limestone</th>
<th>Crinoidal Limestone</th>
<th>Limestone</th>
<th>Impure Limestone</th>
<th>Oolitic Limestone</th>
<th>Micritic Limestone</th>
<th>Dolomite Limestone</th>
<th>Siliceous Sandstone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel to bedding</td>
<td>105</td>
<td>154</td>
<td>143</td>
<td>179</td>
<td>218</td>
<td>251</td>
<td>287</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical to bedding</td>
<td>131</td>
<td>161</td>
<td>141</td>
<td>176</td>
<td>233</td>
<td>240</td>
<td>296</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oblique to bedding</td>
<td>113</td>
<td>161</td>
<td>140</td>
<td>166</td>
<td>225</td>
<td>261</td>
<td>288</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TABLE I: The Uniaxial Unconfined Compressive Strengths of seven lithologies from the Carboniferous Limestone of the Bristol region, in MN/m²; using cores of 38 mm diameter and 76 mm long.

AGGREGATE IMPACT TEST

The aggregate impact test is the measurement of the resistance of the aggregate to pulverisation under repeated impact by a heavy weight. The standard test reported in this paper was carried out in accordance with the British Standard 812: 1975. This involved using samples with particle size range between 10-14 mm and subjecting them to impact from a 13.5 kg hammer falling freely from a height of 380 mm. The percentage of the material passing the 2.36 mm sieve after 15 blows is known as the Aggregate Impact Value.
Different factors, such as petrography, grain size, cementing materials, flakiness, angularity number etc, affect the results obtained during the aggregate impact value (AVV) test. These have been discussed by different authors such as Ramsay (1965), Hocking and Tudi (1969), William and Lees (1970), Chir and Ramsay (1971), Attewell (1971), Spence et al (1974) and Hartley (1974).

Technical factors associated with the actual testing must also play a part in the accurate determination of the aggregate impact value. Two of these include:

a) The necessity of having the vertical guide runners of the apparatus perfectly aligned, otherwise an impeded fall may occur which would result in a lower figure than the true value. It was recognised at an early stage of the work that if the hammer did not fall freely due to any distortion or poor lubrication on both sides of the runner, the AVV would be less than expected; consequently both sides were greased with oil to facilitate the free movement of the hammer.

British Standard 812: 1975 does not give any guide to the required standard of maintenance and lubrication to ensure an efficient free fall.

b) The nature of the floor on which the test is performed. It was noted that when different thicknesses of concrete floor were used during the test, the results indicated that with thicker concrete the data achieved were more consistent and reproducible.

This could be attributed to the vibration elasticity of the floor, ie the thicker the floor the more rigid it is likely to be and hence the lower the possibility for vibration during the impact test. Whilst it is accepted that a rigid test floor may not represent the potential situation in which the aggregate will be used, it does ensure a more constant and reproducible test condition.

Although the British Standard indicates the floor should be over 450 mm thick, it should be remembered that this is something rare in the floors of most laboratories. The British Standard does not give any guide as to the rigidity required of the base on which the test is performed, only that it shall be over 450 mm thick.

Results of the test

The test was performed on each of the six main calcareous lithologies and the siliceous sandstone which occur within the Carboniferous Limestone formation in the Bristol region. The results are listed in Table II. These figures are the average of at least two tests; any abnormal figure is usually ignored, although such a situation occurred only rarely. As can be seen, the AVV ranges between 12.9 and 20.41%, the lower figure being exhibited by the dolomite and the higher figure by the shelly limestone.

<table>
<thead>
<tr>
<th>Rock Lithology</th>
<th>Original mass(gm)</th>
<th>AVV% (av)</th>
<th>AVVR% (av)</th>
<th>AIM% (av)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelly Limestone</td>
<td>320</td>
<td>20.41</td>
<td>40.20</td>
<td>39.39</td>
</tr>
<tr>
<td>Crinoidal Limestone</td>
<td>304</td>
<td>18.79</td>
<td>36.51</td>
<td>7.70</td>
</tr>
<tr>
<td>Impure Limestone</td>
<td>327</td>
<td>19.26</td>
<td>41.90</td>
<td>38.84</td>
</tr>
<tr>
<td>Oolitic Limestone</td>
<td>320</td>
<td>20.31</td>
<td>22.18</td>
<td>57.50</td>
</tr>
<tr>
<td>Micritic Limestone</td>
<td>311</td>
<td>15.11</td>
<td>43.73</td>
<td>41.16</td>
</tr>
<tr>
<td>Dolomite</td>
<td>333</td>
<td>12.91</td>
<td>52.6</td>
<td>34.23</td>
</tr>
<tr>
<td>Siliceous sandstone</td>
<td>306</td>
<td>14.05</td>
<td>49.02</td>
<td>36.93</td>
</tr>
</tbody>
</table>

TABLE II: Aggregate Impact Test results - standard tests - for the different limestone lithologies and siliceous sandstone of the Carboniferous Limestone of the Bristol region.

Ramsay (1965) used the term aggregate impact value residue (AVVR) to express the percentage of the original aggregates which is retained on the 10.00 mm sieve after the test. Ramsay et al (1973) described the importance of the value and suggested that it is correlateable with the normal aggregate impact value. The authors also showed that this value is more sensitive to the change of particle size/shape than the ordinary impact value.

Ramsay et al (1977) drew attention to the importance of the median fraction (their M; the Aggregate Impact Median of this paper), which represents that percentage of aggregate whose range in size is between 2.36-10.0 mm. The AVVR and AIM values of the different lithologies of the Carboniferous Limestone of the Bristol region are included in Table II.
Discussion and Conclusion

The results of the aggregate impact value test on samples of Carboniferous Limestone from the Bristol/Mendip area have demonstrated that the dolomite and the siliceous sandstone are more resistant to the impact test than the other lithologies.

Attewell (1971) mentioned that the partial dolomitisation of limestone may lead to weakening of the aggregate; he attributed this to the changes in volume during the process of dolomitisation. Attewell’s explanation is similar to that given by many authors to explain some unconfined compressive strength results in dolomitic rocks. It is unfortunate that Attewell (1971) does not record the porosity of the samples he quoted. It is noted that the results given by other authors for the unconfined compressive strength of dolomitic rocks are such that their explanation cannot be applied in the Bristol region where the porosity of the dolomite is only 3.5% (Al-Jassar and Hawkins, 1977); this has been discussed in more detail by Al-Jassar (1979). The same is true for the aggregate impact test on dolomite, which shows a higher resistance to the hammer impact than the other calcareous lithologies.

The test results obtained during the present study for the dolomite, micritic limestone and crinoidal limestone fall below the figures of 17-33 given in the British Road Note No 24 (1959). This implies that these three types of limestone are more resistant to the impact test than the other limestone included in both this study and those considered when preparing Road Note No 24.

Spence et al (1974) show that a good inverse relationship exists between the AIIV and the AIVR. Ramsay et al (1977) in an attempt to show the data pictorially, proposed the use of a triangular diagram and placed the 100% of the three components (AIIV, AIVR and AIM) of the aggregate impact test at each apex. This diagrammatic form gives an impression of the relative percentages of the main components of the tested aggregate.

Following the suggestion by Ramsay et al (1979) for the use of a triangular diagram it is now proposed that the data can be more clearly presented if the triangle is divided into four equilateral triangles at 50% of each component (Figure 1). This subdivision is valuable in that it allows a classification of different types of rock aggregates, depending in which of the four triangles the plotted results occur, ie according to the percentage of the three components. Thus plotting the results of the aggregate impact test on this modified triangle gives a very clear indication of the relative proportions of the components of the tested aggregate.

Plotting the aggregate impact results of the main Carboniferous Limestone lithologies shows that the majority of the tested samples for the standard mass lie in Triangle 3; while the dolomite lies near the boundary between Triangles 3 and 4, and the oolitic limestone in Triangle 2. Ramsay et al (1977) did not provide the actual data used when preparing their diagram. Consequently it was not possible to replot their points and therefore the points given by them have simply been redrawn on the modified diagram.

When the data from Ramsay et al (1977) were plotted on the modified triangle (Figure 2) it showed that the weak crushed brick material lies mostly on the boundary between Triangles 1 and 3, whereas the majority of the different types of rock aggregates (lithologies unspecified) lie within Triangle 2, some rock in Triangle 3, but only a small number in Triangle 4.

The following points are noted:-

1. In dealing with the aggregate impact values (AIIV) it is important to determine the other two components (AIVR and AIM).

2. Different rock types may show a similar AIIV but different AIVR and AIM, implying that there is no consistent relationship between the three components and that even with the same AIIV, the AIVR and AIM may be very different. As it is important that any good aggregate should exhibit a minimum comminution to particles of any size, the determination of these components and their clear presentation on a diagram is important to a rapid appreciation of the characteristics of the materials. The results of the Carboniferous Limestone aggregates, including the siliceous sandstone, are given in Table II and Figure 1.
The oolitic limestone and shelly limestone show approximately the same AIV but they lie in different triangles which implies that the shelly limestone has a higher percentage of AIVR than the oolitic limestone. The crinoidal limestone, micritic limestone and the siliceous sandstone show relatively the same AIV but different percentages of AIVR. The lowest AIV is represented by the dolomite.

From the above discussion it can be concluded that even when using the same process to cause the disintegration of the rocks, the proportions of the various coarser aggregates (>10.0 mm) are different even for material having the same aggregate impact value. This supports the contention that the proposed modified diagram is a useful way to facilitate the interpretation of the behaviour of aggregates as they undergo breakdown during the test. To give a more detailed indication of the aggregate impact test it is recommended that the number of the respective triangle should be added to the AIV in order to indicate an approximate proportion of the other components (AIVR and AIM).

It has been found that the mass of the tested material has a significant effect on the result obtained in the impact value test and hence on the related components (AIVR and AIM). A sample mass of approximately twice the original sample used in the standard test (BS 812: 1975) was taken to obtain sets of non-standard test results. The non-standard test results are listed in Table III. It can be seen in Figure 1A that in the standard test the calcareous lithologies of the Carboniferous Limestones and the siliceous sandstone have usually a higher AIV and AIM and lie generally in Triangle 3, whereas in the non-standard test there is a relatively low AIV and AIM; consequently the results lie in Triangle 4, Fig 1B.

<table>
<thead>
<tr>
<th>Rock Lithology</th>
<th>Original mass (Gms)</th>
<th>AIV% (av)</th>
<th>AIVR% (av)</th>
<th>AIM% (av)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelly Limestone</td>
<td>606</td>
<td>9.32</td>
<td>56.42</td>
<td>34.25</td>
</tr>
<tr>
<td>Crinoidal Limestone</td>
<td>698</td>
<td>5.84</td>
<td>64.47</td>
<td>29.67</td>
</tr>
<tr>
<td>Impure Limestone</td>
<td>638</td>
<td>8.50</td>
<td>55.97</td>
<td>35.52</td>
</tr>
<tr>
<td>Oolitic Limestone</td>
<td>616</td>
<td>8.60</td>
<td>55.08</td>
<td>36.26</td>
</tr>
<tr>
<td>Micritic Limestone</td>
<td>604</td>
<td>7.54</td>
<td>29.80</td>
<td>32.26</td>
</tr>
<tr>
<td>Dolomite</td>
<td>674</td>
<td>4.94</td>
<td>65.84</td>
<td>25.21</td>
</tr>
<tr>
<td>Siliceous Sandstone</td>
<td>610</td>
<td>5.33</td>
<td>66.90</td>
<td>25.76</td>
</tr>
</tbody>
</table>

**TABLE III**: Aggregate Impact Test results of the different Carboniferous lithologies, for non-standard test.

![Figure 1: Triangular diagram for Aggregate Impact test](image-url)

**A**: for standard samples  
**B**: for non-standard samples.
Figure 2: 500 test results from 52 rock types by Ramsay et al (1977) shown on the modified triangular diagram discussed in this paper.

To express the results of the impact test as determined for the three components (AIV, AIVR and AIM) as well as the mass of the original tested sample (in grammes) another diagram, plotting the results on double log cycle/arithemetic paper can be used. Figures 3A and B give the respective results for both the standard and non-standard samples. The advantage of using this type of plot is that it gives a clear indication of the relationship of the three components of the impact test and allows the opportunity to correlate these components. The dolomites sample, for instance, is seen in the standard test to have the lowest AIV, the lowest AIM, but the highest AIVR figure.

Figure 3: Logarithmic/Arithmetic diagram for Aggregate Impact Test:
A for standard samples
B for non-standard samples.

The disadvantage of the logarithmic diagram is that it cannot be used to depict more than about eight aggregate types; otherwise the distinction and the correlation between the different aggregates will be difficult. The triangular diagram, however, whilst depicting the same information, can be used to show the results of many aggregates.
Plotting the results of the non-standard test on the logarithmic diagram (Figure 38) it can be seen that the pattern of the lines connecting the values of the three components is very different to that of the standard test.

Relationship to the Uniaxial Unconfined Compression Test

If the aggregate impact value is plotted relative to the unconfined compressive strength only, a fair relationship is obtained (Figure 4). The lack of a clear relationship is considered to be due to the fact that the unconfined compression machine measures the strength of the bonding between the grains of the intact rock, while the aggregate impact test determines the effectiveness of disaggregation, i.e., the resistance to pulverisation which reflects the toughness and tenacity of the aggregate under loading, beyond the point of failure.

Relationship to the Schmidt Hammer Test

Schmidt hammer tests were carried out on the different lithological rock types of the Carboniferous Limestone; the results are listed in Table 4. For further detailed description of the Schmidt hammer test see Al-Jassar (1979) and Al-Jassar and Hawkins (1979).

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Perpendicular to bedding, Rebound No</th>
<th>Parallel to bedding, Rebound No</th>
<th>Perpendicular to bedding (Laboratory), Rebound No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelly Limestone</td>
<td>50.0</td>
<td>54.2</td>
<td>46.2</td>
</tr>
<tr>
<td>Crinoidal Limestone</td>
<td>56.5</td>
<td>59.4</td>
<td>49.0</td>
</tr>
<tr>
<td>Impure Limestone</td>
<td>48.6</td>
<td>51.0</td>
<td>43.2</td>
</tr>
<tr>
<td>Oolitic Limestone</td>
<td>54.1</td>
<td>56.6</td>
<td>50.0</td>
</tr>
<tr>
<td>Micritic Limestone</td>
<td>59.0</td>
<td>60.6</td>
<td>52.1</td>
</tr>
<tr>
<td>Dolomite</td>
<td>60.2</td>
<td>63.5</td>
<td>56.2</td>
</tr>
<tr>
<td>Siliceous Sandstone</td>
<td>64.2</td>
<td>67.0</td>
<td>56.4</td>
</tr>
</tbody>
</table>

Table IV: Schmidt Hammer test (N=15) carried out on specimens vertical and parallel to the bedding in the field and vertical to the bedding in the laboratory.

The Schmidt hammer test is the measurement of the resistance of the rock, i.e., the hardness of the intact rock whereas the aggregate impact test measures the toughness and tenacity of the aggregate. Again, for the different types of limestone and the siliceous sandstone only a fair relationship exists between the aggregate impact value and the Schmidt hammer number (Figure 5) for samples tested perpendicular to the bedding. When samples parallel to the bedding are measured the line has a similar slope but moves above the line given in Figure 5 for the perpendicular direction.

![Figure 4: Relationship between Unconfined Compressive Strength and the Aggregate Impact Value for the seven lithological types.](image1)

![Figure 5: Relationship between Schmidt Hammer Number and the Aggregate Impact Value.](image2)
AGGREGATE CRUSHING TEST

The aggregate crushing value measures the resistance of the aggregate to crushing under static load. During the test a continuous loading is applied to the aggregate, ranging between 10.0-14.0 mm in size, through a piston set in a compression testing machine; the load is raised from zero to 400 kN in 10 minutes (British Standard BS 812: 1975). The percentage of material passing the 2.36 mm sieve at the end of the test is known as the aggregate crushing value.

Similar factors affect the aggregate crushing test as the aggregate impact test including petrography, texture, grain size, cementing material, hardness of the constituent minerals, shape of the grains, flakiness etc. Pattoni (1973) has reported that the crushing value is also influenced by the weathering state of the material tested.

Results and discussion

The results of the aggregate crushing test on the different types of limestones and the siliceous sandstone are listed in Table V. These averages indicate an ACV range between 13.87 and 23.74, the values lying within the range given for limestone (11 and 37) in Road Note No 24 (1959).

<table>
<thead>
<tr>
<th>Rock Lithology</th>
<th>Original mass (gms)</th>
<th>ACV% (av)</th>
<th>ACVR% (av)</th>
<th>ACM% (av)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelly Limestone</td>
<td>2840</td>
<td>23.74</td>
<td>26.63</td>
<td>47.61</td>
</tr>
<tr>
<td>Grinoidal Limestone</td>
<td>3000</td>
<td>19.07</td>
<td>33.07</td>
<td>47.84</td>
</tr>
<tr>
<td>Impure Limestone</td>
<td>3125</td>
<td>23.35</td>
<td>31.15</td>
<td>45.48</td>
</tr>
<tr>
<td>Oolitic Limestone</td>
<td>3035</td>
<td>23.55</td>
<td>25.06</td>
<td>51.38</td>
</tr>
<tr>
<td>Micritic Limestone</td>
<td>2932</td>
<td>22.32</td>
<td>26.24</td>
<td>49.43</td>
</tr>
<tr>
<td>Dolomite</td>
<td>3015</td>
<td>13.87</td>
<td>36.87</td>
<td>47.14</td>
</tr>
<tr>
<td>Siliceous Sandstone</td>
<td>2825</td>
<td>13.96</td>
<td>43.50</td>
<td>42.73</td>
</tr>
</tbody>
</table>

TABLE V: Aggregate Crushing Test for the different calcareous lithologies and the siliceous sandstone of the Carboniferous Limestone of the Bristol region.

The dolomite and siliceous sandstones exhibit a much lower aggregate crushing value (ACV) than the values exhibited by the shelly limestone lithologies. The aggregate crushing value residue (ACVR) which was introduced by Shah et al (1971) was calculated from the proportion of the material held on the 10 mm sieve listed in Table V. The median fraction suggested for the aggregate impact test by Ramsay et al (1977) to express the percentage of aggregate retained on the 2.36 mm relative to that on the 10 mm sieve was determined for the aggregate crushing test and has been named in this paper Aggregate Crushing Median (ACM); the results are listed in Table V for the different types of limestone and the siliceous sandstone. In the same way as in the impact test, the crushing value (ACV), as well as the ACVR and ACM, will indicate a better understanding of the behaviour of the aggregate samples under static loading.

The three components of the aggregate crushing test were plotted on a similar triangle to that suggested by Ramsay et al (1977) for the aggregate impact test. In this paper the same modifications as proposed above, Figures 1 and 2, have been incorporated when preparing Figure 6, as they provide a better appreciation for correlating and identifying different aggregates. The results show that all the different types of Carboniferous Limestone lithologies lie in Triangle 3, near the boundary with Triangle 2. The oolitic, impure and shelly limestones show a similar ACV but different ACVR and ACM. The same advantages can be acquired from this diagram as for the impact test diagram, in that it helps in determining the behaviour of the aggregates under static loading.

In a similar manner to the aggregate impact test, ie a logarithmic diagram, Figure 7, can be used to express the three components (ACV, ACVR and ACM) of the aggregate crushing test and the mass of the original sample.

Plotting the unconfined compressive strength and the aggregate crushing value it can be seen that only a fair relationship exists, Figure 8. The same explanation is believed to account for this lack of a clear relationship as that given for the aggregate impact test.

Attewell and Farmer (1976) have reported that both the aggregate impact value and the aggregate crushing value are related closely to the unconfined compressive strength, and also that a high aggregate crushing value is equivalent to a high strain under uniaxially confined loading. From the previous discussion it has been demonstrated that in the case of the Carboniferous limestone lithologies the relationship between both the aggregate impact value and the aggregate crushing value with the unconfined compressive strength is not clear.
Attewell and Farmer (1976) also mentioned that the aggregate crushing and impact values are similar numerically. For the lithologies of the Carboniferous Limestone of the Bristol area the differences between the Aggregate Impact Value and the Aggregate Crushing Value for the dolomitic is 0.96% and for the siliceous sandstone only 0.08%. For the micritic limestone the range is 7.12%, yet only 3.98% for the crinoidal limestone. Hence caution must be adopted before assuming that values obtained for the aggregate impact test can be used to obtain the value of the aggregate crushing figures.

![Figure 6: Triangular diagram showing Aggregate Crushing Values for the seven lithological Types.](image)

![Figure 7: Logarithmic/Arithmetic diagram for the Aggregate Crushing Values for the seven lithological types.](image)

![Figure 8: Relationship between unconfined compressive strength and the Aggregate Crushing Value.](image)

CONCLUSIONS

The Aggregate Impact Value (AIV) of the Carboniferous Limestone samples collected from the Bristol region ranges between 12.91 and 20.41. The Aggregate Crushing Value (ACV) of the different lithologies of the Carboniferous Limestone ranges between 13.87 and 23.74.

This work has demonstrated the need to consider not only the AIV and ACV but also the other components of the test, i.e. the AIVR, AIM, ACVR and ACM. It has also shown the possibility that different limestone rock types may have the same AIV and ACV but different AIVR, AIM, ACVR and ACM. The modified triangular diagrams suggested in this paper for both tests give a clear picture of the aggregate behaviour under repeated impact loading and static loading. The authors suggest that the subtriangle number (in which the results fall) is added after the Aggregate Impact Value and Aggregate Crushing Value to indicate the proportion of the other two components.
It has been shown that there is a fair relationship between the AIV and ACV and the uniaxial unconfined compressive strength of the samples; this could also apply to the relationship between AIV and ACV and Schmidt hammer number. This work has shown the importance of the mass of the test sample on the AIV; the larger the mass (non-standard test) the higher the AIV.

A logarithmic diagram has been suggested by the authors to express the results of both tests. Not only can the relationship between the three components be shown on these logarithmic diagrams, but also they facilitate correlation between the different types of aggregate. The diagrams are only useful for a limited number of rock types.

This work is still in progress, notably correlating other lithologies with the strata from the Carboniferous Limestone.

ACKNOWLEDGEMENTS

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REFERENCES


Up-to-date valuation of the results of the building material investigation

Resultats des recherches sur les matériaux de construction

Sándor Karácsony & Zoltán Bereáth, Institut for Geodesy and Geotechnics, Hungary
Mihály Mészáros, Central Geology Office, Hungary

ABSTRACT: Growing demands of the building material industry requires the raw material in constant and appropriate quality. Therefore it is necessary to have more data with higher information content about the no homogeneous raw materials. The up-to-date elaboration of the data with higher information content can be done using calculation methods. But using only calculation methods for itself is not enough. It is necessary it would be in accordance with the system of investigation i.e. with the distribution of bare holes with sampling and with the representative quality testing of the samples. For the necessary utilization of the raw material it is not enough to know the static distribution of its quality but it is necessary to determine the exploitation program ensuring the required quality of the raw material.

RÉSUMÉ: Les matières à construction sont dans une qualité permenente et appropriée exigées par l'industrie de construction. C'est pourquoi on a besoin d'avoir plus de données avec un niveau plus élevé informatique concernant les matières premières hétérogènes. Les données d'information plus élevées peuvent être élaborées par les nouvelles méthodes de calcul, seul ne pouvant point satisfaire les exigences. Pour l'utilisation future de la matière première ce n'est pas satisfaisant de seul connaître la distribution statique et sa qualité, mais il faut projeter le programme d'exploitation assurant la qualité exigée de la matière première.

1 DISCUSSION AND NEW OBSERVATIONS

The layers with rough detritus are occurring in several areas, so in Hungary too in a relatively great extension near the surface and often in a thickness more than 200 m. This enables their widespread building industrial utilization. The gravelly series from the mostly Quarter period are products of fluvial accumulation, both the settlement and the quality of which is very capricious and changing. Their marking the most favourable for a mine opening occurs through comprehensive and systematical research /from the prognosis until production/.

The most important aim of the geological prospecting on the prognostic raw material occurrences is the suitable pre-indication of the plant-parameters. For indicating the space tendencies the interpolation and extrapolation of the exploration informations is necessary. The most suitable way of this is to realize an evaluation reflecting well the geological structure and the genetics and approaching well the reality. In case of the settlement conditions our case is relatively simple because we can make our isoline maps precise on basis
of geological profiles which can be generally geologically well to be interpreted. The processing and geological interpretation of the quality parameters always more important from the point of view of utilization is already not so simple. The complexity of these is already much greater. In this case the application of other methods may be absolutely necessary too for assuring the acceptable exactitude. Such a method is rendered to us by the geostatistics by which the inner structure of the digitalized geological and quality informations can be followed too.

The method elaborated and under development resp. - based on geostatistics - will be illustrated by the example of the gravel occurrence Leasencetomaj-Billége. As connecting to this we have to remark that the indicable regularities present themselves generally in a stronger way in layer series with rough detritus where from the clay to rough gravel all fractions beside a certain classification appear. The presented area is favourable from the point of view too that its exploration by drilling is equal, the same time the connection of the layer series with detritus and the basic mountain as the lowest layer can be investigated directly.

Previously the estimation of the quality informations within the explored occurrence happened mostly by a nearly linear interpolation. One of the most modern variation of this is the so called kriging method based on geostatistics. This is based on taking the inner quality structure having a correlation domain /r/ to be indicated by a variogram into consideration /Fig.1/.

![Variograms of the settlement parameters](image)

**Legend:**
1. empirical variogram of gravel contents,
2. of clay-silt-contents,
3. of lower layer level and
4. their approach by a spherical model

With the interpolation the absence of distortion of the estimation and the minimalization of the estimation varyancy is an exigence by which the best linear distortionless estimation is rendered. It reckons with the conditions of
variability and covariability of the known and unknown parameters what requires a very great computing capacity. In spite of this the method is schematic and equalizing.

From the variations of kriging we have applied the interpolation by point kriging, its running was implemented on a computer MADIX. The clay-silt contents /volume %/ maps prepared by linear and stochastic densing are shown by Figure 2.

The two variations do not differ essentially from each other in spite of the great calculation investment of the latter. The geological interpretation of quality tendencies which can be illustrated by maps is not unambiguous. According to our investigations the more detailed analysis of the inner structure can render a possibility for the further precising of above methods. Already the same frontier zone of the variograms /Figure 1/ supposing the shown isothropy is justifying the tight connection of the structure, settling and quality parameters. The "hole effect" indicating the changes in space and cycles of the settlement parameters /lower layer, clay-silt contents, gravel contents/.

Fig. 2 Depiction by isolines of settlement parameters by a linear and stochastic interpolation /gravel occurrence Lesencetona/Billega/

Legend: A. depiction with linear, B. by stochastic interpolation, 1. drilling, 2. condensation by point-kriging, 3. isoline of clay-silt-contents /volume %/, 4. areal extension of clay-silt-contents 12 volume %
can be recognized. In all this we can recognize the effect on the previous river water and by this on the accumulation of the alluvium of the space position of the lower layer cut by tectonics. After all this the question arises if a system of the change of the parameters was indicable and if it could be restored back in space. For the analysis of this we have produced variograms along direction /Figures 3 and 4/. These are reflecting the anisotropy $/a_1, a_2/$ in the frontier zone and variancies occurring generally on the one hand and are rendering a possibility for the more detailed analysis of the "hole effect" on the other. On the variogram curves reflecting the direction of the greater variability the variances of the clay-silt- and gravel contents are showing the different character. For the investigation of the causes we must know that the variogram renders a certain superposition of changing of the parameters with the increase of the distance. Following from this interpreting the variogram approaching as an integral curve there is a possibility for the approaching modelling the formation along profile of the parameter /Figure 3/.

\[ \text{Fig.3 Variograms and analysis of the direction of a greater settlement parameter changeability} \]

Legend: A. empirical variograms along direction: 1. gravel contents, 2. clay-silt-contents, 3. level of lower layer, B. interpretation of the "hole effect": a. change along direction and b. integral curve of settlement parameter.

The two qualitative variables are giving so the mirror system of each other, the positive and negative anomalies of which are changing each other in a definite cycle. Its explanation can be searched the regularities of fluvial accumulation and in the contrary appearance generally more gravelly of the river bed and clayey-silty of the banks. The same character in the direction
of the smaller changeability indicates the identity of the cycles. Following from all this the outlines of a cycle model restorable in space can be seen. For justification the totality of the gravel occurrence was divided to storeys 10 m each and in all slices /5 slices/ the map of the gravel-, silt- and clay-contents was drawn. The relative maxima presenting themselves with a relative frequency greater than 50% were contracted on a single map. The so appearing anomalies complementing each other and the anisotrophy ellipses and system of the variograms forming a coinciding picture justified the rightness of our suppositions. It can be stated that on basis of the relative maxima and minima appearing on the occurrences a cycle model in principle can be restored which appears with a different intensity but in all parameters. By the way this is justified also by the correctly drawn geological profiles.

Fig. 4 Variograms and analysis of the direction of a smaller settlement parameter changeability.

Legend: A, empirical variograms along direction: 1. gravel contents, 2. clay-silt contents, 3. level of lower layer, B, interpretation of the "hole effect": a. change along direction and b. integral curve of settlement parameter.

The isoline maps interpolated taking the above described model into consideration show a picture reflecting more reliably the geological structure and genetics. We illustrate this by the comparison of the isoline maps which are linear and made exact by a model /mass %/ of the gravel contents of the area. The procedure renders the base of elaboration of simulation models which seem to assure a prognostics more exact than all ones until now. Their formation and mathematical conception is going on. On basis of above a model in principle can be formulated gravel fields in all cases which renders a possibility for the production of maps and profiles reflecting genetics the best possible corresponding to the given prospecting phase. By the investigation in the horizontal slices the change of the major orientation of the fluvial accumulation can be followed. All this forms the base of the elaboration of mathematical models which helps mining first of all by forming out a production following better the required and areal quality average resp.
Fig. 5 Structural analysis
Legend: 1. drilling, 2. the relatively great clay-silt contents /volume %/ appearing with a greater frequency than 50 % in slices of 10 m of the occurrence, 3. extension area of much rough detritus /gravel, mass %/, 4. center in principle of zones of different quality /relatively rough and fine/, 5. area of the qualitatively coherent unit

Fig. 6 Characteristic geological profile in the direction of the greater settlement parameter changeability /profile trace-line on Fig. 5/
Legend: 1. clay, silt, 2. sand, gravel is the following in mass %: 0-15, 3. 15-40, 4. 40-60, 5. 60-80, 6. 80-100, 7. limestone, dolomite
Fig. 7 Settlement parameter's isoline depiction by interpolation beside a linear and modelled inner structure

Legend: A. depiction by linear, B. taking model into consideration interpolation, 1. isoline of drilling, 2. of gravel contents (mass %), 3. areal extension of gravel contents where gravel contents is than 40 mass %
The impact of geomechanical properties of working environment of the technology of making water resistant screens for draining surface excavations

L'effet des propriétés géomechaniques des terrains environnants, sur la technologie de fabrication des filtres hydrauliquement stables pour le drainage des fouilles a ciel ouvert

Radomir Simić, Slobodan Vujčić & Vladimir Pavlović, Faculty of Mining and Geological Engineering, University of Belgrade, Yugoslavia

ABSTRACT: The paper deals with the technology of making water resistant screens for the system of drainageing surface excavations. Particularly is treated direct impact of geomechanical properties on the selection of adequate technology of drainageing structures. The methodology for solving the problem of mutual impact between the geomechanical properties of working environment and the technology for making water resistant screens was developed for the case study of surface excavations on the location "Čirkovac" in Kostolac, Yugoslavia.

THE INFLUENCE OF GEOMECHANICAL PROPERTIES ON THE TECHNOLOGY OF WATER-TIGHT SCREENS FOR DRAINING OPEN PIT MINES

1. INTRODUCTION

The protection against the infiltration of water is a most important factor for the successful drainage of open pit mines.

A great number of open pit mines in Yugoslavia, especially of lignite, are closely located to watercourses and contain permeable layers through which mine water easily filtrates. Examples are the mines at Kolubara, Kostolac, Suvođol, Jelovce, Kosovo, Duzla, Gacko, Kovin, etc.

The problem of mine water infiltration on the zone of open pits requires a careful analysis of hydrogeological, hydraulic and other deposit conditions decisive on the intensity of infiltration. In other words, the degree of infiltration can be assessed only if the following elements are analysed: hydrologic and hydrodynamic relationships in the open pit zone and in its surroundings, geotechnical characteristics of mineral deposits and the manner of opening and the choice of the open pit exploitation technology for the prevention possibilities of infiltration in mining technology.

The study of these relationships gives a basis for the successful investigation of the problem of infiltration of mine waters in an open pit zone.

Because of the fact that the numerous open pits are opened near to permanent or periodical watercourses, in alluvium, or in such environments which enable an intensive hydraulic connection between the source and the open pit zone, in open pits in Yugoslavia we have prominent infiltration problems.

In drainage systems the infiltration is prevented by: the displacement of the watercourse out of the zone of influence onto the open pit (Fig.1.), canals for cutting smaller water-bearing layers (Fig.2.), by cutting water-bearing layers by impermeable screens (Fig.3.) and by placing wells between the source and the open pit (Fig.4.).
Fig. 1. - Dislocation of the watercourse outside of the zone of influence of the open pit

Fig. 2. - Protection by canals

Fig. 3. - Protection against infiltration by a water-tight screen

The most successful way to stop the infiltration of water into the open pit zone is the cutting off of water-bearing series, or the flow of mine waters from the source to the open pit. The cutting off is brought about by an object technically called the screen, diaphragm, curtain, etc.

The use of screens in the draining system of open pit mines is even more relevant, bearing in
mind that the screen offers a reliable protection against the infiltrating waters, that it stops mineralization of waters which flow in the direction of the source and it reduces the negative effects of depression which are more pronounced in the drainage systems without the screen.

However, screens can be built only in particular conditions, and in addition, a special technology and mechanization is required; the funds for their building take an increasingly important part in the investments for an open pit mine.

Because of all this, the building of screens for draining has to be a result of a thorough comparative analysis with other ways of stopping the mine water infiltration in open pit mines.

A series of wells is the most frequent alternative to the planning of screens and the final decision depends on the technical and economical comparison of the objects in question. The advantages of the screens come from the facts that they are permanent in use, do not require maintenance, nor the use of energy. However, besides of using screens, other means of drainage can also be considered, especially the use of wells for lowering the level of the mine waters in the zone of the planned screens. This can decide in favour of the wells, because of the comparatively high initial investment for the construction of the screens.

Two problems concerning the building of screens are covered in this paper: the determination of the stability of a wider zone in the function of the screen, and the hydraulic penetration of mud in the side of the screen.

The selection of these two problems does not mean that the authors underestimated the importance of a complex influence of geomechanical characteristics of a working medium on a technology of making the screens.

2. THE DETERMINATION OF THE STABILITY OF A WIDER ZONE IN THE FUNCTION OF THE SCREEN

It is well known that the presence of water degrades the geomechanical properties of the working medium. Introducing a body such as is the screen into the structure of the working medium, the condition of ground water changes in vicinity of the screen, thus modifying also the geomechanical properties. From the point of view of slope stability the situation is favourable, but it is rather complex for geostatic calculations. The example of the open pit mine "Drmo" IEK "Kostolac" will show the possibilities of the computer use for the determination of the
stability of the structure "slope-screen".

In the computer centre of the Faculty of Mining and Geology in Belgrade a number of programmes has been developed for the determination of slope stability. Bishop's method is appropriate for the determination of the stability of a wider zone in the function of the screen.

Bishop's method is most frequently used for the determination of the stability of slopes. It is an analytical lamellar method with a circular sliding surface. The method takes into consideration the lateral forces onto the strips and the pore pressure, due to the hydrostatic pressure of groundwater in front of the slope and the level of the water table, and due to consolidation. The method is used for the determination of the stability of homogenous and heterogeneous working media for sliding at the level of the foot of the slope or within the ground.

The mostly used expression for the determination of the stability factor by Bishop's method is of the form:

\[ F = \frac{1}{W \sin \phi} (c'b + (W \cos \phi - Ub) \tan \phi) \quad (1/m) \]

where:
- \( \phi \) - the angle of internal friction
- \( c' \) - cohesion
- \( W \) - the weight of a strip
- \( \phi \) - the angle of the normal force in relation to the vertical axis of the strip
- \( b \) - the width of the strip
- \( U \) - the pore pressure
- \( m = \cos \phi + \frac{\sin \phi}{\tan \phi} \)

The programme for determination of the stability of slopes by Bishop's method was developed in BASIC on a Wang-2200 computer at the Faculty of Mining and Geology in Belgrade. This programme can be applied widely.

The flexibility of the programme can be assessed by the following:
- it is possible to treat slopes of any shape,
- the programme takes into consideration the hydrostatic pressure of water in front of the slope (slopes on river banks, lakes, etc.),
- it is possible to treat drained or undrained slopes with arbitrary seepage pattern,
- it is possible to treat slopes within complex geologic structures,
- the programme takes into consideration the influence of supplementary loads,
- for each given centre of a sliding circle, the computer automatically finds a sliding arc (R) with minimum stability factor,
- by entering values for the coordinates \((X,Y)\) of the centre of a sliding arc and its radius \(R\), cases of sliding at the foot level or within the ground can be analysed,
- for given geometric, geomechanical, and hydrogeological elements which define a slope, it is possible to repeat for new sliding arcs the determination of the slope stability as many times as necessary.

Cross-section BK-20 on Fig. 5 shows that it is heterogeneous. In the structure of the cross-section there are five types of materials with the following geomechanical characteristics:
<table>
<thead>
<tr>
<th>Material</th>
<th>C(KN/m²)</th>
<th>K(KN/m³)</th>
<th>θ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>19</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>21</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>20</td>
<td>34</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
</tbody>
</table>

Fig. 6 shows a numerical model of the cross-section from Fig. 5 as well as other input information necessary for computation. Results are shown on Fig. 7.

![Diagram](image)

**Fig. 5.** - The cross-section with a screen and slope

**Fig. 6.** - The determination of slope stability using Bishop's method

Project: example for demonstration
Geomechanical properties of a working medium

1. $\gamma = 19 \text{ (KN/m}^3) \quad \phi = 25^\circ \quad c = 20 \text{ (KN/m}^2)\)
2. $\gamma = 20 \text{ (KN/m}^3) \quad \phi = 30^\circ \quad c = 10 \text{ (KN/m}^2)\)
3. $\gamma = 21 \text{ (KN/m}^3) \quad \phi = 20^\circ \quad c = 20 \text{ (KN/m}^2)\)
4. $\gamma = 20 \text{ (KN/m}^3) \quad \phi = 34^\circ \quad c = 0 \text{ (KN/m}^2)\)
5. $\gamma = 20 \text{ (KN/m}^3) \quad \phi = 30^\circ \quad c = 10 \text{ (KN/m}^2)\)

Coordinates of the centre

$x = 35 \quad y = 55$

$R = 30$

$F_1 = 1,704 \quad R = 30,00$

$F_2 = 1,905 \quad R = 32,70$

$F_3 = 1,829 \quad R = 31,35$

$F_4 = 1,704 \quad R = 30,00$

$F_5 = 1,682 \quad R = 28,65$

$F_{\min} = 1,6827508904$

Coordinates of the centre

$x = 45 \quad y = 55$

$R = 35$

$F_1 = 2,425 \quad R = 35,00$

$F_2 = 2,581 \quad R = 37,70$

$F_3 = 2,494 \quad R = 36,35$

$F_4 = 2,425 \quad R = 35,00$

$F_5 = 2,385 \quad R = 33,65$

$F_{\min} = 2,38512660654$

Fig. 7 - The characteristics of a working medium with a screen

3. HYDRAULIC PENETRATION OF MUD INTO THE SIDE OF THE SCREEN

When building cuts for the screens, there is a flow from the cuts to the massif caused by the difference of the level of mud in the cuts and the groundwater level considering also the density of the mud.

The diagram in Fig. 8 shows the value of the mud infiltration for various grain sizes with the same mud and the pressure of $p = 9,81 \times 10^6$ Pascals, which have been determined for various depths of penetration into the outer screen crust. If these points are plotted onto a diagram, it results in an approximately interpolated curve. By increasing the coefficient of permeability, the depth of penetration into the sides of screens increases, and with it the thickness of the outer crust of the screen, i.e. the total thickness of the screen.

During the building of cuts the liquid of the mud is enriched with grains which have been excavated from the soil in some layers of the massif (rock). Parts of these soil grains are carried by streams from the cut to the side of the massif. These soil particles will probably fill the pores of the existing layers of the massif so that the full depth of penetration which would be accomplished by pure water cannot be attained.

As it has been mentioned, the element of influence of the flow of mud after the outer crust will form an internal crust of filters.
The depth of penetration of the crust of filters depending on the coefficient of permeability of the rock material (massif)

The thickness of the outer crust of filters greatly depends on the composition of the soil layers (and also on the time of the influence of the mud onto the side of the cut), and since it can be mathematically determined only approximately, it has first to be tested as to how much the internal crust of the cut is able to guarantee the effect of the screen.

Depending on the technology of cuts and the type of the backfilling, the material in the cut can have a higher coefficient of permeability than the outer or the inner crust of the cut. Therefore this should be taken into consideration, so that the inner crust of the filters is formed with adequate thickness.

A research has been made into the relationship between the outer and the inner crust of the cut, the hydraulic gradient and the hydraulic penetration. In these experiments a physically similar model has been applied. By using compressed air, the samples were loaded with a constant raise of pressure until hydraulic penetration occurred. Sand was the protective material of the samples, and its thickness was between 15 and 75 mm. The crust of the cut had a $k_d = 4.8 \times 10^{-9}$ m/s.

If an artificial influence occurs in the body of the soil, then the geomechanical parameters of the soil can be modified. For example, this may happen when building the elements of drainage or when setting the screens. Then, the flow of water acts as a sole cause of the local mechanical deformations of the cohesionless material of the soil and can produce the above characteristic phenomena.

When dealing with foundation failure, the existence of geometric and hydraulic criteria of foundation failure has to be determined first.

During the building of the screens, there will be no foundation failure, because of the flow from the cuts to the massif (rock). Foundation failure can occur in the screen itself and in front of it, as a result of high hydraulic gradients.

4. CONCLUSION

This paper covers the general problems of building watertight screens, and in particular, the stability of a wider zone in the function of the screen in the open pit mine "Greme" in Yu-
goslavia was determined. The problem of hydraulic penetration of the mud onto the sides of the screen has also been discussed.

The problems covered in this paper are a part of a complex appreciation of the influence of geomechanical characteristics on the technology of building watertight screens for draining open pit mines.

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ABSTRACT: The Spanish seismic norm has a direct application in building construction but it recommends specific studies in the case of important public works such as large dams or bridges. For this reason, and to establish specific criteria in its field of activity, the Dirección General de Obras Hidráulicas of the Spanish Ministerio de Obras Públicas y Urbanismo commissioned us a seismotectonic and seismic risk study applicable to Spain, materialized on a series of maps of immediate and direct use. In this paper we explain the methodology pursued to obtain these maps. It has required, firstly, investigations with the aim to improve the seismic information corresponding to the historical or preinstrumental period, that allowed more precise quantifications. Secondly, these data have been processed by probabilistic methods, using intensity as fundamental parameter. The corresponding maps have been developed. Finally, other maps of seismic accelerations have been compiled.

RESUME: La normative sismorresistente espagnole à une application directe dans l'édification, mais elle recommande la réalisation des études spécifiques dans le cas des travaux publics importants telles que ponts ou barrages. Pour cette raison et pour établir des critères spécifiques dans son champ d'activité, la Dirección General de Obras Hidráulicas du Ministerio de Obras Públicas y Urbanismo espagnol a commandé un étude sismotectonique et de risque sismique applicable à l'Espagne, materialisé en une série de cartes de utilisation directe et immédiate. Dans cette communication on explique la méthodologie a suivre pour la réalisation de ces cartes. Celà a fait nécessaire, d'abord, des recherches pour améliorer l'information concernant le période historique ou préinstrumental, ce qui permet des quantifications plus précises. En second lieu ces faits ont été traités avec des méthodes probabilistes, employant l'intensité comme paramètre fondamental. En conséquence nous avons développé des cartes d'intensité et d'accélération sismique.

INTRODUCTION

The elaboration of a recent research made in behalf of the Dirección General de Obras Hidráulicas of the Spanish Ministerio de Obras Públicas y Urbanismo (Dirección General de Obras Hidráulicas M.O.P.U. - Ingeniería 75, S.A., 1.986) has allowed to pursue an analysis of the seismic risk distribution all over the Spanish territory. The results of this research have shaped a serial of maps that facilitates in a direct way the finding of the basic seismic parameters that are to be considered in structural projects. The methodology that has been followed for this study is a generalization of regional character from other previous researches by the authors, referring to seismic risk studies, regarding specific construction locations (Sáenz Hidroel, C., Arenillas, M. & López Marinas, J.M., 1.978; Arenillas, M., Sáenz, C. & Samartín, A., 1.982; Arenillas, M., Fernández Montero, A. & Torcal, R., 1.982). An immediate precedent of this work is a doctoral thesis made by one of the authors (Bisbal, L., 1.984) that has helped to set a suitable working methodology. (Arenillas, M. y Bisbal, L., 1.985).

In the following chapters of this paper, the systematic methodology followed through the seismic risk analysis is shown in the Spanish territory. The Canary Island area has been excluded due to its special seismotectonic characteristics, that need slightly different out-lines.

AREA OF STUDY

It's generally admitted that earthquakes stop producing noticeable effects further than approximately 50 km from its epicenter. This simplification is usually advisable in most cases, but there are others where it's not like that. The Iberic Peninsula is one of those cases, as the effects of the earthquakes related to the transforming Azores-Gibraltar Fault, are felt all over the occident.
of the Peninsula with figures that, in some areas, match the highest known, in spite of their
being located at a fairly bigger distance than the emitting 300 km focus previously mentioned.
This is normal because, on one hand, these areas are situated on the ancient hercynic oraton
that has a very low seismicity and, on the other hand, the indicated atlantic earthquakes focuses, are
generally quite deep and cause important energy liberation which effects are transmitted to very
remote distances from the corresponding epicenters.

So, in the spanish case—and specially in peninsular Spain—it's not possible to ignore the exis-
tence of some remote focuses, and therefore, do without them over all, at least all over the West
Area of the Peninsula. Accordingly, and in order to cover all possible epicenters whose effects
can reach the spanish territory, we've generally extended the study area to 300 km from the spa-
nish territorial boundaries, but we've extended this limit to the west of the Cadiz Gulf, to take
in consideration the important seismic focuses related to the Azores-Gibraltar structure

CALCULATION METHOD

Seismic risk studies has been approached from two different angles. One one hand a method is follo-
wed, normally called determinist, that involves the utilization of the biggest real earthquakes,
using these events for the definition of the most unfavorable stage. The other calculation proc-
dure is the probabilistic one, in which earthquakes are taken as aleatory variables, defining
the relationship between return periods and the values of the adopted seismic parameters, through
the corresponding statistical treatment. Even if in some minor studies is possible to accept the
validity of any of the two previously indicated calculation procedures, the fact is that in a re-
gional analysis, deterministic methods can introduce important distortion in the final results,
sometimes on the safe side, and sometimes on the contrary. That's why the isoseismal curves
that are finally obtained are only formal synthesis of different seismic situations that are not
comparable among them (Arenillas, M. & Biañal, L., 1.988).

The previous reasons have led us to approach the seismic study of the spanish territory by a
probabilistic procedure that, basically, has been developed as follows.

SEISMOLOGY CALCULATION PARAMETERS

In order to select the seismic sample that is used later in the risk quantifications, we set out
with a list of earthquakes from a publication of the Service of Seismology and Seismic Engineering
of the Instituto Geográfico Nacional, called "Sismicidad del área ibero-magrebí" (Mexequa, J. &
Martínez Solares, J.M., 1.983) where an extensive information is found on earthquakes happened
within all our study area and further in some sectors.

This publication, that covers a period of almost twenty centuries, and goes up to 1.980, has ma-
ny references to preinstrumental seism, in which it was possible to verify the existence of qui-
to a lot of errors and omissions, since it originates from different classic catalogs, whose rea-
son was not the precise quantification of the earthquakes, but a mere descriptive classification
of these events.

This is why it was necessary to go through all this information critically, leading to a
thorough research of all documentary sources available. Logically, this research was centered in
the earthquakes happened in spanish territory, and the study was done according to the methodolo-
gy exposed in a previous work (Arenillas, M., López Marinas, J.M. & Biañal, L., 1.983) to which
we refer.

According to that research, it was possible to extend the list of Instituto Geográfico Nacional,
that includes 6.830 seismic references within the study area, to 7.818 earthquakes after adding
1.048 non-referenced events and eliminating 60 figures that corresponded to phenomena of other
kind (43) or to duplications in the first list (17). So then, from those 7.818 seisms, only 3.734
were found valid for the calculation, as they where the only earthquakes for which it was possi-
ble to set with reasonable precision, the two parameters that we consider essential to do the lat-
ter calculations, that are the location and the epicentral intensity.

In our opinion, it's compulsory to use the intensity as a parameter of seismic definition and
not the magnitude that is the factor usually used in some studies. As we said in some other ocasio-
nion, (Arenillas, M. & Biañal, L., 1.985) it's convenient to keep in mind that "on one hand, the
seismic magnitude, in spite of its teoric direct relation with the liberated energy in each phe-
nomena, responds to empiric formulations whose general validity is not totally demonstrated yet.
Moreover, instrumental support being needed to define the parameter, restriction of the period of
information to a few decades would result in this case, unless a-priori assignation of the focal
depths of the historical earthquakes was chosen. The impresiones derived from using this proced-
ure would be, in our opinion, higher than those caused by the direct use of the seismic intensity.

We have to consider also that it doesn't look reasonable to limit our calculation sample to ins-
trumental earthquakes, as records of this kind correspond to an exceptionally calm period of the
spanish seismic history, and that would suppose to disregard in the calculations the biggest earth-
quakes that took place in the area of study. On the other hand, the intensity, as defined in M.S.
K. scale, which is the one used by us, shows in fact, the influence of a factor of undoubted imper
tance in the structural calculation that is response to seismic action. This matter, that we are not analyzing now (see Lomitz, 1.981) helps us support the decision that we made about the parameters of calculation.

SEISMOTECTONIC UNITS

The concept of seismotectonic units is old and usually applied to every portion of the territory with homogeneous tectonic characters and where seismic activity follows systematic time and space sequences.

The seismotectonic compartmentalization is also a device used when sufficiently precise data are missing for the correlation between earthquakes and faults. In areas where seismicity is not often shown, like the case of the Spanish territory, even in its more active sectors, using this type of compartmentalization is justified more, because it allows to obviate the imprecisions that come from theoretically more exact correlations, like punctual or linear type. These correlations, only in few cases can be met with a degree of enough detail, for the real difficulty to assign probabilities of occurrence of earthquakes in certain faults of a particular structural family.

Nevertheless, this kind of compartmentalization requires a complete combined analysis of the tectonics and the seismicity of the area of study, and it obliges to choose or the other, according to the higher or lower definition of their respective characters. Thus, for instance, there are areas of determined tectonic units—particularly those of high seismicity—where the concentration of seismic focuses and the systematic character of the occurred earthquakes are shown. This leads to the individualization of these areas within its respective structural units. In other occasions, a disperse and low systematic seismicity in time, can advise the definition of a big unit determined by similarity of tectonic features, so that a sufficient united homogeneity is achieved, as the concept of seismotectonic unit itself requires. This type of considerations, that proved valid in numerous works that support this present one, have taken us to compartmentalize the area of study in numerous seismotectonic units of variable surfaces. This is consequence of the notorious geological-structural variety of the peninsular territory, matter known enough to be insisted on.

Therefore, this type of compartmentalization of the area of study has required a previous division in tectonic provinces, characterized by uniformity of their structural conditions within reasonable limits. One this division was made, the seismotectonic units were obtained overlapping seismic information and outlining areas in which uniformity of both conditioners is fulfilled.

According to the above, the area of study has been finally divided in 103 seismotectonic units of very variable importance.

SEISMIC ATENUATION

Dampering of seismic actions from the epicenters where earthquakes area superficially located, and, as it's logic, from the hypocenters themselves, or deep focuses, depends on many factors, some of them insufficiently shown and some others difficult to known in detail. Among the first ones, there are the focal mechanisms, or processes according to which energy is liberated, and among the second ones the geological parameters, or the particular characteristics of the materials through which seismic waves are transmitted. In order to quantify these complex phenomena, it was attempted to define various mathematical formulations that were not satisfactory in any case and, anyway, can't be generalized with enough guarantee. That's why atenuations deduced from isoseismal curves are usually used, corresponding to well-known earthquakes. This is the procedure we've followed in the present study.

To accomplish the above, in the first place we've consulted the publication by Mescua, J. & Martínez Solares, J.M. "Catálogo general de isoseismas de la Península Ibérica", 1.982. This publication is a recompilation of isoseismal curves corresponding to 261 earthquakes, studied by many different authors, according to available data in the Servicio de Sismología e Ingeniería Sísmica del Instituto Geográfico Nacional. The publication does not claim the critic analysis of the introduced information, and therefore it's not possible to assign the same degrees of validity to the different graphics that are found there. This has led us to revise these graphics and object those that didn't depict the corresponding phenomenon correctly, in our opinion. Comparative analysis of earthquakes related to the same seismotectonic unit has been very useful for this topic.

In addition to the graphics of the previous publication, other data were taken from some monographic studies of different earthquakes where it was possible to define the corresponding isoseismal curves.

At this rate, in order to obtain the different atenuation curves, in the first time graphics corresponding to earthquakes that took place in the same seismotectonic unit were clustered, and then those graphics were analyzed separately.

In most cases, it was possible to verify that earthquakes from each seismotectonic unit showed privileged orientations of maximal or minimal atenuation, usually different for each unit. In a
first adjustment, following the criteria that we've applied in the mentioned doctoral thesis, medium values of the directions were used, turning out to be, in general, four different orientations for each of those units, and, therefore, four attenuation curves. Defining the effects of each emitting focus on each receiving point through the linear interpolation in function of the direction between certain values allows to establish a sufficient adjustment. These values would be set through the application of the attenuation curves that correspond to the two directions between which the line defined by both points runs.

Nevertheless a posterior adjustment allowed to verify that choosing four standard directions for every seismotectonic unit, the final result were basically identical to the first ones. As this last simplification makes calculation much easier it was finally adopted. Orientations North, South, East and West were set as preferred seismic attenuation directions, for all seismotectonic units (except those related to structures linked with the Azores-Gibraltar fault).

Because of the marginal situation of the Gibraltar-Azores area compared to the Peninsular and the peculiar order of the isoseismal curves of the earthquakes related to this units, other preferred attenuation directions were kept, in order to avoid important distortions in the final results. This does not complicate excessively the calculation procedures because the units that are related to the Azores-Gibraltar fault lead to a specialized treatment of their effects in the Peninsula due to the lack of 300 km limitation previously referred and explained.

Afterall, attenuation curves of all the earthquakes with epicenter in each seismotectonic unit that correspond to the four mentioned preferred directions, were obtained for each unit. Each of these groups are formed by a number of curves (not a lot, in general, as there are not many isoseismal, graphics available) from which the calculation curves are established. These have been defined, either choosing the one that assumes lower attenuation, or setting the involving of each one in the group, if this is more operative due to the existence of noticeable differences among them. With this criterion we are on the safe side, because given equal epicentral intensities, lower attenuations correspond to deeper focuses.

All these curves are continuous and could be easily defined, with enough approximation, with polynomials, as it has been done in some cases. We've chosen to do without this step, as the calculation with computers is simplified substituting each one of the original curves with a group of straight tracts, that is the way they are all finally defined.

A certain improvement on the quantification of the seismic attenuations is still possible. This comes from the fact that in certain areas where many earthquakes took place (which in the end enables us to have a big sample of isoseismal curves) it's possible to prove the existence of different types of damping related to different degrees epicentral intensities. Particularly net and -systematic variations are observed among the attenuations of the highest and lowest seisms. These differences have to show different focal depths, and, probably, the respective mechanisms of energy liberation, matter on which there is still a lot to investigate. Nevertheless, different attenuation curves were used for cases where we could find clear differences between attenuations of most intense seisms and those with lower epicentral intensities, keeping the criteria mentioned above for the preferred directions of seismic attenuation.

"Anyway" -quoting the conclusions on the matter included in a previous work (Arenillas, M. & Bisbal, L., 1985)-, "all these adjustments and improvements that have an undoubted scientific interest, never lead to important variations in the final result of the quantification of the seismic risk, at least when it's in an analysis of territorial character, worked out with probabilistic criteria. The differences out of method are more notorious than those originated in isolated modifications during the complex procedure of calculation of attenuation coefficients."

After having made numerous verifications all along the development of the present study, we can underline the previous conclusions. However, we emphasize the need for using in any case attenuation curves deducted from occurred seisms, properly studied, avoiding the use of theoretical formalizations that don't match the actual attenuations, as they can introduce important distortions in the final results. In certain areas, historical data on maximal sensed intensities are used for contrast, that allow to adjust the corresponding attenuation curve in many cases, as we have verified. For those same reasons, some simplifications that have been done sometimes, like substituting isoseismal curves by circular tracts that encircle equivalent areas, are not advisable either, as in certain areas the consequent variations, in more or in less, in regard to minimal and maximal attenuations, can also cause important differences in the final results. And also all these simplifications of a pretended "mathematical" character, tend to ignore -or at least undervalue- the fundamental base of the seismic phenomenon that is all but the geological structure -and tectonic after all- of the territory where these seismic actions take place.

According to the utilization of these criteria, we've finally defined 71 seismic attenuations curves, that have been used to calculate the seismic dampening corresponding to the emitting focuses located in the different seismotectonic units that compartmentalize the area of study. In the units where it was not possible to set suitable attenuation curves, due to the lack of enough data on earthquakes happening in them, curves of some close unit with similar tectonic features have been used.
CALCULATIONS

The mathematical development of the procedure of adopted calculation and the basic antecedents can be seen in Samartín, A. & Martínez, J. "Un método de obtención del terremoto de diseño en un emplazamiento", 1985.

The criteria that have been used for the concrete utilization of this calculation procedure in the definition of the seismic risk in Spain are briefly exposed as follows:

- A network of "receiving points" had been defined on the Spanish territory distributed with higher density in areas of higher and more frequent seismicity. In these spots is where the relation intensity/return periods have been calculated. The total number of receiving points has been 476, from which it has been possible to interpolate with enough precision the isoseismal curves that define the general distribution of the seismic risk.

- The area of study has been initially divided in the 109 seismotectonic units indicated above. Due to its own definition, it's admitted that the seismic activity is systematic and uniform in each of these seismotectonic units, which compares to admit that the probability of a Fp epicenter of an occurred earthquake within the unit is located in a differential area from it, is independent from the situation of this area within the unit. In our case, these areas have diminished, being defined as limited elements between meridians and parallels distant 0.1 degrees in both cases. These areas are elements of about 100 km² of average surface, slightly variable from North to South, even if real surfaces have been used for the calculations.

- The seismic sample used in the calculations is constituted by the 3,734 earthquakes mentioned above, that have been defined by its epicentral coordinates, their intensity in that point and the date of occurrence, limited to a year as a higher precision does not lead to any improvement in the results.

- The utilized attenuation curves are the 71 mentioned above. According to the already made considerations regarding seismic dampening, all the attenuation curves, except those corresponding to units located on the S.W. of the Peninsula, cancelled themselves atmost 300 km away from the emitting epicenter. The curves corresponding to the units indicated in the peninsular S.W. have been extended to 1,200 km according to the available data on the seismicity's influence of these units.

- It's admitted that in all the seismotectoncic units, the apparition of seisms in temporarily distributed according to a Poisson model. This hypothesis is usually accepted and in any case the utilization of other models, as Markow's have not shown to lead to more trustworthy results up to now.

- The seismic sample has been statistically verified to check its matching with the Poisson model, which has allowed for a selection of a representative part of the sample. To accomplish that, we have done as follows. If the available sample was bigger than it is, (which applies to territories with higher seismicity than Spain), the contrast of Poisson hypothesis, could have been made in each seismotectonic unit. This not been possible in our case, due to the small size of the sample, the extreme option on the other hand would be to treat all the area of study as a whole (which can be used in regions of general low seismicity). We've verified that in the Spanish case this solution is not advisable either, as the parameters of the Poisson distribution obtained in this way define a "a-priori" seismicity "medium" type, excessively softened in its maximum and minimum. And even if these parameters are corrected later, this correction is far from being efficient due to the big influence of the starting situation, resulting in "a-posteriori" parameters that does not show properly the real seismicity of each of the seismotectonic unit. In the present study it has been verified that following this pattern the high and frequent seismicity areas remain under valued, while those of low seismicity are overvaluated. Accordingly, in order to correct this error and, at the same time, to work with the only available sample, the seismotectonic units have been clustered in a group of bigger regions, valuating two concepts that underlay in the Poisson model itself. On one hand, it's obvious that the own structural variety of the area of study determines notorious differences in the seismic "topology" of the big geological areas that constitute it. On the other hand, is also undoubtedly that the seismic information gathered in different regions of the area of study, can differ notoriously from one to another, for the criteria applied in the corresponding recopilations, as well as the variable interest arouses by the seismic fact. This two conditions have helped to group the seismotectonic units in 14 bigger regions. It's in them where the contrast of the Poisson model has been made, that has helped to define the part of the seismic sample that has to be considered in the mathematical calculation procedure.

- Once the valid seismic sample is selected the seismic rate of each one of the 14 above mentioned regions has been obtained from it. Then applying Gutenberg-Richter law, an adjustment of least squares was done, that has allowed for establishing the coefficients of the mentioned law in the 14 regions. The seismic rates deduced in this way are the ones applied as "a-priori" values in each of the seismotectonic areas belonging to a certain region. Applying the Bayes theorem, these values are modified then, based on the actual earthquakes happened in each of these units during the validity period of the sample.

- The relations intensity/return period in each of the 476 "receiving points" mentioned above were finally obtained, from these seismic rates and the intervention of the seismic attenuation.

These punctual results allowed for the definition, by interpolation, of different groups of isoseismal curves, that represent, with a good approximation level, the distribution of the seismic
risk in Spain. In the graphics that are found at the end of this work, the situations corresponding to return periods of 50, 100, 500 and 1,000 years have been represented, widely covering the range of times that have to be taken in consideration in structural projects. In any case, the relations intensity/return period referred to each of the "receiving points" enable us to deduce easily the corresponding values to other return periods and also, to get by interpolation within the acceptable limits, the values relative to points different from the ones considered.

It is evident that the seismic intensity is not a parameter that can be directly introduced in the structural calculations. Therefore, the values obtained through the procedure that we have indicated, require a posterior elaboration, that allows for the definition of other parameters of immediate utilization in the corresponding formulations. This is the procedure that is usually followed in different seismoresistant standards, in which the relations between the intensity and other more operative parameters are set. One of them is the seismic acceleration that has been often tried to be correlated with the intensity through different methods with quite different results up to now.

In our study we've used one of this relations, Murphy and O'Brien's (1.979), that relates both parameters through the following empiric formula: $\log a = 0.25 I + 0.25$.

Even if the validity of this formula for the spanish territory is not verified at all, its application could anyway be of interest in some cases, if only for the contrast with other numeric methods that relate intensity with acceleration. In the last graphic of this paper, the isoacceleration curves obtained by application of this formula have been represented corresponding to a return period of 500 years, which is the figure generally adopted to define the seismic intensities that have to be considered in the structural calculations of constructions of higher risk.

Finally, we can point out that the procedure we've using for the calculation of the seismic risk in Spain, in generalizable for any territorial area, whenever enough geological information is available and a reasonable representative seismic sample exists.

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Flood-hazard mapping in Spain and its incidence on land use management
La carte des risques d’inondation en Espagne et son incidence sur l’aménagement du territoire

C. Prieto Alcolea, Equipo de Asistencia Técnica, Madrid, Spain
F. Ayala Carcedo & J.J. Durán Valsero, Spanish Institute for Geology and Mining, Madrid, Spain

ABSTRACT: Several types of flood-hazard maps recently carried out in Spain are described: 1) Maps covering complete basins, usually at a 1/100,000 scale, where a flood vulnerability analysis is featured, as well as recommendations for land use management and 2) Maps of flood-hazard in urban areas (scales from 1/10,000 to 1/25,000) including floodways for several return periods.

The methodology used is presented in connection with two practical cases: the Nervión river basin, in the northern oceanic or wet climate and the area of Puente Genil in a dry mediterranean climate in the South.

RESUMEN: Se describen los distintos métodos de la cartografía de los riesgos de inundación recientemente desarrollados en España. 1) Mapas cubriendo áreas completas con escala 1/100,000 en donde se realiza el análisis de vulnerabilidad, así como recomendaciones para el manejo del uso del suelo y 2) Mapas de riesgo de inundaciones en áreas urbanas (escalas de 1/10,000 a 1/25,000), incluyendo el paso de crecidas con diferentes duraciones de retorno. La metodología de trabajo está aplicada a dos casos prácticos: El río Nervión, en el norte con clima oceánico y el área de Puente Genil, en el sur con clima mediterráneo seco.

1. INTRODUCTION

Although flooding is a very frequent hazard in many inhabited areas, only in recent times scientific treatment has been devoted to such a problem.

Customary measures to mitigate flood damage as defense or training works have little incidence on land use of hazardous areas and occupancy of floodplains. Instead of attempting to keep rivers away, the trend is towards preventing hazardous environments out of flood plains.

The present approach in developed countries is towards elaborating Flood-Hazard Maps as basic documents for:

1) Delineating areas bordering watercourses which would become inundated by floods of a certain magnitude or with a statistical chance of occurrence.
2) Anticipating the changes to be excepted in the flood plains under floodwaters.
3) Establishing safe development plans consistent with good flood plain management and
4) Suggesting corrective measures or regulations in order to minimize potential losses to lives and properties. These measures should not affect other hazardous areas.

In Spain flood conditions show wide variations from a basin to another as concerns discharges, flow velocities, return periods, etc. Rainfall in northern areas ranges from 1200 to 1800 mm/year, whereas in the South rarely exceeds 500 mm/year. In the first case, there are more than 160 rainy days a year as compared with only 15–20 days in the Southern regions; very frequently half of annual rains fall in 24 hours. Other differentiating features are of geomorphological type:
narrow young valleys without coastal plains in the North, Coastal plains formed - by connected alluvial fans in the Mediterranean Coast or wide river flood plains in the Guadalquivir (South-West) or Ebro (North-East) basins.

These features imply the use of several methodological approaches in the preparation of hazard maps. For instance, in flood areas with thick deposits, sediment analysis and archaeological research have been used to define active areas under flood action; erosion patterns are useful in evaluating areas with thin deposits of limited extension. Historical and hydrological data are always necessary for checking the geomorphological conclusions.

2. FLOOD HAZARD MAPS

In connection with a recent program of the Spanish Institute for Geology and Mining two types of flood-hazard maps have been prepared:

- Map of flood hazard factors (main basins)
- Maps of flood hazard in urban areas.

2.1 Maps of flood hazard factors

These six-coloured maps are published at a 1:100.000 scale. An example is given in fig. 1. The map includes an overall land classification with respect to flood hazard as: highly endangered areas, occasional hazardous areas and safe areas. As concerns urban communities an evaluation of risk is also given as high, mean, moderate, and low.

Areas where slides or ground movements have occurred as consequence of floods are also included.

Each watercourse is divided into sections according to their depositional or erosive condition under flood waters. Specific signs mark locations of obstructions to the free flow of water as bridges, small dams, etc.

This map is combined with two companion maps:

- Map of hydrological factors and inundable areas
- Map of runoff and drainage.

The first one (fig. 2) defines a) the widths of flood plains and b) the values of natural slopes. Due to the small scale a symbolic representation has been chosen for the widths. The degree of occupancy of flood plains by dwellings or industrial buildings is also stated according to the following percentages: < 20%; 20 to 60% and > 60%. As refers to the slope grades they are classified into high (over 40%), mean (20 to 40%) and low (< 20%).

In addition to the main watercourses the map includes permanent and temporary streams. When information is available the discharges corresponding to the 50 years flood are represented by arrows to scale. Historical damages by flooding in urban areas are figured by signs representing the percentage of surface inundated (< 25%; 25-50%; 50-75% and > 75%).

The Map of Runoff and Drainage contains information on permeability of surface ground formations. Open granular soils, fissured or karst rock are mapped as pervious ground. Moderate drainage conditions prevail in cases of thick permeable layers on impervious subgrades or bedrock, or when free draining layers alternate with non-permeable ones, as in some sedimentary formations. The third group includes impervious soils or rocks as well as areas with emerging water table (marshes, wetlands, etc.).

In this map each watershed is also classified according to its drainage conditions into three categories, from well developed drainage networks to areas with ponded water.

The unit basins dealt with in these maps rarely exceed 2000 km², as basins of greater surface show marked differences in climatic, hydrological and morphological conditions.
2.2 Detailed maps (urban areas)

These maps are the application of flood-hazard mapping to urban areas, at scales of about 1:5000. (fig. 3)

The mapping techniques differ according to the origin of the inundations: overflow of well developed watercourses or temporary streams into adjacent areas; braiding of creeks discharging out the plains and forming alluvial fans; temporary ponding in poorly drained areas; water level rise in marshlands by storm surges, etc.

In normal cases the mapping is limited to flood prone areas, zoning the following domains: a) areas endangered by 1.5 year floods (red) b) inodurable areas by 5 year floods (orange) c) as above by 25 year floods (yellow) and d) flood liable land. These domains broadly correspond to those stated in the "Principles of floodplain management" (United Nations, 1984): Floodway - Floodfringe - Regulatory flood limit - Standard Project flood limit.

Within each domain information is given as refers to facilities whose location in these areas should be avoided.

The main difficulty is the assessment of the frequency of flooding and to delineate the limits of the hazardous areas. Continuous or dotted lines are used according to the permanent or temporary character of the streams.

This detailed mapping also includes the surroundings of the urban area at scales between 1/10,000 and 1/20,000, reflecting other features as poorly drained areas, springs, marshes or wetlands, any permanent or ephemeral stream, obstructions to water flow, training works, dykes and levees, channels, irrigation facilities, etc. Stability problems related to exceptional water levels as slides, erosion, undflows sediment migration, etc. are also mapped. (fig. 4)

3. WORKING METHOD

As a first step available data related to geology, hydrogeology, drainage and movements of terrain are mapped at the working scale. On this basis, an air photo evaluation of soil covers, river channel shifting, abandoned meanders and any interesting features are superimposed.

Significant streams are classified according to their morphometry, following Strakler and Horton laws. This procedure simplifies further field surveys.

Flood-routing studies are carried out for the 50 and 100 year floods, attempting to justify hazardous conditions as reported in existing records or historical data. This calibration of flood levels is associated with a considerable degree of uncertainty at locations subjected to intensive encroachment of floodways, thus requiring to model the original morphology.

Field surveys include the establishment of characteristic cross sections, the location of flood traces, the observation of stability problems and the identification of river deposits. Man-made obstruction are also recorded.

A similar procedure is followed for the detailed maps but logically emphasizing floodline analysis in the main watercourses, storm outlets, etc. The effects of floods of different return periods are extensively investigated in connection with hydrological data and site inquiries.
GENERAL FLOOD-HAZARD MAP

LEGEND:
- WATERSHED LIMIT

FLOOD-HAZARD LEVEL IN URBAN OR INDUSTRIAL AREAS:
- NON-HAZARD ZONE
- MODERATE RISK
- MEAN TO HIGH RISK
- MAXIMUM RISK

DISPERSED RISK:
- HIGH-HAZARD AREAS FOR HOUSING AND PUBLIC UTILITIES
- OCCASIONALLY UNSTABLE GROUND
- STABLE GROUND

SEDIMENT TRANSPORT
- EROSION
- DEPOSITION

- SMALL
- MEAN
- HIGH
- VERY HIGH

OBSURCTIONS
- SMALL
- MEAN SIZED
- IMPORTANT

PUBLIC UTILITIES
- HIGH-HAZARD SECTION BY FLOODING OR GROUND MOVEMENTS
- LOCALIZED LANDSLIDES

FIG. 1

MAP OF HYDROLOGICAL FACTORS AND FLOOD PRONE AREAS

LEGEND:
- FLOOD PLAINS WIDTHS EQUIVALENCES
- AXIS OF PERMANENT STREAM
- AXIS OF TEMPORARY STREAM
- WATERSHED LIMIT

URBAN FLOOD RISK
- SAFE AREAS
- AFFECTING <25% OF THE URBAN AREAS
- " " OF 25% TO 50%
- " " 50% " 75%
- " " 75% " 100%

FLOWWAY ENCROACHMENT
- < 20%
- > 20%
- > 60%

MORPHOLOGY
- HIGH-SLOPED GROUND
- MEAN-SLOPED GROUND
- LOW-SLOPED GROUND

433 MAXIMUM 50-YEAR FLOOD DISCHARGE

FIG. 2
4. **EXAMPLES**

4.1 The Nervion river basin

The Nervion river flows towards the northern Spanish coast and its basin occupies 1692 square kilometers. The main watercourse has 72,5 km in length and the mean slope is 0,116%. Mean annual rainfall is 1100 mm, with 155 rainy days a year.

The morphometric features are as follows:

- Deficit of lowest-order streams with respect to the bifurcation ratio of the basin.
- Short relative length of lower-order streams as well as the main channel.
- Schumm's sinuosity factor = 1,41; average sinuosity (1,53 lower river section, 1,17 upper section).
- Single channel.
- Drainage density: 1,63 km/km² - low
- Shape index 1,78 - very straight basin, low circulacit.
- Channel gradient very variable. Some chutes.
- Fournier's erodibility index 144 t/km²/year; it seems high for oceanic climates.

According to these data it can be expected a rapid concentration of discharges. This leads to the building up of hydraulic barriers at the confluences of the main streams. Bed load is important along the year with very high sand transport capacity even with mean discharges. At high waters cobble and gravel transport is significant.

The analysis of sediments confirmed the above expectations. Outside the final section, bed deposits only include gravel and cobbles with some sand lenses. This applies to the side bars as well as to the point bars; overbank deposits are also of coarse materials, and mean-sized sands are found at more than 100 m from the river channel, as a result of the high-energy flow. Alluvial fans at the exit of tributary streams are very scarce.

Hydraulic data have been checked against mean slopes, storm rainfall and hourly intensity, peak discharges and time of concentration for 8 points in the basin. The watersheds of 5 tributaries have been also studied, amounting 36 control points for three return periods. Peak values established at the river mouth are:

<table>
<thead>
<tr>
<th>Return period</th>
<th>50 years</th>
<th>100 years</th>
<th>500 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak total rainfall (1/m²)</td>
<td>115,8</td>
<td>132,7</td>
<td>154,0</td>
</tr>
<tr>
<td>Peak storm intensity (1/m²/h)</td>
<td>4,2</td>
<td>5,0</td>
<td>5,9</td>
</tr>
<tr>
<td>Discharge (m³/sec)</td>
<td>1444</td>
<td>1726</td>
<td>2162</td>
</tr>
</tbody>
</table>

Time of concentration (hours) River month: 19,8
Entry to the estuary: 12,1

The available hydrographs show short times of concentration, with small lamination of floods and high velocities of flow. These high velocities make uncertain any prediction of flooding in the lower river sections from the observed increase of discharges at the upper section.

Historical evidence shows that the Nervion river has suffered important floods at least 17 times in the last 500 years, with rainfall exceeding 130 mm.

Any evaluation of the damage potential of existing properties is subjected to the uncertainty derived from new developments at unsuitable locations or the floodplain. More often than not the worse damages are due to the high velocity currents of water in narrow passages and the amount of debris carried by flood waters. This probably explains the increasing number of catastrophic flood in the last 30 years (especially that one of August 1983 with losses evaluated in $ 1250 millions).
FIG. 4. DETAILED MAP SURROUNDINGS OF PUENTE GENIL URBAN AREA.
4.2 The urban area of Puente Genil

The town of Puente Genil is located near the Genil river, the main tributary of the Guadalquivir river, the most important river of Southern Spain.

The town occupies part of the floodplain and a upper level ground, at 70 m over the watercourse. The name of the town refers to a favourable river crossing, through a bridge with approach embankments raised on the floodplain.

The study carried out for preparing flood-hazard maps included the following steps:

- Evaluation of flood elevations with several return periods in four checking sections. The original conditions have been changed by new reservoirs upstream of the town, but their effect was not taken into account for explaining ancient morphological features.
- Air photo zoning of several alluvial areas showing active forms (visible deposits, channel paths in vegetated land, etc.).
- Field checking of the tentative zoning above, relating the observations with historic records, mainly the 25-30 year floods (1912, 1943, 1968). The outermost limit seems delineated by 100 year floods. The lower terrace is inundated by 1.5 to 20 year floods but the gentle slope makes difficult to establish the floodway. The limits for high return periods are very close (3 to 5 m) due to the river banks.

The afflux created by the upstream bridges during 700 years has also created distinct morphological features.

5. CONCLUSIONS

The methodology used in Spain for preparing flood hazard maps is described, as well as its application to two cases: a general map at a 1:100,000 scale and a detail map at a 1:5,000 scale.

These maps include the areas which can become inundated by floods of different frequency, the points of potential damage to slopes, civil works, roads, etc. Information is also given as refers to protective measures or limitations to siting of dwellings or industrial facilities, etc.

A minimum quality standard requires climatic, historic and hydrological data for at least 50 years.

These maps have proved very useful for city and urban planners and they are also intended for establishing insurance rates, the need of engineering works, etc.
Erratum

Erratum

Paper 4.2.6: Stability analysis of scarpments on the left margin of the reservoir in the Paraná Medio Project - Chapetón closure dam
N.P.Morbidoni & D.Larangeira
p. 1293

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NOTE: Geological and Geomorphological Aspects, Massive-Wasting Phenomena and Some Geotechnical characteristics have been taken by the authors from:


Paper 6.1.2: Pedological maps interpretation for soil planning in the west zone of Rio de Janeiro
J.A.Barroso
pp X and 1725

Names of co-authors have been omitted. The names of the authors should read:
Josué Alves Barroso
Franklin dos Santos Antunes
Angelo Eurico Silva Pedroto