VOLUME III

THEME 1

Auxiliary papers
Des articles auxiliaires
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The Organising Committee of the 4th International Congress of Engineering Geology feels particularly happy to be able to release in time the nine volumes of the Proceedings of the Congress, running to about 3000 pages. The first eight volumes encompass over 270 scientific and technical papers, one-third of which are from India and two-thirds from 32 other countries of the world. The ninth volume contains the Panelists' Reports, the Sessional Reports and the Special lectures planned to be delivered during the Congress. The tenth volume, containing the record of the Technical Sessions and the discussions to be held during the Congress, is proposed to be released at a later date.

The seven Themes as chosen for discussion during the 4th Congress cover a wide range of topics, all of which are of special interest to the present-day developmental activities in the world.

The listed Themes are:
2. Engineering Geological Problems of Tunneling and Excavation of Cavities;
3. Soil and Rock as Construction Material;
4. Engineering Geological Problems of Natural and Man-made Lakes;
5. Engineering Geological Problems of Sea-coast and Shelf Areas;
6. Seismic and Seismo-tectonic Investigations of Engineering projects; and

The themes as listed above have been further sub-classified into 31 sub-themes, so as to bring into their fold several related problems and to focus attention on specific aspects.

Although the Organising Committee feels very much gratified by the overwhelming response to the call for scientific and technical papers for discussion during the 4th Congress, and nearly 300 papers were received from different countries of the world, the major and somewhat perplexing task faced by the Committee was to classify the papers received and to pigeon-hole them into the various Themes and sub-themes. In fulfilling this onerous task, the Committee has done its best in good faith and has offered a classification in the eight volumes of the Proceedings which may be taken as one of the best-fits for the Themes and the sub-themes and not as the only best possible fit. Again, by stretching the ambit of some of the themes, a number of papers, which were of great topical interest and were considered to be of an auxiliary nature as well as related to the Themes posed for discussion, were accommodated, as is particularly the case with Theme 1. In some of the papers, minimal language corrections were made, taking care, however, to retain the style of the authors' presentations to the maximum possible extent.

A very large number of papers were received for Theme 1 and these have been distributed in the first three volumes: the first volume containing all the papers falling "sensu stricto" under Theme 1 while Volumes II and III containing all the papers of an auxiliary or supporting nature and which are, in one way or the other, related to Theme 1. Theme 2 had also more than 50 contributions; therefore, these have been accommodated in two volumes, Volumes IV and V. While the papers for Theme 3 have been included in Volume VI, the papers for Themes 4 and 5, put together, have been incorporated in Volume VII and those of Themes 6 and 7, put together, in Volume VIII. The Organising Committee thanks all the contributors for their generous cooperation. The Committee is grateful to the various National Groups for the trouble they had taken to collect the papers from their respective countries and to forward these to the Organising Committee for its consideration. The Geological Survey of India has been the backbone of all activities relating to the Congress and the work of publication of the Proceedings of the Congress has not been an exception in its contribution. The Organising Committee is also under a deep debt of gratitude to the Director General, Geological Survey of India and to the Officers working under him in different geoscientific disciplines, for their generous and unfailing support.

The main burden of the review of all the papers and the editing of some of them was shared by M/s.

New Delhi
12 September 1982

The Organising Committee is obliged to all these geoscientists for their unremitting help and support. M/s. Oxford & IBH Publishing Co., New Delhi, did a commendable job in bringing out all the volumes of the Proceedings of the Congress in very good shape and in good time, despite several odds faced by them. They deserve the sincere thanks of the Organising Committee.

V.S. Krishnaswamy
Chairman, Organising Committee
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APPLICATION OF SEDIMENTOLOGICAL GEOCHEMICAL AND COMPUTER TECHNIQUES IN THE ENGINEERING GEOLOGICAL INVESTIGATION OF DAMSITES, LOWER GORDON AREA, TASMANIA, AUSTRALIA

APPLICATION DES TECHNIQUES SEDIMENTOLOGIQUES, GEOCHIMIQUES ET CELLE DE MACHINE A CALCULER AUX INVESTIGATIONS GEOLOGIQUES DE L'ART DE L'INGENIEUR DES EMPLACEMENTS DE BARRAGE, A REGION GORDON PLUS BAS, TASMANIE, L'AUSTRALE

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ABSTRACT

The application of sedimentological, geochemical and computer techniques assists in recognition of variation of rock types, stratigraphic correlation, folding, faulting and porosity.

Fifty-two limestone facies, ten dolomite facies and ten terrigenous facies show wide variation in a short distance of 8 km along the lower reaches of the Gordon River in Tasmania. In a previous study, factor analysis resulted in the identification of a depositional model consisting of subtidal, shoal, bar, intertidal and supratidal carbonate environments together with channels, and a dune and flat terrigenous environment. Discriminant analysis has now been employed to extend the paleo-environmental model laterally. One hundred and forty-two samples from five new drill holes were examined for 101 variables and the data compared with the earlier data by this technique. The results confirm a prograding tidal flat complex, anticlinal structure, faulting and two major carbonate units in the sequence.

The variation of elements is related to depositional and diagenetic environments and to the influence of fresh water on the marine sequence. These variations are particularly useful in distinguishing early and late diagenesis, and several stages of secondary dolomitization, which in the sequence replace both limestones and sandstones. The distinction between stages of dolomitization aids in interpreting faulting in the sequence.

As the porosity is facies-controlled large caverns are unlikely to occur. Present day flows of water through the sequence occur predominantly through the faults and the anticlinal structure.

Because this computer-based technique intensively uses information from each length of drill core, the possibility exists of more confident interpretation of new data from less extensive drilling, with consequent saving in cost.

ABSTRACT

L'application de techniques sédimentologique, géochimique et d'ordinateur assiste a la reconnaissance de variation de types de roches, correlation stratigraphique, plissement, formation de failles, porosité et regime hydrologique.

Cinquante deux faciès calcaires, dix faciès dolomites et dix faciès terrigene démontrent des grandes variations sur une petite distance de 8 km le long de la Riviere Gordon en Tasmanie. Dans une etude antérieure, une analyse de facteur a résultée a l'identification d'un modele de déposition consistant de plusieurs milieux (calcaires, terrigene). Une analyse discriminate est maintenant employée pour étendre lateralement le modele. Cent quarante deux échantillons de cinq nouveaux forages ont été examinés pour 101 variables et les données ont été comparées par cette technique avec les données

III.1
antérieures. Les résultats confirment un milieu estran progressant, structure anticlinal et formation de failles dans la sequence.

La variation des éléments est associée aux milieux depositionel et diagénétique et à l'influence de l'eau fraîche sur la sequence marine. Ces abondances sont particulièrement utiles pour distinguer le phase de diagénèse, et plusieurs stades de dolomitisation secondaire, qui dans la sequence remplace des calcaires et grès. La distinction entre les stades de dolimitisation aide à l'interprétation de formation de failles dans la sequence.

Comme la porosité est contrôlée par faciès, l'existence de grandes caves est improbable.

L'écoulement d'eau à travers la sequence est actuellement lié à la structure.

Du fait que cette technique emploie intensivement l'information de chaque longueur de carottage, la possibilité existe pour l'interprétation plus sûr de données de forage moins extensive, résultant à une reduction de frais.

INTRODUCTION

The Hydro-Electric Commission has investigated damsites along the Lower Gordon River from Butler Island up to the Gordon-Franklin junction in Tasmania (Fig. 1). Generally we knew we had a north-south trending anticline with Gordon Limestone of Ordovician age on each limb and the Butler Island Formation below, containing limestone horizons. There was a consequent possibility of gross leakage if a limestone horizon was continuous, permeable or cavernous, and outcropping along the river. Engineering geology investigations, largely based on megascopic description of outcrop and subsurface samples, have not been fruitful in correlation of subsurface strata both regionally between damsites and in some cases between adjacent drill holes. This affects the interpretation of geological structure and thus reduces confidence in evaluation of the potential for gross leakage from the proposed storage.

We looked at Damsite 2B in the area covering drill holes 3, 4, 5 and 6 (Fig. 1). We found it difficult to Correlate the sequence using megascopic properties. Therefore, we were unable to understand the distribution of rock types. So we did a sedimentological, geochemical and factor analysis study to try to understand the palaeoenvironments and the correlation so we could predict the facies changes along the river (Rao & Naqvi, 1977).

We succeeded in delineating palaeoenvironments and correlation of strata. For engineering reasons the Damsite 2B was abandoned, but the effort wasn't wasted because we kept running into just the same difficulties in macroscopic correlation of core at other possible damsites. We could use our ideas about the sedimentary environments and the data already in the computer, to extend regionally and untangle the new problems rather cheaply.

Fig. 1. Area of study and location of drill holes. Surface geology modified after Roberts (1977).

Without going into technical details we briefly restate the results at Damsite 2B. We had a classical prograding subtidal-beach-dune sequence and this had been folded into an anticline. There was no faulting at this damsite. More importantly as we will see, we had established a depositional model that we could use to predict rock types at new sites.

This paper will briefly show how we did our study at Damsite 2B and how this study gave us the palaeoenvironment model. Later on we will explain the extension regionally, and of the way it confirmed our sedimentological model and how we could unravel the structure. Finally we will show how these sophisticated techniques really gave us confidence that the site would be suitable for dam construction.

At Damsite 2B we started with 101 variables including petrography, porosity and chemistry. These variables were analysed by using factor analysis techniques. Significant variables were used in the discriminant analysis which combines the variables in an optimum way to extrapolate the facies. This confirmed our
previous environmental model and produced a more extended prograding tidal-depositional model (Rao & Naqvi, 1981). This study has been able to distinguish two limestones that in hand specimen seem correlatable as one horizon between closely spaced drill holes. You note that we have used the information from each length of drill core intensively, thus saving cost in extensive drilling in the area.

REGIONAL SETTING & STRATIGRAPHY

In the area under investigation the Gordod River (Fig. 1) cuts through structurally complex Lower Ordovician rocks which have been folded and faulted into a broad N-S trending antcline with gently dipping limbs. Broad, flat, synclinal valleys occur to the east and west of this area. The Gordon River flows generally in a westerly direction (towards Butler Island) from the Franklin River junction through a 300 m deep sandstone valley with thick scrub and medium-dense timber.

The stratigraphic sequence (Fig. 2) consists of, in ascending order, sandstone (> 20 m thick), overlain by lower limestone and dolomite (85 - 100 m), middle sandstones with mudstones and conglomerates (190 m), upper limestone (60 - 70 m) and the upper sandstone (220 m). The contacts between these units are gradational.

This sequence of Early Ordovician age was defined as Butler Island Formation (Roberts & Naqvi 1979, Rao & Naqvi, 1981). The drill holes in the present study include two major carbonate units (Fig. 2). The recognition of these units is vital for the interpretation of the structure and the evaluation of gross leakage problems. Both carbonates and terrigenous sediments show marked lateral facies variation in the studied sequence. This variation, together with folding and faulting in the area (Fig. 1) makes correlation by conventional megascopic methods extremely difficult, thereby reducing the confidence in the interpretation of the structure and consequent possibility of gross leakage. For these reasons detailed petrographic, geochemical and computer methods have been used.

METHODS

Two hundred and sixty-two samples from ten drill holes were collected at significant lithological changes. Sedimentary structures were noted. Thin sections were stained (Katz and Friedman, 1965) for identification of carbonate minerals. Codes used to describe sedimentary features and the drill cores are shown in Fig. 3.

Eighty-eight petrographic variables (Fig. 4) quantified were: depositional textures (17), diagenetic fabrics (18), terrigenous grains (13), allochons (8), intraclast composition (3), fossil types and their relative abundance (11), algal mat thickness and type (2), structure (9), color code (1) and porosity (6). Full details of the quantified variables have been reported elsewhere (Rao and Naqvi, 1977).

One hundred and ninety-seven samples were dissolved in hydrochloric acid and these solutions were analysed by atomic absorption spectrophotometry (AAS) for Ca, Mg, Sr, Na, Mn and Fe. Sixty-five terrigenous clastics were analysed by x-ray fluorescence (XRF) for SiO2, Al2O3, CaO, Fe2O3 and K2O. X-ray diffraction (XRD) analyses were made on selected samples for carbonate minerals.

R-mode and Q-mode factor analyses (Parks, 1970) were initially performed to derive sample relationships in a dendogram on samples from drill holes 3, 4, 5 and 6 (Rao and Naqvi, 1977). Samples from these four drill holes were also analyzed by Q-mode factor analysis (Klovan and Imbrie, 1971) to study the affinity between successive samples and to recover environmental gradations. This resulted in the identification of subtidal, shoal, bar, intertidal and supratidal carbonate.

Fig. 2. Stratigraphy: drill hole numbers as in Figure 3.
### Microfacies Legend

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<th>Major Microfacies</th>
<th>Microfacies</th>
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<td>XI</td>
<td>51. MICRITE (&lt;101 allochems)</td>
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**Purity**

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**Dolomitic Facies**

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<td>XVI</td>
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**Silty and Sandy**

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**Fig. 3.** Legend for microfacies and major microfacies. Note code numbers of drill holes.

Environments, and channels together with a dune and flat terrigenous complex (Rao and Naqvi, 1977). Discriminant analysis (Demitirmen, 1969) was used to allocate samples from drill holes 1, 2, 7, 8, 9 and 10 to the previously established environmental groups (from factor analysis) of drill holes 3, 4, 5 and 6.

**RESULTS**

**Sedimentology**

Wide variation of rock types, microfacies (Figs. 3 and 5) and extensive dolomitization is a feature of the sequence studied. Coarse secondary dolomites (Fig. 5) arrows are confined to drill holes 2, 6, 8 and 10. These coarse secondary dolomites have replaced both limestones and terrigenous sediments; they have been found only in drill holes showing brecciated and fractured zones. The microfacies variation indicates a tidal-flat carbonate and terrigenous sequence. As porosity is facies controlled (Rao & Naqvi, 1977, 1981) large caverns are unlikely to occur.
Geochemistry

Detailed geochemical features of the sequence are presented elsewhere (Rao & Naqvi, 1977, 1981). Briefly, the major features are:

1) Sr and Na define the carbonate facies and salinity variation.
2) Dolomitizing brines were saline and contained three times more Mn than the depositional facies.
3) The high concentration of Mn in dolomites indicates a mixing of continental brines higher in Mn and Fe and lower in Sr, with the marine-origin brine.

Factor Analysis

The factor analysis (Rao & Naqvi, 1977) grouped petrographic and geochemical variables from samples of Damsite 2B. This resulted in four carbonate and seven terrigenous clastic clusters related to major depositional environments, stages of dolomitization, silica-iron cementation and formation of evaporite and mica. The recognition of the distinction between stages of dolomitization is important in the structural interpretation of the sequence. Certain secondary dolomites are associated with faulting. These results also showed chemical variation of solutions during diagenesis including Ca, Mg, Sr, Na, Mn and Fe abundances. Porosity was less well defined than others, the amount, type and stages of development of porosity and some petrographic and chemical variables were included. The porosity is facies controlled and fabric selective.

The vertical facies variation, Q-mode factor groupings along with salient characteristics, porosity and core recovery of four investigated drill holes (3, 4, 5 and 6) at Damsite 2B are shown in Fig. 6. Interpreted environment variations from Q-mode factor analysis are also shown. From east to west a distinct lateral and vertical variation of the environment can be seen. In the bottom of the sequence marine transgression (biofacies) took place with open marine conditions in the eastern part and restricted marine intertidal (agal) in the west. This was followed by bar facies (oncolite) and tidal channel facies (oolite) throughout the Damsite 2B area. Major marine regress-
Fig. 5. Regional facies variation. Facies symbols and drill holes as in Figure 3. Note presence of secondary dolomites (arrows) confined to drill holes 2, 6, 8 and 10.

ion resulted in progradation of supratidal and intertidal terrigenous clastic facies in the west while subtidal conditions still persisted in the east. Subtidal, intertidal and supratidal channels extended throughout the area with the deposition of non-bioturbated clay pellet-bearing coarse arenites and rudites and well sorted arenites. Thin or wispy mudstone layers overlie these units. Finally aeolian conditions prevailed with very high angle cross-bedded fine arenites with abundant heavy minerals and occasional broken fossils. These units were subsequently folded into a broad north-south trending anticline.

For engineering reasons the Damsite 2B was abandoned, and further damsites along the Gordon River were investigated by using the paleoenvironmental model developed at Damsite 2B.

**Discriminant Analysis**

Discriminant analysis computes group characteristics and establishes discriminant parameters for the groupings. It then attempts by iteration to relocate individual samples in the data set so as to reduce the variance within groups and to increase the variance between groups. Each iteration will continue to improve the groupings and associated classification until no more items can be relocated to improve the existing groupings. After three iterations the core items (those items which have been retained in their
Fig. 6. Environmental variation at Dam site 2B. Drill hole distances not to scale.

Initial groups) have not been reshuffled appreciably indicating that no further improvement in the groups is possible. Most of the samples from drill holes 1, 2, 7, 8, 9 and 10 were grouped into Dam site 2B environmental groups. Brief descriptions of these groups (Subsets) and their environment of deposition are as given elsewhere (Rao and Naqvi, 1981).

Eleven subsets were delineated by the discriminate analysis (Rao and Naqvi, 1981). These subsets are related to subtidal (Subset 1), bar (Subset 2), shoal-intertidal (Subset 3), supratidal (Subset 4), dunes (Subset 5), sandy flat (Subset 6), mud flat (Subset 7), subtidal channel (Subset 8), intertidal channel (Subset 9), supratidal channel (Subset 10), and secondary dolomite (Subset 11). The depositional environments of the lower carbonate unit A as delineated by the discriminant analysis of samples from drill holes on either side of Dam site 2B are shown in Fig. 7.

In summary, the present work confirms and extends our earlier model (Rao & Naqvi, 1977) that a major basal transgression from E to W was followed by a regression at the top of the sequence. The regional depositional environment was a prograding tidal complex (Fig. 7).

The interpretation of these complex field relationships began by correlating strata formed contemporaneously. Four faults had to be postulated to bring the observed dips into concordance with the correlation, and we have drawn these (Fig. 7 bottom), close to the holes showing secondary dolomitization, brecciation and stratal discontinuities. The resulting smooth anticline (Fig. 7, middle) was then unfolded to reveal a continuous prograding tidal flat complex. The simplicity of the final result contrasts with the complexity of the field picture, confirming earlier ideas of a major anticlinal structure and establishing major faulting as well.
Fig. 7. Progradation (Section 1), folding (Section 2) and faulting (Section 3) in the Butler Island Formation below carbonate B unit. Note present topography and horizontal to vertical exaggeration = 1:4.
Discriminant analysis also showed that the carbonate from drill holes 9 and 10 was different from the others, as illustrated by contrasting depositional facies (Fig. 8). The lower carbonate Unit A is characterised by an algal facies, whereas the upper carbonate unit is highly fossiliferous, oolitic and oncoidal.

The regional distribution of these carbonate units are shown in Fig. 9. Following the deposition of the lower carbonate unit A a regression resulted in terrigenous clastic facies which were succeeded by a transgression resulting in the deposition of upper carbonate unit B followed finally by a regression resulting in the deposition of terrigenous clastics. The whole sequence was subsequently folded and faulted.

CONCLUSIONS

The application of sedimentological, geochemical and computer techniques were found to be useful in understanding the variation of rock types, stratal correlation, folding and faulting. This has assisted in assessing the likelihood of gross leakage. Specific conclusions from this study include the following:

1. Wide variation of rock types is due to the paleoenvironment of open marine

Fig. 8. Environmental comparison of A & B carbonate units within Butler Island Formation.

Fig. 9. Regional distribution of A & B carbonate units within Butler Island Formation. Note present topography and horizontal to vertical exaggeration = 1:4.
conditions to the east and land towards the west.

2. Two major carbonate units occur within the Butler Island Formation.

3. The continuity of depositional environments (both lateral and vertical) assist in recognizing the folding and the faulting in the sequence. As there are no marker beds in the sequence, facies variation provides a simple means of correlation.

4. The observed variation of the thickness of carbonates in the sequence is largely due to dolomitization as both sandstones and limestones are dolomitized.

5. Geochemical variation aids the recognition of environmental facies variation, and early and late diagenetic dolomites.

6. As porosity is facies controlled, large caverns are unlikely to occur.

7. Computer coded data assist in interpreting a complex sequence. Because this computer based technique intensively uses information from each length of drill core, the possibility exists of more confident interpretation of new data from less extensive drilling, with a consequent potential saving in cost.

ACKNOWLEDGEMENTS

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REFERENCES


SITE INVESTIGATIONS FOR THE BEOK DAM, JOHOR, MALAYSIA

INVESTIGATIONS D'EMPLACEMENT POUR LE BARRAGE BEOK, JOHOR, LA MALASIE

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ABSTRACT
The Bekok Dam is one of the several earth dams proposed for flood mitigation in southern Peninsular Malaysia. Detailed engineering geologic and soil investigations have been carried out at the proposed dam site and related areas. Among other soil investigation works carried out were in situ sounding tests using the Swedish Weight Sounding, the Mackintosh Probe and the Standard Penetration Tests. These tests were performed at the main dam sites, along the diversion channel, at the borrow area, etc. to supplement boring works in determining and correlating the underlying soil strata. This paper presents details of the site investigations carried out at the Bekok Dam project area, with discussions on the results of the various in situ soil tests mentioned which were found to be very useful particularly in differentiating the alluvial from the residual soils.

INTRODUCTION
As part of the flood mitigation scheme for the southern regions of peninsular Malaysia several earth dams have been proposed for construction in the near future. Preliminary and detailed site investigations have been carried out for the various dam sites and their associated structures to determine the nature and distributions of the foundation soils, the availability and suitability of construction materials, etc.

Various site investigation techniques have been utilised at the various dam sites. In the case of the Bekok Dam (Figure 1), the methods include the Swedish Weight Sounding, the Mackintosh Probe and the Standard Penetration Test. This paper presents the results of some of the site investigation works carried out at the Bekok
Fig. 1 Location map of the Bekok Dam.

The project site, i.e., covering the main dam, saddle dam, spillway, borrow area and the diversion channel, is underlain by sedimentary rocks consisting predominantly of siltstone and shale, with minor occurrence of sandstone. General strike directions of the sedimentary rocks range from NW to NW, with dips varying between 30° to 60° towards the west. Metamorphic equivalents of the sedimentary rocks, namely slate, schist and quartzite have also been recovered from boreholes in the project area. Minor occurrences of volcanic rocks (andesite and rhyolite) are also found further south of the damsite area.

The bedrocks are covered by residual soils of varying thicknesses in the higher grounds such as the hills and hill slopes, and by alluvial deposits in the low lying valleys and swampy areas. The depth to bedrock ranges from a few metres up to about 15 metres in the valleys.

Table 1 summarises the stratigraphy of the Bekok Dam project area.

<table>
<thead>
<tr>
<th>Age</th>
<th>Lithologic Unit</th>
<th>Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quaternary</td>
<td>Alluvium</td>
<td>Unconsolidated clays, silts, sands &amp; gravels</td>
</tr>
<tr>
<td></td>
<td>--- unconformity ---</td>
<td></td>
</tr>
<tr>
<td>Mesozoic</td>
<td>Volcanics</td>
<td>Massive, fine grained andesite with associated tuff; rhyolite further south</td>
</tr>
<tr>
<td></td>
<td>Sedimentary</td>
<td>Predominantly siltrocks &amp; shale, with minor sandstone. Also slate, schist, equivalents phyllite &amp; quartzite, highly weathered</td>
</tr>
</tbody>
</table>

Of direct relevance to the site investigation works are the Quaternary alluvium and the other surficial soil materials. The differentiation between alluvial and residual soil deposits, for example, is of practical engineering significance in view of their contrast in properties and behaviour.

3 SWEDISH WEIGHT SOUNCING

The Swedish weight sounding method was used at the main damsite as well as at the saddle dams to help better define the soil strata at these places. Altogether, 33 soundings were made in this project. At the Main Dam (Figure 2), three boreholes spaced at 200 metres apart were sunk along the Main Dam axis, and three Swedish soundings were also carried out, two in between the boreholes and one towards the left abutment. In addition, three soundings were done slightly upstream and another three downstream from the boreholes to determine the variations of the soil layers upstream and downstream from the dam axis.

Results of the Swedish soundings are shown in Figures 3 and 4, from which it was possible to interpret and define the soil profiles at the Main Dam site, both along the dam axis (as shown in Figure 5) and also perpendicular to the dam axis (part of which is shown in Figure 6).

In the interpretation of the Swedish sounding results, observations made with regards to the sound produced when penetrating sand layers were very useful.
Fig. 2 Main Dam test locations showing locations of boreholes (BH) and Swedish Weight Soundings (without BH designation). Boreholes spacing = 200 metres.

Correlations with adjacent borehole data were also necessary in interpreting the soundings results. The surficial Organic Clay/Peat layer characteristically shows very low resistance to penetration. The alluvium as a whole shows lower sounding values when compared to the residual soils underlying the alluvium. It was also possible to define two lenses of Organic, Silty Clay based on borehole as well as the Swedish sounding data.

Fig. 3a Swedish sounding no. 105
Fig. 3b Swedish sounding no. 107
4 MACKINTOSH PROBE

The Mackintosh Probe was used mainly to determine the soil types along the diversion channel, a major excavation work in the project. The method was also used in parts of the Main Dam and the saddle dams.

The diversion channel was aligned close to the present river, as well as traverses some old river beds. In places it also cuts into residual soils. Some of the typical probe results are shown in Figure 7 (Mackintosh probes marked as EMA). A total of 48 probes were carried out in the project.

In general, as can be seen from the examples shown in Figure 7, the alluvial soils show lower blow counts than the residual soils. Also, probe penetration is usually much deeper in alluvial soils than is possible for the stiffer residual soils. Furthermore, due to the great variability of alluvial soils with depth, the probe data shows an erratic variation of blow counts with depth. In the case
Fig. 5 Soil profile along the Main Dam axis. Interpretations between boreholes are based on Swedish soundings.

Fig. 6 Soil profile perpendicular to the Main Dam axis.

of residual soils, the stiffness generally increases with depth. Based on such features and coupled with correlations with a few borehole data, the Mackintosh probe data were found useful in defining the soil strata along the diversion channel.

Figure 8 shows an example of the soil profile along the diversion channel obtained with the help of Mackintosh probe data. Once again, direct correlations with nearby borehole or augerhole data were necessary at the initial stage before further interpretations of other probe data can be made.

5 STANDARD PENETRATION TEST (SPT)

Standard Penetration Tests were carried out in all the boreholes made throughout the project area, i.e. at the Main Dam, the diversion channel, the spillway and the borrow area. The Standard Penetration Tests not only yield soil samples for visual examination, but also indicate the consistency of the soil materials. The SPT values are also useful in differentiating alluvial soils (softer materials) from the stiffer residual soils. Thus, alluvial soils are found to have SPT values of less than 10, while the residual soils have values of greater than 10, with some as high as 40-50 (upper limit of test value).

At the Main Dam, for example, the alluvial soils all show SPT values of less than 10, and the underlying residual soils show SPT values of greater than 10. At the spillway and borrow pit areas which are sited entirely on residual soils, SPT values of greater than 10 are obtained throughout the borelogs.

Some typical results of the SPT values in relation to the soil types are shown in Figure 9 (spillway) as well as in Figure 5 previously (Main Dam). The residual soils being derived from weathering of the underlying shale bedrock.
Fig. 7 Mackintosh Probe data (BMA1, 6, 5, 8) obtained at part of the diversion channel.

Fig. 8 Soil profile at part of the diversion channel (BA=augerhole)

Fig. 9 Soil profile at the spillway show residual soils with high SPT values.
The differentiation of alluvial from residual soils is of great engineering significance since these two groups of soils have widely contrasting properties. The alluvial soils, for example, are much softer or weaker materials and are more compressible than the residual soils.

The three methods of site investigations discussed, namely the Swedish Weight Sounding, the Mackintosh Probe and the Standard Penetration Test are most useful in differentiating these two soil groups. The alluvial soils show low test values for all the three methods mentioned, while the residual soils generally show much higher test values. For the softer alluvial soils the depth of penetration for testing using the Swedish sounding or the Mackintosh Probe is deeper than is possible for the case of residual soils. Finally, the erratic variations of test values (all three methods) with depth is characteristic of alluvial soils, while residual soils would show gradual increase of test values with depth.

7 CONCLUSIONS

The experience at the Bekok Dam shows that the Swedish Weight Sounding method, the Mackintosh Probe and the Standard Penetration Test are very useful and can be utilised successfully in damsite investigations in difficult, swampy or waterlogged terranes in tropical Malaysia.

The Swedish Weight Sounding and the Mackintosh Probe are relatively light tools and highly portable. These tests can thus be carried out relatively fast and at low cost. These features are particularly important in a developing country like Malaysia where highly sophisticated, heavy and expensive test machinery may not be feasible for one reason or another. However, the test results of these two methods must be verified or correlated with borehole data to be meaningful.

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ENGINEERING GEOLOGIC STUDY FOR THE FIRST STAGE CONSTRUCTION OF GEZHOUBA PROJECT

ETUDE DE TECHNIQUE GEOLOGIQUE POUR LA PREMIERE PHASE DE CONSTRUCTION DU PROJET GEZHOUBA

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ABSTRACT

The first stage structures of the Gezhouba Project, which are located on the left side of the Yangtze River, have already been commissioned. The foundations are composed of red fluvioelastic rocks of the lower Cretaceous system. The gentle strata as well as several tens of sheared or non-sheared claystone intercalations, actually shear zones, within the rocks made the engineering geological conditions rather unfavorable and problematic for the stability of these structures. In addition, construction disturbed further and worsened such conditions. On the basis of the first stage construction practice and the accumulations of the results geological mapping explorations, observations, tests and studies, this paper describes in detail foundation rock blasting, quick weathering of clayey soft rock, deformation of rockmasses in the foundation pit, changes in hydrogeological conditions, foundation treatment and measurements in the initial operation period. This paper presents valuable informations and data for construction of the second stage and for other dam constructions on soft rock foundations.

INTRODUCTION

The Gezhouba Project is situated 2.3 Km below Nanjingguan the lower mouth of the Three Gorges of the Yangtze River. At this site, the river becomes wide and was divided before construction by two small islands, Gezhouba and Xiba, into three water channels known as the first, second and third channels. The completed first stage of the project involves the second and the third channels and includes 2 shiplocks, a 6-bay sluice gate and a short length of non-overflow dam on the third channel and at Xiba island one of the powerplants equipped with 7 turbo generator units and a 27-bay spillway on the second channel, all of these structures totalling over 1,600 meters along the dam axis.

The foundation rocks are of the lower Cretaceous red fluvioelastic rocks, composed mainly of moderate-and-small-grain sized gray sandstone and purple silt stone with several lenses of soft clayey rock intercalations in between. The strata dip SE, i.e. towards the downstream and the left bank at an angle of 50°-80°. Along the weak intercalations shear zones developed and low angle small faults were caused due to the action of slight tectonic stresses of the Himalayan movement. Most of the shear zones, being argillized and softened,
constituted the major engineering geological problem to control the stability of foundations.

In the pre-and-under construction periods, large amounts of geologic investigations and studies were carried out on special problems concerning the engineering geological conditions of the dam region. They involved large scale geologic mapping, a number of small diameter core holes ( = 90 - 130 mm) and over 50 large diameter boreholes ( = 1000 mm), 9 experimental adits, several hundred sets of laboratory and in-situ rock mechanical tests, permeability tests, shear zone behavior evolution tests, comprehensive geophysical prospectings, and borehole television inspections. As a result of the above work, the engineering geological characters of the weak foundations which are characterized by nearly horizontal strata had been made clear and a sufficient scientific basis had been provided for the design work. Descriptions in this paper concentrate on the blast influence to bedrock in the process of construction, quick weathering of argillaceous soft rocks, foundation rockmasses deformation, permeability increase in rockmasses, foundation treatments and deformation measurements during the initial operation of the first stage structures.

FOUNDATION ROCK BLASTING

Blast excavation of foundation rocks which had multiple shear zones should have to be carried out in consideration of the degrees of impairment to the preserved rockmasses. Because blasting would cause the rockmasses surrounding the blasting area to split and damage their integrality, increase the permeability, make the shear zones displaced in the blast-influenced strata, and reduce the shear strength of the shear zones. At the beginning of the first stage construction, elaborate blast tests were conducted, and in actual excavation the technology defined through the tests were strictly followed. The results show that the blasting was of no serious consequence to the foundation structure.

QUICK WEATHERING OF THE ARGILLACEOUS SOFT ROCK

When fresh integral argillaceous rocks were exposed on the surface of the foundation pit, it would contract and develop desiccated cracks. And as they get soaked with moisture, they would expand, become slacken and disintegrated, such a physical weathering action developing at a great speed. In-situ observations proved that rock cracks could happen several or more than ten minutes after the rock had been exposed; a number of hours later they developed so many as a net; and within one or two years a new 1-2 m thick weathered crust would take place. The new crust was obviously and extremely unfavorable for ensuring the concrete-rock bonding surface to have proper strength. If the surface of fresh silt stone was under ten-day weathering, the shear strength of the concrete-rock bonding surface would be reduced approximately by 20% - 30%, and the tensile strength be reduced by 90%. Quick weathering had also harmful effects upon the behavior and mechanical strength of the shallow shear zones. In order to avoid such an influence, it was finally determined through protection tests to protect the foundation rock surfaces by timely placing a 20-30 cm thick layer of fine concrete, preserving a protective rock layer (which was to be excavated just before the placing of the concrete), covering them with wet straw bags and so on.

DEFORMATION OF THE ROCKMASSES IN THE FOUNDATION PIT

The first stage foundation pit was excavated 10-20 meters below the level of the river bed, with a maximum depth of 54 meters in the powerhouse section. During the period of excavation, the unbalance of in-situ stresses caused by unloading resulted in vertical and universal rockmass rebounding and those in the deep pit undergoing horizontal weak surface deformation. In consequence, the rockmasses deteriorated in integrality, the permeability increased and the engi-
neering geological characters became from bad to worse. On this respect, there are special descriptions in Reference (2). Here are some supplements to the development trend of how the rock structure turned loose and rockmasses deformed after stress relief.

1. Structural relaxation of the rockmasses in the pit

Because of the unloading influence, new relaxed rockmass zones occurred universally in the pit after excavation, and the crustal stresses 20-30% below the bottom of the pit was found through insitu stress survey comparison 1/3 or 2/3 smaller than the initial stresses (Fig. 1).

![Fig. 1. The variation curve of the insitu stress with the excavation depth of foundation pit.](image)

- 1. Original insitu stress in the riverbed rockmass
- 2. The insitu stress of rockmass below the bottom of foundation pit

Sonic sounding method which was used in 23 boreholes also indicated that the longitudinal sound wave velocity (Vp) was significantly decreased about 15 meters below the foundation rock surface; the maximum decrease was even up to 50% (Fig. 2). It is certain that in regard to such rockmasses which were in the relaxed zones the engineering geological conditions were not as good as that before load-off. Therefore, this point will have to be considered in evaluation of the engineering geological conditions.

2. Deformation development trend of rockmasses in the pit

During the time when the pit's insitu stresses were undergoing readjustment, the development trend of the rockmasses in horizontal displacement towards the direction of pit-side were different according to their different locations in the pit and different used treatment measures. Take the powerhouse's deep pit for example:

1. While the lower portion of the pit was taking shape the insitu stresses were readjusted, and followed by the occurrence and development of the rockmass deformation. When the crustal stresses in both the up and downstream rockmasses contradicted through the media of the concrete and attained balance, the deformation tended to stop as a whole. The development process can roughly divided into four periods like this: slow-fast-slow-generally stable (see Curve I, Fig. 3). The first period presented by the portion before the point "a" on Curve I is called the pre-split blast period, in which rockmasses deformed in the form of elasticity and creeps with low speed and small value. During the second period or the blast excavation period, shown on Curve I as the portion between...
"a" and "b", the rockmass deformation developed faster and faster as the depth of the pit was increased. This process continued until the rockmasses suffered large-scale shear displacement along the shear zones, and accompanied by the expansion of the argillaceous range. In the third period ("b-c" portion on Curve I), the deformation speed became relatively slow for relaxation had happened in the wake of the large-scale displacement of the rockmasses and the concrete was being placed. The last period represented by the portion after the point "c" on Curve I, the rockmass deformation was generally ceased when concrete was continuously placed and to make a load about 2 kg/cm² on the bottom of pit.

(2) At the upper part of the foundation pit, at which the structures are not thick, a "soft joint" or buffer joint was designed and placed between the upstream pit wall and concrete of structure to eliminate crustal-stress harms to the structures. The rockmass deformation here is still slowly continuing at present, with a rate of movement of about 1 mm per year (see Curve II, Fig. 3), it is much smaller than the deformation during construction of the "soft joint".

PERMEABILITY INCREASE IN THE PIT ROCK MASSES

Before excavation, the majority of the foundation rockmasses were relatively impermeable (water absorption rate was less than 0.01 l/min./m), except two permeable portions respectively in the second and the third channels where the water absorption rate was greater than 0.1 l/min./m. According to the statistics of water pressure tests, the maximum rate of the second channel was 0.14 l/min./m and that of the third channel was 0.51 l/min./m. After excavation, water pressure tests were conducted again in newly drilled boreholes, and the results indicated that permeability of the rockmasses increased universally and substantially in the ranges 2-5 meters and 5-10 meters below the foundation rock surfaces in the second and the third channels. The maximum water absorption rates were then 4.11 l/min./m for the second channel and 2.32 l/min./m for the third channel. Here are the laws by which the per-
meability increased:
(1) Significant increase in permeability and the thickness of permeable layers were found in the area which was composed of intercalated sandstone and siltstone with rather developed shear zones.
(2) The permeability of low angle fault zones were significantly increasing.
(3) As excavation went on deeply the permeable depth of rocks further extended. When the loads of structures were added, impermeability restored in the region where the compaction effect was enormous, except at the structure bearing surface.

for protecting the shear zones.
(4) Reinforced concrete anchor piles were placed in the pre-drilled large diameter boreholes for consolidating the resistant blocks downstream of the spillway's left six bays and the powerplant's No 1 - 3 units.
(5) To protect structures against damage to be caused by the release of crustal stresses in the deeply-excavated powerhouse pit where the rockmass underwent obviously horizontal displacement, a ten cm thick compressive buffer joint was universally built above the Ele. 16.2 m platform.

2. Analysis of measurements in initial operation
The results of outer deformation survey in half year since the first stage was commissioned indicated that foundation settlement was relatively large and horizontal displacement was small. For instance, the total settlement at the powerhouse was 12 - 24 mm and the horizontal displacement was only 3 mm in general (see Fig. 4); at the spillway total settlement was 5 - 8 mm and horizontal displacement was only about 2 mm. Although the total settlement was large, yet it accounted only for 20% - 50%, with the balance of 80% - 50% finished during construction. The settlement occurred during construction period was identified as different settlement, while that which happened after reservoir impoundment was even. Horizontal displacement measured in the half year was within the design range and its change were found coinciding with the difference of water head, suggesting that this deformation was small and did not exceed the elastic limit.

Measurements of seepage flow made the following known: No uplift water heads went beyond the design value. Except at shear zones and in the sandstone area where there were relatively plenty of emergence points of somewhat large seepage, no water was found in 80% - 90% of the drain holes under the spillway whose foundation was composed mainly of siltstone. A similar case was also found in the drain holes in the sandstone foundations of the shiplocks and the sluice. The percentage was about 70. Every founda-
tion section had a very little seepage, such as the apron section of the spillway where the total seepage was less than 1% of the design total. In short, the above descriptions show that although the engineering geological conditions of this project were not good, yet the careful treatment adopted had still brought about good engineering geological properties and therefore design requirements were met.

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ABSTRAIT

Les structures de première phase du Projet Gezhouba, qui sont situées au côté gauche de la Rivière Yangtze, ont été déjà commissionnées. Les fondations sont composées des roches fluvioclastiques rouges du système crétacé abaissé. Les couches modérées aussi bien que plusieurs tonnes d'intercalation de pierres argileuses cisaillées et non-cisaillées, en effet des zone de cisalement, entre les roches ont fait les conditions des techniques géologiques plutôt défavorables et problématiques pour la stabilité de ces structures. De plus, construction a dérangé et a empiré davantage telles conditions. Sur la base de la pratique constructionnelle de première phase et les accumulations des résultats, des explorations géologiques, de cartographie, des observations, des essais et des études. Cet article décrit en détail fondation d'abattage de roche, désagrégation vite de roche d'argile molle, déformation de masse de roche dans la fosse de fondation, changements dans les conditions hydrogéologiques traitement de fondation et des mesurages pendant la période initiale d'opération. Cet article présente des informations précieuses et des données pour construction de la deuxième phase et pour des constructions des autres barrages sur des fondations de roche molle.
PREDICTIONS AND REALISATIONS WITH REGARD TO THE SUB-SURFACE FEATURES IN CIVIL ENGINEERING STRUCTURES INCLUDED UNDER KALINADI HYDRO ELECTRIC PROJECT, KARNATAKA, INDIA

PREDICTIONS ET DES REALISATIONS CONCERNANT LES STRUCTURES SOUTERRANIENNES DANS LES STRUCTURES D'INGENIEUR CIVILS DANS LE PROJET HYDRO-ELECTRIQUE DE KALINADI DEL'ETAT KARNATAK EN INDE

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ABSTRACT:

The demand for more and more dependable and accurate geological predictions of foundation conditions of major civil engineering structures is on the increase in the present day. But the geology of the Earth still presents an enigma rolled in a mystery. It is never possible to predict geology of any area with an exact degree of certainty and be sure of realisation.

Kalinadi Project in Karnataka, India, is one of the major hydro electric projects which involves huge civil engineering structures. The project, when completed, is envisaged to supplement 1241 M.W. of electric power to the national grid. The project is taken up for execution in three stages and the first stage works are nearing completion.

The main components of Civil Engineering works executed/being executed are located in a terrain hitherto considered as composed of quite competent rocks. The problems encountered during execution were quite strange and abnormal and not usually to be anticipated in the shield areas. The rock encountered at the foundation of Supa Dam proved at many places to be incompetent on the left flank demanding elaborate treatment to withstand the load of the mighty superstructure. The head race tunnel is driven in metagreywacke rock associated with bands of phyllite and posed some surprises during the execution. The hill slope behind Nagjhari Power House turned hostile after cutting the toe of the hill for accommodating the Power House structures. The open channel reaches of the Tattihalla Water Conductor System experienced many slope failures requiring major remedial treatments to stabilize the slopes.

The pre-construction stage predictions are compared with construction stage realisations with regard to the sub-surface features. The main items reviewed are (1) Supa Dam Foundation (2) Head Race Tunnel (3) Hill slope behind Nagjhari Power House (4) Tattihalla Water Conductor System.
ABSTRACT

La demande pour des prédictions géologiques précises et fiables des conditions de fondements des structures de l'art d'ingénieur civil principaux s'augmente tous les jours. Mais la géologie de la terre pose un énigme entouré d'un mystère. Il est impossible de prédire la géologie d'une région avec certitude et d'être sere de l'en réaliser.

Le Projet de Kalinadi dans l'état de Karnataka en Inde est l'un des projets hydro-électriques majeurs qui s'agit des structures de l'art de l'ingénieur civil. Le projet quand il sera terminé, envisagera d'alimenter la gril national avec l'électricité de 1241 M.W. Ce projet sera réalisé dans trois étapes dont la première est presque achevée.

Les parties principales des chantiers civils terminés ou en train d'être terminés sont situées dans le terrain qui était considéré auparavant, composé des roches sédimentaires. Les problèmes auxquels on devait faire face pendant l'excavation étaient étranges et anormaux et qui ne sont pas généralement attendus dans la région péninsulaire de l'Inde. Les roches rencontrées à la rive gauche du fondament du barrage de Supa se manifestent hostiles à plusieurs endroits et en conséquence exigeraient un traitement élargi afin de supporter le poids de la super structure énorme. Le tunnel d'entrée est creusé dans la roche metagreywack associée avec les rayures de phyllite et a posé quelques surprises pendant l'excavation. Le côté derrière 'Nagazari Power House' a devenir hostile après avoir tranché les pied de colline afin de installer les structures des unités d'alimentation. Les canaux ouverts du 'Système de conduit d'eau de Tattihalla' a subi les problèmes d'inclinaison ce qui obligeait à effectuer les remèdes pour stabiliser les inclinaisons.

Les prédictions de l'étape de préconstruction sont comparées avec les réalisations de l'étape de construction concernant les structures souterraniennes. Les structures de l'art de l'ingénieur civil étudiées dans cette communication sont (1) Supa Dam Foundation (Fondement du Barrage de Supa) (2) Head Race Tunnel (Le tunnel d'entrée) (3) L'inclinaison de lacolline derrière 'Nagazari Power House' (4) Le conduit du système d'eau de Tattihalli.

INTRODUCTION

Earth is born out of violence and the adjustment of Earth following its explosive formation is not yet complete as can be evidenced by several volcanic eruptions, cluster movements, earth quakes and formation of new islands in sea.

The process of adjustment between "Man and Nature" has been very tedious and long and has never been easy. The man has been demanding higher prize from nature for his existence and nature has been periodically notifying its resentment by shaking off from the face of earth many magnificent structures dreamt, conceived, and brought into existence by man.

Geology of earth still presents an enigma rolled in a mystery. Nobody can with any exact degree of certainty predict the geology of any area and be sure of realisation. The nature has exhibited infinite variety which is almost paranoid and it does not believe in duplication of type sets. The nature does not appear to be adopting the technique of providing fascimiles and hence any engineering solution to a geological problem must have inbuilt consistency and compatibility. In this respect nature always remains truant and not easily accessible to man.

KALINADI HYDRO ELECTRIC PROJECT:

Salient Features:

Karnataka is one of the pioneer states in exploiting Hydel Potential. As early as 1902, the east bound river of the state, Kaveri was tapped to generate 4.3 MW of electric power at Shivasamudram.
The main precautions made after the detailed subsurface investigations are summarised as under (Oct.1974) :-

1. The foundation conditions in the river bed proper (Between Ch 781 M and Ch 832 M) are favourable while the flanks require deeper excavations for founding the high dam.

2. The foundation rock is available at much higher level in the right flank than in left flank.

3. The rock at the foundation grade on the flanks is the highly folded magnetite quartzite rock containing number of clay filled joints.

4. The longitudinal dyke and the transverse dyke are crossing each other in the bed portion and these dykes are fresh at river bed level and highly weathered and disintegrated in the flanks. The base width of the dam on the flanks is to be so adjusted to keep clear of the weaker portions of the foundation.

5. The rock at the foundation requires heavy grouting due to the presence of several sets of joints and shear zones.

6. The site is not feasible to construct an arch dam as the abutment rocks are very weak and weathered to great depths.

7. Construction of an earthen or rock fill dam at the Supa garge site is not feasible due to non-availability of suitable saddles for heavy flood dispossals during the rainy seasons. The deep pools existing at the upstream and downstream of the axis are to be filled up in case of a rock fill or an earthen dam, the quantities of which are enormous. The diversion of water during construction calls for large diameter diversion tunnels through the weak rock on the flanks. The depth and width of cut off trench for the earth or rock fill dam are to be as much as it should be for a masonry/concrete gravity dam.

8. Concrete gravity dam with liberal provision for grouting the foundation and two lines of curtain grouting is the ideal design for the Supa garge.

9. The power house which is planned to be on the left bank is to be shifted to the right bank due to unstable hill slope above the Power House cutting.

10. Due to the narrow extent of competent rock in the river bed, the spillway should be of 3 spans (15 m x 10 m) instead of 4 spans (15 m x 7.5 m).

THE FOUNDATION CONDITIONS REALISED DURING EXCAVATION:

River bed:

Blocks No.9,10,11 and 12 are coming in the river bed proper (between Ch 781 M and Ch 832 M). Fresh phyllitic quartzite, calcareous quartzite and banded magnetite quartzite are the rock types met with at the toe portion beyond the middle two thirds of the base width.

The longitudinal dyke was found displaced at the contact of the main transverse dyke and these two sets of dykes form the main rock type at the heel of the dam. The contacts of the dykes were found to be sheared (Plate 2).

The foundation rock was not found entirely satisfactory at the heel portion. A week zone struck at the heel portion of Block Nos.6 to 8 continued in Block No.9 to an extent of about 4 to 6 M from Block Joint 8/9. The weaker rock was excavated to depth of about 4.5 M from RL 466.5 M which was the general foundation level in the gorge portion. Water seepage in the II/s pit created a great problem during placing concrete which was overcome by dividing the pit into compartments and cover-concrete was laid by shifting sump well from one corner to another.

In Block No.11 deeper excavation in an area of 15 M by 20 M was to be attended at the middle two third portion to a depth of about 4 M to remove the crushed and weathered phyllitic and calcareous quartzite.

A 1.5 M wide bedding shear zone was encountered in the toe portion of Block Nos.9 to 11 comprising of highly limonitised weathered ferruginous quartzite with clay filling. T trench was to be excavated up to a depth of about 6 M below the average foundation level.

Although it was predicted that the river bed would be free of all foundation problems, some amount of difficulties
were to be faced during the actual excavation. The foundation level predicted at the river bed was RL 468.0 M and the average level reached for founding the dam was RL 466.5 M.

**Left flank:**

The depth of excavation required for exposing the foundation rock was too huge for all the blocks in the left flank. (Plate-3) At the beginning of Block No.2, the depth of excavation is of the order of 45 M from the original ground level and at the Block joint 7/8, the depth of excavation is of the order of 20 m, the maximum being at the Block Joint 3/4 to as much as 30 m. The acceptable rock profile, predicted in the geological drawings in the above locations was at depths of 45 M, 25 M and 47.5 M respectively.

The foundation conditions encountered for Block Nos.4 to 8 were of such abnormal nature which went very much beyond the imagination or predictions of the earlier investigators, Geologists and Designers. The site conditions called for radical changes in the design of the dam.

The heel portion of Block Nos.7 and 8 was comprised of weak rock to an extent of about 25% due to the intersection of the longitudinal dyke and the transverse dyke. The pit excavation and concrete back-filling were done very carefully by providing masonry baffle walls in the trench and after providing hammock reinforcements to transfer the loads to the shoulder rocks.

Heavy dewatering was to be attended as the seepage was considerable. As the entire base width of the dam was found to be not resting on sound rock foundations, it was decided to provide a strut at the heel in Block Nos. 7 & 8 to transfer the load of the dam to the good rock available at the upstream side. The upstream rock where the load of the dam is to be transferred through the strut was found to be adversely dipping at shallow angles towards the D/S side and some bands of manganese layers were observed along the bedding plane. Closer examination indicated that the joint planes were not smooth but folded downwards creating a favourable situation against sliding. Low pressure consolidation grouting was however carried out to stabilize the rock and to seal off the open joints.

Heel portion in Block Nos.5 & 6 contained weathered rock at the general foundation grade level. The conditions were similar to that existing at the heel portion of Block Nos. 7 & 8. This zone of weathering was due to the shearing caused at the contact of the longitudinal dyke. The shear zone material was scooped out to a depth of 8 to 10 m below the general foundation level. The good rock existing near the heel portion was protected by supporting the overhanging rock with masonry baffle walls. Hammock reinforcements were provided in addition to the anchor rods in the shoulder rocks. At the toe portion of Block Nos.4 & 5, highly weathered zone was encountered at the border of the transverse dyke. The pit for the removal of weathered rock commenced at the toe of Block No.5 was continued in Block No.4. The bore holes drilled in this location indicated the weathering to extend to more than 20 to 25 M depth. The pit excavated was later back filled with concrete after providing hammock reinforcement.

The area occupied by the pits in Block No.4 at the heel and at the toe portion was covering a major portion of the foundation of the Dam, and the construction of the original design would have made the toe of the dam to rest on the highly weathered zone. Considering the site conditions, the section was finalised by giving an upstream overhand in the concrete section to overcome the problem of stability.

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Fig.1: Foundation treatment for blocks 7 & 8 of Supa Dam - Heel Portion.
At the left edge of Block No.4, a shear zone existed at the heel portion which consisted of highly crushed rock 18/19 (plate 3). The earlier predictions in this regard were proved to be correct.

It was observed that the foundation rock once exposed to the atmosphere for a long time, gets a weathered coating on the surface. This phenomenon is attributed to the alteration of magnetite to hematite and secondary goethite by absorption of oxygen and water molecules from the atmosphere. To overcome this difficulty, the foundation blocks on the right bank were covered by bed concrete immediately after the fresh rock is exposed (Fig.3).

Major foundation treatment attended in Block Nos.13, 14 and 15 was for the shear zone existing in the middle third of Blocks 13 & 14 continuing in Block 15. After laying the foundation concrete in Block No.14 the shear zone material was scooped out to a depth of 8 M in Block No.13 in the form of a trench extended below foundation of block No.14. Concreting the scooped out portion was carried out along with concret ing in Block No.13.

Blocks No.16 to 18 contained a highly weathered longitudinal dyke near the toe of blocks. Deeper excavation by 1 to 2 M in Block No.16 in the dyke portion exposed fresh rock while the excavations in Block No.17 was to be made to a depth of 8 to 10 M to expose the fresh dyke rock. Chasing the dyke further into the right flank by the open excavation method would have endangered the side slopes in Block 17 and would have necessitated wider excavation at the top of

Fig.2: Excavation by Drifts below Banded magnetite rock.

Although the several problems faced during the execution were not individually identified during the investigation stage, the postulation made that the left flank will have difficult foundation problems has been proved to be correct.

Right flank:

Block Nos. 13 to 20 are located on the right bank.

The depth of excavation for all the Blocks on the right bank was much shallower when compared with left bank excavations and it varied from 18 M at Block Joint 13/14 to 30 M at Block Joint

Fig.3: Excavation of weathered dyke by drift - Block No.16 to 18 Toe portion.
the trench encroaching on good rocks on either side of the weathered dyke. Hence scooping out of the weathered dyke was taken up by forming intermittent gallery and through drift. A concrete arch was created near block joint 17/18 making use of the good rock available further downstream. (Fig.3). This facilitated scooping of the dyke up to Block joint 17/18 without inviting any slips from the side slopes. The cave formed after scooping out of the highly weathered dyke material, was later plugged by using concrete.

In the middle third of block Nos. 18 to 20 and at the heel portion in Block No.17, 2 to 3 M wide shear zone existed which was to be scooped out to twice its width and back filled with concrete.

Besides the above stated few unfavourable features, the foundation conditions in the right bank, were generally satisfactory.

The earlier predictions made about the availability of the better rock conditions on the right bank at much higher levels than in left bank proved to be very much correct.

The consolidation grouting taken up in the Blocks already covered by concrete has confirmed the earlier prediction of heavy grout intake in the highly jointed rock.

The consolidation grout holes are spaced at 3 M C/C in a grid pattern in the overflow section and at 1.5 M C/C in a grid pattern on both the flanks. The depth of holes in general has been restricted to 1/5 of the height of the dam.

Heavy grout intake was observed in the heel portion of Block No.7, 8 and 11 in the river bed, where the rock at the base of the dam is highly fractured due to the intersection of the longitudinal dyke and the transverse dyke.

Throughout on the right bank in Block No.12, 13, 14, 15 & 16 the rock at the heel upto middle third has been found to be consuming grout intake more than 23 kg/M upto 36.7 kg/M. At the toe portion, the grout intake varied between 2 kg/M and 17.5 kg/M.

The efficacy of the consolidation grouting has been tested in Block Nos.16, 17 and 18 on the right bank. The pre-grouting water loss tests were carried out in all the grout holes in stages of 1/2 M. Post grouting water loss was carried out in one test hole per Block. The results of the pre-grouting and post grouting water losses are tabulated as below:

<table>
<thead>
<tr>
<th>Block No.</th>
<th>Pre-grouting water loss in lugeons</th>
<th>Post-grouting water loss in lugeons</th>
</tr>
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<tbody>
<tr>
<td>Block No.16</td>
<td>0.30 to 11.50</td>
<td>0.07</td>
</tr>
<tr>
<td>Block No.17</td>
<td>2 to 11</td>
<td>0.08</td>
</tr>
<tr>
<td>Block No.18</td>
<td>1 to 19.5</td>
<td>0.90</td>
</tr>
</tbody>
</table>

These tests indicate that the consolidation grouting carried out below the foundation has effectively sealed off all the joints.

The predictions made with regard to the instability of the hill mass downstream of the dam site on the left flank were taken due note of during the time of execution and the power house was shifted to the right bank side to ensure safety of the Power House.

The existence of favourable foundation conditions restricted only to the river bed portion as postulated has proved to be true and as such the recommendations on the change in design of gates and reducing the number of gates have been very well appreciated and duly complied with during execution.

**HEAD RACE TUNNEL**

Another important component of Kalinadi Hydro Electric Project is the Head Race Tunnel which leads the waters of the Bommanhalli Pick-up Dam to the Surge Tank. The length of the tunnel as executed is 944.8 Mtrs. in a horseshoe shape to carry a discharge of 243 M^3/Sec.

Along the alignment of the tunnel 25 Nos. of bore holes were drilled in the investigation stage to fix up the fresh rock levels. The bore holes were closely spaced in the initial and exit reaches of the tunnel to fix up the tunnel portals. The bore holes indicated that the rock through which the tunnel would be driven consists of meta-greywacke with associated bands of phyllite cut up in places by dolerite dykes and vein quartz.
The surface investigation had indicated the existence of a dolerite dyke in the initial reach of the tunnel and that the width of the dyke would be of the order of about 30 m. During the execution of the tunnel, the width of the dyke encountered was of the order of about 105 m. Several concealed dykes which did not have surface manifestations were encountered during the driving of the tunnel. In all, 9 dykes were identified and their total width was of the order of 154.5 m.

The bore hole logs had indicated that fresh rock would be available throughout the length of the tunnel with sufficient rock covers and nowhere in the execution of the tunnel there were soil and weathered rock encountered at the tunnel grade, proving the predictions as correct.

Although at the investigation stage, by looking at the core pieces obtained from the bore holes, it was said that there was not much of change in lithology, it was observed during the execution of the tunnel that the initial length of about 4.5 km of the tunnel is comprised of more massive metagreywacke rock with phyllite bands spaced far and wide. The later half of the tunnel was found to be more foliated and contaminated with closely spaced bands of phyllite bands.

The alignment of the tunnel is N 25° E-S 25° W, while the general foliation trend is NNW - SSE. The tunnel alignment being about 40° skew to the foliation trend of the rock, it was predicted earlier that there might be heavy overbreaks at the right half of the crown portion. But the shape and amount of overbreaks occurred during excavations bore no corollary to the predictions made. This was attributed to the massive nature of rock not yielding along the crudely developed foliation planes.

The unpredicted predicaments during the driving of the tunnel were encountered at the following locations:-

1. A fault zone was met with at Ch 1260 M in the metagreywacke rock. The vertical displacement of bed was of the order of about 2 to 3 m as evidenced by the marker horizon of a quartz band. The movement of the rock across the plane was so intense to form a clay gouge of a 50 cms width.

The rock for a reach of 45 M was highly shattered and the roof falls occurred to an extent of about 6 m at the crown. This reach was heavily protected by steel girders at 30 cms O/C and the roof to rib contact was established by placing concrete.

2. At Ch 2111 M of the tunnel, the metagreywacke rock met with was highly blocky at the roof and also at the sides. By heading and benching and supporting the rock over the bench by pipes temporarily, until levelling course was laid for full section ribs followed by permanent supports, this zone was tackled successfully.

3. Another shear zone accompanied by highly jointed rock in a reach of about 15 m was struck at Ch 2457 M of the tunnel. Rock falls in this case were heavy and were so frequent that for a good length of time the labour could not dare to go even for supporting. Temporary supports by way of joist pieces and pipes were erected on the rock blocks fallen down and touching specific blocks of rocks in the roof which were cut up by intersecting joints. A concrete wall plate was laid over rock bolts driven at close intervals of 50 cms and ribs were erected over this. Joist pieces and pipes which were erected earlier were cut once the concreting of the roof is done. Technique of cantilevering a number of joist pieces was adopted from the preconcreted zone to blocks which were tending to come down with a little blasting disturbance. After removing the bench, the underpinning was done below the wall plates placed at the springing level.

The total length supported between Ch 2111 M and Ch 3207 M came to 71.5 M.

4. A shear zone was encountered at Ch 3207 M of the tunnel where the rock was moderately blocky and seamy. The overfalls from this zone of about 27.5 M was of the order of 3.5 M above the roof. Permanent supports were erected on the bench from above the springing levels and the ribs were spaced at 1.25 M centre to centre followed by concrete backfilling.

5. At Ch 3557 M of the tunnel, another major shear zone comprising of highly blocky rock for a length of about 50 m was encountered. The rock was crushed phyllite with clay gouge. This shear zone was found bringing too much
seepage water from the sides through the open joints. This reach was successfully tackled by providing full supports from the invert to the roof at intervals of 30 cms centre to centre.

In all, the reaches where permanent supports were erected came to 235.7 m which formed 2.5% of the entire length.

Another adverse feature that was encountered during driving of the tunnel was the difficulty in dewatering while the pumps fell due to power failures. The seepage of water, mainly from the shear zones was totally unanticipated. During the execution, dewatering to an extent of 6 to 74 ltrs/sec. was to be done from various adit mouths.

**HILL SLOPE BEHIND NAGJHARI POWER HOUSE**

The Nagjhari Power House and the other components are accommodated on the right bank of Kali Nadi by cutting the hill slope vertically to a depth of 43 M and the enroachment on the hill slope is of the order of 64 M. This cutting of the hill mass created an imbalance in the stability of the hill slope. While the excavation for the pits to accommodate the generators were under progress a huge slide containing material like soil rock and overburden mass slid down the hill slope, during June, 1975 after a heavy down-pour of rain. This huge slide from the hill slope behind Nagjhari Power House was totally a surprise, although it was anticipated that some minor slides could occur in the soil and overburden mass.

Several protective works were carried out to make the hill slope completely out of danger and they are:-

1. Diversion of major valleys to lead the water, precipitating on the hill slope, far away from the Power House area.
2. Construction of deeply anchored dwarf wall across the hill slope bifurcating the gunitied reach and the reach above.
3. Providing deep prestressed anchor pads at places where soil and boulders zone are considerably thick.
4. Constructing concrete pedestals all around loosely perched rock boulders to bury them in concrete so that the should not rool down in due course of time.

5. Anchoring rock boulders by means of deep perfo-bolts.
6. Constructing rock fall barriers at the upper reaches to hold back and arrest the motion of loose rolling boulders.

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**Fig.4:** Protective works behind Nagjhari Power House.

**TATTIHALLA APPROACH CHANNEL:**

The Tattihalla Water Conductor System was investigated by taking diamond drill holes totalling to 17 Nos. The alignment was fixed to pass through the ridge on a straight line so as to cover the major distance by tunnel and to maximise the deep cutting for open channel and exit channel. The lengths which were contemplated to tackle for the Tattihalla Water Conductor System were:

1. 2050.80 M of open channel at the approach side.
2. 4293.00 of tunnel.
3. 2882.00 M of exit channel.

During the execution of the channel at the approach side, many slope failures were encountered in top soil, weathered material and also in highly jointed fresh rock of matagreywacke and phyllite. The rock at Ch. 2898.15 M where it was earlier planned to open the tunnel portal was found to be highly jointed and surcharged with ground water. The rock with joint planes stained due to movement of ground.
water was not found to be fully competent to establish the portal at the intended location. In order to save time and to achieve economy and safety, it was decided that a further length of about 460 M was to be opened out with step side slopes so that this reach could be further covered by RCC to overcome obstruction of water flow by side collapse.

While excavation progressed, it was found that the left side slope was maintaining a 48° to horizontal slope (incidentally the dip of the slabby rock mass) and not the steep slopes contemplated by the designs.

This particular prediction in the open channel excavation was combated by resorting to stitching of the rock slabs by a series of prestressed anchors located at the toe portion with passive supports to prevent incipient movement. Each anchor was of 145 tons capacity and they were arranged in a grid pattern in 3 rows each connected with the other by cross beams.

CONCLUSION:

Geological predictions of subsurface features are the possible best fit concepts of an experienced Engineering Geologist derived from the best judgement and deductive reasoning of the limited information that is available to him. Beyond the depths explored and between the points explored reliance is made on the extrapolation of the available data. At best, the predictions are qualitative and cannot be quantitative.

The present State of the Art in the Engineering Geology is such that it is not possible to quantify or provide quantitative forecasts about the types of difficulties that the engineers might come across in constructing dams or tunnels where the time-cost-escalations are too heavy and are necessarily to be avoided.

There is large scope to improve upon the present state of the Art so as to provide an engineering geologist working in Civil Engineering projects better tools of prediction to enable him to achieve what he is expected to achieve. The geological process and products are not always quantifiable in specific, measurable terms and laboratory of nature is so difficult to understand that evaluation quantitatively is beset with many imponderables.

ACKNOWLEDGEMENT:

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Special thanks are due to Sri H.S. Bhat, Chief Engineer, Supe Dam Construction and Sri B.T. Rajamani, Divisional Engineer, for their fruitful discussions and for furnishing the details.

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- Soil
- Dolerite Dykes
- Shear Zone
- Fault
- Drift

- Assumed Foundation Grade Level
- Assumed Acceptable Rock Level From Drift Data
- Modified Foundation Level
- Actual Foundation Level

Scale: 1:2000

I - Section of Supa Dam

Predictions and Realizations: Kalinadi Hydroelectric Project - D.G. Chinodi, V.S. Upadhyaya, K.R. Pattabhiraman
INFLUENCE OF DYKES ON CONSTRUCTION OF CERTAIN DAMS IN MAHARASHTRA AND MADHYA PRADESH, INDIA

INFLUENCE DES FOSSES SUR CONSTRUCTION DES CERTAINS BARRAGES A MAHARASHTRA ET MADHYA PRADESH, INDE

K.L. MISHRA
Geological Survey of India
Nagpur, India

ABSTRACT

Geotechnical investigations of certain dam sites have shown occurrence of dolerite dykes in rocks belonging to Pre-cambrian, Deccan basalts and Gondwanas. Dolerite dykes have been observed at Tillari in Dharwar granulites; Matnar in Peninsular gneiss; Nilvand and Maniokdoh in deccan basalts and Makra dam site in Barakar sandstone of Gondwana system. Dykes occurring along, across and askew to river flow have been observed. Joints are closely spaced but tight, showing little or no water loss during tests. Apparently, surface contact of dykes with the country rocks appears tight, but percolation tests conducted in angle holes have indicated leakage along the contact. At one of the dam sites, holes located 29 meter away from each other got connected, resulting in complete water loss. No pressure was possible to develop during the test. Dykes occurring across the river flow provide a resistant barrier while those occurring along and askew to river flow are amenable to high under-seepage. This requires complete sealing of the dyke and country rock contact.

ABSTRAIT

Des investigations Géotechniques des certains emplacements de barrages ont montré l'occurrence des fosses dolérites en roches apparemment à pre-cambrian, basaltes Deccan et Gondwanas. Des fossés dolérites ont été observés à Tillari aux granulites de Dharwar; Matnar à peninsulaire gneiss; Nilvand et Maniokdoh aux basaltes Deccan et emplacement de barrage en Makra au grès de Barakar du système Gondwana. Des fossés se présentent le long, à travers et de côté à l'écoulement de rivière ont été observé. Des joints sont étroitement espacés mais serrés, montrant très peu ou absolument pas de Perte d'eau pendant des essais. En apparence, contact de surface des fossés avec des roches de compagnie semble très serré, mais des essais de filtration conduits dans les trous d'angle ont indiqué un coulage le long du contact. A un des emplacements de barrage, des trous situés à une distance de 28 m de chacun à l'autre se sont rapportés, resultant en perte complète d'eau. Pas de pression était possible à développer pendant l'essai. Des fossés se présentant à travers l'écoulement de rivière fournissent une barrière résistante tandis que ceux qui se présentent le long et de côté de l'écoulement de rivière sont responsable à sous infiltration à un degré considérable. Cela nécessite une action complète de sceller du fossé et contact de roche de campagne.

Presence of Dykes in various rock groups of Indian peninsular shield has been mentioned by earlier workers (1). The Author had the opportunity to investigate or study a few Dam sites in Maharashtra and Madhya Pradesh States where dykes have
been found to penetrate the country rocks. Cases of such occurrences have been observed in:

A. Precambrian of Peninsular India
B. Permo-Carboniferous of Gondwanas and
C. Cretaceous basalts i.e. deccan traps.

A. Precambrian Rocks:

In the Dharwar Group of rocks, are located two dam sites, forming a part of Tillari Irrigation Project which is joint Venture of Maharashtra and Goa States. One Dam under construction is 14 m high pick up weir across Kharari river. Here the country rocks comprise sericite schist and granulites. The rock formation of this site have been intruded by dykes in two phases. In the first phase four dykes have been intruded, which occur along the flow of river or slightly askew to flow.

Similarly the second pair of hole Nos. 3-V and 5-V located 29 m apart developed similar connections and water loss was complete and no pressure was possible to develop for testing the permeability. Flowing conditions developed between test sections at R.L. 51.4 m to 49.3 m and 46.3 m to 44.8 m.

Second Dam site of this Project for a 72 m high structure was investigated. Here a 15 m thick dolerite dyke occurs in the river channel along the direction of

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<table>
<thead>
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<th>Sr. No</th>
<th>Rock Group</th>
<th>Rock Types</th>
<th>Name of Dam site/Project</th>
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<tbody>
<tr>
<td>A.</td>
<td>Precambrian</td>
<td>Granulite, Amphibolite, granatiferous and sericite schist</td>
<td>1. Pick-up weir and main dam, Tillari Irrigation Project, Maharashtra</td>
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<td>B.</td>
<td>Gondwana</td>
<td>Supra Barakar sandstones</td>
<td>2. Matnar Hydel Project Bastar District, Madhya Pradesh</td>
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<td>C.</td>
<td>Deccan basalt/Trap</td>
<td>Basalt/Traps</td>
<td>3. Makra dam site, Kudresana Project, Shahdol District, Madhya Pradesh</td>
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<td>4. Manikdoh dam, Kudadi Project, Poona district, Maharashtra</td>
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<td></td>
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<td>5. Nilvande and Mhaladevi Dam sites upper Pravara Project, Ahmednagar District, Maharashtra</td>
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</table>

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In the second phase a dyke has been intruded across the river flow (Fig. 1). The country rocks are soft to moderately hard, thinly foliated and sparsely jointed. The dykes, dolerite, in composition are very hard and compact and the one across the flow is standing out as highly resistant barrier. During the lean period water flows through reaches where it meets the dykes along the flow. The dykes are closely jointed but the joints are tight. During subsurface investigation by means of drill holes, open nature of the country rock and intruding dyke contact was conclusively proved by two pairs of holes (2).

Hole No. 10-I(Inclined) and 2-V (Vertical) when tested for permeability river flow. Another dyke about 2 m thick joins it from upstream right side aligned askew to river flow.

Although enough exploration has not been carried out so far, one 70° angle hole which reached the contact zone at Ch. 3100 m and 90 m downstream of the axis showed complete water loss. The test water came out through another hole 15 m upstream which had not entered the dyke. This observation has also indicated the open nature of contact zone.

2. Dykes in Peninsular Gneisses:

A case of dolerite dyke occurrence in precambrian gneiss has been observed...
during preliminary investigations of Matnar Hydel Project Dam site across Indravati river in Bastar District of Madhya Pradesh, (India). A nearly 100 m thick dolerite dyke extends from the upstream side of the right bank to the downstream side of the left bank. At the dam axis most of the river channel is occupied by this dyke, whereas the country rock here is peninsula gneiss. The dip of the dyke is about 85°. Dolerite shows transition to epidiorite (3). The gneisses show contact effect and are brecciated and broken. Weathering in gneisses is up to 12 m depth causing decomposition of gneisses. Joints up to 6 cm opening are common in gneiss. In dyke, rock joints are closely spaced but tight. Deep weathering and broken nature, of the gneisses indicates an open contact zone. Further investigations are proposed.

B. Dykes in Gondwana Group:

During preliminary investigations of Makra dam site, Kudrenesa project in Shahdol District of Madhya Pradesh, presence of a bun shaped pluton in the Johilla river (a tributary of Son river) channel and two dykes on each flank was observed (4). It was found that petrological composition of all the three intrusions is that of dolerite. Country rocks at this dam site are conglomeratic pebbly sandstones and variegated shales/clay. They are grouped into persora formation of Suprabakaraks. The sandstones are horizontally bedded and poorly cemented. It was noted that the sandstones are deeply weathered up to 12 m depth while weathering depth in the dolerite is 2 meter only.

Main intrusive comprises 15 m thick bun shaped pluton as compact and dark grey rock with baked margins and caught up blocks of sandstones (Fig. II). On the left flank a dolerite dyke 20 m wide trending parallel to the river flow is exposed and extends into the reservoir area. The contact between barakar sandstone and dyke is open as revealed from drill hole data. Both sandstones and dolerite rocks show vertical to subvertical joints. Also in the contact zone both rock types are weathered. The compact nature of dolerite gives 80% core recovery while poorly cemented sandstones gave less than 20% core recovery. Four bore holes have been drilled across the contact zone which indicated water loss during drilling. Although percolation test data is not available loss of drilling water indicates open nature of contact zone.

C. Dykes in Deccan Basalts:

Several cases of dyke intrusions have been observed in Deccan basalt/trap, three of which are being discussed here. One dam where construction work is nearing completion and grouting is in progress is Manikdhon dam on Kukadi river in Poona District of Maharashtra State. Here horizontally disposed cretaceous trap suite of rocks from the rock type (5). Two dolerite dykes have been observed along the right flank (Fig. III) trending 10° to 15° askew to river flow. Dykes are about 2 m are 40 cm thick and dip at 80° towards river. For investigation of subsurface condition of dyke trap contact zone, above the dip plane of each dyke, two holes have been drilled. Core recovery from the contact zone was poor and permeability tests revealed almost complete water loss. Grout intake in the contact zone is higher and this data is given in appendix-1.

Two dam sites involving dykes in deccan basalts are investigated on Pravara river for Mhaladevi (6) and Nilvande (7). Dam sites in Ahmednagar District, Maharashtra State. At Mhaladevi site two dykes, one along each abutment has been observed. The left bank dyke is 5 to 6 m thick and dips towards river channel. It shows well defined contact with trap - which is sheared and brecciated on both sides. Along the right bank a 5 to 7 m thick dyke also has similar contact zone. Both dykes are parallel to the flow direction.

The Nilvande dam site is located about 4 km upstream on the same Pravara river. Here a 6 to 7 m thick dolerite dyke traversed the left abutment for about one kilometer distance from Nilvande to Thakurwadi village. It has a dip direction of 80 degrees away from the river channel. This dyke passes into covered ground near Nilvande Village and another dyke is seen at right angle to it along this village. The river also takes a Right angle turn forming a "U" shaped loop with the village located inside the "U" shape. This dyke also is about 6 m wide appears to be almost vertical. The trend of dykes in this valley appear to control flow direction river.

Investigations of these two dam sites are at preliminary stage and drilling data for the dykes is yet to be obtained. However, the broken nature of the contact zone here indicates possible high permeability values.

Seven cases of dam sites, with occurrence of dolerite dykes show open nature.
of the contact zone with country rocks with high permeability. Generally the dykes are along the flow direction of river and control valley development. Wherever dykes are observed, the nature of this contact with the country rocks must be studied by angle holes. The vertical contact suggests possibility of high under seepage through the contact zone. Observations necessitate intense consolidation grouting to seal the path of percolation, with a view to prevent seepage/reservoir loss. High grout intake of the orders of 15 to 50 kgs for 5 m stretch at Manikdoh dam site confirms the need for grouting. This also needs testing the effect of grouting against seepage. For this various grout techniques would be called for.

REFERENCES

1. Auden, J.B. Dykes in Western India, Transactions National Institute of Science, G.S.I., Record, Vol. 3 No.III.


Appendix 1
Grout and water intake data, Manikdoh Dam

<table>
<thead>
<tr>
<th>Dyke at CH. 815 m Thickness 1 to 2 m</th>
<th>Dyke at CH 857 m Thickness 0.25 to 0.40 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hole Location w.r. to dam axis</td>
<td>Grout Intake kg</td>
</tr>
<tr>
<td>CH. 816 m 15 m d/s</td>
<td>16</td>
</tr>
<tr>
<td>CH. 816 m 9 m D/S</td>
<td>50</td>
</tr>
<tr>
<td>CH. 816, 75 m 3 m D/S</td>
<td>20</td>
</tr>
<tr>
<td>CH. 813 m On Axis</td>
<td>12</td>
</tr>
<tr>
<td>CH. 812.2 Four holes in four directions</td>
<td>15, 15, 20, 15</td>
</tr>
<tr>
<td>CH. 811.8 m 17.2 m D/S</td>
<td>10</td>
</tr>
</tbody>
</table>
CONSTRUCTION OF SHEAR KEYS BELOW FOUNDATIONS OF INDIA'S KADANA DAM

CONSTRUCTION DES JOINTS A EMBOÎTEMENT AU DESSOUS DES FONDATIONS DU BARRAGE KADANA DE L'INDE

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ABSTRACT

The five low angle faults detected in the foundations posed threat to stability of 66 m high straight gravity dam across the Mahi river in Western India. The stability computations against sliding revealed that power dam blocks 2 to 5, spillway blocks 7 to 9 and 15 to 18 resting over low angle faults had inadequate safety. Thus, it became imperative to introduce stabilizing forces in form of "Shear Keys" along these faults below foundations to achieve requisite shear friction factor against sliding. Shear keys, 3 to 6 m wide, in a total length of 402 m were constructed by excavating drifts below existing structure. Controlled blasting using safe charges based on ground vibration characteristics determined by site experiments were adopted for tunnelling. These drifts were then back filled with concrete having 210 kg/cm² strength and contact grouting was done one year later to seal off the shrinkage gaps. The driving of drifts below the partly completed dam was unique in a way that tunnelling work, as close as 5 m to the structure, using safe charges consistent with the safety of dam was to be done along faults which were slightly undulating while the construction of the dam proceeded above it. Continuous dewatering and working in the restricted space with inadequate ventilation made the execution very difficult. The entire operations, both tunnelling and concrete filling, were done manually.

This paper describes the criteria for design and construction of shear keys, blast experiments to determine the safe charges for controlled blasting and problems experienced during the excavation of cavities below foundations of the masonry dam and back filling them with concrete.
ABSTRAIT

Les 5 failles décourvertes dans les fondations d'excavation posaient un danger à la stabilité du barrage à gravité de 66 m de hauteur sur la rivière Mahi située dans l'Ouest de l'Inde. Les calculs de stabilité contre le glissement ont montré que les blocs de béton 2 à 5; les blocs de déversoir 7 à 9 et 15 à 18 qui reposent sur les failles m'étaient pas de suffisante sécurité. Ainsi il était impératif d'introduire des forces stabilisatrices du type « joints à emboîtement » le long de ces failles en dessous des fondations pour achever le facteur requis de friction due au cisaillement contre le glissement. Des joints à emboîtement, 3 à 6 m, de largeur sur une longueur totale de 402 m ont été construits en approfondissant des mandrins en dessous de la structure existante. Le travail aux explosifs utilisant des charges de sûreté basées sur les caractéristiques de vibration du terrain qui étaient déterminées par expériment sur le chantier même ont été adoptées pour le percement d'un tunnel. Ces mandrins ont été alors remplis de béton ayant 210 kg/cm² de force et le jointoiement au mortier liquide à été fait un an plus tard pour boucher les brèches dues à la contraction. L'entraînement des mandrins en dessous du barrage inachevé a été unique. Le percement du tunnel, aussi près que 5 m de la structure, utilisant les charges explosives de telle façon que le barrage reste secure, a été fait le long des failles qui étaient un per ondulés pendant que la construction du barrage continuait en haut. L'assèchement continu et les travaux dans une espace limitée avec insuffisante ventilation avaient rendu l'exécution difficile. L'opération entière, c-a-d- le percement du tunnel et le remplissage du béton ont été fait manuellement.

Cet article décrit le critérium pour le module et construction des joints à emboîtement, les expériences d'exposition et les problèmes pendant l'excavation des cavités au dessous des fondations du barrage et les remplir de béton.

1. Introduction

The 1500 m long rubble masonry and earth fill Kadana dam across the Mahi river in Gujarat, India impounds 1545 hm³ water for irrigation and power generation. The straight gravity dam constructed in uncoarsed rubble masonry, located in the river portion comprises 405.7 m long spillway flanked by 106 m long power dam and 70.4 m long transition dam on its left and 39 m long non-overflow dam on the right side. The earth fill dam wraps around the left transition section and extends 762 m to tie up with the left abutment ridge. The masonry dam is 66 m high above the lowest foundation. The construction of the dam has been completed in 1979 at a cost of ₹.900 million while the works for
the hydro power project to install 4 reversible turbine units, each of 60 MW capacity, are under progress and planned to be commissioned by 1983 at an estimated cost of Rs.1010 million.

The intricately folded sequence of steeply dipping metamorphic rocks of Arvali Super group of the pre-Cambrian age, traversed by several faults, bedding shears and joints form the foundation rocks. In addition to the general measures adopted for the foundation improvement such as dental treatment to soft beds and high angle faults, curtain grouting and consolidation grouting, shear keys along the low angle faults below the dam were constructed for strengthening foundations of the masonry dam.

2. Dam site geology

The surface geological mapping of the dam vicinity revealed that folded quartzites, quartzose phyllites and schistose phyllites, trending in N60°W-S60°E direction and dipping 65° in southerly direction to vertical were traversed by shears, closely spaced joints and faults. About 19 faults, having steep upstream dips were recorded in the initial stages of investigation.

3. Sub-surface exploration

The pre-construction stage sub-surface exploration by 48 drill holes, ranging in depths from 7 to 92 m aggregating to 1675 m, nine test pits aggregating to 77 m depth in the earth dam reach and one 15 m long trench into the right abutment ridge was carried out to assess the foundation conditions. In addition, 36 core holes aggregating to 1077 m depth were drilled during construction for the exploration of faults, power house and additional spillway.

4. Rock types

Alternate beds of quartzites and phyllites varying in thickness from a few centimetres to 3 m make an angle of 40° to 70° with the NE-SW aligned dam axis. The quartzites are strong to very strong, fine grained, fresh, jointed, fractured and grey to pinkish white in colour. While grey to greenish grey or purple grey quartzose phyllites are moderately strong and less jointed; the 3 to 15 cm thick schistose phyllites are soft, weak and decomposed at places.

5. Faults

Seventeen faults were encountered in the final grade foundations of the masonry dam. Of these, seven dip 60° to 75°, five dip 45° to 60° and remaining five have dip less than 45°. They were marked by breccia and clay gouge of varying characteristics and thickness.

The high angle faults, generally trend in N-S direction, running diagonally to the dam axis from upstream to downstream, dip in westerly direction or towards upstream. The thickness of clay gouge varied from 2 to 15 cm, occasionally upto 30 cm and the associated fractured zone measured upto 1 m.

The low angle faults varying in composition and in thickness from a few centimetres to 1.2 m consisted
of clay gouge with angular rock fragments of quartzites and phyllites of varying size, shape and quantity.

6. Low angle faults

As the construction progressed with conventional foundation treatment like consolidation and curtain grouting, detal treatment to high angle faults, etc., five low angle faults affecting stability of the structure were discovered during the progressive excavations. These low dipping faults defied their detection during investigation stage by conventional exploration methods; mainly due to their concealed nature and insignificant size.

Two faults gently dipping upstream in the power dam blocks 2 to 5, two faults gently dipping downstream in the spillway blocks 8 and 9 and a saucer shaped fault with variable dip directions in the spillway block 15 onwards posed a threat to the stability against sliding. The attitude and nature of gougy material along these faults are described below.

Case I : Faults gently dipping upstream

Two near parallel low angle faults, 12 to 15 m apart, one over the other with an apparent dip of 15° to 21° towards upstream occurred below the foundations of the power blocks 2 to 5. As the lower fault was deep seated, the stability analysis revealed that it does not require any treatment.

The upper fault trending in N-S to N5°W-S5°E direction and dipping 35° to 38° due West consisted of 2 to 20 cm thick clay gouge. At some places, the clay was mixed predominantly with little gritty material while at other places rock fragments were mixed in it making the thickness of gougy material even upto 55 cm. The adjoining fractured zones on the hanging wall and the footwall were 0.15 to 1 m thick and here some joints were filled up with yellow clayey material.

Case II : Faults gently dipping downstream

Two parallel faults, one above the other and 6 to 10 m apart dipping 25° to 30° towards ENE and making 70° to 75° angle with the dam axis were recorded one each in the foundations of the spillway blocks 8 and 9. The apparent dip of these faults was 7° to 9° in the flow direction and 20° to 22° along the dam axis towards the left bank. The faults, in general, showed the following section.

- 20-60 cm thick fractured zone on the hanging wall
- 1-10 cm thick predominant clay-discontinuous
- 5-20 cm thick clay and breccia-rock powder
- upto 1.3 cm thick clay-discontinuous
- 15-65 cm thick fractured zone on the footwall

Case III : Fault with variable dip directions

In the perennial river channel, a fault striking N55°-60°W-S55°-60°E and dipping 25° to 30° (occasionally 10°) towards S35°W i.e. towards the right bank was observed initially during progressive excavations up to 30 m downstream of the dam axis in the spillway block 14. Its conti-
nuity in the upstream, towards the downstream as well as along the dip direction was established by further foundation excavations, trenches and six drill holes.

This 0.3 to 1.2 m wide fault, wavy in disposition traversed the foundations from upstream to downstream and changed its dip directions as well as gradient at the heel and toe of the dam. The apparent dip in the direction of flow varied from 7° to 20° towards downstream upto middle third of the block and then 40° towards upstream in the toe portion. Thus, in section this significant fault assumed a saucer shaped configuration. The fault is made up of the following section.

15-30 cm thick fractured zone on the hanging wall
1-10 cm thick predominantly clayey seam-yellow, white, grey or brick red in colour—is conspicuously absent at places.
7-40 cm thick clay and breccia
0.5-2 cm thick clay-reducing to a thin film at places and pinching out.
7-60 cm thick fractured zone on the footwall.

To keep up the construction schedule and also since the fault occurred at a shallow depth, large scale excavations of the hanging wall, which could be utilised for treatment later on were carried out till over 10 m of rock cover was attained above the fault.

The clay in the fault material was of low plasticity and had the coefficient of consolidation of the order of 150 to 200 x 10^{-4} cm^{2}/s.

Since the clay layer was only 10 cm thick, most of the consolidation would be over within a period of 2 to 4 weeks and as the masonry blocks get raised. Thus, it was considered that a shear key on the upstream side below blocks 15 to 18 would not be required to guard against settlement.

These low angle faults posed main problem of sliding and it became inevitable to evolve suitable remedial measures. Therefore, in 1973 the Government of Gujarat constituted the 'Kadana Experts Committee' to examine the problems emanated from foundations and to recommend treatment for the safety of the structure.

7. Significance of low angle faults

Faults with gentle dips, either towards upstream or downstream in the foundations of the masonry dam pose problems of leakage, settlement and sliding. The treatment to such faults mainly depend on the following factors.

* Thickness and properties of the fault material, e.g. gouge, breccia
* Orientation and dip of fault relative to loading of the dam
* Delineation of the fault trace below the foundation i.e. depth, geometric configuration, etc.
* Stress pattern due to the fault
* Stability against sliding

The two dimensional stability computations revealed that sliding could occur along these weak planes. To design appropriate strengthening measures, a series of insitu tests
and laboratory studies on the fault material were conducted. Based on these tests, the Experts Committee recommended that

(i) coefficient of friction \( \tan \delta \) for the gougy material along the fault plane should be considered as 0.5 instead of 0.2 assumed earlier;

(ii) value of 20.5 kg/cm\(^2\) be adopted for evaluating passive resistance available from the downstream rock mass below the maximum scour depth and

(iii) cross shear value for foundation rocks be considered as 7 kg/cm\(^2\).

The stability computations against sliding of masonry blocks (each 19.5 m long) resting over low angle faults indicated that the power dam blocks 2 to 5, spillway blocks 7 to 9 and 15 to 18 had inadequate safety coefficient. Thus, it became imperative to introduce stabilizing forces in the form of shear keys, as follows, to achieve the requisite shear friction factor against sliding.

(A) Two 4.5 to 6 m wide drifts parallel to the dam axis and four 3 to 4.5 m wide drifts at right angle to the dam axis were driven for a total length of 182 m along the fault below foundations of power dam blocks 2 to 5. These drifts were then back filled with concrete to act as shear keys (figure 1a & 2).

(B) One 4.5 m wide and 24 m long drift at the right angle to the dam axis in block 9; a 4.5 m wide drift parallel to the dam axis for half the width of block 8 along the lower fault plane and a 4.5 m wide drift along the upper fault in block 7 were driven and back filled with concrete (figure 1b).

(C) Two 4.5 to 6 m wide drifts parallel to the dam alignment at 30 m and 42.5 m downstream of the dam axis were driven for a total length of 135 m along the fault below foundations of blocks 16 to 18 and back filled with concrete (figure 1c).

8. Excavation of drifts for shear keys

The following criteria were laid down by the Experts Committee for the design and construction of shear keys.

(i) The number of keys should not be more than two along one fault.

(ii) The photoelastic studies indicated that the ideal location of the shear key would be the downstream one third point of the dam base.

(iii) The width should preferably be 3 m and should not exceed 4.5 m. Extra width, if required, can be obtained by scooping out the fault material.

(iv) The drifts should be excavated by controlled blasting using safe charges.

(v) Excavation be done at least 1 m into sound rock both at the top and bottom of the fault.

(vi) Thorough washing and grouting of the fault zone and adjoining fractured zone beyond the scooped area on the sides of the drifts should be done through 2 m deep holes spaced at 1.5 m apart where
(a) POWER DAM BLOCKS 2 to 5

SECTION THROUGH CENTRE OF POWER DAM BLOCK 4

(b) SPILLWAY BLOCKS 7 to 9

SECTION THROUGH CENTRE OF BLOCK 8

(c) SPILLWAY BLOCKS 15 to 18

SECTION THROUGH CENTRE OF BLOCK 16

KEY: (1) DAM AXIS (ii) TOE OF MASONRY (iii) DRAINAGE GALLERY (iv) ADDITIONAL GALLERY (v) CROSS GALLERY (vi) LOW ANGLE FAULT (vii) SHEAR KEY (viii) APPROACH SHAFT (ix) DRAINAGE HOLE (x) CURTAIN GROUT HOLE.

FIG. 1 LAYOUT OF SHEAR KEYS
consolidation grout holes had not intercepted the fault.

(vii) Before back filling the drifts, inlet pipes with nipples at 1.5 m interval should be placed along the fault trace and at the crown for contact grouting to seal off shrinkage gap on concrete setting.

(viii) Contact grouting need be carried out at least one year after the concrete filling.

(ix) The drifts should be back filled with concrete having 210 kg/cm² strength using neat cement only; care to be taken no to leave construction joint in the concrete along the fault plane.

9. Controlled blasting

As these drifts were to be excavated below the existing structure or approach shaft near the masonry already laid, controlled blasting using safe charges was necessary.

Blast experiments were conducted for recording the ground vibration intensities from the explosions. Instantaneous blasts using various charges were made and resulting ground vibrations such as particle displacement and particle velocity were recorded at different distances from the blast. From the data thus obtained, the safe charges for different distances from the structure to be safeguarded were adopted as given in table 1.

10. Approach shaft

The excavation of the drift was generally done through 4.25 m diameter and 7 to 25 m deep appro-
ach shaft. In case of drifts for the power dam, two accesses were made to facilitate tunnelling and better ventilation. First, a 4.25 m diameter shaft was sunk to 12.5 m depth from the excavated penstock trench in downstream of the power dam block 4, while the second entry was directly made just below the toe of the power dam block 3 where the fault was exposed 6 m below the foundation level (figure 2).

Table 1

Recommended values of safe charges for excavation of drifts

<table>
<thead>
<tr>
<th>Distance between the centre of explosion and the structure to be safeguarded against vibration in metres. (R)</th>
<th>Amount of charges per delay (not less than half second) in kg. (Q)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.15</td>
</tr>
<tr>
<td>10</td>
<td>0.35</td>
</tr>
<tr>
<td>15</td>
<td>0.55</td>
</tr>
<tr>
<td>20</td>
<td>0.75</td>
</tr>
<tr>
<td>25</td>
<td>1.00</td>
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<tr>
<td>30</td>
<td>1.25</td>
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<td>40</td>
<td>1.70</td>
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<tr>
<td>50</td>
<td>2.30</td>
</tr>
<tr>
<td>60</td>
<td>2.80</td>
</tr>
<tr>
<td>70</td>
<td>3.30</td>
</tr>
<tr>
<td>80</td>
<td>3.90</td>
</tr>
</tbody>
</table>

This second access was made with utmost care using rock bolting, steel supports and concrete lining in the initial reach. In the spillway block 8, about 7 m deep shaft was excavated below the bucket foundation level and then concrete
1. FOUNDATION GRADE ROCK
2. FAULT PLANE
3. APPROACH SHAFT
4. SHEAR KEY
5. MAIN DRAINAGE GALLERY
6. ADDITIONAL DRAINAGE GALLERY

FIG. 2 ISOMETRIC VIEW OF SHER KEYS ALONG FAULT BELOW POWER DAM BLOCKS
was raised up to the invert level leaving a 4.25 m diameter approach shaft for tunnelling operations.

11. Tunnelling

The driving of drifts below the partly completed dam structure at Kadana was unique in a way that tunnelling work using safe charges consistent with the safety of the dam was to be done along the faults while the construction work was going on above it. Continuous dewatering and working in a restricted space with inadequate ventilation made the tunnelling operations very difficult. Usually, the heading and benching method was adopted in driving the drifts i.e. advance heading was driven on the hanging wall and then the fault and the footwall were excavated by benching. Normally, the drilling pattern adopted for heading consisted of 75 cm deep 20 to 22 holes. These holes were then charged with gelatine of 80% strength and half second delay detonators. The pull achieved per round of blast was 0.5 to 0.8 m. The entire operations, both tunnelling and concrete filling, were done manually.

Substantial leakage was observed through sheared rock lying above the clay gouge at many locations in the drifts below the spillway blocks 7 to 9 and 16 to 18. Dewatering by sump pumps (3.5 to 7.5 HP) placed at different locations in drifts, especially near concentrated leakage, and advancing face of the drifts was done and collected at the main sump at the bottom of the approach shaft. From here, dewatering was done by deploying adequate number of high capacity pumps (20 HP) through delivery pipes installed along the wall of the shaft. The leakage was so heavy and concentrated that dewatering pumps were working round the clock keeping stand by generators near the drifts. The leakage observed and capacity of pumps installed are given in table 2.

During the excavation of the drifts by controlled blasting, monitoring of vibration was also carried out. Few observations were made in block 17 where the masonry was three weeks old. The results showed that agreement between the observations and recommended values was quite satisfactory. On completion of the excavation of the drift to the requisite size, the fault gouge along with adjoining crushed rock was scooped out by demolition tools at least to 60 cm depth or more wherever possible. Then 50 mm diameter inlet pipes with nipples at 1.5 m interval for contact grouting were laid along the fault trace on walls of drifts and one at the crown (figure 3). These inlet pipes were also connected with the return pipe for removal of air, etc. while grouting from the top of the shaft. The end of each nipple hugging the drift was capped by covering it with thin polythene so as to prevent it from chalking while concrete filling. This polythene was expected to be punctured on applying the grouting pressure.
Table 2
Details of dewatering pumps in the drifts

<table>
<thead>
<tr>
<th>Location of drifts</th>
<th>Total leakage in lit/s</th>
<th>Pumps in drifts</th>
<th>Pumps in shaft</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Power dam blocks 2 to 5</td>
<td>7</td>
<td>7.5 HP - 4 Nos.</td>
<td>20 HP - 2 Nos.</td>
<td>Pumped intermittently and one pump was stand by</td>
</tr>
<tr>
<td>Spillway blocks 7 to 9</td>
<td>17 to 42.5</td>
<td>3.5 HP - 1 No.</td>
<td>20 HP - 3 Nos.</td>
<td></td>
</tr>
<tr>
<td>Spillway blocks 16 to 18</td>
<td>28 to 56</td>
<td>7.5 HP - 5 Nos.</td>
<td>12 HP - 1 No.</td>
<td>Total HP hours 2,32,308</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20 HP - 3 Nos.</td>
<td></td>
</tr>
</tbody>
</table>

5200 kg cement with an average intake of 19 kg/m. Similarly, 5 holes were grouted in the fault zone at the drift 30 m downstream of the axis below the spillway blocks 16 to 18, where the average intake was 26 kg/m.

In the drifts below blocks 7 to 9, it was observed that the cement of the consolidation grouting carried out through the foundation at 1.5 m centres had impregnated the fault material and had set well. Hence, grouting of the fault through these drifts was not considered necessary.

After grouting of the fault, the drifts were back filled with concrete. The concrete mixed near the top of the approach shaft was conveyed through derick cranes and carried manually from the bottom of the shaft to the drift. An additive was mixed with the concrete for the top lift near the crown for minimising shrinkage. The engineering details of the drifts are given in table 3.

(i) Fault plane (ii) Fault gouge scooping (iii) 40 mm diameter inlet pipes with nipples at 1.5 m centres

FIG. 3 ARRANGEMENT FOR CONTACT GROUTING IN SHEAR KEY

12. Grouting of the fault

The grouting of the fault and adjoining crushed rock was carried out at 3.5 kg/cm² pressure through 2 m deep holes spaced at 1.5 m interval beyond the scooped area in drifts below the power dam blocks 2 to 5. Here, 145 holes consumed
Table 3
Engineering details of drifts for shear keys

<table>
<thead>
<tr>
<th>Item</th>
<th>Power dam blocks 2 to 5</th>
<th>Spillway blocks 7 to 9</th>
<th>Spillway blocks 16 to 18</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i) Total length (m)</td>
<td>182</td>
<td>85</td>
<td>135</td>
</tr>
<tr>
<td>(ii) Maximum depth below foundation (m)</td>
<td>29</td>
<td>12</td>
<td>40</td>
</tr>
<tr>
<td>(iii) Rock excavation (m³)</td>
<td>3140</td>
<td>1670</td>
<td>2175</td>
</tr>
<tr>
<td>(iv) Explosives used</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Gelatine (kg)</td>
<td>1020</td>
<td>984</td>
<td>1300</td>
</tr>
<tr>
<td>(b) Delay detonator (Nos.)</td>
<td>6715</td>
<td>9654</td>
<td>7300</td>
</tr>
<tr>
<td>(v) Cement used for concrete (tonnes)</td>
<td>1328</td>
<td>746.5</td>
<td>948.4</td>
</tr>
<tr>
<td>(vi) Cost (Rs. million)</td>
<td>3.152</td>
<td>1.197</td>
<td>1.761</td>
</tr>
</tbody>
</table>

Table 4
Cement consumption in contact grouting

<table>
<thead>
<tr>
<th>Location of drifts</th>
<th>Cement consumption in kg</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left</td>
<td>Right</td>
</tr>
<tr>
<td>(I) Power dam blocks 2 to 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Entry below block 3</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td>(ii) Prior to the grouting, water was</td>
<td></td>
<td></td>
</tr>
<tr>
<td>coming out from the top pipe at the</td>
<td></td>
<td></td>
</tr>
<tr>
<td>rate of 33 litres per minute</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) Entry below block 4</td>
<td>20</td>
<td>11320</td>
</tr>
<tr>
<td>Approach drift in block 4 and main</td>
<td></td>
<td></td>
</tr>
<tr>
<td>drifts in blocks 3 to 5 at 35 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>downstream</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(II) Spillway blocks 7 to 9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(c) Drifts for upper fault</td>
<td>210</td>
<td>215</td>
</tr>
<tr>
<td>(d) Drifts for lower fault</td>
<td>195</td>
<td>380</td>
</tr>
<tr>
<td>(III) Spillway blocks 16 to 18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(e) Drifts at 30 m downstream</td>
<td>36</td>
<td>25</td>
</tr>
<tr>
<td>(f) Drifts at 42 m downstream</td>
<td>427</td>
<td>232</td>
</tr>
</tbody>
</table>
13. **Contact grouting**

A year or more after the drifts were plugged with concrete, the contact grouting was done through embedded inlet pipes for sealing the gaps created due to shrinkage on setting of concrete. Water was first injected through inlet pipes till clear water came out from the return pipe thereby thoroughly washing the pipes to receive the grout. The grout was then injected with an initial mix of water:cement ratio of 20:1 and gradually thickened to 1:1 as required till refusal was achieved at 4 to 7.5 kg/cm² pressure. The details of grout consumption in the various shear keys are given in table 4.

The flow diagram depicting various activities of construction of a shear key is given as figure 4.

14. **Instrumentation**

Incidentally, it is hoped that the shear keys would help reducing the uplift pressures. However, the Experts Committee had suggested that holes be drilled from the various cross galleries and pressure cells be installed for uplift measurements. It is also planned to instrument the block joints to monitor movements on or along the fault planes and also movements along any of the longitudinal construction joints in the blocks resting on faults where shear keys have been provided.
References

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Report on the vibration studies carried out for the assessment of safe charges for excavation of drifts in the Kadana dam, Gujarat. Specific note No. 1404

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MISTRY J.P. AND KULKARNI V.N. (1978)
Abstract

The Warna Project, presently under construction in Sangli District, Maharashtra, consists of a 77 m high and 840 m long earthen dam across Warna river with masonry flanks in the abutments. The dam site is located in the Deccan basalt province of Peninsular Shield, where three basalt flows ranging in thickness from 35 to 45 m are present within a column of 96 m. The top of basalt flows are marked by vesicular, amygdular, tuffaceous horizons and intervening red boulder beds. A fault trending N35°W - S35°E and dipping steeply due west has been demarcated cutting across the dam axis in the river section. The permeability recorded along the fault zone ranges between 70-80 Lugeons. During grouting 702 tons (dry weight) of cement, bentonite and brick powder have been injected to treat a volume of 11280 cubic metre of foundation.

In view of the post-impoundment seismic activity in the adjoining Koyna valley and location of epicentres of some recent tremors (4<M<5) and micro-seismic earthquakes (M 2) recorded here, it is apprehended that this reservoir may also enhance the likelihood of seismic activity in this valley.

Abstrait

Le projet à Warna, à présent sous construction à District Sangli, Maharashtra, consiste d'une barrage de 77 m haut et 840 m long à travers la rivière Warna avec des flancs maçonnés dans les abutements. L'emplacement de barrage est situé dans le province de basalte Deccan de bouclier péninsulaire, où trois basalts coulent s'alignant en épaisseur de 35 à 45 m sont présent entre une colonne de 96m. Le sommet d'écoule-
ment de basalte sont marqué par des horizons vésiculaires, amygdulaires tufacés et intervenant des assises de bol rouge. Une faute s'étendant vers Nord 35° Ouest- Sud 35° Este et plongeant en pente rapide droit à l'ouest a été démarqué tranchant à travers l'axis de barrage dans la section de la rivière. La perméabilité à enregistré le long de la zone faillée rangé entre 70-80 Lugeons. Pendant le jointoiment au mortier liquide 702 tonnes (poids sec) de ciment, bentonite et poudre de brique ont été injecté à traiter un volume de 11280 mètres cubiques de fondement.

En vue de poste-fourrière l'activité sismique dans la vallée Koyna adjacent et location d'épicentre de quelques tremblements récents (4<M<5) et des tremblements de terre micro-sismiques (M 2) ont enregistré ici, il est appréhendé que ce reservoir peut aussi enrichir la probabilité de l'activité sismique dans cette vallée.

The Project

The Warna irrigation project, presently under construction in Sangli district, Maharashtra, consists of a 77 m high and 340 m long earthen dam in the river bed with masonry flanks in the abutments across Warna river in the Krishna valley. This will create a reservoir of capacity 36 T.M.C. to be utilised for irrigation of 85,425 ha of land on either banks. The dam site is located, in the Deccan basalt province of Indian Peninsula, near Chandoli village on the eastern slope of the continental divide in a terrain representing dissected upland with steep sided young valleys.

Presence of a steeply dipping fault across the Warna dam foundation and due to high permeability (15-155 Lugeons) along the contact of tuffaceous, vesicular, amygdular and intervening red-bolé and overlying basalt, below the dam seat has necessitated in foundation treatment in various forms. Extensive grouting along the fault zone, plugging of the fault zone at the upstream, sand filters and relief wells on the downstream of the earthen dam cut-off trench and increasing top and basal width of the dam have been considered necessary to reduce the path of percolation and ensure stability of the dam.

Geology

At the foundation of the dam three Deccan basalt flows ranging in thickness from 35 to 45 m are present. The basalts are fairly hard, grey sparsely to moderately porphyritic in texture. The top of these flows are characterised by vesicular, amygdular, tuffaceous horizons with intervening red-bolé beds. The foundation is cut by six sets of joints. Of these the NW-SE and the NNW-SSE trending joints are
the prominent features in the dam foundation (Fig. 1). These joints cut across the dam foundation and are interconnected by NE-3W and N65°E - S65°W trending joints. The joints are vertical to subvertical dipping due west and have been found traversing down to full thickness of basalt flows. Most of the joints are tight except a few which are open by few mm to 5 cm and filled up with clay material.

A fault with N35°W-S35°E trend with subvertical dip towards west is also present in the earthen dam portion (Rawat, 1979). The throw is of the order of 5 m with 3.5 m width. The upthrow block has a minor dip of 5° to 8° towards west (Fig. 1 and 2).

The flanking hill slopes on either bank are occupied by thin cover of soil. The soil is derived mostly from in situ alteration of basalt. Iron migration
from the laterite has given red hue to the soil. About 10–20 m thick alluvium (riverine deposit) occurs in patches along the banks of the Warna river (Fig. 2).

River Bed Fault

The existence of this fault has been proved by topographical, geomorphological studies and interpretation of the cores from drill holes drilled across the fault zone and also by an exploratory pit.

The fault zone is made up of fault gouge and brecciated material. Innumerable joints have been noticed on either sides of the fault zone for a distance of 3 m, which have developed parallel to the fault zone. Excessive water seepage through the brecciated and highly affected zone on either sides of the fault has also been noticed. The brecciated material is compact and hard and does not allow the water to seep through it. On the other hand, the fault gouge material when comes direct in contact of water, gets washed. The contact of the fault zone and the adjoining sound rock is open from few mm to 5 cm and is pervious.

With a view to determine the nature of the fault material and the extent of the fault zone in downstream (SE) and upstream (NW) directions, traverses were carried out and exploratory pitting has been done (Ravat, 1980). Interpretation of the pit dug 390 m upstream of the dam has projected 2–4 m thick cover of riverine deposit followed downward by 3–4 m of spheroidally weathered, sparsely porphyritic basalt in the upthrow block. And in the downthrow block the top is characterised by 2–3 m of thick riverine deposit followed downward by 3–4 m of weathered basalt and rest fresh, hard sparsely porphyritic basalt, which rests on tuff-breccia (Fig. 3 and 4).

The fault zone has the same characteristics as found in earthen dam portion, except its throw, which is 10 m.

The existence of the fault zone has been traced for a length of 6 km on both upstream and downstream sides of the dam by topographical difference, geomorphological features and making a few exploratory pits. And further in
the upstream of the dam (NW direction) for a length of 15 Km, the fault has been traced on the basis of ground subsidence (Fig. 5).

In addition a graded sand filter covering the fault zone alignment has been provided on the downstream of the dam. A few relief wells have also been provided to release the pressure of water seeping through the fault zone.

Three stages of curtain grout holes spaced 3 m apart with drill holes at 6 m centre to centre have been taken (Ravat, 1981). The first stage holes have been taken on the downthrow block with second on upthrow block and third stage in the centre of the fault zone. About 70 m length of the fault zone (from upstream edge of heaving zone to inclined sand filter) was considered necessary for grouting. Due to high permeability (15-155 Lugeons) along the contact of basalt and fault zone material, the depth of curtain grout holes has been lowered down to 30 m. At this depth the contact of overlying fresh, hard and underlying vesicular, amygdular, tuffaceous zone with intervening red-beds is highly pervious.

Grouting has been done stage-wise in descending manner, each stage of 5 m depth. Initially at a specific pressure (1.5 to 2 Kg/cm²) a mixture of cement and water (1:10) was used for a specific time and 10 mixtures were injected. Later on, the mixture was thickened to 1:5 and 10 mixtures were injected under a specific pressure (1.5 to 2 Kg/cm²), and time again 1:2 and 1:1 mixtures
were used till refusal. The grouting pressure varied from 6 Kg/cm² to 13 Kg/cm².

Bentonite has been added to the grout mixture for stabilizing the grout and reduce the tendency of the cement grains to settle down resulting in premature clogging of the joints or open spaces. And where there was continuous flow of the thick grout mixture at a low pressure, brick-powder has been resorted to plug the large voids. A total of 702 tons (645 tons cement + 33 tons bentonite + 24 tons brick powder) of solid (dry weight) was injected in 61 percussion holes of 1040 m length to treat a volume of 11280 cubic metre of rock.

The pre-grouting water losses in and around the fault zone varied from 70-80 Lugeons. And the post grouting water losses varied from 1-3 Lugeons with some pockets almost nil. Interconnection of the grout holes and surface leakages have been noticed. The average grout consumption per metre of rock treated has been recorded around 883 Kg/m. The maximum grout consumption in one grout hole has been recorded around 3302 Kg/m at 8 Kg/cm² pressure. The pre-grouting permeability of the bed-rock is about 52 Lugeons.

Some recent activity along Warna valley lineament.

The Warna river flows along a NW-SE lineament right from its source to Khujgaon village for a distance of 40 km. The Warna river takes suddenly southerly course. It is also interesting that the throw of the Warna fault decreases towards SE i.e. Khujgaon village (Fig. 6).

The culmination of Warna river SE course near Khujgaon shows that the Warna fault is existing upto this location. In the area upstream of the dam site the throw of the Warna fault increases from 5 to 10 m and more.

Along the fault zone evidences of recent activity, such as mole-tracks and furrows near Lotiv temple (16 km NW of Warna dam site) and ground fissures, subsidences and landslides adjoining the Lotiv and Gave villages have been found (Fig. 7 and 8). Recently, earthquakes of magnitude 4.3, 4.4,
4.7 and 4.9 between September, 1980 to December, 1981 with their epicentres in Warna valley have occurred (Fig.5).

Discussion

The Warna valley lineament identified on landsat imagery has been proved to be a N35°W-S35°E trending subvertical fault traced for a length of 40 km. The 3.50 m wide pervious and soft gougy fault zone coming under the dam foundation was selectively excavated to a maximum depth of 25 m and backfilled with rich concrete before laying the foundation. Besides, this foundation treatment a cut-off shaft was provided at the upstream of the dam and the backfilled/rock contacts were thoroughly grouted.

Evidences of neotectonic movements in the Warna valley and location of epicentres of recent and moderate (4<Ms<5) earthquake shocks in the headwater reaches of this valley possibly indicate its seismogenic nature. This fault is particularly important being located close to Koyna reservoir where sufficiently clear evidences of post impoundment seismicity has been established and a seismicity level as high as M7 has been recorded in December 1967(Koyna Main Shock) following initial filling up of the reservoir(Guha et. al., 1974).

Recently, migration of epicentres from the Koyna region to the Warna valley where a sizeable reservoir will be built up soon, has been the cause of deep concern to the geoscientists community. The entire region being
proved to be prone to reservoir induced seismicity, it is a matter of grave concern whether the Warna reservoir may aggravate the situation further from this viewpoint although reservoirs less than 100 m depth are not known to cause such effect. Monitoring of each tremors being done by a network of seismograph station all over this region, may however, be able to throw some light in future on this aspect.

Acknowledgement

The author is indebted to the Dy. Director General, GSI, Central Region, Nagpur, for scrutinising this paper. Thanks are due to Shri D. Ramachandran, Director, Engineering Geology Division and Shri N. Majumdar, Geologist (Sr.), GSI for their encouragements and useful discussions during preparation of this paper. The help from Shri S.R. Pradhan, Geologist (Sr.), GSI is also acknowledged.

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FOLDED AND DEFORMED METASEDIMENTARY ROCKS INFLUENCING THE PLANNING OF BODHGHAO HYDEL PROJECT, MADHYA PRADESH

ROCHE METASEDIMENTAIRE PLEIE ET DEFORMEE INFLUANT LA PLAN DE PROJET HYDEL A BODHGHAO, MADHYA PRADESH

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Geologist
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Abstract

The project layout comprise of tight NW-SE directional tri-folded metasedimentary rocks with axial plane fractures and planes of dislocation. Shear seams, brecciation and disintegrated zones are characteristic features influencing project planning.

Low crushing strength and deformation modulus of schist and brecciated quartzite at the narrow valley section necessitated shifting the location of gravity structure in granite-gneiss and a revision in the layout of water conductor system.

Introduction

The river Indravati forms a wide loop near Satdhar within a distance of approximately 2.5 km. It has a bed fall of 55 m from elevation 400 m at the dam site to 345 m at the tail water level of the tunnel. There is a natural fall along the river for approximately 750 m downstream of the dam site. The average surface runoff,
approximately 56.6 cumec during lean period, rises to approximately 9490 cumec during flood season. A 95 m high gravity dam in the river section across Indravati and rock-fill section on the abutments, 2.8 km long water conductor system intercepting the loop and a semi-underground power house for generation of 500 MW are under construction (Fig.1).

Lithologic and Structural Framework

Early Proterozoic formations - quartzite with breccia bed, phyllite, schist and associated intra-formational conglomerate constitute the meta-sedimentary rock group superposed over the basement of Archaean granite-gneiss in the order as follows (Fig.1).

Vein Quartz
Basic intrusive (Dykes and sills)
Phyllite, schist with conglomerate
Quartzite with breccia bed

---------Unconformity---------

Basement granite-gneiss,
amphibolite and schist

These interbedded formations show gradational contact, at times preserving primary sedimentary features such as minor ripples and load casts in the phyllite and quartzite.
Andalusite and talcose material in the schist suggest low-grade metamorphism. Thus, a sequence of partially resistant quartzite and weak rocks-phyllite and schist, from the point of view of weathering, occupy ridges and depressions. This has been caused by tectonic heights corresponding to relief heights and depressed structures to low lands. The entire sequence of meta-sedimentary rocks show unidirectional steep foliation dips (50°-60°) towards south-westerly direction. These are tri-folded into tight asymmetric to isoclinal folds, because of floor compression (Fig. 2). Two antiforms with one complimentary synform at Hitalkudam are observed in the area. These F1 folds in NW-SE direction follow the strike of formations. Cross folds (F2) in NE-SW direction superimpose the first generation folding. As a result of basement rise and compressional forces (in a belt of 2.5 km), the tri-folded metasedimentary rocks have developed close spaced jointing and axial plane shears.

Thus, two clearly discernible phases of deformation (D1 and D2) have generated two sets of cross fractures and four sets of attendant faulting and shear seams along axial planes of the folds. The penecontemporaneous intrusives also follow these deformational phases. The prominent fractures are: longitudinal sets parallel to folding and faulting and the cross sets parallel to axial plane cleavages, the details of which are given in Table 1.

Major faults and shear zones of the area are related to first phase of deformation and strike in NW-SE direction along the axis of F1 folds. Shear seams, brecciation and disintegrated zones sympathetic to F1 and F2 folds are characteristic features of the area. These make an askev of about 10-15° and 70°-75° with the dam alignment and water conductor system respectively.

---

**Fig. 2: Geological Section Across Folded Metasedimentary Belt Showing Alignment of Water Conductor System**
Table 1: Showing attitude and nature of fracture system.

<table>
<thead>
<tr>
<th>Strike</th>
<th>Dip</th>
<th>Nature of filling</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. NW-SE and NNW-SSE</td>
<td>70°NE ENE</td>
<td>Clay filled with opening upto 1.5 cm</td>
<td>4-5 cm apart</td>
</tr>
<tr>
<td>2. NNW-SSE</td>
<td>20°-35° ENE</td>
<td>Tight</td>
<td>30 cm apart</td>
</tr>
<tr>
<td>3. NE-SW</td>
<td>Near vertical</td>
<td>Clay filled with opening upto 0.5 cm</td>
<td>15 cm apart</td>
</tr>
<tr>
<td>4. NNE-SSW</td>
<td>70° WNW and ESE</td>
<td>Tight</td>
<td>30 cm apart</td>
</tr>
<tr>
<td>5. N-S</td>
<td>70° W</td>
<td>Clay filled, open upto 10 cm</td>
<td>15 cm apart</td>
</tr>
</tbody>
</table>

Significance of Deformational Features on the Layout

As a result of tight isoclinal folding, intersection of two attendant steep joint planes, shear seams etc., has developed unstable wedge blocks of maximum dimension as 2.5 m longer wedge axis, which are susceptible to slips on the free face and easy disintegration on loading. Delineation of wedge blocks and shear seams, brecciated zones and their low strength, shear properties influenced the selection of layout for this major hydel project in Indravati basin.

Dam Alignment

Three locations were chosen within a distance of approximately 500 m for comparative geotechnical studies according to favourable topography and engineering advantages. Sites A and B are located on the quartzite and associated quartz-sericite schist, while site C is located on the amphibolite and granite-gneiss. All the three alignments are 10° to 15° askew to the strike of geological discontinuities (bedding shear and fractures) which dip 45° to 50°, due southwesterly i.e. downstream. Hence loading on dam foundation is along structural discontinuities. A 25 m wide shear zone, within the quartzite and intervening site A and B, is also a major weak feature at these foundation sites (Fig. 3). Low modulus of deformation in schist and quartzite and lower shear parameter along rock/masonry and concrete contact indicated unstable conditions at site A and B for dam safety. Geotechnical characteristics and engineering properties of foundation sites are given in Table 2 and 3. It is seen that the site-A involves prohibitive foundation treatment by deep excavation to more 30 m, plugging by concrete and consolidation of foundation by grouting. The efficacy of these treatments were considered uncertain in the light of doubtful
<table>
<thead>
<tr>
<th>Site condition</th>
<th>Site-A(NN)</th>
<th>Site-B(N-4N-4)</th>
<th>Site-C(N-6N-6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topography</td>
<td>Flat valley bottom, steep abutments</td>
<td>Flat valley bottom and steep abutments.</td>
<td>Flat valley bottom and moderate abutment.</td>
</tr>
<tr>
<td>Rock type</td>
<td>Quartzite and quartz-Sericite schist</td>
<td>Quartzite and Amphibolite and granite-gneiss</td>
<td>Can also be located in the amphibolite within a depth of 25 m.</td>
</tr>
<tr>
<td>Geological discontinuities</td>
<td>a. 25 m wide shear zone dipping downstream in the dam foundation within 25 m.</td>
<td>b. 25 m wide shear zone and brecciated portions in the quartzite occupy spillway foundation.</td>
<td>a. Brecciated quartzite is partly located in the spillway bucket which shall be removed to found it on underlying amphibolite.</td>
</tr>
<tr>
<td></td>
<td>b. Shear zone and brecciated portions in the quartzite not amenable to cement grouting.</td>
<td>b. As in Site-A</td>
<td>b. Jointed foundation is amenable to cement grouting.</td>
</tr>
<tr>
<td></td>
<td>c. Disintegrated zone, &gt;12 m.</td>
<td>c. Disintegrated zone for a depth of 8-10 m.</td>
<td>c. Depth of weathered rock for 5-7 m.</td>
</tr>
</tbody>
</table>

Remarks: 1. Requires diversion structures.  
2. Provides construction facilities for diversion.  
3. The foundation on granite-gneiss requires local grouting in jointed reaches, embedding of foundation in rock by 0.5 m instead of consolidation which may not be effective.

Consolidation by grouting and low shear parameters of the foundation material. At site-B, though the dam foundation could be located on hard amphibolite within a depth of 25 m, energy dissipating structures on the downstream fell within the zone of brecciated quartzite and the 25 m wide shear zone. Obviously, the upstreammost site is the best choice for a gravity structure as it is located on hard amphibolite.
Table 3: Showing engineering properties of foundation rocks.

<table>
<thead>
<tr>
<th>Test Result</th>
<th>Quartzite</th>
<th>Brecciated quartzite</th>
<th>Schist Shear zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Crushing strength (Kg/cm²)</td>
<td>300</td>
<td>120-160</td>
<td>50-70</td>
</tr>
<tr>
<td>2. Deformation Modulus (x 10⁴ Kg/cm²) within stress range of 20 Kg/cm²</td>
<td></td>
<td>0.12-0.36</td>
<td>0.03-0.06</td>
</tr>
<tr>
<td>3. Dynamic Modulus x 10⁴ Kg/cm²</td>
<td>6-7</td>
<td>0.07-0.19</td>
<td>2-3</td>
</tr>
<tr>
<td>4. Specific gravity</td>
<td>2.08</td>
<td>1.94</td>
<td>1.85</td>
</tr>
<tr>
<td>5. Cohesion along masonry/concrete and rock (Kg/cm²)</td>
<td></td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>6. Angle of friction</td>
<td></td>
<td>40°</td>
<td>45°</td>
</tr>
</tbody>
</table>

Remark: Poor deformation character is mainly because of inherent fractures in the rock units, loading being parallel to foliation and discontinuous planes.

N.B. 1. Tests were conducted by C.M.P.R.S., Pune.
2. Rate of loading not known and this is significant for understanding deformation pattern.
3. Cyclic loading tests not carried out, hence it is not possible to analyse the actual deformation rate and its elastic behaviour.

and granite-gneiss with minimum foundation treatment and shallow depth excavation (within 7-10 m) for founding the structures. This site is away from the folded metasedimentary rock group, being sited in the basement complex. The brecciated quartzite occupying the abutments at site-C result in the dam resting on foundation of differing strength character. Hence, a rockfill dam has been designed for abutment sections to reduce the foundation treatment needed for gravity structure and providing greater length of subsurface seepage, with a 20 m deep positive cut-off. Another significant aspect as a result of deformation of the metasedimentary rocks is the absence of suitable construction material - as aggregate and dimension stone from these rock types. However, granite-gneiss occurring within a lead of 5-10 km provided suitable aggregate material and dimension stone for dam construction because of limited occurrence of deleterious minerals and adequate strength. General physical properties of granite-gneiss in the area are given in table 4.
Table 4 : Showing engineering properties of granite-gneiss

<table>
<thead>
<tr>
<th>Sample locality</th>
<th>Sp.gr.</th>
<th>Absorption %</th>
<th>Poro- sity %</th>
<th>Crushing strength</th>
<th>Abrasion %</th>
<th>Impact %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koderu Quarry</td>
<td>2.64</td>
<td>0.31</td>
<td>0.8</td>
<td>&gt; 500 Kg/cm²</td>
<td>39.8</td>
<td>7.1</td>
</tr>
<tr>
<td>+ 20 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>+ 40 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>+ 80 mm</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Manden Quarry</td>
<td>2.6</td>
<td>0.28</td>
<td>0.7</td>
<td>&gt; 480 Kg/cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>+ 20 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>49.4</td>
<td>10.6</td>
</tr>
<tr>
<td>+ 40 mm</td>
<td></td>
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<td></td>
<td></td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td>+ 80 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>Erpunde Quarry</td>
<td>2.6</td>
<td>0.26</td>
<td>0.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>+ 80 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

Water Conductor System

The 2.8 km long head race tunnel passes through a folded sequence of schist, quartzite and basic rock (Fig.2). The alignment in a general NE-SW direction, is 70°-75° skew to the trend of geological discontinuities and major shear zones for negotiating these at shortest width and maintaining rock cover over the tunnel, which varies from 20 to 80 m. The steep oblique attitude of foliation and major weak planes, intercepted by the tunnel, shall render pressure concentration mainly on the sides of the tunnel. The tunnelling media can be classified as stratified and schistose to moderately blocky and seamy rocks. The ratio between the two may vary from 50:50 to 40:60. The tunnelling condition would be similar to the Bailadila conveyor tunnel in the region. Diameter of the tunnel has been proposed by the designer as 13 m for one tunnel or 8 m for twin tunnels to accommodate the desired power draft. In view of the proposed large diameter tunnel and prevailing disposition of weak features, a possibility of wedge shaped block detachment from the sides and roof collapses as a result of loosening pressures are expected in such folded set up. Since the tunnel is off-spur, it passes through crusts of antiforms and synforms. Therefore, the tunnel would be subjected to low pressures except at the entrance sections of the fold axes, where high pressures are expected. Such areas are around 5% of the tunnel length.
Stress system is regarded as non-rotational and in horizontal plane as is indicated by the parallelism of the folds to the trend of deformed belts (NW-SE). Since the overburden depth is at most places around 40 m, the pressures as a result of residual stresses are not expected to be hazardous in the tunnel excavation.

Tectonic control of relief is demonstrated by an analysis of the direction of linear relief with structural trends and because of which the drainage follow the weak trend lines of the topography in the area. The respective lengths of seepage zones could thus be correlated with joint system and stream network. An interesting drainage characteristic is the formation of a natural bridge or drainage barrier away from Hitalkudam nala and thus minimising seepage into the tunnel.

Power House

An underground power house was proposed by designer at the toe of the outermost linear ridge across the loop (Fig. 1 and 2). The 100 m (l) x 25 m (w) x 50 m (h) size cavity is located in the highly jointed effusive rock of the meta-sedimentary group. Open criss-cross jointing shall render the high cavity roof unstable on account of loosening and detachment of rock wedges. The power house site has, therefore, been shifted into a 40 m deep open pit in the hard amphibolite and associated basic intrusive of the basement complex to minimise support system and housing it on stable foundation away from the folded set up.

Conclusions

Tightly folded metasedimentary rocks—quartzite with breccia bed phyllite and schist and associated intra-formational conglomerate over the basement of granite-gneiss and pre-contemporaneous intrusives have shown intense fracturing, brecciation and shearing and disintegrated zones as a result of stressing and cataclasis.

Deformed schist and quartzite have shown low modulus of deformation around 0.03-0.36 x 10^5 Kg/cm^2 and poor shear parameters along rock/masonry and concrete contact.

While analysing the significance of folded set up on the project layout, it is assessed that more tests on cyclic loading and reexamination of the test values, with rate of loading are necessary.

Acknowledgement

The authors express their grateful thanks to the Director General, Geological Survey of India, Calcutta, for his kind permission to present this paper. Thanks are also due to the Dy. Director General and the Director (Geology), Engineering Geology Division, Central Region, Geological Survey of India, Nagpur, for their kind scrutiny and valuable suggestions in preparation of this manuscript.
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ASSESSMENT OF FOUNDATION OF MASONRY DAM AND PART RESERVOIR RIM OF THE
HASDO-BANGO DAM PROJECT, MADHYA PRADESH

EVALUATION DE FONDEMENT DE BARRAGE DE MACONNERIE ET PART DE BORD DE RESERVOIR
DU PROJET HASDO-BANGO, MADHYA PRADESH

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Geologist (Jr.)
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Shillong, India

Abstract

Five tier engineering grade rock classification has been presented of
the granite-gneiss forming the foundation of 85 m high masonry gravity
dam. Fracture zones and associated shear seams have not been properly
consolidated during test grouting. Hence greater emphasis is laid on
selective dental treatment and drainage facility in the dam foundation.
Suggestion has also been made for designing mat foundation for uniform
loading as an alternative to deep dental treatment.

30% of the reservoir area occupying Barakar sandstone shows seepage
losses and submergence of coal under impoundment conditions. Seepage
losses through sandstone and impact on coal mining has been assessed.

Abstract

Classification de roche d'une grade de cinq loge d'art de l'ingénieur
a été présentée du granit-gnéiss formant le fondement un barrage d'une
gravité de maçonnerie de 85 mètres. Des zones de fracture et des veines
cisaillées n'ont pas été proprement consolidées pendant l'essai de jointo-
toiement au mortier liquide. Ainsi une emphase plus grande est mise au
traitement selectif dentaire et à la facilité d'écoulement dans le fonde-
ment de barrage. Proposition a été aussi faite pour projeter le fonde-
ment de natte pour un chargement uniforme comme un alternatif à traite-
tement dentaire profond.

30% de la région de reservoir occupant des grès de Barakar montre des
pertes d'infiltration et une submersion de houille sous des conditions
de confiscation. Des pertes d'infiltration par des grès et impact sur
exploitation de la houille ont été évalués.
Introduction

Hasdeo-Bango Multipurpose project includes construction of a 85 m high composite dam (rock fill section on left bank and gravity dam in river section and right bank) across Hasdeo and 27 m high earth dam across low saddles on the right bank for irrigation and power generation of 40 MW.

Geologic Setting

Archaean granite-gneiss, Proterozoic group constituting phyllite, quartzite and basic intrusives and the lower Gondwana Coal measures comprising the conglomerate, sandstone, shale and coal seams occupy the dam site and reservoir area (Fig.1). The granite-gneiss is deeply weathered, felspathic and micaceous with occasional pegmatite and kaolnised zones. The phyllite is fissile and thinly cleaved. The sedimentary sequence constitute inter-bedded coarse and fine clastics which are soft and laminated. The sandstone is friable, pervious and easily disintegrates on saturation.

Granite-gneiss shows crude foliation while the associated metamorphites strike in E-W to WNW- ESE with dips varying from 45° to 80° towards north and NNE. The sedimentary units are near horizontal. The boundary between the basement complex and the lower Gondwana coal measures, is faulted in an E-W to ENE-WSW and NW-SE directions. A number of faults within the Gondwanas, parallel to the boundary fault, shows block faulting in the area.

Floor compression and movement along pre-existing fractures in the basement have caused the block faulting and these basinal fractures being channels for basic intrusion. A major deformation of such nature has caused sympathetic fracturing and development of anti-thetic shear seams/faulting in the basement complex and the overlying sedimentary pile. Since the sedimentary pile is free from any accountable fold movement, except some local undulations north of Kema the Gondwana faults owe their origin to reactivation along pre-existing faults in the basement. Therefore the ENE-WSW and NW-SE trending faults are significant features in the area.

The area falls in zone III of the isoseimal map of India (1983-1993). One earthquake of magnitude 6.5 has been reported 150 km north of the dam site. The longer axis of the isoseismals of this earthquake shows near parallelism with the ENE-WSW directional folding and fractures in the basement granite-gneiss which formed a buttress towards east and west of the region.

Foundation Condition

Mechanical impact of heitherto described large scale tectonic movements aided by weathering has led to disintegration and decomposion of the basement complex to great depths (Ghosh, 1980). Hence avoidance of such features in the
foundation of the dam at this site is rather impossible.

Sub-surface exploration by drill holes in the dam area penetrated mainly two types of granite-gneiss - i) intensely weathered and fractured, ii) moderately fractured and stained along joints, in a total depth of 50 m. Determination of ‘RQD’ and assessment of permeability of the medium provided a basis for five-tier engineering grade rock classification (Table-1).

At the gravity dam site, rocks of category II, III and IV are common and their engineering properties (Ghosh, 1980) have shown that alteration of felspar has resulted in increase of clay and mica content and consequent decrease in the permeability values in the higher weathered profile. The gorge
Table – 1: Engineering Grade Classification of Granite-Gneiss

<table>
<thead>
<tr>
<th>Grade</th>
<th>Degree of weathering</th>
<th>Field Recognition</th>
<th>Engineering Utility</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>Granitic Soil</td>
<td>Completely altered to clayey, silty soil</td>
<td>Not suitable for founding large concrete, masonry structures, erodes easily. May be suitable for low earth dams.</td>
</tr>
<tr>
<td>IV</td>
<td>Highly weathered</td>
<td>Pulverised and decomposed to soil and Kaolin pockets with embedded corestones, crumbly patches.</td>
<td>Suitable for low earth and rockfill structures but not reliable for masonry, concrete structures, due to erratic boulders. Requires protection against sub-surface erosion.</td>
</tr>
<tr>
<td>III</td>
<td>Moderately weathered</td>
<td>Partially decomposed and highly stained.</td>
<td>Suitable for low concrete, masonry and medium earth-rockfill structures, depending upon consolidation. Doubtful aggregates.</td>
</tr>
<tr>
<td>II</td>
<td>Slightly weathered</td>
<td>Fractured and stained along joints.</td>
<td>Can be rendered suitable for all types of structures by grouting and for aggregates by selection.</td>
</tr>
<tr>
<td>I</td>
<td>Fresh Rock</td>
<td>Less fractured and tough.</td>
<td>Suitable for all civil engineering structures and aggregate material.</td>
</tr>
</tbody>
</table>

section of Haedo at the dam site is characterised by highly weathered rock profile (Category-V) on the left bank and partially to moderately weathered rock profile (Category II and III) on the right bank. In view of this erratic weathering on either banks, the decision of a gravity structure had to be modified to a composite structure with a rockfill section in blocks 1 to 9 on the left bank. This necessitated a revision in the rock classification of the remaining portion on the basis of RQD and K-values (Ghosh, 1980). A very good core recovery of over 80% has indicated a 'RQD' of less than 70% which has been seen to decrease up to 33% in some cases. However, there is a general higher 'RQD' with depth and the quality of rock improves below 10 m (Table-2).

Data on 'RQD' and core recovery percent when plotted against permeability values (Fig.2) show exponential relationship. Poor RQD has recorded higher K-values up to 56 lugeon but occasional low K-values, as in drill holes 4, 7, 25 and 26, are because of tight
### Table 2: Showing nature of rock at depth and 'ROD'

<table>
<thead>
<tr>
<th>Depth</th>
<th>Nature of Rock</th>
<th>'ROD'</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5 m</td>
<td>Highly weathered and fractured</td>
<td>Upto 30%</td>
<td>Very poor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(0.30)</td>
<td></td>
</tr>
<tr>
<td>5-10 m</td>
<td>Moderately weathered, fractured and stained</td>
<td>26-56%</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(0.56-0.26)</td>
<td></td>
</tr>
<tr>
<td>10-40 m</td>
<td>-do-</td>
<td>58-89%</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(0.89-0.58)</td>
<td></td>
</tr>
<tr>
<td>Below 40 m</td>
<td>-do-</td>
<td>75-95%</td>
<td>Very good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(0.95-0.75)</td>
<td></td>
</tr>
</tbody>
</table>

### Fig. 2: Correlation between Permeability and Recovery Percent

**Remark:**
- Zone of complete wetting
- 75% ROD in percent
- 89% Recovery
- Borehole

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Log permeability</th>
<th>Recovery %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>26</td>
<td>75</td>
</tr>
<tr>
<td>5-10</td>
<td>26.6</td>
<td>75</td>
</tr>
<tr>
<td>10-40</td>
<td>16.6</td>
<td>81</td>
</tr>
<tr>
<td>Below 40</td>
<td></td>
<td>89</td>
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<table>
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<tr>
<th>Depth (m)</th>
<th>Log permeability</th>
<th>Recovery %</th>
</tr>
</thead>
<tbody>
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<td>72.4</td>
<td>95</td>
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<tr>
<td>10-40</td>
<td>75</td>
<td>89</td>
</tr>
<tr>
<td>Below 40</td>
<td></td>
<td>79.6</td>
</tr>
</tbody>
</table>

<table>
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<tr>
<th>Depth (m)</th>
<th>Log permeability</th>
<th>Recovery %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>0.33</td>
<td>83.2</td>
</tr>
<tr>
<td>5-10</td>
<td></td>
<td>79.6</td>
</tr>
<tr>
<td>10-40</td>
<td></td>
<td>84</td>
</tr>
<tr>
<td>Below 40</td>
<td></td>
<td>84</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Log permeability</th>
<th>Recovery %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>19.7</td>
<td>82.8</td>
</tr>
<tr>
<td>5-10</td>
<td></td>
<td>54</td>
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<tr>
<td>10-40</td>
<td></td>
<td>95</td>
</tr>
<tr>
<td>Below 40</td>
<td></td>
<td>95</td>
</tr>
</tbody>
</table>
jointing and or shear seams along them, which did not allow penetration of water at pressures of \(1\) H (where \( 'H' \) is hydraulic head). As a result, the consolidation tests have also not shown encouraging behaviour in improving strength of foundation. Composite width of fractured and seamy zones intercepting the dam foundation (Fig.3) is \(0.5\) to \(12\) m and the individual seamy zones range in width from \(15\) cm to \(2.5\) m. Dental treatment of the seamy and tight fractured zones along with drainage facilities for relieving uplift, constitute the main foundation treatment at the gravity dam site.

In the remaining portion occupying open joints, consolidation grouting has been successfully attempted. A study of dam foundations on granitic and basaltic terrain (Ghosh, 1980) has shown that consolidation of foundation by cement grouting is not fully achieved because of lack of penetration beyond \(1.5\) m to \(2\) m and hence a reliance is needed for proper drainage. Since the overall strength of the rock is much higher than imposed stresses, it is felt that a change in the design is called for in such rock media by providing a reinforced mat in the fractured and seamy zones for uniform loading conditions and also providing proper drainage facilities. Thus deep dental treatment and consolidation grouting preparations can be eliminated altogether in hard granitic rocks such as on Hasdo-Bango gravity dam site and similar type hard basalt foundations also.

Reservoir Competency

The reservoir spread of the Hasdo-Bango dam shall extend over lower Gondwana sedimentary rocks - sandstone, silt-stone and shale (Talchir) and coarse, gritty, sandstone with coal seams and shale (Barakar). The coal bearing area would be approximately \(50\) sq.km. Principal reservoir characteristics include: i) seepage losses through permeable coarse and gritty Barakar sandstone, ii) submergence of coal (0.3 m to 2 m thick seams) and flooding of upcoming mines in the vicinity of lake and iii) basin tightness.

The Barakar sandstone is pervious with permeability values ranging from \(8\) to \(20\) lugeon. In the adjacent Korba coal field, the rate of percolation in the underground mine is approximately \(0.23\) m/24 hours (personal communication from Western Coalfields Limited). In view of the granular medium of sandstone, the condition of seepage is laminar and non-turbulent (Fig.4) and following Darcy's law, the quantum of seepage has been assessed as \(428\) kilo litre/day through one bank (Ghosh, 1980). In case of future coal mining in the area after impoundment, seepage losses through \(3\) m x \(3\) m mine cross section has been estimated to be \(9\) litre/day.

Thus there are two aspects - direct submergence of coal by
1. Estimated seepage loss from reservoir through Barn Kal Sandstone in one bank = 428 Kilo Ltr/day.

2. Estimated seepage loss in mine cross-section of 10 ft x 10 ft = 9 Ltr/day.
reservoir and the other because of indirect influence of impoundment away from valley vis-a-vis ground-water table and flooding of mines.

Four areas - Katma, Gach and Rampa on the left bank and Marhali on the right bank show water table lower than full reservoir level at 390 m. Such areas are expected to cause loss of storage for short duration as the full reservoir level may not be available at these sites for most of the year.

Conclusions

Fracture zone and shear seams coupled with erratic weathering have influenced varying strength character of the granite-gneiss. The need for consolidation grouting has been assessed. It is seen that selective treatment by plugging and drainage facility are more effective than consolidation. It is also necessary to design gravity dam on such foundation by embedding into rock.

Quantum of seepage through Barakar sandstone has indicated that efforts should be for extraction of coal as far as possible from the vicinity of impoundment and later a sandstone barrier of at least 500 m in extent be left as a protection against seepage.

Acknowledgement

The authors are grateful to the Director General, Geological Survey of India, Calcutta, for kind permission to present this paper. Thanks are also due to the Deputy Director General, and Director (Geology), Geological Survey of India, Central Region, Nagpur, for kind scrutiny and valuable guidance in preparation of the manuscript.

References


FOUNDATION PROBLEMS AND REMEDIAL TREATMENTS—NORTH KOEL DAM, INDIA

DES PROBLEMES DE FONDEMENT ET DES TRAITEMENTS CURATIFS—BARRAGE A KOEL NORD, INDE

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ABSTRACT

Hard, moderately jointed granite and granite gneiss, permeated with pegmatite and quartz veins belonging to Archaean constitute the foundation rock of the 68.88 m. high masonry overflow dam under construction across North Koel river.

A linear zone of basic intrusive trending N75°-80°W-S75°-80°E direction and dipping 75°-80° in upstream or downstream direction occurs in the river bed foundation blocks. Weathered at the surface, it poses problems of leakage leading to piping underneath the dam and foundation settlement.

The foundation treatments required selective excavation of weathered and decomposed material along the basic intrusive and back-filling it with concrete and special grouting to treat the sheared rock zones on either side of this weaker unit.

The bed rock topography necessitated roughening of the smooth surfaces of the numerous pot holes. One of the pot hole had an unusually large diameter of 3.25 m with nearly vertical walls down to 4.8 m depth and resembled a natural well. It was emptied of its riverine contents and back filled with concrete embedding four vertical anchor rods. A number of 1 to 2 mm wide cracks observed in the partially concreted blocks and the loosening of the bond along the rock-concrete interface were taken care of by grouting along their traces.
ABSTRACT

Dur, granit articulé modérément et granit gneiss, pénétrés avec des filons pegmatites et quartz appartenant aux constituants Archéans, la roche de fondement d'une hauteur de 68-88m de barrage maçonnerie débordé sous construction à travers la rivière de Koel Nord.

Une zone linéaire d'intrusion basique s'étendant à direction Nord 75-80° ouest-sud 75-80° Este et plongeant à 75-80° à direction en amount ou en aval se présent aux blocs de fondement dans la couche de rivière. Résistant à la surface, il pose des problèmes de coulage menant à tuyautage sous le barrage et ajustement de fondement.

Les traitements de fondement avaient besoin une excavation sélective des matériaux résistants et decomposes le long de l'intrusion basique et bourrage en arrière avec jointoirement au mortier liquide spécial et béton pour traiter des zones rocheuses cisailées à l'un des côte de cette unité faible.

La topographie de la couche rocheuse avait nécessité rendant rude des surfaces polies de nombreuse trous de pot. Un des trous de pot avait un diamètre rarement large de 3.25 avec des murs presque verticaux jusqu'une profondeur de 4-8m et ressemblait un mur naturel. C'était vide des contenus ravins et était rempli en arrière avec des enforcants à béton quatre verges d'ancres verticaux. Un nombre de 1 à 2 mm de large crevasses observé dans les blocs partiellement en béton et en détachement de l'entrepot le long de béton-rocaille d'interface avait eu soin de jointoirement au mortier liquide le long de leurs traces.

INTRODUCTION

North Koel Dam, presently under construction, is located across North Koel river about 25km west of Barawadih Railway Station in Palamau district, Bihar. The reservoir created by this dam will have a gross storage of 1,17,000 cumeecs. The regulated discharge from the dam will be utilized to generate 60 MW of hydral power. The tail water will be picked up at Mohammadganj barrage, also under construction, about 95 kms downstream of the dam, to provide irrigation facility to 1,76,000 ha of land.

The 328 m long masonry overflow dam is flanked by a 213 m long earthen dyke on the right abutment. The dam is 68.88 m, high from the deepest foundation level and is designed to pass a peak discharge of 16,000 cumeecs. The total width of the overflow blocks is 163 m. The crest level of the spillway is at 21.367.28 m and is provided with nine gates of 15 m x 15 m. The spillway has a ski-jump bucket for dissipation of the energy.

GEOLOGY OF THE DAM SITE

Hard, moderately jointed gra-
nite and granite gneiss permeated with pegmatite veins ranging from a few centimetres to a metre in thickness and quartz veins belonging to Archaean Complex constitute the foundation rock around the dam site. The bedrock has gneissic banding with the foliation planes trending in N75°-80°W-S75°-80°E direction and dipping 75°-80° both in the upstream and in the downstream. These granitic rocks are intruded by thin bands of basic rock mainly along the foliation planes.

FOUNDATION CONDITION OF THE SPILLWAY

The average foundation grade of the spillway overflow Block Nos. 6,7,8 and 9 in the river bed portion is 81.303 m. The bedrock exposed at the foundation grade in the river bed portion consists of fresh and firm granite and granite gneiss intruded by bands of basic rock and pegmatitic permeations (Fig.1). The bands of basic rocks follow the foliation direction or cut slightly oblique to it and are comparatively more weathered and decomposed at or near the surface.

A linear zone of highly decomposed clayey material, 2.5 to 3 m in thickness, was encountered in the river bed in the foundation Block Nos. 9 and 10. This zone trends in N75°-80°W-S75°-80°E direction dipping 75° to 80° in the upstream as well as in the downstream direction (Fig.1). In its upstream extension, it is a highly decomposed basic rock band maintaining more or less the same thickness and orientation. In its downstream extension it continues with much reduced thickness across Block Nos. 8,7 and 6 in the river bed foundation. In parts of Block Nos. 9 and 8, it is partially filled with calcite veins and farther downstream in Block Nos. 8(partly), 7 and 6 with pegmatite, which at the surface is weathered to highly altered whitish sticky clay. The contacts of this zone with the granitic bedrock is apparently sheared. The effect of shearing in the granitic bedrock is seen for about 3 to 4 m on either side of this zone. Towards downstream from Block No. 9, this zone tends to taper off in Block Nos. 8, 7 & 6 where its thickness varies from 15 to 25 cm, accompanied by 3 to 4 m wide zone of moderate shearing on either side.

FOUNDATION PROBLEMS

Linear weak zone :

The problems expected to be posed by this linear weak zone are:

1. By virtue of its extension in the reservoir and its continuity across the dam foundation and farther downstream in the river section, it may guide leakage through the foundation

2. The head operating on the extension of the zone in the
reservoir may lead to piping.iii. The incompetence of the material within the zone and its poor bearing capacity may pose problem of differential foundation settlement.

Bedrock topography:

Deep and uneven scouring, due to abrasion, of the granitic rock came to light when the rock surface was exposed on removal of sandy overburden from the main channel of the river bed. The bottom of the main channel was found to be dotted with a number of pot holes of circular, oblong and elliptical shapes, generally ranging up to 0.5 m. in diameter and 1 m in depth (Fig.1). An unusually large pot hole of 3.5 m diameter and 4.8 m. depth with nearly vertical walls, resembling a natural well, was encountered in Block No. 7 about 48 m. downstream of the dam axis. The pot holes were filled with riverine material consisting of coarse sand, gravels and pebbles of quartzite, granite and epidiorite.

Cracks in concreted blocks:

Development of cracks in the concreted blocks of the overflow section of the spillway in the river bed portion were noticed after the first working season. Detailed studies of the causative factors leading to the development of the cracks were undertaken to evaluate the magnitude of the problem and to evolve remedial treatment. The studies carried out included drilling of 23 shallow vertical/inclined holes in close proximity of the observed cracks and analyses of data obtained by pressure testing small segments intercepting concrete, concrete-rock interface and rock portions. Standard size concrete cube chiselled out of already laid concrete was tested for its compressive value.

Majority of the cracks had developed longitudinal to the shear key trench and transverse cracks branched off from the longitudinal ones. Diagonal cracks were few and these were deflections of both the longitudinal and the transverse cracks (Fig.2). The studies indicated that the cracks extended depth but were confined to the concrete. However, concrete/rock interface was observed to be disturbed. No field evidences indicating any settlement along the linear weak rock zone in the granitic foundation were noted. Excessive water losses were recorded in majority of the holes drilled in close proximity of the observed cracks over the segments intercepting concrete and concrete-rock interface. There was negligible or nil water loss in the foundation rock and there was no correlation between the trend of the linear weak zone, the orientation and the pattern of the cracks. Therefore, it was concluded that the cracks have not resulted due to incompetence of the foundation rock which
was intact and the damage was confined to the concreted structure only.

FOUNDATION TREATMENT:

An elaborate programme of foundation treatment was executed in the spillway blocks which included:

i. Excavation of weathered and decomposed rock along the weak zone and back-filling with concrete.

ii. Special grouting of sheared rock zone.

iii. Consolidation grouting.

iv. Roughening and chiselling of the smooth surfaces of the numerous pot holes in the river bed portion.


Linear weak zone:

The weathered and decomposed rock along the linear weak zone was excavated down to 2-3 m. depth on the guide lines of Shasta’s formula and back-filled with concrete so as to transfer the superincombent load on the competent rock on either side of this zone in foundation Block Nos. 10 and 9. The rock encountered along this zone beyond that depth was fairly fresh and marginally sheared.

In addition, a special grouting programme to treat the sheared rock zone on either side of this weak zone was also executed. This included two rows of grout holes, each approximately 4 m. away on either side of the contact. The drill holes in each row were originally proposed to be kept 3 m centre to centre spacing and inclined at angles so as to intercept the weak zone at H/5 or 12 m. depth whichever is more (Fig. 2). Before executing this programme, experimental grouting was carried out under which a total of 17 inclined holes were drilled in two rows located about 4 m. away on either side of the weak zone spaced 9 to 12 m. centre to centre. Based on the permeability characteristics of the weak zone and its groutability, additional grout holes were drilled reducing the spacing to 3 m where considered necessary. In all, a total of 920 m. of rotary core drilling was done through 42 number of holes, pressure tested and grouted under maximum pressure of 6 kg/cm² on the basis of 0.35 kg/cm² pressure per metre of rock segment. Grouting was done after the foundation had been concreted and raised 4-5 m above the general foundation grade. A total of 2833 kg. of cement was consumed under the special grouting programme of the shear rock zone in Block Nos. 6 to 10. A critical analysis of the grout intake suggests that the fracturing and shearing of the bed rock was of the higher magnitude along this weak zone in Block Nos. 9 and 8 whereas in the adjoining blocks, it possessed comparatively less factured nature and was relatively tight. Further, grouting along the
trace of this weak zone is envisaged through the vertical grout pipes embedded in the concrete along its outcrop at the foundation grade. This will be carried out at high pressures after thorough washing at a later stage when the structure is raised to about 10 m. in height.

Consolidation Grouting:

Consolidation grouting was carried in four parallel rows 6 m apart, in the overflow Block Nos. 6 to 11 of the spillway to cover a distance of 20 m. downstream of the batter line. The holes in a row were spaced 6 m. apart and staggered in plan to ensure consolidation grouting to the maximum extent possible (Fig. 2). Thus, a total of 12 Ax size rotary coring holes were drilled and grouted in each block. A total of 9150 Kg. of cement was injected during consolidation grouting in these six blocks down to an average depth of 10 m. from the foundation grade. An analysis of the consolidation grouting data indicates that the bedrock was traversed with open joints mainly in Block Nos. 11 and 8 which consumed 3,617 and 3,193 Kg. of cement respectively whereas the bedrock in adjoining blocks were comparatively less jointed with limited grout intake.

Uneven Bedrock Topography:

In order to achieve perfect bond between the smooth bedrock surfaces of the numerous pot holes and the concrete, roughening by chisel of such smooth surfaces was done. The surfaces so prepared were covered with rich cement slurry before concreting. The riverine contents of the 4.8 m deep pot hole encountered in the river section foundation Block No. 7 (Fig. 3) were removed and it was back-filled with concrete embedding four vertical anchor rods. A concrete cap was provided above its mouth in continuation to the last lift of the concrete filling the top level of the well.

Cracked concrete and concrete-rock Interface:

Based on the studies conducted, the concrete already laid over the spillway Block Nos. 6 to 10 appeared generally acceptable on strength consideration. Hence, it was not considered necessary to dismantle the cracked concrete. Instead, it was decided to be treated by an elaborate programme of grouting. Under this programme, a total of 47 Nos. Ax size vertical/inclined holes at 3 m spacing were drilled along the trace of each crack down to depths of half the thickness of the concrete at the particular drill point (Fig. 2). Ten metre long pipes, perforated to the extent equal to the thickness of the already laid damaged concrete, have been left in these holes. Low pressure grouting is to be done after the masonry is raised over the cracked concrete upto the
level of the top of the embedded pipes.

The interface between the foundation concrete and the bedrock, the bond along which had been disturbed, had been grouted while executing consolidation grouting. Wherever, the water loss at the interface was observed during the post-grouting pressure testing to be in excess of the permissible limit, additional holes were drilled and grouted and/or pipes left for subsequent grouting. These measures are considered adequate to take care of the problem arising due to the development of cracks in the concrete and loosening of the bond reducing the shear resistance at the concrete-rock interface.

CONCLUSION

The geological set-up at the North Koel Dam site posed a variety of problems, namely, leakage and consequent development of piping tendency across the dam foundation and differential settlement along the linear weak zone of basic int-

Fig. 3. Looking down into 4.8 m deep and 3.25 m diameter pothole in granitic bedrock at North Koel Dam Site, Bihar.
Invasive. The development of cracks in the concreted blocks of the spillway posed a structural problem and was probed in detail to establish that the foundation rock had remained undamaged. Geological set-up and foundation conditions necessitated formulation and execution of special grouting of the sheared rock zone on either side of the linear weak zone by inclined grout holes. Likewise, an elaborate grouting programme had to be executed to take care of the problem arising due to the development of cracks in the concrete and reduction of the shear strength at the concrete-rock interface caused due to loosening of the bond at the interface.

The effective solutions to these foundation problems were evolved by the multi-disciplinary studies and interaction of the Geologists and Engineers at the various stages of execution of the project.

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REFERENCES


DELINEATION OF A BURIED RIVER COURSE BY GEOPHYSICAL METHODS AT NARAYANPUR DAM SITE, KARNATAKA (INDIA)

DELINEATION D'UN COURS DU FLEUVE ENTERRE PAR LES METHODS GEOPHYSIQUES A L'EMPLACEMENT DU BARRAGE DE NARAYANPUR, KARNATAKA (INDE)

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ABSTRACT

The construction of the right flank dyke of the earthen portion of Narayapur dam on river Krishna has been completed. Excessive seepage of water has been observed through the drainage arrangement provided below the dyke. It is feared that on impounding of the reservoir the seepage may increase posing danger to the dyke. The bedrock in the area is granitic gneiss/pink granite overlain by overburden. A geophysical investigation was carried out there with a view to determining the probable cause of the seepage so that suitable remedial measures could be adopted.

A dolerite dyke (geological) is seen exposed at few places. A probable cause for the excessive seepage could be the subsurface continuation of this impervious dyke in the valley which may be a hindrance to the flow of groundwater and thus responsible for the observed seepage. However, magnetic survey carried out there indicated the non-continuation of the dolerite dyke.

Seismic refraction and electrical resistivity methods were then employed so as to get information about the different subsurface layers and the bedrock configuration which might throw light on the cause of excessive seepage. For seismic refraction work, in addition to the usual inline reverse shot profiles a modified technique was adopted which gives continuous bedrock configuration along a line. This new technique involves summation of arrival times at a common refractor from two shot points and subtraction of reciprocal time below the shot points. The seismic and electrical results revealed a general depression in the bedrock topography below the dyke which extends both towards downstream and upstream sides. This depression in the bedrock topography indicates the possible existence of a buried channel which might be the old river course of the river Krishna and the pervious material of this might be responsible for the observed seepage. Remedial measures in the portion of bedrock depression were suggested.

ABSTRACT

La construction de la digue du flanc droit de la partie de terre du Narayapur sur le fleuve "Krishna" a été complétée. Le suintement excessif de l'eau a été observé par l'arrangement du système d'égouts
prévu sous la digue. On craint qu'après l'endiguement du réservoir le suintement puisse augmenter ainsi le danger posant à la digue. La roche de fond dans la région est le gneiss granitique/granit rose couvert de la surcharge. L'étude géophysique a été exécutée la en vue de déterminer la cause probable du suintement pour qu'on puisse choisir les mesures réparatrices.

A quelques endroits on voit exposée une digue de dolérite (géologique). Une cause probable du suintement excessif pouvait être la continuation de la sous-surface de cette digue imperméable dans la vallée ce qui peut être un obstacle à l'écoulement d'eau sous-terre et ainsi responsable aux suintement observés cependant, le levé magnétique exécuté a indiqué le non-continuation de la digue de dolérite.

Les méthodes de la réfraction sismique et de la resistivité électrique ont été ensuite employées afin d'obtenir le renseignement sur les différentes couches de la sous-surface et sur la configuration de la roche de fond qui pourraient jeter de la lumière sur la cause du suintement excessif. Pour le travail de réfraction sismique, en outre les profils usuels en ligne des points inverses des charges (d'explosif) a été adaptée une technique modifiée qui donne la configuration de la roche de fond continue le long d'une ligne. Cette nouvelle technique engage l'addition des temps pris pour l'arrivée au réfraction commun de deux points des charges (d'explosif) et la soustraction du temps reciproque au-dessous des points des charges (d'explosif). Les résultats sismiques et électriques ont mis à jour un creux général dans la topographie de la roche de fond au-dessous de la digue qui étend à la fois vers la côte d'aval et la côte de montant. Ce creux dans la topographie de la roche de fond indique l'existence possible du canal enterré qui pourrait être le vieux cours du fleuve, "Krishna" et la matière perméable de ceci peut être responsable pour le suintement observé. Les mesures réparatrices la partie du creux de la roche de fond ont été proposées.

INTRODUCTION

Narayanpur dam is one of the two dams contemplated in the Upper Krishna Project complex in Karnataka State (Fig.1). The construction of the dyke portion of the earthen dam on the right flank has been completed. Water through the drainage arrangement provided below the dyke is collected in the six interconnected gauge wells located along the downstream toe of the dyke (Fig.2). Excessive seepage of water has been observed, which has necessitated pumping of the water from the lowest level gauge well. It is feared that on completion of the Narayanpur dam the seepage may increase further due to the impounding of the reservoir and may pose danger to the dyke structure. A geophysical investigation was carried out both on the upstream (u/s) and downstream (d/s) of the dyke with a view to determining the cause of high seepage so that suitable remedial measures could be taken.
FIG. 2: Plan showing dyke portion of Narayanpur dam along with geophysical results
the gauge wells located on the d/s toe of the dyke as well as the few boreholes drilled in the area. The three continuous seismic profiles taken on the upstream side are marked as AA', BB', and CC' with shot points as S1, S2 etc. (Fig.2). The reverse profiles with their shot points SH1, SH2 are also shown in this figure. Similarly the shot points for the reverse profiles taken on the downstream side, (most of them were along the line DD') are marked as S1D, S1R etc. in the same figure.

At each shot point ground level (GL) together with the bedrock level (RL) as estimated by seismic results are marked in the Fig.2. The different depths to bedrock (D) (difference between ground level and the bedrock level) are also indicated at the respective locations.

The places on the upstream side where electrical soundings were taken are shown as E1, E2 etc. along with their results (Fig.2).

Figs. (3) and (4) show in section the typical bedrock topography along the seismic lines AA' and DD' respectively as obtained from seismic and electrical results.

DISCUSSIONS OF RESULTS

It is seen from the seismic results that both towards upstream and downstream of the dyke the depths to bedrock (thickness of overburden) vary from about 4 to 13 m (Fig.2). The depths to bedrock obtained at different electrical sounding stations (most of which were situated near the seismic shot points) compare reasonably well with those obtained by seismic methods.

The results along the continuous seismic line AA' taken 50 m u/s of the dyke reveal that the thickness of overburden which is only about 4 m near point A' first goes on increasing along this line as seen at shot points S2, S3 etc. and is maximum at S5, S6 (located at distance of about 200 m and

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SCALE:

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FIG.3: Section along Seismic Line AA' showing estimated Bedrock line
The bedrock in the area is granitic gneiss/pink granite overlain by overburden consisting of black cotton soil, whitish yellow and red clay. It is suspected that the accumulation of water in the gauge wells may be due to the existence of a subsurface geological dolerite dyke (intrusion) acting as a barrier to the groundwater flow or due to the excessive seepage because of the presence of an aquifer zone.

METHODS EMPLOYED

Geophysical methods using magnetic, seismic refraction and electrical resistivity techniques were employed for these investigations so as to get information about the different subsurface layers and the bedrock topography which might be helpful in determining the cause of the excessive seepage.

(1) Magnetic Method:

The magnetic method is based on the fact that the different rocks exhibit different magnetic susceptibilities. Local deviations of magnetic field from their normal values i.e. magnetic anomalies, in either total or its vertical or horizontal components are measured by sensitive magnetometers and these are interpreted in terms of local geology.

(11) Seismic Refraction Method:

Seismic refraction method utilizes the existence of recognizable difference in the compressional wave velocities with which elastic waves are transmitted. The arrivals of the elastic waves usually produced by the explosion of gelatine charge placed at the bottom of a shot hole, are picked up by different geophones (detectors) placed on the ground at regular intervals in line with the shot point. The seismic arrivals are then amplified and recorded. The travel times of these arrivals are measured from the record on which a time scale is maintained and a time-distance graph plotted.

In the usual in-line reverse profiles two shots are taken one at each end of the geophone spread (to take into account the dip of the formation). From the slopes of the straight line segments of the time-distance graph the compressional wave velocities corresponding to different subsurface layers are calculated. The thicknesses of the different layers at each of the two shot points are calculated either by time intercept method (Nata, 1954) or by the critical distance method (Heiland, 1940).

However, for applying seismic refraction method to such problems as encountered at Narayanpur it is vital that the velocities are more accurately determined and the depths calculated at many places along a line rather than at only two shot points located at the two ends of the profile. Therefore, a different technique (Sjögren, 1980) was adopted for this problem. In this method a number of shots are recorded both inside and outside of the geophone spread which is of small length (geophones are usually kept at 5 m or 10 m intervals). To obtain a continuous seismic profile the geophone spread is gradually moved so that each new spread overlaps a part of the previous one. This technique involves summation of arrival times from a common refractor at a particular geophone from the two shot points on the opposite sides of that geophone and subtracting the shot point to shot point travel time which is the same in the either direction of travel between those shot points and is known as 'reciprocal time'. The time thus obtained refers only to the geological conditions in the vicinity of the geophone and varying conditions outside this region are automatically eliminated resulting in more accurate determination of depths at all shot points as well as below each geophone (except those shot points which are taken far away from the spread for getting bedrock velocity) (Sjögren, 1980).

The depth determination by this technique is proceeded by detailed velocity determination of
different subsurface layers. In this method of velocity determination at each geophone position, the travel time from the shot point on one side of the geophone is consistently subtracted from the travel-time of the equivalent shot point on the other side of the geophone spread. This is done for each geophone position along the spread and the value plotted yielding true velocities (Hawkins, 1961 and Sjögren, 1972).

(iii) Electrical Resistivity Method:

The electrical resistivity method is based on the principle that different substrata offer different resistance to the flow of electric current. In this method, current is passed into the ground by a pair of current electrodes and the resulting voltage measured by a pair of potential electrodes. The apparent resistivity is derived from the ratio of the measured voltage drop to the current passed. The apparent resistivity will vary as the position and spacing of the four electrodes change and this depends on the electrical properties of the subsurface i.e. on the true resistivities and distribution of the subsurface materials. Various electrode configurations can be employed. Schlumberger configuration was used for these investigations. In this set-up the two potential electrodes are placed between the two current electrodes such the separation between the potential electrodes is much less than the current electrodes separation. The apparent resistivity \( (\sigma_a) \) is determined by the relation

\[
\sigma_a = \frac{\pi}{2l} \left( \frac{L^2 - l^2}{V} \right) \frac{V}{I}
\]

where 'L' and 'l' are half the current and potential electrode separations respectively, 'V' voltage drop and 'I' the current passed.

With the electrical station at the centre, successive values of apparent resistivities are measured by progressively increasing the distance between current electrodes or that of potential electrodes, but only one at a time, during the course of measurement. A curve is plotted between the current electrode separation and apparent resistivity and this curve is matched with theoretical curves drawn for a number of discrete resistivity and layer thickness ratios. By such matching the thicknesses of different layers and also their true resistivities are obtained.

DETAILS OF INVESTIGATIONS

On the downstream side near the main road, granite rock having a dolerite intrusion (geological dyke) is exposed. It was felt that the subsurface continuation of this dolerite intrusion parallel to the dyke portion of the dam might be responsible for the accumulation of water in the gauge wells as dolerite being a impervious rock may act as a barrier to the flow of groundwater.

Dolerite being highly magnetic its intrusion in granite was expected to be indicated in the magnetic traverses taken there by a magnetometer. However, the results of these traverses did not reveal any magnetic anomaly ruling out the possibility of any such dolerite barrier.

On the downstream side of the dyke 13 seismic reverse profiles were taken. Most of these profiles were aligned along a line parallel to the dyke line about 25 m away from d/s toe line. A few such profiles were taken at right angles to this line also.

Towards the upstream side continuous profiling was done along three lines parallel to the dyke line at a distance of 50, 150 and 300 m from the u/s toe line. In the two lines nearer to the dyke the distance between successive geophones was kept 5 m while on the farthest line this distance was kept 10 m and as such the depths were determined at less number of place there. A few in-line reverse profiles almost right angles to these lines were also taken in this area.
225 m respectively from the point A) and then it starts decreasing as seen from the estimated depths at S7, S8 and S9 (Fig.2). This means that there is a depression in the bedrock topography along this line as the ground level all along this line is almost same. The bedrock profile goes down first gradually and then rises. This depression in the bedrock topography is clearly seen along the section AA' in Fig.3 drawn from seismic results and borehole data. The bedrock falls from RL 482.0 at S1 to about RL 473.5 at S5 & S6 and then rises to RL 480.0 at S9. This depression in the bedrock is more pronounced in the area between S3 and S8 i.e. between 150 to 300 m from the point A. The results of the continuous seismic lines BB' and CC' taken further away from the dyke also show the same trend in bedrock topography i.e. bedrock level first comes down and then rises as seen from the results of shot points taken on or near these lines.

Now coming to the downstream side of the dyke line, where only reverse seismic profiles mostly along the line DD' were taken, it is seen that here too the bedrock level first goes down starting from the extreme end D of the line and then it again starts rising after the shot point S7D (Fig.2) located at about 340 m from it. Fig.4 shows in section the bedrock topography there.

Thus from the results of the survey it is seen that the bedrock which is quite shallow at the points A, B, C & D of lines AA', BB', CC' & DD' respectively, gradually becomes deeper but after a certain distance along each line it again starts becoming shallower (Fig.2).

This general depression in the bedrock topography indicates the possible existence of a buried channel. The curve ZZ' (Fig.2) indicates the trend of the buried channel as the bedrock level is about the lowest along this curve.
It is quite likely that the nearby river Krishna in the past was flowing along this bedrock depression.

It was inferred that this depression in the bedrock may be responsible for the excessive seepage observed on the d/s toe of the dyke. This buried river course is probably acting as an outlet for the water because the extra thickness of overburden here may be of permeable strata i.e. sand, jointed or weathered rock etc. It was therefore recommended that suitable remedial measures such as grouting or providing an impervious curtain or cut off may be taken before impounding of the water in the portion of the dyke where the depression in the bedrock has been delineated.

CONCLUSIONS

The geophysical investigations carried out at the dyke portion of the Narayanpur dam revealed the existence of a buried river course. The delineation of this old buried channel was made possible by adopting a suitable seismic refraction technique which involves summation of arrivals times from two shot points at a common refractor and subtraction of reciprocal time below the shot points.

The excessive seepage observed in the downstream toe of the dyke was attributed to the buried channel along which the nearby river Krishna might be flowing in the past. It was recommended that suitable remedial measures be adopted in the portion of dyke where this buried channel exists.

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PRELIMINARY ENGINEERING GEOLOGICAL INVESTIGATIONS OF THE PROPOSED WADI NAMAN UNDERGROUND DAM, SAUDI ARABIA

INVESTIGATION PRELIMINAIRE DE LA GEOLOGIE DES ARTS ET METIERS DU WADINAMAR PROPOSE SOUS BARRAGE DONNE, L'ARABIE SEOUDEITE

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ABSTRACT

Wadi Naman is one of the main valleys that flow westward toward the Red Sea. It is considered as an important domestic groundwater supply to the Holy City of Makka. The existing structures consist of a series of wells penetrating the alluvium and intersecting its freshwater before it gets mixed up with Wadi Uranah brackish water approximately 20km east of the Holy city of Makka.

The idea of underground dam started with the view of creation of an underground reservoir without disturbing the surface facilities and structures. A two phase site investigation program was initiated including the selection of four alternate sites along which soil investigation, drilling and/or geophysical surveying were conducted. Since there was not much variations in the soil characteristics between these sites, the most suitable one was selected according to the valley geometry, geomorphology, structures and land use. The study of the reservoir area included an aerial photographic survey, potential seepage problems, aquifer parameters, existing groundwater levels and water chemical quality.

The second phase of this investigation included the effect of the dam on the aquifer in view of the information obtained in the first stage. Parameters such as changes in storage and anticipated discharge at the dam site were discussed. A water management plan was also reviewed.

ABSTRACT

Le Wadi Naman est l'une des vallées principales descendant vers l'Ouest en direction de la Mer Rouge. Il est considéré comme une importante source d'eau minérale pour la ville sainte de La Mecque. Les structures existantes consistent en une série de puits pénétrant les terrains alluviaux et coupant leur nappe aquifère avant qu'elle ne se mêle à l'eau saumâtre du Wadi Uranah, à environ 20 km à l'est de la ville sainte de La Mecque.

L'idée d'un barrage souterrain a été avancée en vue de la création d'un réservoir souterrain qui ne dérangerait pas les installations et structures de surface. Un programme d'étude de site en deux phases a été élaboré, qui comprend la sélection de quatre emplacements possibles, sur lesquels furent entreprises des études de sol, de forage et/ou géophysiques. Étant donné qu'il n'existait pas de variation notable dans les caractéristiques des sols des différents sites, la sélection du site optimum s'est opérée selon les critères de géométrie, de géomorphologie, de structure et d'utilisation du terrain. L'étude de la zone du réservoir a porté sur un relevé par photographie aérienne, les problèmes potentiels d'infiltration, les paramètres de la nappe aquifère, les niveaux existants d'eau minérale et les qualités chimiques de l'eau.
La deuxième phase de cette enquête a porté sur les effets du barrage sur la nappe aquifère d'après les informations recueillies lors de la première phase. Les paramètres tels que les variations de quantités stockées et la déperdition prévue au niveau du barrage furent débattus. Un plan de gestion de l'eau a également été élaboré.

INTRODUCTION

Wadi Naman is one of the main valleys that flow westward, originating from the scarp mountains, passing east of the Holy City of Makka and ending at the Red Sea coast. Fig. 1 shows Wadi Naman catchment area modified after Gamman (1979). Wadi Naman occupies an area of 710 km² and is considered as an important domestic ground water source for the Holy City. The existing structures consist of a series of wells (Al Zubaydah) penetrating the alluvium and intersecting the freshwater flow before it gets mixed up with Wadi Uranah brackish water, approximately 20 km east of the Holy City. The pumped water which is estimated as 10,000 m³/day is collected in a gallery and is transmitted to the Holy City for domestic use.

The growing demand for water resources especially during Haj time when over 2 million Moslems visit the Holy City, warrants the need to store Wadi Naman's water. The idea of constructing a surface dam was not feasible since the runoff in the valley is intermittent and since the main highway that connects the Western province of Saudi Arabia and the Holy City with the Southern, Central and Eastern Provinces, is located within the valley. The idea of constructing an underground dam then came into the picture where freshwater can be stored without demolishing any civil structures and which causes the minimum degree of evaporation and surface pollution. Few similar dams were constructed in the world (Matsuo and Kono, 1968 and Matsuo, 1975).

The purpose of this paper is to present a preliminary geotechnical investigation of four potential sites for the underground dam and a reconnaissance study on the geometry and geologic structures of the future reservoir which extends approximately 3 km upstream of the proposed dam site.

PHYSIOGRAPHIC AND GEOLOGIC SETTING

The studied section of Wadi Naman ranges in elevation between 336-388m above sea level at the valley floor to a maximum of 727m on the northern flank and of 427m on the southern flank. The valley attains a width of 1000 to 1300m with alluvial deposits ranging between 20-70cm in thickness. Along the banks of the valley, alluvial terraces, fans and scree are present and are generally cultivated using Wadi Naman groundwater reservoir. The northern flanks slope at an angle of approximately 18° while the average angle of slope of the southern flank is 21°. The downstream slope of the valley in the studied section is approximately half a degree.

The studied area is covered by a unit of relatively old amphibolite (Smith, 1980) intruded by plutons of granodiorite and diorite complex. The amphibolite outcrops in few places along the Wadi flanks and is highly weathered. The granodiorite is widely spread in the northern flank of the valley, while the diorite complex dominates the southern flanks. Both units are moderately to slightly weathered and are intersected by dykes of acidic and andesitic compositions.

Alluvial deposits covering the valley floor and forming the principal aquifer consist mainly of sand and gravel. Some low water-bearing zones exist and are marked by alluvium rich in fines.

Structurally, the area is intersected by several faults and shear zones trending either east-west parallel to the main valley, or north-south to northwest-southeast crossing the valley at several locations. Shear zones may range from few meters to 100m in thickness. Joint pattern, in the studied area, was examined both along the proposed dam axes and along the reservoir flanks. Generally three sets of joints were identified with spacing ranging between 20 and 65cm, apertures ranging between 1 and 300mm and are moderately weathered.

FIRST PHASE OF INVESTIGATION

This phase includes the preliminary investigation of four alternate sites for the proposed underground dam, the selection of the most suitable site and the investigation of its future reservoir area.

Dam Sites

The four proposed dam sites are located within a section of the valley having a length less than 1.5 kilometers (Fig. 2). The investigation along sites A, B and C was carried on by Sogreb (1979) and re-investigated by the writers, while site D
Fig. 2. Physiography and Documents of Wadi Naman Proposed Under Ground Dam
was proposed and investigated by the writers.

The alluvial deposits that occur in the four sites mainly consist of moderately graded coarse sand and gravel (Table 1) with average specific yield of 20% and average transmissibility of $5.5 \times 10^{-2}$ m$^2$/sec (Italconsult, 1969).

Table 1. Grain Size Analysis.

<table>
<thead>
<tr>
<th>Percentage of Alluvium</th>
<th>Grain Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Silt and Clay</td>
</tr>
<tr>
<td>8, 10</td>
<td>Medium sand</td>
</tr>
<tr>
<td>25, 30</td>
<td>Coarse sand</td>
</tr>
<tr>
<td>25, 30</td>
<td>Fine gravel</td>
</tr>
<tr>
<td>15, 20</td>
<td>Coarse gravel</td>
</tr>
</tbody>
</table>

Since the alluvium can be considered to be homogeneous within this 1.5 km stretch of the valley, no preference can be concluded based on the soil characteristics. Similarly the fault pattern that exist on the northern flank (Fig.3) is expected to equally continue underneath the four sites. Besides, joint survey showed no serious changes in joint orientations or frequencies on the valley flanks at the four sites. Therefore, structures might not also be considered as a decisive factor to prefer one site over the others.

Although site D indicates the location of the narrowest section of the valley (Fig. 2), it also shows the location of the maximum thickness of alluvium in the area. Resistivity surveys along sites A, B and D, seismic refraction survey along site C and drilling operations along site A indicate the presence of 10 to 20m of dry alluvium with resistivity of 500 to 2200 $\Omega$-m and seismic velocity of 0.4 to 1 km/sec followed by 10-50m of saturated alluvium with resistivity of 17 to 42 $\Omega$-m and seismic velocity of 1.5 to 2 km/sec. The thickness of the weathered rock zone ranges between 6 and 12m with resistivity $>1000$ $\Omega$-m and seismic velocity ranging between 4.6 and 5.1 km/sec. Fig.4 shows the determined cross sections along the four proposed sites and Table 2 shows the estimated cross-sectional areas for these cross sections.

The thickness of the weathered rock zone and consequently its cross-sectional area is known only for site A through three holes drilled to a maximum depth of 50m and penetrating at least 19m in the bedrock. It is feasible to assume equal areas of the weathered rock zones for the other three sites.

Site D, being the farthest site downstream is relatively closer to the Holy City of Makka and dams a relatively larger reservoir ($4.34 \times 10^6$ m$^3$). However, it possesses the largest cross-sectional area of all the other sites ($31.7 \times 10^3$ m$^2$). Site A on the other hand, possesses the smallest cross-sectional area ($26.9 \times 10^3$ m$^2$), dams a reservoir of $4.06 \times 10^3$ m$^3$ and is more closer than sites B and C to the Holy City.

Based on the previous considerations, sites A and D are considered to be the most feasible sites for the underground dam constructions. It now boils down to an economic controversy. If the dam is constructed on site D, it will cost 18% more than that constructed on site A (based on cross-sectional areas). At the same time, the reservoir for the dam constructed on site D is only 7% larger than that on site A. Therefore it might be more feasible to consider site A as the most suitable site.

Reservoir

An aerial photographic interpretation and field mapping of the reservoir area have been conducted in order to identify different soil and rock types, geomorphic features and existing geologic structures.

The main reservoir is enclosed within the valley alluvium while the weathered bed rock may also contribute to the reservoir capacity. The presence of fractured and shear zones in the bed rock was interpreted by extrapolating the surface structures and was proved to exist from drilling data. These zones may cause water seepage below the proposed dam or across the wadi flanks to nearby valleys. Several problematic spots within the reservoir area have been identified and should be grouted. Grouting of the weathered bed-rock zone underneath the proposed dam site should also be conducted.

The reservoir area was also surveyed using the resistivity method in order to outline the geometry of the basement below the alluvium and to draw the water table map. Fig.3 is a combined map showing the contours of ground elevations, water table elevations and basement rock elevations.

In the reservoir area the gradients of the ground surface ($9 \times 10^{-3}$), the bedrock surface ($1.5 \times 10^{-3}$) and the groundwater level ($9 \times 10^{-3}$) continue regularly westward without any unusual obstacle.

Borings drilled in the reservoir area were used to verify the resistivity survey data, to obtain the aquifer constants and to sample water for chemical quality.
Fig. 3. Engineering Geologic Map of Wadi Na'man Proposed Underground Dam
Fig. 4 shows the cross-sections along the proposed sites as predicted from geophysical surveys.
Table 2. Cross-sectional areas of the four proposed sites

<table>
<thead>
<tr>
<th>Site</th>
<th>Area of dry alluvium (m²) x 10³</th>
<th>Area of saturated alluvium (m²) x 10³</th>
<th>Total cross-sectional area (m²) x 10³</th>
<th>Area of weathered rock zone (m²) x 10³</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>16.9</td>
<td>10.0</td>
<td>26.9</td>
<td>1.9</td>
<td>Estimated by resistivity and drilling</td>
</tr>
<tr>
<td>B</td>
<td>16.3</td>
<td>12.7</td>
<td>29.0</td>
<td>unknown</td>
<td>Estimated by resistivity</td>
</tr>
<tr>
<td>C</td>
<td>14.8</td>
<td>13.0</td>
<td>27.8</td>
<td>&quot;</td>
<td>Estimated by seismic refraction</td>
</tr>
<tr>
<td>D</td>
<td>17.3</td>
<td>14.4</td>
<td>31.7</td>
<td>&quot;</td>
<td>Estimated by resistivity</td>
</tr>
</tbody>
</table>

SECOND PHASE OF INVESTIGATION

This phase includes the effect of dam construction on the existing aquifer in view of the information obtained from the first phase of investigation.

In case of an underground dam at site A with its crest situated close to the ground surface, the aquifer will extend to a distance of 2.9 km under natural flow condition. Assuming an average reservoir width of 700m, and specific yield of 20%, the additional storage capacity of the reservoir will be 4.1 x 10⁶ m³.

The present conditions show that water table gradient at site A is 9 x 10⁻³. The estimated groundwater discharge for a cross-sectional area of 10,000 m², water table condition 20m below the ground surface and permeability of 3.2 x 10⁻³ m/sec is 288 ltr/sec or a daily discharge of 24.9 x 10³ m³. During the rainy periods, the water level becomes only 15m below the ground surface, the saturated cross-sectional area increases to 12.9 x 10³ m² and the rate of subsurface discharge becomes around 32.0 x 10³ m³/day. This amount of discharged water together with Ain Zubaydah production should represent the total subsurface discharge of Wadi Naman. In other words, if we assume that the average daily subsurface discharge is 28.5 x 10³ m³ and the average daily Ain Zubaydah pumping is 10.0 x 10³ m³, the average total subsurface flow in Wadi Naman will amount to 38.5 x 10³ m³/day. It will be possible, with the presence of the underground dam, to stop this flow, to rise the water table to become closer to the ground surface and to increase the storage capacity of the reservoir. If the reservoir is well managed, it will be possible to double the existing production of Ain Zubaydah.

CONCLUSIONS

It is feasible to adopt the idea of subsurface dams for an arid country like Saudi Arabia. Wadi Naman makes a good site for an underground dam since the surface civil structures can't be demolished and because of the existence of thick potential reservoir sediments. If an underground dam is to be constructed on site A, it will stop an average subsurface flow of 38.5 x 10³ m³ per day, an amount that may greatly increase the existing daily production if it is correctly managed.

REFERENCES


STUDY ON THE PHOTOGRAPHS OF AERIAL REMOTE SENSING FOR THE EVALUATION OF GEOLOGICAL ENVIRONMENT IN THE ENGINEERING CONSTRUCTION

L'ETUDE SUR LES PHOTOS TELEPHOTOGRAPHIQUES AU VOL AERIEN POUR L'EVALUATION DES ENVIRONNEMENTS GEOLGIQUES DANS LA CONSTRUCTION DES TRAVAUX

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ABSTRACT

This paper deals with the evaluation of geological environment for the exploitation of water energy resources using the data of the aerial remote sensing in the mountain-canyon area. The regional geologic structure and it's formation-development historical analysis, the regional tectonic activities of crust, hydrogeological structure, landforms, valley slope stability and the characteristics of rock mass structure on the concrete construction area have been comprehensively researched, thus, the change tendency of the geological environment can be predicted. The examples, used in this paper, are results of the aerial remote sensing research. The regional geologic tectonic map and stability zoning map have been made, further, the application of engineering construction may be selected and evaluated. The corresponding engineering geologic map has been made.

ABSTRACT

L'idée générale concernant l'évaluation des environnements géologiques en cas de la construction dans une région montagneuse a été décrite, en indiquant le moyen d'utilisation des photos aériennes. L'auteur pense qu'en basant sur l'analyse de la structure régionale et de sa formation et son évolution, une étude synthétique doit être aménée sur l'activité, la structure hydrogéologique et la morphologie de l'écorce régionale, ainsi que sur la stabilité des talus des vallées et sur les caractéristiques structurales des masses au site. Elle a pour but de prédir la tendance de l'évolution des environnements géologiques. Les exemples expliqués sont tirés des résultats de notre interprétation des photos. La carte de la tectonique régionale et celle de la stabilité dans les zones différentes ont été construites avant de choisir la zone valable à la construction. L'évaluation s'est ainsi faite en fournissant une carte de la géologie de l'ingénieur.
The evaluation of the suitability of geological environment for the engineering construction and the prediction of the tendency of the environmental change after the construction are considered as a scientific basis for the planning and projection of the territory. The data obtained by remote sensing is of wide use and can provide a good foundation for such kind of researches. This paper deals with the ways and content of the engineering geomechanics in the researches and describes an example of research works in the early stage for the development of water energy resources on the river Ya Long-jiang in Sichuan Province.

The facts have proved that it is very difficult to make a field investigation in the mountainous area with deep canyons, and a satisfactory effect can be gained using the remote sensing data and comprehensively analysing the field investigation and related recorded data.

1. Evaluation of geological environment for the engineering construction using the aerial remote sensing data

The researches of engineering geomechanics of the geological environment are to probe into a formation-development history of geologic mass on the basis of regional geological structure, to study its present status and to predict its behaviour in future, i.e. to evaluate specially the engineering geologic environment. One has to start with regional geologic background, then to analyse the fundamental factors, determining the engineering geologic environment, and its orogenic connexion; further, to evaluate regional crust stability and engineering geological characteristics of the specific area; to predict environmental change tendency and to give the conclusions for the environment and mineral resources potentiality.

The researches are carried out in three stages, as shown in Fig.1 (on p.III.120).

I stage To obtain data and to explain geologic background

First, to collect geologically recorded data, to carry out field investigation and to explain photographs of the Earth Resources satellite. The purpose of this work is to understand the regional geological structure and principal engineering geologic problems, to establish the lithological and stratigraphical succession and explanatory marks, to put forward the specialized requirement for the whole projection of an experiment, of aerial remote sensing.

The focus of the work at this stage is to explain the given photographs. The principal contents are the geological structure, including the trace of neotectonic movements, geomorphic and physical geological phenomena. Undoubtedly, the visual explanation is considered as a foundation, but it should be supplemented by a series of methods, strengthening explanation, or the accuracy obtained results can be raised by the using electronic computer and automatic explanation. Even then, the photograph explanation should not be isolated. So long as the attention is paid to the field investigation and the given geological, geographical and geophysical data can be referenced, in the abundant information of photographs an accurate explanation and effective utilization may be attained.

II Stage Analysis of the fundamental factors for engineering geological environment

On the basis of the photograph explanation and comprehensive analysis of related data, the factors and its organic connexions should be researched respectively.

1) Large fault

To study the spatial distribution, geologic, geophysical, geothermal fields and some geographical characteristics, to probe into its formation development process; to research its controlling factors for the regional geological history and neotectonic activity of earth's crust.

2) Neotectonic movement and earthquake

Take a large fault as the key link to study the deformation characteristics and loading state for tectonic movement since the Pleistocene epoch, especially, to pay attention to the direct evidences, reflecting neotectonic activity, such as: characteristics of seismicity, terrain deformation, regional ground stress distribution and abyssal geophysical features.

3) Underground water

The underground water whether is considered as the resources exploitation, or as a hazard for engineering
activity, the regional hydro-geological structure and its hydro-dynamic characters should be researched.

4) Geomorphic and physico-geological phenomena

The geological structure characteristics may be reflected in the various geomorphic landscape. The researches for geomorphic characteristics and its development history not only give evidence for neotectonic movement, but also give data for analysis of physico-geological phenomena. The researchers should pay attention not only to the crustal stress, exogenic force, and man-made unfavorable geological phenomena, but also to the potential hazards of easily destroyed geological environment, such as expansive soil, soluble salt.

5) Geological resources

Geological resources include mineral products, natural building materials. In order to utilize in future and protect the resources, these researches are necessary. It is easily comprehensible that there is a close internal connexion between above-mentioned factors. The basis of this connexion is a regional geological structure. The large fault controls the characteristics of a regional geological structure in various degree and controlled its formation-development history, and has constitutes the foundations for neotectonic movement and present seismicity. The regional geological structure controls not only the geomorphic characters and its formation-development history, but also the regional hydro-geological structure, and hydro-dynamic characteristics. The comprehensive process of them controls the depositional character of the geologic resources and active regularity of unfavorable geological phenomena.

III Stage

To give the decisive conclusion

The conclusion of decisive analysis depends on the comprehensive analysis of previous data in the former two stages. A regional crust stability is estimated and the engineering geological condition on the concrete construction region is researched specially. The discussion for the resources and environment protection is made.

According to the explanation and researched achievement in the above-mentioned stages the corresponding maps can be made. A series of maps of this subject has been conducted. They are provided into use for designers.

According to above-mentioned principles and measures, in the investigations for exploitation of water energy resources on the river Ya Longjiang in Sichuan Province, the study on this subject is made. The content and conclusion of these researches are summarized as follows.

2. Researches for explanation of geological background and for evaluation of regional crust stability down the river Ya Longjiang

(1) Explanation and analysis of regional geological background

A scale, corresponding to the aerial remote sensing photograph, has been adopted. The visual method (with the aid of a stereoscope) and the isodensity division method have been used. The principal explanation contents are: the faults in various scale, strata occurrences and folds, strata and magmatic rock distribution. "A geologic-tectonic map based on the interpretation of the aerial remote sensing photographs down the river Ya Longjiang" has been made, as shown in Fig.2.

According to the characteristics of geologic formation, the contact relationship between the formation strata in the various tectonic epoch and according to the tectonic framework characters (Fig.2), the following problems can be considered:

1) The tectonic pattern in area is determined by the fault F1, F2, F3. There is an obviously zoning in accordance with the distribution of the tectonic framework.

2) Synthesizing geological data, aeromagnetic field and gravity anomaly, the faults F4, F2 may be considered as the lithosphere faults formed in the Hercynian movement lithosphere. These faults are activating up to the present. Its presence and action basically control geological development history in the area.

3) According to the geologic tectonic characteristics illustrated by the Fig.2, there is a South-North compressive stress field in the later Proterozoic era to the early Sinian Period. The sedimentary circumstance was tranquil in the later Sinian Period to the Caledonian Period. There was a rise denudation circumstance in the early Hercynian Period, and in local region some sedimentary material (sediment) had been presented. In the later period due to the intense tension of faults F1, F2 the east-west compressive
Fig. 2. Geological tectonic map of the aerial remote sensing photograph interpretation down the Yalong River.


stress had been brought on the present area. In the Indo-Chinese Period during the fault-block uplifting descending, the intermontane terrestrial deposits had been accumulated in the fault trough valley. In the later Jurassic Period tectonic movement was violent and it was represented by the NWW compressive stress, and the right lateral shear has been induced. In the later period of tectonic movement the compressive stress has diverted from the NWW to the North west. The left lateral shear of the faults F_1, F_2, F_3 has been represented. Due to the restraint of crystallized basement an uniform stress distribution and a zonation of tectonic framework have been formed.

(2) Neotectonic movement and earthquake

The study of regional tectonic activity is considered as a research for the trace of neotectonic action of deep fault and present seismicity by means of the methods of photograph interpretation and comprehensive analysis of the related data. The contents of researches include: 1) geological characters of the photograph interpretation, 2) geomorphic characters; 3) changes of water system; 4) distribution of thermal spring; 5) gravity anomaly, terrain deformation characteristics; 6) seismicity.

According to the comprehensive analysis of above-mentioned data, it is described that the fault F_1 is subjected by NWW principal compressive stress since the Pleistocene epoch up to the present. A differential motion of fault blocks with left lateral shear has been formed.

(3) Zoning of regional tectonic stability of earth crust

The main bases for zoning according to regional crustal stability are: 1) the conclusion from regional geologic structure and its geological development history, simultaneously, considering the propagation properties of seismic wave of geologic mass; 2) the conclusion of the neotectonic movement since the Pleistocene epoch, the study on the sedimentary rate in the basins of Cainozoic era is very important; 3) the earthquake data, especially, the solution of focal mechanism and historic macroseism, earthquake degree, frequency, the status of energy stress release, and the geological structure effect from the gramme of intensity iso-

line. The observed data of the weak and slight earthquake in recent years are very useful for zoning; 4) geophysical characteristics; 5) terrain deformation characters; 6) ground stress status; 7) a conclusion on the present activity of major faults. The concrete zoning as shown Fig. 2 and its estimations are described as follows.

Zone A A wide area in the lying wall of fault F_5 is a relative stable zone. It can provide optimal location for the high dam construction and the industrial construction. The problem of earthquake protection should be considered for engineering construction with high standard.

Zone B The block between faults F_1 and F_5 is a sub-stable zone, in which subzone B_1 is relatively favorable, and subzone B_2 is relatively active. It can provide a location for the low dam construction and industrial construction. But according to the rock mass engineering geological characteristics in locations the problem of earthquake protection should be considered.

Zone C The present active area of fault F_1 and the area of laying wall and upper wall are the unstable zone. It is not suitable for engineering construction with high standard.

3. Engineering geological estimation of the reach with ladder exploitation down the river Ya Longjiang

According to the zoning of regional crustal stability the suitability for engineering construction in zones had been dealt with. The researches for valley geological structure and stability analysis of the slopes should be developed still further. Thereby, the optimal location of dam construction can be selected and the engineering geological condition of a concrete damsite may be proved. These can provide bases for design of feasibility stage.

(1) Valley geological structure and the bank slope stability section

The comprehensive results of the photograph interpretation and related data as shown in Fig. 3 and Table.

1) The major outcrop rocks at the both banks of valley; granodiorite, limestone, dolomite, limestone with sandstone and claystone parting, basalt, sandy shale, sandy conglomerate, syenite.

2) Major faults: F_4 strike NNE; F_5 (elongation of regional fault F_2) occurrence: N30°-60°NE SE 80°; F_6 N50°-60° NW
<table>
<thead>
<tr>
<th>Subzone</th>
<th>Engineering construction suitability evaluation</th>
<th>Stability evaluation of the valley bank slope</th>
<th>Geological structure and stability evaluation</th>
<th>Engineering geological rock groups</th>
<th>Engineering geological characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>Relative stable zone (L)</td>
<td>Stability zoning (code name)</td>
<td>Occurrence of statum and major faults is perpendicular to the valley transverse geological structure. Lithology as right column. Main developed strike N50°-60°E fault with minor slips on NNE, NE. Bank slope stable, minor slide presented in mountain slope in accordance with the normal dip and in fault fractured zone.</td>
<td>Thick-great thick laered conglomerate rock group ((A_a^1))</td>
<td>Consists of gravel sandstone and conglomerate, thick-very thick layered, effect of layer surface is not obvious. Rock mass intact. The intact strength is high considered as a intact blocky structure.</td>
</tr>
<tr>
<td></td>
<td>Engineering construction suitability evaluation</td>
<td>Transverse valley bank slope stability area (b)</td>
<td></td>
<td>Sandy shale interpreted by the hard-soft ((A_a^2))</td>
<td>Combination of the hard sandstone and the shale or coal seam, by the hard-soft intercalation. The interformational slide developed, strength and intact inhomogeneous.</td>
</tr>
<tr>
<td></td>
<td>Transverse valley bank slope stability area (b)</td>
<td>(A)</td>
<td>Occurrence of statum and major faults is parallel to the valley geological structure. Lithology as right column. Major fault is a fault N35°-N45°. There are 3 joints: SN, E, NE. Bank slope along the side, slope dip unstable and the slope against dip is more stable.</td>
<td>Basalt rock group, blocky fractured structure ((A_a^3))</td>
<td>Quartz asperine syenite intrudes into the basalt, by the welded contact, intact strength high. The strength heterogeneity induced by the alternation</td>
</tr>
<tr>
<td></td>
<td>Longitudinal valley bank slope stability area (b)</td>
<td>(A)</td>
<td></td>
<td>The limestone with shale, claystone rock group ((A_a^4))</td>
<td>Coarse grain basalt, porphyritic structure, rock mass fractured. Weathering and relaxing are intense</td>
</tr>
<tr>
<td>B_1</td>
<td>Engineering construction suitability evaluation</td>
<td>Longitudinal valley bank slope stability area (b)</td>
<td>Consist of the blocky granodiorite, mainly developed the faults SW, NE. The relationship between the fault direction and the valley strike is more changeable, indirectional fault. In addition to minor slide in part area, the bank slope is basically stable.</td>
<td>Granodiorit rock group blocky fractured structure ((B_a^1))</td>
<td>Limestone, dolomite, dolomitic limestone, medium and thick layered structure, is of dissolution phenomena</td>
</tr>
<tr>
<td></td>
<td>Indirectional valley bank slope stability area (b)</td>
<td>(b)</td>
<td></td>
<td></td>
<td>Due to the polycyclic tectonic movement, the intactness and intact strength of rock mass heterogeneity, the sets of joints is many and infilled by the loosen and soft material, fractured structure in the local area. Blocky structure far from the fault</td>
</tr>
<tr>
<td>B_2</td>
<td>Engineering construction suitability evaluation</td>
<td>Longitudinal valley bank slope relative stability area (b)</td>
<td></td>
<td>Hard-soft intercalated sandy-shale rock group ((B_1a))</td>
<td>Hard sandstone and the shale or coal seam, hard-soft intercalated, interformational slide developed. Rock mass strength and intactness heterogeneous.</td>
</tr>
<tr>
<td></td>
<td>Longitudinal valley bank slope relative stability area (b)</td>
<td>(b)</td>
<td>Fault F_9 and traction folds developed, longitudinal valley geological structure. The bank slope is bad.</td>
<td>Hard-soft intercalated sandy-shale rock group ((B_2a))</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 3. Engineering geological zoning map of the aerial remote sensing photograph interpretation in the step exploration reach down the Yalong River

1. A and B1, B2 the code name of crust stability zoning, a or b the code name of slope stability section, 2-7 the code name of engineering geological rock group. Reference in tab. 1; 2. Landslide; 3. Rock fall; 4. Engineering geological zoning boundary.

Note: The geological legend see the Fig. 2.

\( \angle 60-70^\circ, F, N20W SW \angle 50^\circ \). The zones are divided in accordance with the fault distribution in map.

3) Reach Aa is a transversal valley, reach Ba is a un - orientation valley, the slope is stable. Only some minor landslides and rock falls occur in the part location or in the tributary valley.

4) According to the geological structure characteristics of the two reaches, the reaches A\(_a\), A\(_b\), and B\(_{1a}\) can provide optimal location for dam site.

(2) Engineering geological estimation of concrete damsite

According to the evaluation of regional crustal stability, stability analysis of valley slopes, division of the engineering geological rock groups and the selection of optimal damsite, the basis of ladder exploitation may be provided. Thereby, the engineering geological estimation of concrete damsite may provide necessary data for feasible proof of engineering construction.

Above-mentioned selection of damsites and its characteristics of engineering geological rock groups as shown in Tab. 1. The principal engineering geological problems in the two damsites A\(_a\), B\(_{1a}\) are considered as follows:

1) Damsite A\(_a\) may be although selected in the syenite mass, but some problems should be researched further, i.e. ① The basalt rock mass on the upper wall of fault F\(_c\) on the upper stream of damsite was subjected by the intense compressive action due to the formation of fault and the intrusion of the syenite. Therefore, the rock mass becomes heavy, in addition, the structure characteristics of basalt itself, can provide convenience for later exogenic process. Although the continual inter original weak seam is of angle at 30° with the strike of bank slope a landslide of large scale can not occur, but the minor slide and mud-rock flow often occurred. Therefore, the stability problem of bank slope has been raised. ② According to the explanation of tectonic framework and the study on the neotectonic movement and the landform condition, the high stress zone in area A\(_a\) has been determined.

3) According to the interpretation and site investigation the syenite is of welded contact with the basalt, and the branch basalt, due to the later fault process, is of the alteration in different degree. Therefore the heterogeneity of rock mass in strength is formed. The problem of erosion by spilled
Fig. 1. The flow diagram on engineering geological environment evaluation for aerial remoting sensing data in the high mountain gorge area

water and deformation of dam foundation, induced, by the alteration zone has been raised.

2) Reach B, this area is a pinch out section of the regional fault F, branch. The west branch fault F, is replaced by the interformation slip. The medium branch is thinning out to the folded zone of sandy shale alone the plane of unconformity. The east branch is a joint zone with an echelon distribution in the valley. Due to the tectonic characteristics the rock mass is cataelastic, the intact strength is lower. Simultaneously, the problem of seepage under dam foundation and seepage stability have been presented. In point of geological condition the dam site is better in the granodiorite with blocky rock structure. From the point of regional crust stability the upper course is better than the tail water.

Finally, it must be indicated that the complex engineering geological environment has brought the restraint for engineering planning and projection, and the new changes of environment will be induced by the engineering construction. In present area the new changes should be further researched.
REMOTE-SENSING AND FIELD INVESTIGATIONS, DESIGNING AND EXECUTION OF REMEDIAL WORKS ON A LANDSLIDE NEAR THE TOWN NOVI SAD

EXAMINATION DE L’INGENIEURS-GEOLIGIQUES A DISTANCE ET SUR TERRAIN, ACTION DE PROJETER ET L’EXECUTION DE MESURES REMEDIABLES DE GLISSEMENTS AUPRES DE NOVI SAD

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ABSTRACT
Along the motorway Novi Sad-Ilok which is passing near the Danube river on the foot of the Fruska Gora Mountain a series of landslides occurred. In the paper the results of investigations, designing, remedial works and control of the displacement after the works executed on one of the landslides, are reported.

Engineering-geological investigations covered the registration of all the occurrences of instability by means of aerial photographs in the IC and thermal technique. At the same time also a correlation with the other sorts of investigations was made along with a rather short field reconnaissance.

Hydrogeological works included recording of all the data on groundwater obtained by piezometers, wells, springs etc.

Soil investigations covered the execution of test holes, sampling and lab testing.

On the basis of the investigations performed the results were analysed, the cause of sliding was established and the remedial measures were evolved. The stability computations were made for the existing condition and the condition supposed after the remedial measures whereupon the construction project of remedial works was designed.

Generally speaking, the soil in the landslide areas consists of Quaternary clayey deposits overlying Pliocene clay. Between these layers there is an intercalation of fine-grained sand acting as an aquifer.

The lower part of this sand layer is "plugged" with a layer of clay due to the movements of masses. The remedial measures anticipated the performance of buried plane drains parallel to the slope.

The drains were constructed by the modified method of diaphragm excavation. The trench excavated was backfilled with gravel according to the principle applied for filters.

The control of the efficiency of remedial measures was made by observing the displacement of bench marks as well as the groundwater level.

ABSTRAIT
Le long de la route Novi Sad - Ilok laquelle s'étend auprès de la fleuve Danube, au pied de la montagne Fruška Gora on avaient aperçu plusieurs glissements de terrain.
Cet exposé comprend bien des résultats de l'examen, de l'action de projeter, de mesures remédiées et de contrôle du déplacement et tout cela après des mesures remédiées faites sur un de ces glissements.

Les travaux de l'Ingénieurs et celles géologiques comprennent la registration de phénomènes de l'instabilité et cela par moyen de photographies d'avion en IC et en technic thermale.

En même temps était faite aussi la correlation avec d'autres sortes de l'examen prise en considération une prospection moindre. Des travaux hydrogéologiques ont bien compris la registration de toutes données quant à l'eau souterraine par des piezomètres, puits sources etc.

L'examen géomécanique comprend l'exécution de trou de soudage, pris des échantillons et des épreuves laboratoires.

En base de toutes examen effectuées on avait fait une analyse des résultats, constatation des causes de glissements et l'action de projeter des mesures remédiées.

Par ailleurs sont faites des calculations de stabilité de l'état naturel et bien rétablir, et il est élabore aussi un projet de construction en ce qui concerne des mesures remédiées.

Puis en général le sol sur le (1iu) lieu des glissements de terrain est composé de couches quartnaire-agriales posées sur argile pliocène. Entre ces deux couches vient posée l'intercalation de sable a grain fin en fonction de un collecteur d'eau. La partie inférieure de cette couche de sable vient bouchée d'une couche d'argile et pour cela la même masse glisse. Pour pouvoir arrêter ce glissement il est prévu de fouiller le drainage parallèlement avec cette pente. Les drains sont exécutés par une méthode modifiée de l'excavation des diaphragmes. La tranchée est remplie de gravier selon le règlement pour l'exécution des filtres.

Le contrôle de succès de mesures remédiées est exécuté à l'aide de l'action d'observer les déplacements de repères et de l'eau souterraine.

1. Introduction

Sliding of the ground is one of the rather complex natural processes. Such processes resulted in great damages and sometimes also they may cause loss of human lives. Very often the ground movements are also caused by man's activity as for example at deep excavations, excessive loading of slopes, acting of storage basins etc.

Sliding of the ground itself is a very complicated mechanism in which many elements are interacting as for example the properties of...
soil, acting of water, morphology, removal of plants’ cover and some others. Since the urbanization has already reached the areas where the above problems may arise, it is necessary to solve first these problems.

One of the unstable regions in Yugoslavia is the area along the Novi Sad - Ilok motorway on the northern slope of the mountain Fruska Gora near the river Danube. In this section the active as well as the old landslides have already imperilled for years the motorway, quite a number of houses and agricultural areas. In order to find out the causes of sliding and to enable the designing of repair works very complex explorations were made.

2. Geological Interpretation of Aerial Photographs

The area explored has been photographed several times from the airplane for the purposes of geological interpretation as well as surveying and that:

- in panchromatic technique in 1975
- in infrared, black-white technique in 1972
- in thermal detection in 1978.

Because of the elaboration of topographic maps/scale 1:500/ and to ensure better comprehension of the aerial photographs and thermograms a scale 1:5,000 was chosen.

Besides the elaboration of topographic maps the interpretation of aerial photographs provided the registration of all the engineering geological elements and occurrences (geological boundaries, structures, inclinations of layers, subsurface water, landslides etc.). The occurrence of surface waters and plashes was registered on the basis of shading from the photographs and that from grey to dark shade range.

Based on the interpretation of the aerial photographs the test holes were planned and the piezometers located.

By comparing the aerial photographs and thermograms taken several times on the same scale (1:5,000) the new considerable movements of sliding mass were registered towards Danube. Thus the dynamics of sliding was established and the zone of the active (AK) and the old landslides (SK) determined.

All the data interpreted from the photographs were checked in the field with a rather short field reconnaissance.

3. Engineering and Geological Investigations

The landslide Čeverić (Fig. 2 and 3) which has been chosen as an example of the detailed investigations is one from the series of rather large landslides as Pavliš, Banoštov and Perļjuša I and II (Fig 1).

In the mentioned landslide Čeverić the main slip circle is in an elliptic form 1200 m long and 450 m wide.

The predominant part of the slipping mass is on old landslide in which a series of new landslides has been activated in the direction of the river Danube.

The observed cracks on the landslides were distinguished as:
- vertical wall (dip slip)
- cracks (open and closed)

In its active part the terrain is intersected by fissures and folded. Composition of the soil is disturbed and the increased moisture has been registered (springs and plashes).

The major displacements of the soil were established in the section covering the motorway in the middle part of the landslide.

Although the dynamics of movement of the sliding mass was registered on the aerial photographs, the numerical values of the displacement were obtained by means of recording the benchmarks network.

By means of exploratory drilling and geological mapping it has been established that the soil in the landslide area is composed of grey-blue Pliocene clay and Quaternary loess in hanging wall. Between these layers the intercalations of Pliocene sand were found.

4. Soil Explorations

The soil explorations performed on all the mentioned landslides involved the execution of test holes and extraction of disturbed and undisturbed samples. The test holes were 10 to 80 m deep and in some of them there were also piezometers. At the same time all the other data interesting for some information on sliding were registered as for example the groundwater
level, difficulties when drilling etc.

Laboratory tests made on representative samples covered soil classification tests (Atterberg limits, grain size distribution) and testing of the general soil properties (natural moisture, unit weight, specific gravity). An extra group of tests included direct shear tests, residual shear tests, Krey-Tiedemann shear tests, triaxial compression test and torsion residual shear test. The results obtained were statistically treated and formed a basis for geostatic computations.

5. Landslide Genesis

The series of the landslides explored being the subject of this paper occurs in uniform geological formations and exhibits almost the identical genesis of sliding. The bottom of the entire slope consists of Pliocene high-plastic clay. In the course of time the alluvial sand was sedimented in subsidence throughs on the clay surface. By further geological evolution in the stage of erosive process the Quaternary material (loess) was

--- Fig. 2 An aerial photograph of the landslide Čerević ---

**LEGEND: (for Fig. 3).**

- at Alluvium - sand, gravel
- $LQ^*$ Quaternary - loess
- $LQ$ Quaternary - silty clay, silty-sandy clay, sands
- --- Lithological boundary
- --- Fault assumed
- --- Fault with relative displacement
- SK Old landslide
- AK Active landslide
- Steep cut
- Fissure
- Fissure - subsidence part
- Dip slip
- ⊙ S-1 Test hole (1968)
- ⊙ B-7 Test hole (1972)
- ⊙-⊙ Geological cross-sections S-1 S-2 (1968)
- ⊙-⊙ PROFILE III, Geotechnical B-3 B-2 profiles (1972)
- --- Plane drain
- --- Bored drain
- □ Benchmarks 1972
- ☞ Spring
- ○ Captured spring
- ○ Well
- □ Drinking fountain
- □ Flash
- □ Surface moisture
- □ Increased surface moisture
- ~ Permanent surface stream
Fig. 3. The detailed map of engineering-geological interpretations of remote-sensing and field investigations in the area of the Čerević landslide
deposited overlying the pliocene clay and the sedimented sand. Rainfall waters from the slopes of the mountain Fruska Gora infiltrate the soil passing through the rather permeable sand layers toward Danube which at the same time erosively attacks the toe of the slope observed. The layer of sand in the soil near the Danube banks is disturbed in mechanical transformation and when mixed with clay it ceases to be an acquirer. By its hydrostatic and hydrodynamic energy on its way towards the river the water tends to pass through the mixed soil which together with the properties of soil is the main cause of the sliding occurring in this area.

6. Designing of Repair Measures
The explorations described exhibited the results which after the required analyses and comparisons have indicated the cause of sliding and how to eliminate it.

The remedial measures depending upon the cause of sliding mainly refer to the application of two methods or their combination. One group of remedial works involves the unloading of the upper part of the moving ground and shifting of this mass to the toe of the slope. The other possibility of remedy is to lower the groundwater level and to direct its flow. Also the rainfall and surface waters shall be regulated. Which of these two methods will be applied depends, first of all, on the cause of sliding and if only succeed to check the cause we may say that the desired effect of remedial measures have been obtained.

On the landslides along the mentioned motorway section the second method of remedy was chosen. The remedy anticipated the performance of buried plane drains by means of which the rather permeable intercalations of water-bearing sand layer would be intersected. The position of drains perpendicular to the isolines as well as their length was chosen on the basis of data obtained by the mentioned explorations. By means of drains a low potential would be imposed and thus the stream flow would be directed in a more favourable direction than before. Beside this function due to their better mechanical properties such drains act as rather rigid structures in the soil.

To make numerical analysis of the natural and repaired state a series of cross sections has been chosen at interesting places. The computation itself was made for two cases and that first of natural state and then with acting of the anticipated drain (Fig. 4).

With the parameters of the soil material obtained by the explorations a control on the chosen slip planes was performed.

These planes were chosen according to the arrangement of the soil layers, the registered cracks and the other data. The computation was made by means of the equations by nonviller and Spencer based on the principles of the yield point of sliding mass.

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Fig. 4. One of design cross-sections
After the way of remedy had been chosen for the new limiting conditions an analysis of stability was repeated for the same slip planes. A safety factor increased at least 24 per cent was obtained. In technical practice such an increase of safety factor for more than 20 per cent ensures the efficiency of remedial measures.

Fig. 5. The way of executing the excavation

7. Execution of Vertical Plane Drains
The problem encountered during the execution was how to construct such deep and narrow excavations required for the anticipated plane drains. It was solved by digging with the machines for diaphragm excavating modified in a way as described more detailed hereinafter. Thus the excavation depths down to 18.5 m were reached without timbering or using mud.

The execution of the drain beins with the excavation of the first section down to the design depth (1). Into the excavation a limiting semipipe (2) is inserted (2) and then backfilled with gravel (3) of the next section (4) is excavated with one side supported by the limiting semipipe. Then the next semipipe (5) is inserted, the excavated section backfilled with gravel (6) and the previous limiting semipipe is pulled out.

The work was quickly performed and well organized whereby the rheological aspect of the horizontal arching in the soil around the excavated section was used.

Minor difficulties appeared during execution did not influence the economic aspect of this method either the continuity of the drain or backfilling.

Fig. 6. The excavation of plane drain
8. Control of the Efficiency of Remedial Works

In the entire area in the beginning of the explorations a network of surveying bench-marks was installed. The measurement results of the soil displacement and the groundwater level in piezometers served as a basis when designing remedial works. After the execution of the anticipated plane drains the recorded displacements of the bench-marks do not indicate any general activity of the landslide. In places some smaller secondary parts of the terrain remained active which was repaired in the next stage of the work.

The measurement of lowering of the groundwater level in the piezometers and the existing country wells point at the acting of drains.

REFERENCES:


FIRST APPLICATION OF THE GRAVIMETRIC METHOD TO THE STUDY OF LANDSLIDE BODIES

PREMIERE APPLICATION DE LA METHODE GRAVIMETRIQUE A L’ETUDE DES CORPS DES GLISSEMENTS DES TERRAINS

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ABSTRACT

A gravity survey has been carried out on a landslide area south of Pisticci (Lucania-Italy) to investigate the geometry and density of landslide bodies. In the studied area quaternary clayey formations (Calabrian blue clays outcrop over which yellow transgressive terraced marine sands lie. Using a computerized procedure theoretical models were considered, on the basis of available geological, geophysical and mechanical data, and their gravimetric contribution was compared with the experimental data obtained with a Worden and La Coste and Romberg model D gravity meter. The procedure was iterated modifying geometry and density of models to improve the agreement. This analysis allowed recognition in the studied areas of the presence of lateral heterogeneity in the shape of a more compact block, the presence of sub-superficial anomalous bodies thicknesses and so on.

ABSTRACT

Dans des corps d'éboulements de terrain près de village de Pisticci - Italie du Sud (Lucania)- on a été appliqué la méthode gravimétrique pour définir la géométrie et la densité des corps mêmes. Les terrains impliqués appartiennent à la formation des argiles blu du Calabrian sur laquelle on trouve des dépôts transgressifs de sables jaunes d'origine marine sous forme de terraces. Sur la base des données géologiques, géophysiques et mécaniques on a d'abord considérés modèles théorétiques, qui ont été comparés avec les données expérimentales obtenues par le gravimètre Worden et le modèle D La Coste et Romberg. En modifiant les valeurs de géométrie et de densité on a amélioré l'approximation des modèles théorétiques. Cette analyse a permis de reconnaître dans les masses de terrain étudiées des hétérogénéités latérales, les épaisseurs des corps d'éboulement de terrain, la présence de masses en position anormale, etc.

1. INTRODUCTION

Gravimetric prospecting has rarely been applied on a small scale and in landslide areas. However, in such conditions it can contribute to the solution of some geological and paleogeographical problems. The first applications were performed in Lucania (Fig. 1), one of the regions in Southern Italy most affected by sliding phenomena and seismic activity to make matters worse in order to determine the geometry and differential density of the sliding bodies.

2. GEOLOGICAL SUMMARY

The terrains that form the hill of Pisticci village belong to the Pleistocene-upperpliocene cycle of the Bradanic Trough.
With respect to the sands it has not been possible to obtain any undisturbed samples for geotechnical determinations.

We can only say that we deal prevailingly with fine sands with uniformity degree not exceeding the value of two.

Considering the geostructural features of the above mentioned clayey and sandy soils, they are affected almost exclusively by slump movements of rotational type for first-time landslides and by slide phenomena when renewal of ancient landslide bodies occurs. In the upper part of the landslide body the displaced soil is affected by cracks of variable width which fill with debris immediately; as one comes down the slope, the landslide body is affected by fissuration and by considerable reworking leading to the formation of long mudflows. Moreover we must observe that, owing to the works of reclamation by means of reprofiling and the construction of drainage measures, at least in the superficial part, the landslide bodies have been remarkably altered.

**GRAVITY MEASUREMENTS**

A Worden and a Lacoste and Romberg model D gravity meter were used. The reading accuracy is around 0.01 and around 0.001 mgal respectively; the precision of the field survey being better than 0.010 mgal where the gravity measurements are concerned and 0.005 m for the topographic levelling.

The field data analysis performed in the area shown in Fig. 2 following the procedure outlined by Calcañile et al. (1982) which can be briefly summarized as following.

a) The gravimetric survey should be carried out whenever possible in the form of mapping rather than in profiling, especially if there is a complex regional gravity field.

b) In the case of survey carried out in profiling, the reconstruction of the behaviour of a complex regional field, in topographically complex areas, is obtainable by compensating the experimental anomalies using the gravimetric effects computed for theoretical models. The interpretation can be, subsequently, developed modifying such models in order to remove possible compensation residuals.

c) The computation of the models gravimetric effects must be
d) In modifying the theoretical model to improve the agreement between experimental data and theoretical effects, it is possible to recognize if changes must be made to the density or the geometry of the outcropping anomalous bodies model, knowing their horizontal width. If sliding bodies are buried beneath other sliding bodies, possible interpretation ambiguities can be solved only with the aid of different methods.

The reduction of gravity measurements (Faye, Bouguer, terrain) was made easy by the availability of 1:2,000 and 1:25,000 topographic maps; such reductions were fulfilled through the use of a computerized procedure (Calcagnile et al., 1982).

Several density values were used in the reductions, among these the value of 2.0 and 2.1 g/cm³ were assumed to be more pertinent to the actual geological setting. Namely in the area we studied there are two types of outcropping terrains (Calabrian clays and Post-calabrian sands) and two types of sliding bodies (in the shape of blocks and debris, respectively). Since clay represents the prevailing lithotype, their density was taken up in the Bouguer and terrain correction; the remaining bodies were considered anomalous.

Two sub-areas were studied (Fig. 2). The gravimetric data were filtered to remove the short wave-length noise due to the rather complex topography.

In Fig. 3a and 3b are shown the Bouguer anomalies-filtered curves for the first survey (Croci landslide) and, respectively, the Bouguer map for the second survey (Campo Sportivo landslide), both with a 2.1 g/cm³ density.

**INTERPRETATION**

A starting theoretical model was set up for the anomalous bodies of Croci landslide, whose density distributions were obtained on the base of available stratigraphic sections (Guerricchio and Melidoro, 1979) of density measurements carried out on samples taken in correspondence of the outcrops of clay, sands, sliding body, and V.E.S.

The assumed density contrasts were: sliding body in the shape of blocks -0.6 g/cm³, sliding body in the shape of debris -0.5 g/cm³, sands -0.6 g/cm³. Using the values of thicknesses and density contrasts obtained in such a way the agreement be-
Even after modifying the density of sands and sliding bodies, short wavelength anomalies compensations were not quite satisfactory. A further improvement of the compensation was not attainable by varying density values; at this stage changes in the geometry of the model were required.

It was necessary to shift the maximum deepening zone of the sliding bodies southwards, bringing it from profile A toward profile B. The agreement could be furtherly improved by increasing the total thickness of the sliding body or shifting the ratio of the thickness of the two parts of the landslide toward the one less compact. The V.E.S. carried out in the area favours the latter solution. The final models are shown in Fig. 4.

In Fig. 5 the experimental (continuous line) and the theoretical anomalies (dashed line, obtained on the basis of the previous models, are given for different profiles.

There is good agreement in profile A between the theoretical curve and the experimental one both for the eastern large negative anomaly (due to the landslide body) and the western small one (caused by sands)

Profile B theoretical and experimental curves are consistent to reproduce the two gravimetric minima yielded by the land sliding body and the interposed maximum corresponding to a more compact portion of the same body.

The southwestern experimental minimum centered on station 125 is much likely amplified by a regional gradient effect associated to the profile irregularity (Calcagnile et al., 1982).

In profile C the left part has been interpreted in terms of the landsliding body gravimetric effects while the right part is due to the effect of the sands.

For profile D, it has to be remarked that the difference between the two curves corresponds to a relative shift of 50-60 m. This anomaly is likely due to a debris lens located 50-60 m north of the assumed position.

The interpretation difficulty due to the regional field decreases if it is possible to survey in mapping instead of profiling (Calcagnile et al., 1982). For the second sliding area (Camposportivo landslide) no stratigraphic data were available for setting up the starting theoretical model, however, there were a few data obtained from V.E.S. carried out
Fig. 4. Croci landslide. Final geometric models: a) whole landslide, b) block landslide only (isopach spacing 2 m, arbitrary kilometric reference).

Fig. 5. Croci landslide. Comparison of the residual anomalies curves (open circles) against theoretical ones (full circles).

for this purpose.

In Fig. 6 theoretical residual anomalies maps are shown; they were obtained assuming the following density contrasts: sliding body in the shape of debris -0.5 g/cm³; sliding body in the shape of blocks -0.2 g/cm³ (Fig. 6a); sliding body in the shape of blocks +0.2 g/cm³ (Fig. 6b).

In order to obtain a relative maximum in correspondence of the sliding body in the shape of blocks it has been necessary to assign to it a positive density contrast whose geological meaning is not very clear since quantitatively it is still too small to reproduce the experimental anomalies. On the other side the landslide foot gravimetric effect is too mild to explain the large bidimensional anomaly (Fig. 3b) unless one assigns to it a strongly positive density contrast (around 0.5 g/cm³) including it among the anomalous bodies.

It is then evident that the anomalies field caused by sliding bodies is disturbed by sub-surficial sources whose anomalies are hardly distinguishable if supplementary data on them are not available.

CONCLUSIONS

The gravimetric method is sensitive enough to detect sliding bodies effects and to give constraints on their thicknesses and density contrast, provide that other shallow-seated anomalies are not
Moreover the gravity measurement are advantageous in that the field work is of relatively short duration, applicable in areas where other geophysical methods must generally be excluded, the measuring coverage of the area is more detailed without increasing too much the cost; the interpretation, e.g. combined with a few vertical electrical or mechanical soundings, is generally simple and it can be applied even for complex shaped bodies.

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Fig. 6. Campo Sportivo landslide. Theoretical residuals: a) for a negative density contrast, b) positive density contrast for the block landslide, whose boundary is given by the dashed line (spacing 0.05 mgal).

present since they are not distinguishable if other supplementary data are lacking. If disturbance of that kind are not present, it is possible to obtain a quantitative interpretation using a few V.E.S. as reference, through the construction of theoretical models of sliding bodies and comparison of their gravimetric effect with the experimental data. In this way densities and thicknesses of outcropping anomalous bodies are obtainable. The attainment of this result is easier if geomorphological, lithostratigraphical and geotechnical data are avail-
ABSTRACT
Terrestrial photogrammetry has been advantageously applied for a quick and detailed investigation of a few landslide areas in Nilgiris district of Tamil Nadu. The technique employs taking a stereo-pair of photograph of each area with the help of a Terrestrial photogrammetric camera and the preparation of large scale maps from them with the help of a plotter, showing various topographic and thematic features, connected with landslides.

A stereo-pair of photographs is a complete and permanent record of the terrain and on the basis of their study a critical and comprehensive evaluation of the terrain has been attempted and discussed in this paper. A number of thematic maps showing erosional processes, vegetation distribution, geomorphology, etc. have been prepared. Also, from the large scale maps, geomorphometric maps related to surface roughness parameters have been prepared. The inter-relationship of all these maps has, besides identifying hazardous and safer areas for developmental activities, led to a proper evaluation of the terrain.

Discussion of the case histories has clearly demonstrated the significance of this advance technique in swift and detailed evaluation of the landscape and hence calls for a large scale application of this technique in projects requiring detailed appreciation of terrain.

ABSTRAIT
Terrestrielle photogrammétrie a été appliquée/advantageusement pour une investigation rapide et détaillée de quelque important glissement à la région de Nilgiri, Distt. Tamil Nadu. La technique se sert de prendre
un stereo-pair photographic de chaque région choisie par une appareillage photographique de Terrestrial photogrammetric et par ces photographies on a préparé les cartes sur une grande échelle au moyen de tracé, en exposant les différentes caractère de topographique et thématique qui se sont reliés avec glissement.

Les photographies de stereo-pair est un dossier de terrain complète et permanente et ici en cette fascicule on a été essayé et discuté une delicate et l'évaluation compréhensive de terrain sur la base de ces études. Les différentes écarts ont été préparé où on explique les procedures dérosion, les distributions de végétation, geomorphology, etc. En a préparé aussi les écarts sur grande échelle, les différentes geomorphometric écarts se relier avec une paramètre de surface raboteux. Une co-relation de évaluation de terrain, de plus on a identifié une région hardieuse, une région de sécurité pour une activité developmental.

Après avoir discuter par le dossier en a été constaté clairement la signification de cette technique avance sur l'évaluation de glissement rapide et sur gr et requise une application sur grande échelle de cette technique pour le requisement en detail d'appréciation de terrain.

INTRODUCTION

The Nilgiris district of Tamil Nadu is a well developed hilly area of peninsular India where increased developmental activities are resulting into cultural encroachments of more and more virgin areas, even involving steeper slopes. Over a long period of time, weathering has produced a thick soil cover over charnockites—the dominant rock type in the area. Subsequent to heavy rainfall during 1978 and 1979, the district was ravaged by numerous landslides, causing considerable damage to life and property. This necessitated a quick investigation of landslides in the region for which Terrestrial photogrammetric survey was employed to prepare large scale maps. Stereo-pairs thus obtained have proved to be highly useful in the overall morphological study of slides and for designing surface drainage alignment. Results have demonstrated the significance of this technique in projects requiring detailed study of selected problem/research areas. Firstly, a Terrestrial stereo-pair—being a complete and permanent record of the terrain affords possibility of map preparation with detailed appreciation of any particular feature of the area, at any time. Secondly, large scale maps of the areas can be prepared with speed, economy and precision, particularly for the areas which are not possible to be covered by the
conventional methods. Thirdly, simultaneous to the preparation of maps there is scope for the digitization of data on a tape which can be fed into a computer for detailed morphometric analysis. This paper has been written to demonstrate the potentials of this technique for a critical and comprehensive evaluation of the landslide terrain.

As a case history two areas viz. Glenmore and Runnymade slide areas have been selected. The selection of the above areas was guided by the striking similarity between them, so that deductions made from one area could be checked from the other and generalizations made. Talus slopes of both these areas are culturally well developed and were affected by slides in 1979, involving no large scale movement of the material. They lie south of the Kateri river with slopes facing towards north.

The paper deals with the terrestrial photogrammetric mapping of slide areas, detailed appreciation of Glenmore slide areas and morphometric studies of Glenmore and Runnymade slide areas. The above studies has helped in evaluating the terrain effectively, identifying the hazardous and safer areas in the context of prevailing set up.

TERRESTRIAL PHOTOGRAMMETRIC SURVEY

The technique of Terrestrial photogrammetry involves taking of two overlapping photographs called a 'Stereo-pair' of the area, from two selected photo-stations, with the camera axis parallel to each other. The photographs (Photo, P13) are taken with the help of Terrestrial photogrammetric camera in which the camera axis can be accurately controlled. A few control points are established in the area whose ground co-ordinates are obtained and the same are also identified in the photographs. The stereo-pair is oriented on a plotting equipment with the help of control points. And, finally the maps are prepared on required scale with required contours, with thematic details (Agarwal, 1980).

A significant advantage of applying this technique in slide prone areas was that the entire area could be mapped out as existing at a particular moment. Whereas, due to slow and imperceptible movements, relative inaccuracies could have crept in the maps if prepared by the time consuming conventional methods. This fact has particular relevance to geomorphometric studies.

APPROCIATION OF GLENMORE SLIDE AREA

A detailed appreciation of Glenmore slide area (Photo, P13) has been attempted with the help of the concerned terrestrial stereo-pair. The area under discussion lies to the south of the Kateri river and extends right upto the ridge crest. The studies include geomorphic analysis, operative erosional processes and existing land use pattern (P12). Geomorphology:

The entire area can be classi-
fied into following four slope classes on the basis of the resultant geomorphic forms and processes, operating in the area (Bloom, 1968).

1. Creep Slopes: It covers the upper most convex and nearly straight segment of the profile, starting from ridge crest down to the starting point of the free fall face. Slope angle is generally of the order of few degrees only and creep is the dominant mass wasting process affecting these slopes. Absence of surface water drainage suggests that the rain water is absorbed in the soil with very little amount flowing down as surface run-off.

2. Free fall face: It covers the steep slope profile immediately below the creep slope and is marked by conspicuous old slide scars and cliff faces. It is also referred to as cliff or deflectional slope. The slope angle is more than 40° in the area. A number of rivulets have appeared at the top of this zone to accommodate the subsurface water from above and for additional rain waters. Gullies are also developing along depressions. Slides and free fall are the dominant mass wasting processes affecting this zone.

3. Talus Slopes: The area below the free fall face and extending right up to the river bed has been designated as talus slope. It predominantly consists of talus material brought down from above, mainly due to slides and creep.

The average slope angle of this land form is about 22°, ranging between few degrees to a maximum of 39°. The area is marked by wide undulations without any conspicuous surface drainage. Most of the water from above and falling onto this portion becomes subsurface. Active slides and subsidences threaten this portion.

4. River bed: It occupies the lowest ground and is wide and flat in the major portion of the area under discussion. In this segment, the river bed consists of a rather stabilized flood plain with river flowing through a bouldery channel.

Erosion:

Sheet wash is the dominant erosional process affecting the creep slopes, whereas, the free fall face is largely affected by slope wash and selective gully erosion. Talus slopes are protected from active surface erosion, for being under tea cultivation, except for the portions which have got bare due to sliding and subsidences. Chemical and mechanical eluviation must be operating in view of prevalent tropical climate and heavy rainfall.

Existing land use:

The creep slopes and a sizeable portion of the free fall face are almost totally deforested, and are presently occupied by bushy to grassy vegetation. However, a portion of the free fall face is occupied by patchy growth of trees. The talus slope is largely under tea cultivation, enclosing a hous-
ing colony in the central portion. Two patches of dense trees occur on talus slope, the one on the east is planted while the other one on the west is natural. The portions of the talus slopes, affected by slides and subsidences are bare to grassy. Flood plain of the river bed is largely covered under bushes, however trees have also grown at places.

GEOMORPHOMETRY

Nilgiris landslides under study are mainly debris slides and gravity has been the essential cause of all landslides. Since slopes are directly related with gravity, therefore, it can be inferred that steeper slopes are relatively more prone to landslides, if other parameters are same.

Slope is the most conspicuous characteristic of any terrain and steepness of the slope and its variation is an important indicator of terrain irregularities or roughness. In the present study, the various parameters considered for slope study included surface area ratio, numbers of contours cutting a straight line, and the difference between maximum and minimum height in a grid. It has been found that surface area ratio provided the most useful information, as such only that parameter is discussed here.

Surface area ratio:

This parameter indicates the amount of similarity between the test area and a planar surface.

It hypothesizes that the surface area increases with surface irregularity.

(Fig. 1)

\[ \text{Surface area ratio} = \frac{AC}{AB} \]

This ratio shows a curvilinear relationship which asymptotically approaches infinity with increase in AC (Hobson, 1972).

The surface area ratio maps of Glenmore and Runnymade slide areas have been prepared from the large scale terrestrial photogrammetric maps of these areas, earlier prepared. The areas were divided into grids of 2cm x 2 cm. In this size of the grid all prominent changes in the slope were accounted. The maximum height difference within each grid was read out from the contour values, and then the value of the angle \( \theta \) (fig.1) was calculated. The value of sec \( \theta \) gave the surface area ratio which was indicated in the centre of each grid. Likewise, values for each grid were calculated and finally the desired contours of equal surface area ratio were drawn. All discreet shear fractures, which were already mapped on Terrestrial photogrammetric maps, were transferred on surface area ratio maps to study the relationship between slides and the surface area ratio.

Slope analysis:

Stability of talus slopes, which are culturally best developed, has been studied in detail for
Glenmore and Runnymade slide areas. The slope analysis was attempted with the simple assumption that the probability of slope failure along prominent breaks in slope is maximum. It was further considered that breaks in slope can best be identified in surface area ratio profiles, since these correspond to the second derivative of altitude with respect to horizontal distance and hence depict rate of change of slope. However, natural profiles which are the first derivative of altitude and show change of altitude with horizontal distance, have also been drawn along same section lines, to make a visual comparison with the surface area ratio profile.

A look at both the profiles of the two areas (Pl3,4) reveal that all prominent changes in slope are highlighted in surface area ratio profiles e.g. the steeper segment in the central portion of the Runnymade slide area, along the section line A-B (Pl4) is prominently exhibited in surface area ratio profile than in natural profile. Surface area ratio maps indicate that old slide scars are confined within closed contours of values 1.2 and more. Whereas, fresh slides are enclosed within values of 1.1 and 1.2. Surface area ratio profiles also reveal a prominent break in slope around 1.1 value, corresponding to the slope angle of 25°. Significantly a large discreet shear fracture, in the central portion of the Glenmore slide area (Pl3) is enclosed within surface area ratio values of 1.1 and more and has terminated just at the contact of 1.1 and lower value. It is thus inferred that slopes are changing rapidly beyond 25° angle and hence areas steeper than 25° are in a state of critical equilibrium.

Surface area ratio profile also reveal that areas falling below the value of 1.05, corresponding to 18° slope, are almost uniform. Further no slides with discreet shear fractures are associated with these areas. It can, therefore, be inferred that slopes gentler than 18° are safer, provided subsurface water is appropriately dealt with. Therefore, in the surface area ratio maps (Pl3,4), delineation of areas steeper than 25° and gentler than 18° have been made, indicating hazardous and safer areas, respectively, purely from the view point of gravity.

Statistical analysis:

Surface area ratio values of the two areas have been analysed statistically, and compared with each other. The surface area ratio values falling in the old slide scar areas have been ignored for this purpose. Thus, a total of 66 and 144 grid values were analysed for Runnymade and Glenmore slide areas, respectively. The summarized data is given in Table-1.

The mean slope values of the two areas are nearly the same which
TABLE - 1  Showing statistical data of slopes of Glenmore and Runnymade slide areas.

<table>
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<td>0.056</td>
<td>18° to 27°30'</td>
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</table>

is also revealed by test of significance of differences between the two means (D = .006; CR = .779; insignificant).

Test of significance of differences between the two standard deviations (D = .01; CR = 1.68; insignificant) reveals that the dispersion of slopes of two areas are also equal. This indicates that the extent of variability of slopes from the mean is equal in both the areas.

DISCUSSIONS

The detailed appreciation of Glenmore slide area, with the help of Terrestrial stereo-pair has brought out that the talus slopes are culturally best developed. These, however, are affected by slides and subsidences. There has been a large scale deforestation of the creep slopes and only a patchy growth of trees now remains in the free fall area. This has accentuated the downward movement of the weathered material into the talus slopes from above, which can, however, be retarded by afforestation of the creep slopes and free fall face. Apart from selective rapid mass wasting in areas of slides and subsidences, creep is the dominant process of slow mass wasting affecting all geomorphic forms. In view of heavy rainfall and prevalent tropical climate, mechanical and chemical eluviation appears to be significant in the entire area. The significant erosional process operative in the area is the slope wash of barren areas. No appreciable toe erosion is noted along the river channel. The river flows along a fixed channel as the flood plain is stabilized where bushes and trees have grown.

The talus slopes are almost wholly under tea cultivation. There exists a patch of planted dense trees on the eastern margin of the area, amidst tea bushes (Photo P13), even steeper slopes have not failed in it's vicinity. A housing colony has come up in the central portion (P12). Immediately above the colony area, a prominent, discreet, semicircular shear fracture has developed for a length of about 225m. The fracture serves as a loci for seepage of water in this critical portion, accentuating the unstability of the slope.

The drainage in the area is largely subsurface, the water from creep slopes and free fall face percolates into the talus material along the contact. Existing gull-
ies can be used for draining off the percolating water. The subsurface water tends to flow down towards the river bed, as a result the lower talus slopes get oversaturated with water, resulting into the flowing of the material and opening up of a number of fissures in this portion. Earth flow has also occurred, due to oversaturation, at the head of some of the fissures. Forestation, in the lowermost portion of the talus slopes, for some distance above the river bed, can serve as a toe buttressing of the slopes.

Geomorphometric studies have brought out that a prominent break in slope profile occurs around 25° slope value. And the slopes steeper than 25° have failed when tempered with e.g. the shear fracture above the colony area (Pl2) has resulted mainly due to overloading of the slope. Also excavation of the road through steeper area was responsible for the slide occurring above it (Pl2). Thus the slopes steeper than 25° are in a state of critical equilibrium. Further slopes gentler than 18° do not enclose slides having discreet shear fractures but at places fissures have been opened up due to oversaturation of water which, if tackled with, will impart stability to these slopes.

Statistical analysis of the slope values of Glenmore and Runnymade slide areas, has revealed that as far as slopes are concerned two areas have almost identical set up, and therefore, the deductions can safely be generalized and extended to similar land forms.

CONCLUSION

The inter-relationship of morphometric studies and photo appreciation of the terrain has helped in an effective evaluation of the terrain. Hazardous and safer areas could be identified in the context of prevailing geomorphological and environmental set up and suitable remedial measures could be suggested.

Study has brought out that gravity and subsurface water are the two main factors responsible for sliding. Whereas the gravity is most important for steeper slopes, oversaturation of gentler slopes is equally dangerous. Slopes steeper than 25° are in a state of critical equilibrium and, therefore, should not be overloaded unless adequate protective measures are taken. Slopes gentler than 18° are safer for developmental activities, provided sub surface water is adequately dealt with.

The striking similarity of morphometric deductions between the two selected areas indicates a uniformity of slopes of the resultant land form. As such the deductions can be generalized and extended to similar areas.

The result of the case studies, amply demonstrate that terrestrial photogrammetry is the most appropriate technique for a swift and
comprehensive appraisal and analysis of the terrain involving selected problem/research areas. Another highly significant advantage of this technique is scope for the digitization of data, on a tape, simultaneous to map preparation. This would permit a detailed morphometric study with ease, using digitized data.

ACKNOWLEDGEMENT

The senior author wishes to record his sincere thanks to Shri V.S. Krishnaswamy, Retd. Director General, Geological Survey of India for providing him the opportunity to carryout the Terrestrial photogrammetric survey of Nilgiri Landslides. He is also grateful to Shri V.K. Raina, Director, Glaciology Division, G.S.I. for the support and encouragement. It is a pleasure to thank S/Shri G.S. Srivastava and Gautam Ghosh, Geologists (Sr), G.S.I. for fruitful discussions on the subject dealt with.

REFERENCES


MAPS OF GLENMORE SLIDE AREA
PREPARED FROM TERRESTRIAL STEREO-PAIR (UNCONTROLLED)

Geomorphological map

Tea Factory

Metalled Road

INDEX
- Creep slope
- Free fall face
- Talus slope steep/gentle
- River bed
- Fracture line
- Gully

Erosion map

Tea Factory

Metalled Road

River Channel

INDEX
- Sheet wash
- Slope wash
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- Protected slope
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Existing land use map

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INDEX
- Dense Trees
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- Barren
- Tea Plantation
- Built up areas

N.K. AGARWAL & R.P. SHARMA
Photograph of Glenmore area taken from Terrestrial Photogrammetric Camera.
GEOMORPHOMETRIC EVALUATION OF LAND SLIDE AREAS OF KINNAUR DISTRICT, H.P.

EVALUATION GEOMORPHOMETRIQUE D'ÉBOULEMENT DE TERRE DES REGIONS DE DISTRICT KINNAUR, H.P.

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Senior Technical Assistant
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ABSTRACT
The slide areas of Sutlej valley in the neighbourhood of Wangtu in Kinnaur distt. H.P. were studied in details. Large scale mapping of the selected areas on 1:1000 scale was taken up for the better understanding of geomorphic forms/processes and study related to hazards coming in way of developmental activity in the area. The present paper deals with geomorphic/geomorphometric analysis of land slope in Wangtu area.

Geomorphometry provides an extremely useful basis for bringing out important spatial variations over an area which more or less reflect geology and structure. Various geomorphometric maps have been prepared using different morphometric parameters i.e. area ratio, relative relief and bumps. Inter relationship of these quantitative maps with geomorphic/land use map has led to the proper evaluation of terrain. These informations can be translated into parameters that can be used by Engineers/Scientists for long terms planning and development of an area. By comparison/superposition of these maps it is possible to delineate safer areas, hazardous areas and areas in the state of limiting equilibrium.

The discussion of case history under reference has demonstrated that geomorphometric analysis of land scape provides useful information for channelising the developmental activity and planning optimum utilisation of land forms.

ABSTRACT
L'étude en detail sur la région d'éboullement de la vallée de Sutlej
qui est au voisinage de Wangtu en Kinnaur dist de H.P., a été fait. La carte de la région choisie à été trace en détail. Sur l'échelle 1:1000 pour le meilleur comprehension de forme geomorphique/procedures et l'étude de hasard qui se rapport à une activite de development de cette region. A cette presente fascicule donne une analysis sur geomorphique/geomorphometrique de la pente a la region de Wangtu.

Geomorphometrie fournit une base entierement tres utilisable pour representer une importante variation spatial en le region ou la géologie et la structure plus ou moins est negligible. Les cartes differentes de géomorphometrique ont été preparé se utilisant les différente parametre morphometrique comme la ratio de la region, relief relatif et bumps.

La corelation des ces cartes quantitatifs avec les cartes geomorphique carte de utilisation de terre amene une evaluation propose du terrain. Ces information peuvent être traduire en parametres qui peuvent être utilise par l'Ingénieurs les savantes pour une planification de longue terme et en voie de development de la region. En comparisation/superposition de ces cartes, il est possible pour marquer le region de sécurité la region hasardeuse et la région où l'équilibrium est limité.

La discussion de etui pour référence à constate que l'analyse geomorphometrique des les pentes fournit une information utilisable pour canaliser une activite de development et planification d'utilisation optimum de forme de terre.

INTRODUCTION

Authors had the opportunity of studying the Wangtu slide area of Kinnaur district, H.P. in detail both in the field and at Headquarter. The land scape consists of curved sloping surfaces which are largely being shaped by mass wasting. How these slopes are formed, how these are maintained and how they change with time and human interference are some of the questions that the authors have tried to answer by drawing a geomorphic profile and detail analysis of the talus cone of Wangtu slide. A correct understanding of slope is extremely difficult because these are transitional in process and in form, climatic conditions play havoc, increasing saturation converts a soil creep (a very slow process of movement) to earth flow or even mud streams in other wise stable talus cone in Wangtu area. Slope of an area is a combination of highly irregular surfaces that cannot be described
by a simple mathematical equation. A mosaic of these slope segments offer a landscape that we are to deal with. The area under reference has been mapped on 1:50,000 scale for delineating various geomorphic units and the stability of hill slopes on the talus cone of Wangtu has been analysed on topographic maps prepared at 1:1000 scale. The material involved in the study is rock debris that veneers pre-cambrian formations. The debris is partially stabilised and is of holocene to recent age.

HILL SLOPE ANALYSIS

The terrestrial slope profile has an upper convex segment towards the sky, starting from the ridge line with a slope ranging between 5° to 10°. There is a development of soil profile that supports grass and pine vegetation. The dominant geomorphic process operating in the area is soil creep. Any amount of water falling on the surface is absorbed in to the soil cover and there is sub-surface soil and water movement. The degree of sub-surface water soil movement and associated creep increases away from the centre of the ridge (Fig. 1).

Immediately below the upper convex segment of profile is the free fall face with a slope angle of the order of 60° and above. In this zone bare rock is exposed and the fall of material in the form of irregular and angular blocks is a continuous process. Rain water falling on these surfaces moves as a sheet wash without any controlled surface/sub-surface drainage. The dominant geomorphic process acting are falls, slides, chemical and mechanical weathering. This zone is the source of debris that has formed talus cones on the down slope which are the centres of cultural activity.

The segment of slope below the free fall face is talus. This is a constructional landform, built of coarse-rock debris, forming a wedge of the slide rock at the base of steep cliff. This represents, a layer of slide rock that is creeping down. The debris that falls form the rock face, armors the talus in the upper part of the cone and replaces the fragments that are lost by chemical weathering and creep. The angle of the talus slope is a function of fragment size, angularity of the slide rock, climate, vegetation and rate of slide rock supply and removal. It has been observed to be of the order of 16° to 26°. Disturbed zones within the talus however show slope angles of the order of 50°. In these slopes transportation by flowing water assumes dominance over creep. There is a development of rills which join to form rivulets which takes care of the drainage over the area.
The lowest parts of the slope profile are covered by channel wall of the Sutlej river and the channel bed. Channel wall area are comparatively steeper slopes. This segment of profile is exposed to alternate wetting and drying leading to sub-surface soil water movements in the zone of periodic saturation. The dominant geomorphic processes acting are conversion, slumping and fall. Removal of material in this part leads to erosion of the supports and triggers of a series of slide movements in the talus cone above.

In the channel bed, transportation of materials takes place by surface water action. Periodic aggradation and corrosion are also noticed in different stretches of the river profile.

GEOMORPHOMETRY OF TALUS CONE:
Geomorphometry provides an extremely useful basis bringing out important variations in an area comprising otherwise uniform materials. These variations more or less reflect the role of geological/structural elements that are hidden underneath the veneer of debris cone. It deals with surface altitude, gradients, distances and areas in any setting. The variables are in some or the other way a function of altitude of the point being considered. The parameters chosen for the detail analysis are:

1. Surface area ratio
2. Relative relief and
3. Length of the contour line.

Before getting into the analysis of landscape it is felt necessary to briefly describe the parameters as chosen above and their applied aspects.

SURFACE AREA RATIO:
This parameter is designed to determine the amount of similarity between the test area surface and corresponding planar surface. With the increase in irregularities of the surface, increase of slope angle etc., the actual surface area increases. It is a function of altitude differences in the area being considered. The estimated area of the test site 'A' is represented by \( A \), where \( \theta \) is the slope angle and \( A' \) is the corresponding planar area (fig. 2). The ratio \( A'/A \) shows a curvilinear relationship which goes on increasing with the increase of slope angle.

RELATIVE RELIEF:
Variation of altitude in the test area has been taken as one of the parameter to study the ruggedness of the terrain. Maximum number of contours over a uniform grid has been taken as raw data to prepare contours showing an equal ruggedness.

LENGTH OF CONTOUR LINES:
A comparison of contour lengths and grid pattern gives an idea of curved nature or otherwise of the test surfaces. Higher
contour lengths over similar grids reflects the bumpiness of the surfaces. A combination of this parameter with surface area ratios is an effective tool in describing the surface irregularities.

The plate I refers to the detail topographic map of the Wangtu slide area. The total area has been divided into a uniform grid at the interval of 25 Cm. The individual squares could be numbered from any direction. The table-I shows the raw data with the slope of the top followed by number of contour length in centimeter in each square.

GEOMORPHOMETRIC MAPS:
SURFACE AREA RATIO MAPS:
Surface area ratio maps of the Wangtu slide is shown in Plate II. This map has been prepared using secant function of the slope measured over different grids (Tab. I). A perusal of the Plate-II reveals that most of the area covered fall under surface area ratio value of the order of 1.03 to 1.05. Selected areas/strips shows the surface area ratio of the order of 1.07. Values of the order of 1.09 form close contours of comparatively smaller dimensions. Active slide scars in the toe zones of the talus cone show exceptionally high values of the order of 1.7 or so. The mapped slide zones are enclosed by area ratio contour of 1.10.

The secant function is representing first derivative of altitude with respect to horizontal distance and there is a variability of area ratio value from stable zones to the active slides. The area ratio gradients versus horizontal distance i.e. second derivative of altitude with respect to horizontal distances has been considered and shown by the area ratio profiles. Three such profiles have been drawn. These profiles show a sudden change of gradients while approaching the slide areas. The rate of change of gradients is pronounced very much as the profile move from stable areas to critical areas which on the surface may appear as stable slope but will fail with slightest overloading or human interference. The nick point for the change of gradient on the three profiles drawn comes approximating to area ratio values of the order 1.08 which corresponds to a slope value of 22°. The area ratio contour of 1.08 can be taken as the boundary separating stable slope areas from the critically stable slopes with or without active slides.

RUGGEDNESS MAP:
Relative relief has been taken as another parameter to evaluate the terrain. Maximum number of contour lines in a grid giving the picture of altitudinal variations over the test areas (Table-I). Using this parameter
a map has been prepared showing equal number of contour lines shown over the grid as plate-III. Most of the area covered is falling between 3 or 4 contour of 5 M each in the grid. Flatter areas with a maximum two contours have also come and extremely rugged areas with 5, 6 and 7 contours per square grid have also been separated. An association of active slides with areas isolated within 6 or 7 contours per square grid has been noticed.

MAP SHOWING BUMPINESS:
Contour length in the individual square is an index of concavity or convexity of the area. The flat surfaces will have minimum contour length, with increasing curvature the contour length will also increase. From the raw data (table-I) bumpiness map has been prepared and shown as plate-IV. Higher contour length areas associated with flatter terrain lowest contour length areas are on the actively sliding portions of the debris however all the lower order contours cannot be extrapolated for identifying critical slope with or without slides.

CONCLUSION:
It is possible to extract valuable information regarding the utility of terrain under consideration by properly evaluating the maps. The hazard areas with surface area ratio higher than 1.06 can be isolated. Cultural activity in these critical zones has to kept to a bare minimum or with preventive safeguards, Road alignment etc. as far as possible can be abandoned. Combination of information stability of slope with ruggedness and bumpiness can be used for planning settlement, afforestation, road building activity to avoid unsuitable areas or alternatively for preventive measures to be taken up for any project in these areas.

ACKNOWLEDGEMENT:
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THE SEVEN CLASSES AND SEVEN MODELS OF LANDSLIDE’S CLASSIFICATION AND THE LAW OF LANDSLIDE’S DISTRIBUTION IN CHINESE RAILWAYS

LES SEPT CATEGORIES ET SEPT MODELES DE LA CLASSIFICATION DE L’ÉBOULEMENT DE TERRE ET LA LOI DE DISTRIBUTION DE L’ÉBOULEMENT DE TERRE AUX CHEMIN DE FER CHINOIS

FU ZHUAN-YUAN
Engineer
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China

ABSTRACT

Between 1973 and 1976, the author had taken part and managed the special topical work of preparing a monograph on the study and classification of Railway Landslides and the law of their distribution in China and in this regard had accumulated abundant data. This paper based on the author's study and experience and utilising the data on investigations of material of all railway lines in China, has advanced a new suggestion for the classification of the slides, besides outlining briefly the law of their distribution along Chinese railway lines.

INTRODUCTION

I. The starting point of classification and the outline of project:

Landslides as a kind of geological phenomena, have their conditions of formation and sliding properties strictly controlled by geological conditions. For any material landslide, its main property is always ruled by specific geological condition. The basic property of slip is mainly controlled by geological conditions are through the material component of slip body and the genesis of slip band. Hence, this classification of landslides synthesizes the component of material of slip body and the genesis of slip band. To classify landslides as the first series according to difference in the component materials of slip body, is extensively used in China. In this paper, I shall not describe this again, since slip band is an attaching band which the sliding body depends upon to slide and its character and change of component materials will directly determine the degree of stability of the slide and many specialities of deformation. The characters of slip band (space-figure, thickness, material composition, texture, structure, law of variation for attenuation of strength in slip band, etc.) are basically determined by geological genesis.
In fact, many fundamental features are controlled by the properties of the slip band. Therefore, classification of landslides according to genesis of slip band into different models is more reasonable than classification into classes.

Many fundamental features are of course controlled by different properties of component materials of the slip body and varying properties of slip band (sometimes by the nature of slip bed). Similar materials of slip bodies may have different sliding properties because of their different modes of genesis of the slip bands. In the same manner, similar genesis and property of slip band may also induce different property of sliding, because of the difference in their materials. Only when the materials of slip body and the genesis of slip band are all same, the sliding will have similar properties. Hence, this classification based on the materials of slip body makes the first series of classifying-class. Then based on the nature of genesis of the slip band, a second series-Model is made. For this reason we termed this classification as a combined slip body component with genesis of slip band or synthetic classification.

Specific character of this classification is that the different types of slides (which combine class with model) will have specific condition of their origin and the feature of their movements.

II. The first series classification of landslides-Class.

1. Bedding plane rock slide: It is an integrated rock slide in which the inclination of bedding surface is the same (or nearly the same) as the inclination of the slope. This slide generally occurs in those geological conditions where hard and soft rock layers are interlayered or where soft layers lie at the bottom of hard rock layers of large thickness. When the soft and weak rocky layers have been disturbed by structural shift to form fault bands between these layers and when the strength is considerably reduced, bedding plane slides will occur more commonly. Bedding plane rock slides are mainly distributed in interbedded sandstone and shale sequences containing coal, sand-shale, conglomerate with shale (or mud stone) intermediate layer, limestone with calcareous shale or calcareous marl, and is noticed rarely in very few metamorphic rock areas.

2. Cutting plane rock slide: This is a whole rock slide with inclination of bedding plane of the rock in reverse direc-

tion to the inclination of the natural ground slope. Generally this slide occurs in the rock body in which faults have developed well or which have borne metamorphism in different degrees. The slip planes include many kinds of soft and weak structural planes except the rocky bedding surface, such as, fault planes, joint planes, etc. Cutting plane slides may occur in various conditions of rocks, but in metamorphic rock areas, this class of slides is the most common phenomena.

Considering the whole degree of rock in combination of slip body, the above bedding plane slide and cutting plane slide can be all called integrated rocky slides.

3. Broken stone slide: In contrast to whole rock slide, the slip body which is composed of broken stones is termed as broken stone slides. It is often encountered in broken structural belt or in areas of strong physical weathering.

4. Clayey slide: These slides were called "Clay slide" in the past. They occur mainly in the deposits of bays, lagoons, lakes and shore. In these deposits, the particle size is mainly that of clay or various clays and clayey earth from heavily or deeply weathered soft rock and easily weathered rocks.

According to the nature of their internal texture, they may be classified as uniform clayey earth and non-uniform layered clayey earth; the former as weathered remnant deposit clay or sedimentary clay of large thickness, are distributed in rainy, dampish, strong chemical weathering areas and bay, seashore, etc. in our Southern Country. The latter can be found in inland lake basin area of middle and small pattern of Cainozoic era in North, Middle and South-West China, as in Chengtu basin of Szechuan province, Tzen district basin of Shansi province, An Kuang basin, Han zhung basic of Shian S Province, Meng Tsi basin of yuen Nan province. The slides within these two kinds of clay have significant difference in their sliding features because of the differences in characters and textures.

5. Loess slide: Slides occur in loess and loessial soil. The loess and loessial soil are of wide distribution in North and Northwestern China. The special lithological characters and texture cause a large number of slide development. Hence these slides in the above
The project of classification is as Table 1. Table 1

<table>
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<th>rock slide</th>
<th>earth slide</th>
<th>embankment soil slide</th>
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<td>Class C (according to main component of slip body)</td>
<td>integrated rock slide</td>
<td>broken stone slide</td>
<td>clayey slide</td>
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<td>Model (according to main genesis of slip band)</td>
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| squeezing out sliding | 0 | | | *
| outwashing sliding | | | |
| plastic flow sliding | | 0 | Δ |
| shearing sliding | Δ | | Δ |
| liquifying sliding | | | 0 |

Remark: * the most extensively spreading model; Δ the general spreading model; 0 the non extensive model; blank space is no spreading.

Areas are classified separately.

The composing materials of slip body in the loess slides are Quaternary (from early Pleistocene to Holocene) accumulation of loess (alluvial, pluvial, aeroliou, delivety).

6. Accumulated soil slide: The materials which compose the slip body are of varied genesis (declivity, collapse, pluvial, falling and interrelated genesis) and include loose grounds on the slope and at the foot of the slope. Their main lithological characters are sand-clay with broken stones. These slides are mainly distributed in bank slopes of mountainous rivers where geological structures are more complex.

7. Embankment soil slide: The materials of the slip body are composed of artificial fill.

The production of slide in filled soil areas has two fundamental reasons: First, due to cutting into the toe of man-made embankment which has been in relative equilibrium, or due to heavy rain which adds weight to the slip body and soaks the slip band, causing the sliding force to be greater than sliding resistance, and forming shear plane within the embankment, thus sliding downward along the shear plane. Secondly, near the contact band between artificial fills and old earth's surface, due to bad quality of the earth, when the collection of underground water increases enormously, thus causing the embankment sliding along the old earth's surface.

III. The second series classification of landslides—Model.

The classification of landslides into seven classes according to the component materials of slip body is obviously very easy to distinguish and has obvious distribution law. But it cannot fully reflect the characters of sliding for each material slide, because among the factors for this determination, besides the composition of slip body, (there still exist the lithological characters and genesis of slip band). In other words, in any class of the seven classes which are classified according to the composition of slip body, the models of sliding are not in general only one kind. For example, in the same loess slides, due to the distribution area and genesis, their internal
texture and lithological characters are different. Therefore, they have great differences in the characters of sliding, viz., some of slides are because of the outwashing for water-bearing sand bed in loess strata (the outwashing model); some of them are because of strong permeable strata along impervious bed (the contact model); and still some more are of sliding controlled by structural surface which faces on air space and produce rapid and strong sliding (the structural model). Other classes of slides have similar things, too.

For the full expression of these sliding features of each actual slide, after we have made the first series classification of landslides-Class, according to the genesis difference of slide band we have made the second series classification of landslide-Model.

1. Structural model sliding: This sliding which occurring along soft and weak textured plane produced by geological tectonic movement is called "Structural model sliding." This kind of soft and weak textural plane includes consequent low angle bedding surface of soft and weak rock, interlaminar fault planes, fault planes and great joint plane, etc. Their inclinations are generally from 10° to 30° and rarely more than 30°. When this inclination is greater than 30°, slides come under the category of "falls".

Structural model slides are in general produced in slopes which are formed by bed rock. They are the main model of sliding in rock slides, and widely distributed in China. From preliminary statistics in railway department, structural model slides are almost one-fourth of total slides. Structural model slides often occur in areas of bedded rocks, consisting of alternate soft and hard rocks, in areas which have developed fault structure and in metamorphic rock terrain. In Szechuan basin, Shian Pei basin and other medium or small structural basins of Mesozoic era and Cenozoic era structural bedding rocky slides are distributed in concentrations.

2. Contact model sliding: This sliding which takes place along the contact band or plane of two kinds of rock formations or earthy layer is called "Contact model sliding". This band (or plane) has different lithological characters and permeability. Its slip band is generally the result of recent geomorphic process. The slip band includes the piedmont depression planes, the pedestal planes in river pedestal terrace, the contacting planes between different lithofacies deposit layers in the Quaternary period, accumulation layer and the rocky (or earthy) contacting bands of different degrees of weathering, etc.

The production of contacting model sliding is mainly related to the large quantity of groundwater aggregated in the vicinity of contacting band, this groundwater can soften the earth and decrease the mechanical strength of the earthy matter.

Sometimes the contact model sliding may be transformed from the structural model sliding.

The contact model sliding is the most important model of various earthy slides. It is widely distributed in China.

According to statistics of the railway department, the number of contact model sliding is more than half of the total of slides that occur in railway lines.

3. Plastic flow model sliding: According to the substance of the sliding process, there is no essential difference between plastic flow model sliding and contact model sliding. The slip band is also moulded from modern geomorphic process. But this model sliding is different with the gentle slip plane of contact model sliding, its slip plane appears more distinct trough due to the linear erosion. Besides, the water content of slip body of plastic flow model sliding is greater than that of contact model sliding; hence in times of sliding, it will appear more or less as of flowing character. But the property of moving, the landform or the place of origin, etc. are different in essence from that of mud-rock flow.

When there is tractive condition and a great compensation of groundwater, the plastic flow model sliding can be transformed from contacting model sliding.

4. Outwashing model sliding: This is a special model of sliding in loess areas (mainly in plateau model loess area). This model of sliding is determined by the intercalations of the powdery earth, silt, silver sand, coarse sand and fine gravel, etc. In loess plateau area the intercalations are distributed very widely. When it possesses enough hydrodynamic pressure, because it accepts the full alimentation of fault crevice-water, the sandy soil within the intercalations will be washed out gradually and be carried to facing space, thus causing sudden downward move-
ment of overlying earth's body and sliding forward.

In North and north-west loess plateaus, the middle Pleistocene loess distribution area has many thick and large loess slides. These slides belong to outwashing model sliding according to genesis of slip band.

The loess distribution and character of loess sedimentation in having specific regional nature, the outwashing model sliding possesses the character of universal distribution in a certain range, and rarely has fragmentary distributions.

5. Liquifying model sliding: A portion or whole lot of intercalation within the sloping body which is liable to liquify, will be liquified due to vibration and will then suddenly lose its strength (thixotropy). This action will cause the sloping body to slide along the liquified layer and this phenomenon is called liquifying model sliding.

The slip band of this sliding is formed in sedimentary process (or man-made embankment process). The materials of slip body have a series of properties conducive to liquification. i.e., a certain particle size (<0.01 mm, while <0.001 mm particle must have an enough percentage content), shape (schistose), mineral composition (clayey minerals with great hydrophilicity) and texture (dispelling unstability with Change of outside conditions).

6. Squeezed out model sliding: A soft layer (with a certain thickness) in the interior of slope body may be squeezed out under the weight of overlying huge thick rock formation so that the overlying rock formation slides downward. This phenomenon is called Squeezed out model sliding. Generally this phenomenon occurs in such conditions where the inclination of rock formation is flat (only some degrees) and the rock formation inclines towards the open space face. This kind of sliding is a specified model of bedding rock sliding. One of the differences from the structural model sliding under bedding condition is that great quantity of earth of slip band is squeezed. In the facing space the squeezed soft rock and earth can be seen frequently.

The most basic reason of production of squeezed out model sliding is that the material of slip band has a certain thickness. The shearing strength of slip band (with large content of moisture) is smaller than that of rock slip body and slip bed, and the difference of shearing strength is very great. For this reason, before the formation of shearing plane between the material of slip band and its upper or lower contact bands, the material of slip band is squeezed forward, gradually decreasing its strength, and causing the sliding of the overlying rock body.

7. Shearing model sliding: The phenomenon of sliding alone burst plane in the interior of slope body due to sliding force being greater than resistance is called shearing model sliding. It is produced in an earth body in which the lithological characters and texture are uniform or more uniform. Generally speaking, in the slope body there is no soft and weak band existing as to behave as yielding band for the future sliding, but mainly due to man-made dissection of slope sudden unloading is caused or due to infiltration and soaking of water into slope body partly or wholly, the load of slope body is increased. When the stress in the vicinity of shear belt constantly exceeds the shearing strength, there will produce a neogenic broken plane along the hidden shear belt. Then sliding will produce with evolution of shear plane at the same time. Just for this reason, some call this neogenic shear plane as contemporaneous plane. The feature of this plane is generally in the form of cylinder.

The mechanism of formation of the cylindrical shaped shear plane has been studied specially by predecessors and it will not be repeated in this paper. We confess the fact that in homogeneous clay areas, man-made cut will cause transition of moisture in slope body. At the same time, the change of gradient of pore pressure causes the groundwater in the slope body transit itself to certain latent cylindrical shape shear plane with the slope foot as the broken end. This action will constantly decrease the strength of earth's body in the vicinity of this plane. The sliding force will begin (and gradually) to exceed the resistance maintained by the shearing strength. Thus it causes the slope body which is in equilibrium or limit of equilibrium state to evolving gradually a continuous shear plane. The earth body above will slide forward along this plane.

IV. Terminology for defining the type of landslide

This plan for division of landslides was advanced to synthesize and summarize the phenomena which had existed in the natural state. The term includes both
the class and the model simultaneously so that it reflects the movement integral to the sliding quality and the main character of a certain landslide. When we want to make such a term of a landslide, we must put the model term first and its classificational term, later. For example: Shearing clayey landslide; structural layer rocky landslide; etc.

V. Law of distribution of landslide in Chinese railways.

1. General situation

Distribution of natural landslides are very widespread in China. The railways passing through all kinds of geological and geomorphological environment encountered a vast amount of landslides especially along the mountainous rocks. After the reconnaissance survey for the landslide along the railways in China, the incomplete statistics showed that there were more than one thousand main landslide.

According to the above-mentioned divisional and naming principles for landslides (as shown in chapter III), we tabulated them as table 2.

2. The law of landslide distribution:

1) The relation between the strata and the rocky characters: Table 2 points out briefly the situation of every kind of landslide. It shows that the strata and the rocky characters are the fundamental conditions for controlling the slide distribution, and are the main factors for controlling the landslide's distribution in concentrated areas and ranges, too. This case may be seen clearly from "Landslides distribution and lithological characters of strata map in Chinese railway lines" which is compiled by author (but this map has not been published yet). According to the relation between the lithological characters of strata and landslide's distribution, we divided the exposed rocks (or earth) along the railway lines into ten rock groups, that is: clayey, loess, accumulated soil, conglomerate rock with intercalation of mud stone, sandstone with intercalation of shale, interbedding of sandstone, coal and shale, carbonate rock, metamorphic rock, intrusive rock and volcanic rock. Of these five rock groups have greater prominence considering the distribution of landslides, they are: accumulated soil, sandstone with intercalation of shale, interbedding of sandstone, coal and shale, clayey media and metamorphic rock. Among these five, specially slides in the accumulated soil rock group, account for about half of all landslides along railway lines of our country.

2) The relation between the geological structure. In brief, this relation mainly appeared as follows:

a. The structural model and contacting model of landslides were in concentrated distribution at the vast structural areas with strengthened activities along the contacting band of different structural areas.

b. Nearby the big fault zones, are usually distributed a large amount of structural model broken stone slides. For example, Bai Guo to La Bai section of Chang Kun line is situated at the influenced zone of big fault zone along Pu Xong river, and there are more than 20 slides, either big or small in a range of 40 km.

c. Along the railway lines running parallel to the main structural line in the area there occur a large number of landslides distributed in clusters. The New Hua Xia structure in NNE direction of Fu Jian province is the main structural line at that place. In Zhang Ping-long Yan-Kan Shi, Sha Xian-Yong An, Zhuo Zei-Mai Yuan and Hua An districts, the railway lines runs along the direction of NNE too, and therefore the landslides are mainly distributed in these sections.

3) The relation between the landforms:

a. Partial gentle slope areas along steep mountain valley slope section are the main distribution areas of Contact accumulated soil slides.

b. The structural bedding slides and contact clayey slides are mainly distributed in the areas of gentle hill landform at the edge of the intermontane basin.

c. Convex slope or spur with the trend towards the space, could produce structural model bedding slides. If there is certain fault passing through, then this may produce structural broken rocky slides. For example, in the district of Xiao Long Dong station of Xiang Qian railway, situated at the fault zone extending from Nan San river to Ma La Zei, the railway crossed five convex spurs with short ravines one by one, and along all these spurs there appeared correspondingly five structural broken rocky slides.

d. The gentle slope area of cuesta produced structural model bedding slides and contact accumulated slides. The slides at K373 and K343-345 sections of Bao
Cheng railway are just the two typical examples of such a case respectively.

e. Linear prolonged fault cliff of the accumulated landform under the fault cliff usually coincided with the distribution of the contact accumulated soil slides. The examples are the slides clusters in K161-163 section of Ying Xia railway and K116-118 section of Chuan Qian railway, etc.

(4) The relation with the hydrogeological texture:

a. A large amount of loose ground in the Quaternary Period piled along the gentle slope or concave slopes of hills and valleys. These having a homogeneous texture, and possessing interstratified beds between the water-bearing and the water-resisting layers gave rise to a great deal of ground water flowed along the top surface of bed rock due to the difference of permeability between the loose ground and the top surface of bedrock, and decreased the strength of the earth in the band. This is the main hydrogeological condition which produced the contact model slides.

b. The mountain slopes that are changed with groundwater will usually produce such slides.

c. The buried ancient trough having accumulated ground water may lead usually to the production of great contact accumulated soil slide and embankment soil slide. The peculiarity in Bao Cheng, Cheng Kun, Yang Xia, Xian Qian and Chuan Qian railway, etc. is that the accumulated of soil slides and embankment soil slide were in concentrated distribution.

d. Gravel and sand intercalation are the main water-bearing horizon in the loess stratum. In Jin and Shaan and other provinces where such a type of hydrogeological conditions (such as: Dou ji Tai slide, Wo Long Shi slide, etc.) exist it has given rise to many a large scale outwashing model loess slides.
Table 2. The distribution of every type of landslides along Chinese railways.

<table>
<thead>
<tr>
<th>Degree</th>
<th>type of slide</th>
<th>area of distribution</th>
<th>name of railway line</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>structural bedding rocky slide</td>
<td>Areas of layered sedimentary rocks</td>
<td>Cheng Kun, Chuan Qian and Xiang Qian lines</td>
</tr>
<tr>
<td></td>
<td>structural broken rocky slide</td>
<td>Fault developed structure areas</td>
<td>Xiang Qian, Ying Xia and Cheng Kun lines</td>
</tr>
<tr>
<td></td>
<td>Contact accumulated soil slide</td>
<td>Areas of well developed fold and fault structures (sloping land before the hill)</td>
<td>Bao Cheng, Xing Xia, Chuan Qian, Kun He, Zhang Long and Mei Qi lines</td>
</tr>
<tr>
<td></td>
<td>contact clayey slide</td>
<td>inland basin of the Cenozoic era</td>
<td>Tai Jiao and Jiao Zhi lines</td>
</tr>
<tr>
<td></td>
<td>contact embankment slide</td>
<td>Southern rainy areas where filled up dykes (materials filled up on the old sloping earth surface)</td>
<td>Qian Qui, Chuan Qian, Cheng Yu and Ying Xia lines</td>
</tr>
<tr>
<td></td>
<td>outwashing loess slide</td>
<td>plateaual type loess areas</td>
<td>Long Hai and Xi Yan lines</td>
</tr>
<tr>
<td></td>
<td>structural cutting rocky slide</td>
<td>layer cutting areas of sedimentary rock and metamorphic rock</td>
<td>Bao Tian, Xiang Yu, Xiang Qian and Wai Fu lines</td>
</tr>
<tr>
<td></td>
<td>contact loess slide</td>
<td>loess areas of the upper Pleistocene</td>
<td></td>
</tr>
<tr>
<td></td>
<td>plastic flow accumulative soil slide</td>
<td>Areas of loosely piled materials distributed at the foot of hill with plenty of ground water sources</td>
<td></td>
</tr>
<tr>
<td></td>
<td>structural loess slide</td>
<td>loess areas of the upper Pleistocene</td>
<td></td>
</tr>
<tr>
<td></td>
<td>shearing clayey slide</td>
<td>areas of strong weathering, southern limestone and granite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>shearing embankment slide</td>
<td>filled up dykes and areas of piled materials with homogeneous quality and having more thickness</td>
<td></td>
</tr>
<tr>
<td></td>
<td>squeezing out bedding rocky slide</td>
<td>areas of Flysch growth (big, thick and hard rocky layers were mixed up with thin, soft rocky layers)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>liquifying embankment slide</td>
<td>areas of poor earth quality in the embankment bases</td>
<td></td>
</tr>
<tr>
<td></td>
<td>plastic flow clayey slide</td>
<td>clayey areas with quite enough of groundwater sources</td>
<td></td>
</tr>
</tbody>
</table>
SLOPE STABILITY PROBLEMS OF THE ACROPOLIS HILL OF ATHENS, GREECE

PROBLEMES DE STABILITE DES PENTES DU ROCHER D'ACROPOLIS D'ATHENES, GRECE

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ABSTRACT

The Acropolis hill of Athens, on which have been founded the well known historic monuments, presents serious problems of slope stability. The upper part consists mainly of limestones, underlain at the base of the steep slope, by the schist-sand-marly series.

The intense and multifarious fracturing of the limestones, the erosional-weathering processes, the steep slopes and the undermining at the base, in conjunction with the varying geomechanical behaviour of the underlying phase, contribute towards the loosening of the rock, resulting in the manifestation of rockslides and falls.

On the basis of the engineering geology study executed, the mechanism of weakening and loosening of the limestone mass has been studied, the critical zones were located and a series of proposals, for more investigation work and some immediate protection measures to be taken, were made.

ABSTRAIT

Le rocher d'Acropolis d'Athènes sur lequel sont situés les fameux monuments historiques a des sérieux problèmes de stabilité des pentes. La partie supérieure principale est constituée des calcaires qui sont reposés à la base des pentes sur des couches de faciès schiste-gréseux-marneux.

La tectonique intense et diversifiée des calcaires, les mécanismes d'altération et d'érosion, la raideur des pentes, le sous-creusement physique de la base du rocher et le comportement différencié géomécanique de la phase sous-jacente, ont favorisé la relâchement de la roche et par conséquence la manifestation de glissements et chute rocheuse.

A partir d'une étude de géologie de l'ingénieur qui est faite, on a étudié le mécanisme d'affaiblissement et relâchement de la masse calcaire, on a décelé les zones de susceptibilité élevée et on a proposé une recherche plus détaillée et une série de mesures d'urgence à prendre pour la sécurité du rocher.
INTRODUCTION

The investigation of the geological-geotechnical conditions of the foundation sites of ancient monuments has been included in an effort, recently made by the State, for taking the appropriate protective steps for their preservation.

Concerning the geological structure and tectonics of the Athens Acropolis hill many geologists have, at times, been occupied, within the framework of mapping the wider area of the Athens basin (Marinos et al. 1971, 1973; Doumas & Gaitanakis 1981). Certain thoughts, also, on the tectonics and the preservation ways of the present morphology of Acropolis refer exclusively to the hill (Trikkalinos 1972, 1975).

A geotechnical study of the hill, which has been recently undertaken, was aiming at the investigation in depth, of the geological-hydrogeological conditions, the appreciation of any, per chance, existing risks connected with the geology of the area, and the formulation of proposals for taking the necessary steps which are indispensable for further securing the stability conditions of the hill (Andronopoulos & Koukis 1976).

Within the framework of this last study a detailed mapping of the area, on a scale of 1:500, has been effected with a study of the tectonics and tracing of the main and secondary faults, compilation of microtectonic, hydrogeological and seismicity maps, appreciation of certain physical-mechanical features, study of the natural conditions of the rocks, appreciation of the erosional-weathering processes action and indication of the unsafe zones.

The results of this study are mentioned in the present work, and some thoughts are formulated on the stability conditions of the hill.

GEODEtical STRUCTURE OF THE WIDER AREA

The geological basement of the town of Athens is formed, up to a considerable depth, by the formations called Athens schists, of flyschoid type in general, consisting of argillaceous schists, sandstones, cherts, marls and limestones, mainly in the upper horizons, as well as of small outcrops of igneous rocks (diabases, spilites, serpentinitized peridotites).

The Acropolis hill, as well as other hills of the town of Athens, are covered by compact, thickbedded and massive limestones, of Upper Cretaceous age. The relationship of these limestones with the underlying schist-sandstone-marl series is interpreted by the various researchers, in different ways, that is tectonic cover by overthrust, conformable and regular contact, and transgression. Of the above aspects only the first one can have repercussions on the general stability conditions of the hill, through a probable activation of the tectonic contact.

GEODEtical CONDITIONS OF THE ACROPOLIS HILL

GEOLOGY

The geological conditions of the Acropolis hill are more especially described below (Figs 1 & 2).

The Athens schists occur at the base of the slopes of the hill and consist of the schist-sandstone-marl phase (mainly sandstones, less argillaceous schists). They are moderately weathered and intensively ruptured formations, faulted in a general E-W up to N 60° E-S 60°W direction.
Fig. 1 Map of critical locations on the Acropolis hill and the proposed investigation places.

Carte des sites critiques de la colline d’Acropole et les endroits proposés à étudier.

multifolded—crumbled at places, where schist is more frequent. At the southern and western slopes a formation of conglomerates appears, which regularly appears in the formations of the system, having thickness up to 10 m.

At the southern slope of the hill phenomena of cataclastic deformation are observed at places, with mylonitization and brecciation to the contact of the conglomerates and the limestones. Their importance is local and they are connected with tectonic displacements. On the presence of this conglomerate, to which tectonic origin has been attributed, was based the attitude of certain researchers about overthrust of the limestones on the Athens schists, which however is, in our opinion, not true.

The contact is regular, with an obvious evolutionary transition of the underlying lithological phases to the overlying limestones. An evident superposition of beds is, everywhere, not distinct
Fig. 2 Geological sections of the Acropolis hill.
Coupes géologiques de la colline d'Acropole.

and this is due to differential bendings (on account of irregular mechanical behaviour during the faulting) while at other places the contact is modified on account of subsequent vertical movements (Fig. 3).

On the upper parts of the hill, altitude +118 up to +156, unbedded massive limestones prevail. They locally appear
Fig. 3 South part of Acropolis. Irregular contact of the limestones and the underlying phase, due to differential bendings and faulting.
Partie Sud d’Acropole. Contact irrégulier entre les calcaires et les faciès "schistes-grès" sous-jacents, due à la stratification différenciée et à la fracturation.

Fig. 4 NW part of the hill. Caves of big dimensions in the limestone rock, extended under the walls.
Partie NW du rocher. Les cavités à grandes dimensions observées dans le rocher se continuent sous le muraille.
layered in thickplaty beds, with a usual dip towards the interior of the hill. They appear with an intense and multifarious rupturing and are strongly karstified, with the presence of cavities and caves, a fact which can be explained by the existence, initially, of an extended and thick limestone formation and residues of which consist, due to erosion, the limestones of the hill (Fig. 4).

TECTONICS

The Athens schists are included in the sedimentary series of the Eastern Greece zone, which have been folded during the lower Tertiary, with the main axis being of a NE—SW direction. The rupture of the region followed and the final stage of geomorphogeny is completed during the Dilluvium, mainly by the manifestation of epirogenic movements.

Basic elements of the tectonics of the hill are the condensation and uniform structure of the limestones and the underlying schist—marl—sandstones, with a fold axis fluctuating around the E—W direction. The contact as above mentioned, however, shows evident bending and is not flat everywhere. The beds dip gently in a syncline structure (Fig. 2).

During the main phase of the folding or immediately afterwards, the development of horizontal stresses of balancing has taken place, resulting in differential mass displacements on horizontal or small dip planes, mainly at the contact of the two units. The local mylonitization, to which wider importance has been attributed by other researchers, is due to these movements, as well as the striations of an E—W direction in limestones.

During the phase of rupture that followed, a net of fractures has been developed, mainly in the limestones, with a main E—W axis as well as a posterior rupture system of a N—S direction (Figs 5 & 6), both without any important displacements of beds. Also in the upper part of the hill the traces of rather insignificant features, mainly of E—W direction and without any serious vertical displacement, were observed.

Finally, in the last tectonic phase, there was a crushing and displacement of limestone masses, which are due to gravity or the influence of remaining stresses. Crushing and falls of isolated limestone rocks have also been observed in recent years due to destruction of the support and excess of shearing strength. The zones of important loosening of the limestone masses can be observed on Fig. 1.

From the microtectonic analysis, which has been effected on the limestone rock, it has been shown that in certain positions of the slopes unfavourable conditions occur, which are due to unfavourable orientation of main planes of weakness, in conjunction with the presence of major ruptures, karstic voids, increased density of joints towards various strikes and presence of open, intersected fissures of considerable width and length. The characterization "unfavourable conditions" based only on the tectonic data, does not suggest an imminent danger of crushings, detachments or slidings of limestone masses.

HYDROGEOLOGICAL CONDITIONS

The hydrogeological conditions are basically determined by the behaviour of the two main lithological phases.

The limestones are characterized as a permeable, karstic formation, which is
Fig. 5 Intense and multifarious fracturing and loosening of the limestone rock in the NE part of the hill.
Fracturation intense et diversifiée et relâchement de la masse calcaire vers la partie NE de la colline.

Fig. 6 As in Fig. 5, the jointing pattern of a limestone block is shown in more detail.
Dans le rocher on voit en détail le réseau des diaclases.
due to the intense tectonism and the karstification, while the schist-sandstones are practically impermeable.

Taking into consideration the various parameters and the infiltration coefficient 0.1 and 0.3, for the earthfill and the limestones respectively, the infiltrating water is amounting to 4-500 m³ yearly, which is discharged through three small springs in different parts of the contact.

SEISMICITY

Concerning the seismic risk of the area, from the collected data it is derived that it is included in a zone almost aseismic, where there is no danger of seismic vibrations of great intensity because: (1) local earthquakes of great magnitude are not mentioned, (2) the lithological units consist of cohesive material with homogeneous mechanical characteristics and high E-values, moreover these units show regular contacts and are characterized by small natural period, (3) the distant strong earthquakes are relatively harmless on account of the high absorption of the seismic energy.

The influence of the small vibrations to which the area is exposed, in a constantly increasing scale, is quite interesting.

EROSIONAL - WEATHERING PROCESSES

The mechanical properties of the rocks and their instability are considerably influenced by the weathering and erosional processes of the lithological formations. The action of the weathering factors appears more intense on the northern and eastern slopes, where the rupture net is dense.

The climatic and lithological conditions favour the combined action of the natural and chemical weathering factors (surface water, atmospheric air, seasonal temperature changes, wind, frost, atmospheric pollution), resulting in the widening of the fissures, loosening of the rock and proportionate decrease of the shearing strength. In certain cases a detachment and free fall of limestone blocks follows.

Concerning the action of underground water, it should be characterized insig-
ificant, on account of the small quantity infiltrating and its easy drainage. An abrupt fluctuation of the water level and sudden increase of the hydrostatic pressure, with a proportionate increase of the shearing stresses is, therefore, not expected. However, there is a slow and weak action of the water, which influences the mechanical strength.

ROCK RUPTURE

Concerning the rupture, as above mentioned, in the greater part of the western and southern slopes, the limestone rock is cohesive and compact and it is crossed by relatively sparse fractures net (faults and joints), while on the upper surface the relatively dense joints are filled with calcitic material.

On the northern and eastern slopes, however, the conditions are more unfavourable on account of the intense rupture of the rock, which at certain places is considerably loose. These conditions are due to the density of the ruptures (Figs 5 & 6), the unfavourable orientation and dip of the rupture planes, which are combined with planes of small dip or horizontal ones, open fissures along the planes and presence of discontinuity planes with dip
parallel to the slopes, resulting to the loosening of the rock. The initially high mechanical properties of the limestones are, thus, considerably reduced and correspond to the residual strength of the fractured rocks.

**GEOMECHANICAL CHARACTERISTICS OF THE ROCKS**

As far as the mechanical characteristics of the rocks, which partake in the structure of the hill, are concerned, it is emphasized that a partial sampling and carrying out of laboratory tests has been effected, aiming to the determination of indicative values for the various lithological horizons, which however are not regarded as representative of them.

Especially, concerning unconfined compression strength for limestones the values were 401-455 Kg/cm², for schistose sandstone-marls, with a considerable nonuniformity observed, the cores vertical and parallel to the bedding showed values of 335 and 116 Kg/cm². The strength in sandstones has been estimated, for moderately weathered material, to be 123 Kg/cm², while for fine grained, slightly weathered 382 Kg/cm² and is clearly influenced by the texture, the degree of weathering and the orientation of the bedding planes. In the conglomerate the values are ranging from 233-253 Kg/cm².

In connection with the modulus of elasticity, in limestones the estimated values were 100.000 up to 275.000 being relatively low, in the schistose sandstone-marls 150.000, the conglomerate 625.000 and in sandstones 250.000 up to 625.000. The Poisson's ratio is for the limestones 0.29, the sandstones 0.15-0.22 and the sandstone-marls 0.21. The above values of the samples in question are indicative of satisfactory strengths in general.

**STABILITY OF SLOPES. SAFETY OF THE MONUMENTS.**

On the basis of the above analyses we could outline the picture of the area, which has been studied from the security conditions and slope stability aspects as follows:

a) On the flat top part of the hill no manifest activation is observed from the fractures which are of E-W and N-S direction and without any visible vertical displacement, the joints are filled, the karstic erosion process of the rock is not visible neither any zones of important looseness of the structure have been observed due to the action of the weathering factors.

The foundation bedrock consequently is not justifying any fear for the safety of the monuments. On the places of artificial earthfill, of a thickness up to 14 m, the foundation of the monuments is made either directly on the limestones or through cubic limestone blocks and this is the reason that not even the slightest vertical displacement of them has ever been observed.

b) The expressed views for any creeping movements on the entrance part of the Acropolis (western side), on account of damages of the buildings, cannot be justified and the damages should be attributed to other reasons, for instance vibrations due to explosions. However, in order to eliminate any doubt the carrying out of systematic precise measurements is suggested.

c) As far as the slopes are concerned they present certain characteristics like:
- The heterogeneous contact at the base of the slope does not show indications of movements activation on a level along or close to it.
- The dips of beds are favourable.
- The existing hydrogeological conditions do not justify an important action of the underground water neither any abrupt fluctuation of it.
- On the northern, southern and eastern slopes mainly, zones of a dense net of discontinuity surfaces and looseness of the limestone mass are observed. Here there is a danger of crushing and detachment of the limestone masses.
- The action of the natural-chemical weathering factors is more intense on the northern side and even where the voids are localized. On this side the looseness of the rock is possible to become critical, without however suggesting an imminent danger.
- At the zones of special weakness the taking of protective measures against weathering is indispensable, mainly on the northern side and especially at certain sections and less in the eastern and southern sides.

PROTECTIVE MEASURES. SUGGESTIONS FOR FURTHER INVESTIGATION

On the basis of the results of this investigation, certain thoughts, in connection with the protection measures have been made, which must be taken for the improvement of the stability conditions of the rock and the preservation of the monuments, as well as for a programme of further investigation.

The proposed measures, more analyti-

Fig. 7 Protective measures taken in the NE part against loosening of the rock, by filling the fractures and joints and supporting.
Mesures prises dans la partie NE contre le relâchement du rocher. La protection consiste à remplir les fractures et les diaclasses et en même temps d'assurer le rocher.
cally, are:

a) The rainfall waters, representing run-off in the flat part, can be drained outside the hill.
b) The caves and voids of the slopes should be filled and their external side should be suitably finished.
c) Supporting of the isolated rocks with support or falling down, or both.
d) Protection against further action of the erosional-weathering processes and weakening-loosening of the rock at certain zones, by filling the cracks, anchoring etc. Here it is emphasized that the most suitable methodology should be chosen to avoid alteration of these historical monuments.

The above measures have already started to be materialized (Fig. 7).

The proposed research programme includes:

a) Carrying out precise measurements for checking any small displacements.
b) Elaboration of a special study of seismic risk and for the action of micro-tremors more especially.
c) Execution, at selected places, of in situ tests for residual stresses.

REFERENCES


STABILITY ANALYSIS OF THE SPACE ROCK SLIDE BY ROCK MASS ENGINEERING GEOLOGY MECHANICS

ANALYSE DE LA STABILITE DU GLISSEMENT DES ROCHES EN PENTE

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ABSTRACT

This paper deals with slope stability. On the basis of learning of the strong points from other theories about slope stability, the author advances the new idea and new method.

The paper takes the rigid body mechanics as its theoretical basis. From the beginning of fundamental conception about the sliding of rock mass on the slip plane by gravity, this paper will try to find the maximum slide force acting on the rock mass and its direction. Therefore, the author advances a new formula and illustrates with example. (Please refer to the formular 44 and 45 in the main body of the paper)

These formulas are also suitable for the landslip stability analysis based on rigid body mechanics.

At the end of the paper, the author advances his doubts about the reliability of the following methods or theories that are recommended to judge the stability of the space rock slide.

1) To judge the stability of rock slide by the safety coefficient;

2) To determine the stability of rock slide by the relation between the attitude of the rock mass constructional surface and the slope of the ground surface in the slide area;

3) To use the inclination elements (azimuth and dip angle) of the intersecting series of slip planes as the sliding direction of the slide mass, and to calculate its sliding force along this direction.

ABSTRAIT

Cet article traite sur la stabilite de la pente. C'est en étudiant les points principaux des autres théories sur la stabilite de la pente que l'auteur conçoit une idée nouvelle et propose une méthode nouvelle.
La mécanique des corps solides est prise comme base fondamentale. En partant du concept du glissement d'une masse de roche sur une pente, sous l'action de la pesanteur, l'auteur essaye de trouver la force maximale agissant sur la masse de roche et sa direction. (Voir formules 44 et 45 du texte)

Ces formules sont applicables pour l'analyse sur la stabilité du glissement des terrains en pente.

À la fin de l'article, l'auteur exprime ses doutes sur les méthodes suivantes pour analyser la stabilité des glissements des roches en pente.

1) Juger de la stabilité du glissement des roches par le coefficient de sécurité.

2) Juger de la stabilité du glissement par la rugosité de la surface de contact et de la pente générale.

3) Juger de la stabilité du glissement en remplaçant les directions et inclinaisons des facettes de contact par la pente générale de la mass glissante et la force de glissement.

For the stability analysis of the space rock slide, there have been quite a few papers dealing with this subject by using different theories or methods. Generally, they may be divided into two kinds.

1) To think that the sliding direction of the space rock mass is in accordance with resultant direction of rock mass constructional surface; in the meantime to point out that sometimes in the certain condition, the correct results cannot also be obtained;

2) To think that the sliding direction of rock mass is judged by experience. Stability analysis of the space rock slide is replaced by stability analysis of plane rock slide on the plane rock slide theory.

On the basis of learning of the strong points from these theories, from the beginning of fundamental conception of mechanics about the sliding of rock mass on the slip plane by gravity, author finds out maximum sliding force and its direction.

Author takes the rigid body mechanics as the fundamental conception of mechanics. The rock mass surrounded by constructional surfaces is considered to be the intact body at least before the sliding begins.

Constructional surfaces mentioned here also includes the fracture planes which occur before the sliding begins.

In view of length of the paper, though consideration must be given to the influence of pore water pressure, ground water, dynamic and static water pressure, rapid water level drawdown, earth-quake, tectonic action etc. on rock mass stability, the paper will not enter into details on them.

Because the gravity of rock mass on the slip plane is the primary internal factor which causes rock mass sliding, this paper will deeply discuss the gravity of rock mass on rock mass stability.

After this problem is resolved by force vector analysis method, the influences of pore water, static and dynamic water pressure, earth-quake force etc.
are resolved by force and component vector analysis by analogy.

I. Stability analysis of the rock mass sliding on the single slip plane \( a_0 \) along apparent dip direction:

When the rock mass on the slip plane \( a_0 \) may freely slide, the sliding along true dip direction has the stability characteristics given by following formula

\[
T_r = W_o \sin \theta_o - (f \cos \theta_o W_o + C_o A_o) > 0 \quad \ldots \ldots \quad (1)
\]

Where symbols are identical to the figure 1 and usual practice, but \( f \) and \( c_o \) are isotropic.

Suppose that the sliding of rock mass on the slip plane is limited to the apparent dip direction, then formula 1 will change into following forms (please refer to the figure 1).

\[
T_{ru} = W_o \sin \theta_o \eta_{ou} (f \cos \theta_o W_o + C_o A_o) > 0 \quad \ldots \ldots \quad (2),
\]

\[
T_{oz} = W_o \sin \theta_o \eta_{oz} (f \cos \theta_o W_o + C_o A_o) > 0 \quad \ldots \ldots \quad (3),
\]

where

\[
\eta_{ou} = \frac{\sin \omega_{oz}}{\sin (\pi - \omega_{ou} - \omega_{oz})} \quad \ldots \ldots \quad (4),
\]

\[
\eta_{oz} = \frac{\sin \omega_{ou}}{\sin (\pi - \omega_{ou} - \omega_{oz})} \quad \ldots \ldots \quad (5),
\]

\[
\omega_{ou} = \angle BAU; \quad \omega_{oz} = \angle BAZ; \quad \omega_o = \angle UAZ.
\]

It will be seen from formulae 2 and 3 that

1) Whichever direction the rock mass slides along, \( f \cos \theta_o W_o + C_o A_o \) always has a fixed value and is the attributes slip to plane. Only when \( W_o \sin \theta_o \eta_{ou} > (f \cos \theta_o W_o + C_o A_o) \)

i.e. \( T_{ou} > 0 \ldots \ldots \quad (6), \) and \( T_{oz} \leq 0 \ldots \ldots \quad (7), \)

then the rock mass can slide along a single direction (as \( A \)). It is to be noted that \( T_{oz} \leq 0 \) is caused by external factor.

2) The variation of \( \omega_{ou} \) or \( \omega_{oz} \) values can influence on the values of component force factor \( \eta_{ou} \) and \( \eta_{oz} \).

When \( \omega_{oz} \) decreases in value or \( \omega_{ou} \) increases, this will make

\[
W_o \sin \theta_o \cdot \eta_{ou} \leq (f \cos \theta_o \cdot W_o + C_o A_o) \quad \ldots \ldots \quad (8)
\]

then the rock mass is stable along apparent dip direction \( A \).

Formulae 6 and 7 are necessary condition of the single sliding direction \( A \) only; Formulae 7 and 8 are necessary conditions of the stable rock mass along apparent dip direction \( A \).

If calculation results are \( T_{ou} < 0; \)

\( T_{oz} > 0 \), then the rock mass slides along apparent dip direction \( A \); If \( T_{oz} \leq 0 \), then the rock mass is stable along apparent dip direction \( A \).

3) When \( \omega_{ou} = 0 \) or \( \omega_{oz} = 0 \), \( \eta_{ou} = 0 \) or \( \eta_{oz} = 0 \), \( \eta_{ou} = 1 \), \( \eta_{oz} = 1 \), then formulae 2 and 3 may be reduced to formula 1.

Above mentioned discussion shows that when the rock mass slides on a single slip plane \( A \) by gravity, then the sliding force of rock mass along true dip direction \( AB \) is maximum. This fundamental may be extended to cover the sliding of rock mass on the multiple slip planes in order to determine maximum sliding force and its direction.

On the basis of above mentioned fundamental conception of mechanics, the paper will develop the stability analysis of space rock slide. In the stability analysis of space rock slide on the multiple slip planes, though formulae 4 and 5 are still used on the secondary slip plane to find their component factor, the following simplified formulae are applied to the main slip plane, i.e.

\[
\eta_{ou} = \frac{\sin \omega_{oz}}{\sin (\pi - \omega_{ou} - \omega_{oz})} \quad \ldots \ldots \quad (9),
\]

\[
\eta_{oz} = \frac{\sin \omega_{ou}}{\sin (\pi - \omega_{ou} - \omega_{oz})} \quad \ldots \ldots \quad (10),
\]

\[
\omega_{ou} + \omega_{oz} = \frac{K}{E}.
\]
Such methods are used so as to avoid the interference between two groups of component forces acting on the main slip plane.

II. Stability analysis of rock mass on the two slip planes:

![Diagram of rock mass with slip planes](image)

- $A_1 = abcd$; $W_1 = V_1 T_1$; $V_1 = abhjgicedj$
- $A_2 = befc$; $W_2 = V_2 T_2$; $V_2 = belmkiecf$

Stability analysis on multiple slide planes is based on the stability analysis on two slip planes. This section will make a thorough study of two slip plane subject. Data listed below are obtained from field investigation of the sliding rock mass shown in Figure 2.

Parameter table of the slip plane

<table>
<thead>
<tr>
<th>number</th>
<th>0</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>attitude elements of the slip plane</td>
<td>$N_0 \theta_0 S$</td>
<td>$N_1 \theta_1$</td>
</tr>
<tr>
<td>area (m²)</td>
<td>$A_0$</td>
<td>$A_1$</td>
</tr>
<tr>
<td>total gravity force of rock mass on the slip plane (T)</td>
<td>$W_0$</td>
<td>$W_1$</td>
</tr>
<tr>
<td>coefficient of friction</td>
<td>$f_0$</td>
<td>$f_1$</td>
</tr>
<tr>
<td>cohesive force (t/m²)</td>
<td>$C_0$</td>
<td>$C_1$</td>
</tr>
</tbody>
</table>

Attitude of intersecting line of two slip planes are shown in figure 2. It acts only on the slip plane range or its area don't directly act on the stability analysis of rock mass.

Slip plane $A_1$ is selected as a main slip plane. The surplus downward tangential component of rock mass gravity pertaining to other secondary slip planes acts on the main slip plane $A_0$ as an additional force (which is essentially a body force, but here it is taken as concentrated force). Now the stability of whole rock mass may be determined.

1) fundamental data required for determination of rock mass stability (refer to Fig. 1 and 3).

$\omega_{OU}$ is assumed to be the angle between the sliding direction $AU$ of rock mass and the true dip direction of the main slip plane $A_0$. Then the angle between the direction $AZ$ (perpendicular to the direction $AU$) and the true dip direction is $\omega_{OZ} = \frac{\pi}{2} - \omega_{OU}$.

The formulation of tectonic geology are used to find their projection angles $\omega_{OUS}$ and $\omega_{OZS}$ on the horizontal plane.

\[ \omega_{OUS} = t g^{-1} \left( \frac{t g \omega_{OU}}{\cos \theta_0} \right) \]  \hspace{1cm} (11)

\[ \omega_{OZS} = t g^{-1} \left( \frac{t g \omega_{OZ}}{\cos \theta_0} \right) \]  \hspace{1cm} (12)

Then the azimuth of the $AU$ direction is $N(\xi_0 \pm \frac{\pi}{2} - \omega_{OUS})$, the azimuth of the $AZ$ direction is $N(\xi_0 \pm \frac{\pi}{2} + \omega_{OZS})$.

Then the apparent dip angle of the $AU$ direction is

\[ \alpha_{OU} = t g^{-1} \left[ t g \theta_0 \cdot \cos \omega_{OUS} \right] \]  \hspace{1cm} (15)

the apparent dip angle of the $AZ$ direction is

\[ \alpha_{OZ} = t g^{-1} \left[ t g \theta_0 \cdot \cos \omega_{OZS} \right] \]  \hspace{1cm} (16)

If the slide stability of rock mass on the adjacent slip plane $A_1$ also con-
sidered in the AU and AZ directions given by formulae 13 and 14, then there are the horizontal difference between the azimuth given by formula 13 or 14 and the azimuth of the true dip direction of the slip plane Ai (see figure 4), i.e.

\[ \omega_{1u} = N(\xi_{1e} \pm \frac{\pi}{2}) - N(\xi_{1e} \pm \frac{\pi}{2} - \omega_{bus}) \ldots \ldots (17) \]

\[ \omega_{1z} = N(\xi_{1e} \pm \frac{\pi}{2} + \omega_{obs}) - N(\xi_{1e} \pm \frac{\pi}{2}) \ldots \ldots (18) \]

It will be seen from this that the angles between the apparent dip Ai and the horizontal surface \( \omega_{1u} \) or \( \omega_{1z} \) direction and the true dip ABi direction of the slip plane Ai are respectively.

\[ \omega_{1u} = \tan^{-1}[\tan(\omega_{bus} \cdot \cos \theta_{1})] \ldots \ldots (19) \]

\[ \omega_{1z} = \tan^{-1}[\tan(\omega_{obs} \cdot \cos \theta_{1})] \ldots \ldots (20) \]

\[ \alpha_{1u} = \tan^{-1}[\tan(\theta_{1} \cdot \cos \omega_{bus})] \ldots \ldots (21) \]

\[ \alpha_{1z} = \tan^{-1}[\tan(\theta_{1} \cdot \cos \omega_{obs})] \ldots \ldots (22) \]

The downward tangential component factor of the slip plane Ai (refer to the formulae 4 and 5) is

\[ \eta_{1u} = \frac{\sin \omega_{1z}}{\sin(\pi - \omega_{1u} - \omega_{1z})} \ldots \ldots (23) \]

\[ \eta_{1z} = \frac{\sin \omega_{1u}}{\sin(\pi - \omega_{1u} - \omega_{1z})} \ldots \ldots (24) \]

Together with the formulae 9 and 10 of the downward tangential component factor \( \eta_{1u} \) and \( \eta_{1z} \) of the main slip plane \( A_{o} \), they will provide the fundamental data for space rock slide stability analysis.

2) To establish the fundamental formulae of mechanics on rock mass stability;

Stability of the rock mass on the slip plane \( A_{i} \) in the \( N(\xi_{0} \pm \frac{\pi}{2} - \omega_{bus}) \) direction:

\[ T_{1u} = W_{i} \sin \theta_{1} \cdot \eta_{1u} \cdot \left( f_{1} \cos \phi_{1} \cdot W_{i} + c_{1} A_{i} \right) \cdot \cos \phi_{1} \ldots \ldots (25) \]

Stability of rock mass on the slip plane \( A_{i} \) in the \( N(\xi_{0} \pm \frac{\pi}{2} + \omega_{obs}) \) direction:

\[ T_{1z} = W_{i} \sin \theta_{1} \cdot \eta_{1z} \cdot \left( f_{1} \cos \phi_{1} \cdot W_{i} + c_{1} A_{i} \right) \cdot \cos \phi_{1} \ldots \ldots (26) \]

If \( T_{1u} \) shown in formula 25 is \( T_{1u} > 0 \), then in the \( N(\xi_{0} \pm \frac{\pi}{2} - \omega_{bus}) \) direction, the rock mass on the slip plane \( A_{i} \) gives the additional force (body force) to the rock mass on the main slip plane \( A_{o} \). If \( T_{1z} < 0 \), then the rock mass on the slip plane \( A_{i} \) is stable in itself; it will not give the additional force to the rock mass on the main slip plane \( A_{o} \). When in the special case, the rock mass on the main slip plane \( A_{o} \) gives the additional force to the rock mass on the slip plane \( A_{i} \); then the directions of the friction and cohesion forces on the slip plane \( A_{i} \) should be considered again so as to determine the stability of the rock mass under these forces. This case is often encountered in the stability analysis of the rock mass on the multiple slip plane. This problem is put forward to draw attention.

Similarly, in the \( N(\xi_{0} \pm \frac{\pi}{2} + \omega_{obs}) \) direction, as \( T_{1z} \) given by formula 26 varies, the problem of the additional force applied to the rock mass on the main slip plane is also discussed.

When \( T_{1u} > 0 \) or \( T_{1z} < 0 \), how does the rock mass on the slip plane \( A_{i} \) give the additional force to the rock mass on the main slip plane \( A_{o} \) in the given direction?

This problem will particularly be discussed below (see figure 5):

If \( T_{1u} \) given by formula 25 is \( T_{1u} > 0 \) in the \( N(\xi_{0} \pm \frac{\pi}{2} - \omega_{bus}) \) direction, then the downward redundant tangential force of the rock mass \( W_{i} \) will be transferred to the slip plane \( A_{o} \).
through the rock mass \( W_o \). If the redundant tangential force of \( W_i \) is taken as the concentrated force on the slip plane \( A_o \), then the force vector diagram is obtained as shown in Figure 5.

\[
S_{ou} = \frac{T_{iu} \cos \alpha_{iu}}{\sin (\frac{\pi}{2} + \alpha_{ou})} \quad \cdots \quad (27)
\]

\[
G_{iou} = \frac{T_{iu} \sin (\alpha_{iu} - \alpha_{ou})}{\sin (\frac{\pi}{2} + \alpha_{ou})} \quad \cdots \quad (28)
\]

\( S_{iou} \) represents the first additional force tangential to and acting on the slip plane \( A_o \), which arises from the redundant force \( T_{iu} \) tangential to the slip plane \( A_i \); and given by rock mass \( W_i \), downward in the \( N(\xi_{o} \pm \frac{\pi}{2} - \omega_{ou}) \) direction.

\( G_{iou} \) represents the plumb force on the slip plane \( A_o \), which arises from the redundant tangential force \( T_{iu} \).

It must be noted \( G_{iou} \) is not a normal force with respect to the slip plane \( A_o \). Hence with respect to the main slip plane \( A_o \), \( G_{iou} \) may be resolved into the additional normal pressure \( P_{iou} \) and the additional tangential force \( F_{iou} \) in accordance with true dip direction of the slip plane \( A_o \).

According to the resolution method of the downward tangential force of rock mass on the slip plane \( A_o \), \( F_{iou} \) may be resolved to the second additional downward tangential component \( F_{iox} \) \( \eta_{iou} \) in the \( N(\xi_{o} \pm \frac{\pi}{2} - \omega_{ou}) \) direction and \( F_{iox} \) \( \eta_{ioz} \) in the \( N(\xi_{o} \pm \frac{\pi}{2} + \omega_{oz}) \) direction as expressed by following formulae.

\[
P_{iou} = G_{iou} \cos \theta_o \quad \cdots \quad (29);
\]

\[
F_{iou} \sin \theta_{iou} = G_{iou} \sin \theta_o \eta_{iou} \quad \cdots \quad (30);
\]

\[
F_{iou} \sin \theta_{ioz} = G_{iou} \sin \theta_o \eta_{ioz} \quad \cdots \quad (31).
\]

By analogy, the additional force that rock mass \( W_i \) gives the slip plane \( A_o \) in the \( N(\xi_{o} \pm \frac{\pi}{2} - \omega_{oz}) \) azimuth direction may be resolved into the following components:

\[
S_{ioz} = \frac{T_{ioz} \cos \alpha_{ioz}}{\sin (\frac{\pi}{2} + \alpha_{ioz})} \quad \cdots \quad (32)
\]

\[
P_{ioz} = G_{ioz} \sin \theta_o \eta_{ioz} \quad \cdots \quad (33);
\]

\[
F_{ioz} \sin \theta_{ioz} = G_{ioz} \sin \theta_o \eta_{ioz} \quad \cdots \quad (34);
\]

\[
F_{ioz} \sin \theta_{ioz} = G_{ioz} \sin \theta_o \eta_{ioz} \quad \cdots \quad (36).
\]

In addition, \( W_o \) on the main slip plane \( A_o \) may be resolved into following three-dimensional components:

In the \( N(\xi_{o} \pm \frac{\pi}{2} - \omega_{ou}) \) direction, \( S_{ou} = W_o \sin \theta_o \sin \eta_{iou} \quad \cdots \quad (37) \)

In the \( N(\xi_{o} \pm \frac{\pi}{2} + \omega_{oz}) \) direction, \( S_{ou} = W_o \sin \theta_o \sin \eta_{ioz} \quad \cdots \quad (38) \)

In the normal direction of the slip plane \( A_o \), \( N_o = W_o \cos \theta_o \quad \cdots \quad (39) \)

It may be seen from above mentioned components that

In the \( N(\xi_{o} \pm \frac{\pi}{2} - \omega_{oz}) \) azimuth direction

\[
T_{ou} = S_{ioz} + G_{iou} + G_{ioz} + W_o \sin \theta_o \sin \eta_{iou} - \left[ f_o \cos \theta_o + G_{iou} + G_{ioz} + W_o \right] \xi_0 \quad \cdots \quad (40)
\]

In the \( N(\xi_{o} \pm \frac{\pi}{2} + \omega_{oz}) \) azimuth direction

\[
T_{oz} = S_{ioz} + G_{iou} + G_{ioz} + W_o \sin \theta_o \sin \eta_{ioz} - \left[ f_o \cos \theta_o + G_{iou} + G_{ioz} + W_o \right] \xi_0 \quad \cdots \quad (41)
\]

This paper requires to meet the conditions that:

When \( T_{ou} > 0 \), must be \( T_{oz} \leq 0 \) or when \( T_{oz} > 0 \), must be \( T_{ou} \leq 0 \), because the rock mass on the multiple slip planes can slide only in a single direction.

For formulae 40 and 41, the thorough study will be carried on below. Formulae 40 and 41 have the universal meaning.

In practice, it should be noted that

1) determination of positive and negative sign of force vector in the given azimuth direction:

Generally, the direction of two component vectors on the main slip plane \( A_o \) are taken as the criteria. The signs of component vectors on the other slip plane
\( A \) must be determined according to this criteria. It should be noted that when the rock mass on the slip plane is carried by the rock mass on the other slip plane to slide in the direction different (or inverse) from its original direction, then the vector direction of the friction force and the cohesive force on this plane should be considered again.

2) When \( T_{iu} \) given by formula 25 is \( T_{iu} > 0 \) and \( T_{iz} \) given by formula 26 is \( T_{iz} < 0 \), then \( S_{ioz}, G_{ioz}, F_{ioz} \) and \( F_{ioz} \) which may be obtained from formula 32 to 36 may lose their physical meanings. Therefore formulae 40 and 41 may be reduced to the following forms:

\[
T_{ou} = \frac{S_{iou} + (G_{iou} + w) S_{iou} + (f_{iou} + w) A_{iou}}{S_{iou} + (G_{iou} + w) S_{iou} + (f_{iou} + w) A_{iou}} (42)
\]

\[
T_{oz} = \frac{S_{ioz} + (G_{ioz} + w) S_{ioz} + (f_{ioz} + w) A_{ioz}}{S_{ioz} + (G_{ioz} + w) S_{ioz} + (f_{ioz} + w) A_{ioz}} (43)
\]

Once \( T_{oz} \) obtained from formula 43 is \( T_{oz} > 0 \), then the value of the opposite direction of friction force and cohesive force on the slip plane \( A \) should be considered again and the value of \( T_{iz} \) should be calculated using formula 26 again, and the stability of rock mass should be determined again. If not, the operation value of formula 40 will be negated.

In the special condition, \( T_{iz} \) obtained from formula 26 is \( T_{iz} < 0 \) and \( T_{oz} \) obtained formula 43 is \( T_{oz} < 0 \), this shows the directions of the \( T_{iz} \) and \( T_{oz} \) are negative, this makes the whole rock mass slide in the opposite direction. This problem should be noted in the stability analysis.

3) The formula set of the sliding in the direction of \( T_{oz} \) and the formula set of the stability in the direction of \( T_{ou} \) can be obtained by the same way as above mentioned, when \( T_{iz} > 0 \) and \( T_{iu} < 0 \). These formulae are similar to formulae 42 and 43 in the form and are different in the subscripts. The subscript "U" must be used to replace the subscript "i" each other.

Formulae 42, 26 and 43 are the fundamental formulae for the calculation of the rock mass sliding, from which the downward sliding force of rock mass can be obtained in the given direction. Only in the direction in which the rock mass can slide, the downward sliding force is maximum. This has been stated in section one of this paper. Therefore formula 42 is required to find the maximum value of \( T_{ou} \).

It will be seen from the above mentioned constants that \( T_{ou} \) in the formula 40 is \( T_{ou} = f (\omega_{ou}) \),

\[
\frac{dT_{ou}}{d\omega_{ou}} = \frac{dF(\omega_{ou})}{d\omega_{ou}} = f'(\omega_{ou}).
\]

It may be known from mathematics that when \( f'(\omega_{ou}) = 0 \), \( \omega_{ou} \) is obtained to make the value of \( T_{ou} \) maximum. But this calculation method is too complicated.

The trial and error method or the graphic method in which the relationship between \( \omega_{ou} \) and \( T_{ou} \) is used to find the extreme value is recommended to find the maximum value of \( T_{ou} \). These methods are illustrated with example now.

3) example

Field Investigation Data Table of the Rock Mass Sliding

<table>
<thead>
<tr>
<th>slip plane number</th>
<th>0</th>
<th>1=1</th>
</tr>
</thead>
<tbody>
<tr>
<td>attitude elements of slip plane</td>
<td>N30°≤30°S</td>
<td>N0°≤60°E</td>
</tr>
<tr>
<td>slip plane area (m²)</td>
<td>1154.7</td>
<td>2000</td>
</tr>
<tr>
<td>total gravity force of rock mass on the slip plane (t)</td>
<td>25000</td>
<td>25000</td>
</tr>
<tr>
<td>coefficient of friction</td>
<td>0.202</td>
<td>0.325</td>
</tr>
<tr>
<td>cohesive force (t/m)</td>
<td>1.8</td>
<td>4.0</td>
</tr>
</tbody>
</table>

According to the data listed above,
the maximum downward sliding force and its direction is determined. The results obtained by the trial and error method are tabulated in the table below.

### Calculation Table

<table>
<thead>
<tr>
<th>$\omega_0$ (form. 11)</th>
<th>$\theta_0$</th>
<th>$\omega_0z$</th>
<th>$\omega_0x$</th>
<th>$\omega_0y$</th>
<th>$\omega_0z$</th>
<th>$\omega_0x$</th>
<th>$\omega_0y$</th>
<th>$\omega_0z$</th>
<th>$\omega_0x$</th>
<th>$\omega_0y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
<tr>
<td>$15^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
<tr>
<td>$17.5^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
<tr>
<td>$17.75^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
<tr>
<td>$20^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
<tr>
<td>$25^\circ$</td>
<td>51.9°</td>
<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
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</tr>
<tr>
<td>$30^\circ$</td>
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<td>75°</td>
<td>72.5°</td>
<td>72.25°</td>
<td>70°</td>
<td>65°</td>
<td>60°</td>
<td>55°</td>
<td>50°</td>
<td>45°</td>
</tr>
</tbody>
</table>

Suppose that there is the rock mass possible to slide on the slip plane $\alpha_0$ and $\alpha_i$ ($i=1, 2, 3, \ldots$), according to the similar principle to formulae 40 and 41,

It is obtained from the above table that the maximum downward slide force of the rock mass on the two slip planes is 12031.92 t, its azimuth is N99.71° and its apparent dip is 28.42°. The similar data as above are obtained from the graphic method.

III. The stability analysis of the rock mass on the multiple slip planes:

the sliding force and its direction are determined. That is to say, formulae 40 and 41 are the simplified formulae of the following universal formulae:

$$T_{ou} = \sum_{i=1}^{N} S_{oiu} + \sum_{i=1}^{N} G_{oiu} + \sum_{i=1}^{N} G_{oiu} + \sum_{i=1}^{N} W_{oiu} \sin \theta \eta_{ou}$$

$$- \left[ f_{c} \cos \beta \left( \sum_{i=1}^{N} G_{oiu} + \sum_{i=1}^{N} G_{oiu} + \sum_{i=1}^{N} W_{oiu} + C_{0} A \right) \right] 0.44,$$
in the $N(\frac{\xi + \frac{3}{2}\xi}{2} + \omega_{\xi})$ azimuth direction

$$T_{ox} = \frac{n}{i=1} S_{i ox} + \sum_{i=1}^{n} T_{i ox} + \sum_{i=1}^{n} T_{i ox} + W_o \sin \theta_x \eta_{ox} -$$

$$G \cos \theta_x (\sum_{i=1}^{n} G_{i ox} + \sum_{i=1}^{n} G_{i ox} + W_o) + C_{o A} A \dot{\dot{c}}, (45)$$

The operation procedures are essentially similar to the calculation table in previous section. But symbol subscript $i$ should be replaced by $1, 2, 3, \ldots, n$. In the view of the length of the paper, the operation results will not be listed again.

4. Summary

The maximum downward slide force mentioned here refers to the maximum initial slide force of the rock mass (which begin to slide) on the main slip plane. The sliding may change the original relation between the rock mass and the slip plane. Hence, some problems have yet to be solved just after the beginning of the sliding.

1. The relation between the rock mass and the slip planes;
2. Whether to occur the fracture or break down in the rock mass itself;
3. Whether to occur the plastic deformation in the rock mass itself or whether to split the slip plane (rock mass) by the sliding rock mass.

The new stability analysis of the rock slide has yet to be carried in such condition. Only in such way, it can be determined how the rock mass slides off from the slip plane.

The slip plane parameters of $f_i$ and $C_i$ are assumed to be isotropic in this paper. If the anisotropic value of $f_i$ and $C_i$ can be determined in practice, then these values will be considered in operation. $f_i$ and $C_i$ mentioned here refer to the values used in operation. They come from the test data, but they will not be completely similar to the test data.

The selection of these parameters must be based on the routine.

The fundamental formulae of this paper may also be applied to the stability analysis of the land-slide in the initial stage of whole body sliding. Because the most of the slip planes of the landslide are related to the geological constructional surfaces or the fracture planes formed by new fracture. They are similar to the slip plane below rock mass mentioned in this paper in property. There are the attitude elements, friction force and cohesive force in the soil slip plane. In addition, the rigid body mechanics is taken as its basis of the establishment of the fundamental formulae in the plane stability analysis of the landslide. This point is also similar to this paper.

It may be known from above mentioned discussion that in the complicated conditions, the reliability must be considered of the following methods.

1. To determine the stability of the rock slide by the safety coefficient;
2. To determine the stability of the rock slide by the relation between the constructional surface attitude and the slope of the ground surface;
3. To use the inclination elements (azimuth and dip) of the intersecting series of the slip planes as the rock mass slide direction and to determine the slide force from this.

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THE EQUATORIAL PLANE POLAR-EQUATORIAL PLANE PLUMB DOUBLE PROJECTION IN THE STABILITY ANALYSIS OF THE ROCK SPACE SLIDE

APPLICATION DES DOUBLES PROJECTIONS-PROJECTION POLAIRE ET PROJECTION NORMALE SUR PLAN EQUATORIAL-A L'ANAYSE SUR LA STABILITE DU GLISSEMENT DES ROCHES EN PENTE

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ABSTRACT

This paper is one of the actual explanations to prove the actual effect of the author's another essay, "Fundamentals and Graphic Method of Equatorial Plane Polar-Equatorial plumb Double-Projection".

This method of "double-projection" is made up of the fundamental principle from the stereographic projection and projective geometry. This method can bring the advantages of both into full play and can store up the space attitude of the material object in the plane map. This paper has elected the rock space slide as an instance to explain briefly the advantage of "double-projection"; namely, with the use of the theory of "double-projection", the investigation material about the geological and structural attitude of the substance in space can be prepared, and by means of the graphic method, the mechanical calculation about the stability of the slide can be carried out without another calculator.

At the end of this paper, the result of this graphic calculation may be stored by double-projection diagram, prepared for reappearance as wanted.

ABSTRT

De présent article fournit des explications concrètes à propos de l'article « Théorie et Méthode des Doubles Projections, Projection Polaire et normale sur plan Équatorial

La méthode des doubles projections dérive de la théorie fondamentale de la projection polaire sur plan équatorial et de la géométrique descriptive. Elle en emprunte des propriétés utiles, elle permet d'enregistrer les formes et dimensions des corps solides sur une figure plane.

Prenant pour application le glissement de roches en pente, l'on démontre
les avantages de la méthode des doubles projections. Elle permet de reproduire les configurations des dormées géologiques obtenues au cours d'une prospection, et par là effectuer par une méthode graphique les calculs mécaniques nécessaires à l'analyse sur la stabilité du glissement des roches en pente.

Des résultats obtenus par la méthode graphique peuvent se conserver sous forme des doubles projections, pour usage ultérieur.

The author in his another paper "Fundamentals and Graphic Method of Double-Projection" (It is called "First paper" for short) has already described the basic principle and method of "Double-projection" and pointed out the practical value of its application in various branches of geological science and technology. The reason for the widening of its application is that using this graphic method, the space attitude of substance can be stored in a plane figure, and can be restored as wanted.

This paper is aimed to introduce the graphic method of "Double-Projection" into the stability analysis of the rock space slide and to describe the graphical calculation method of it.

As to the theory and method used here is quoted from another paper of the author, "Stability Analysis of the Space Rock Slide by Rock Mass Engineering Geology Mechanics" (It is called "second paper"). Therefore, for the theory regarding to "Double-Projection" method and stability analysis of the slide, please refer to the above mentioned two articles.

Now, the application of "Double-Protection" method is to be illustrated with the example quoted from the second paper. The fig. 1 is the double projection figure for the data collected in this example. Topographical and geological data may be stored in the Double Projection Figure of the attitude of a rock slide and restored as wanted. The advantage of the application of this method is not notable in the attitude analysis of a simple slide, however in the geological and structural attitude analysis of the Earth's crust in a large area, such as in geomechanics or structural geology, this method is very helpful. This paper is aimed to introduce the Double Projection Method, and deals with this problem as general...
The application of Double Projection Method in the stability analysis of a slide will be described as follows.

I. To restore the height value of each survey point of a space substance (accuracy is in direct proportion to the dimension scale used in the figure);

By using the theory and method described in the section one of the first paper, the height value of each angle apex on the sliding mass can be solved inversely (refers to figure 2).

4. Point B is the plumb projection point from the space point B' on the equatorial plane. It has the same height value (80m) as that of point O. Then, line BB' is the difference of the plumb height values of the point B' and point O. It is known from the figure scale, that BB' = 20m. Therefore, the height value of point B' is 100m as shown in figure 2. As above, it is the origin of the plumb projection point B (100m) on the Double Projection figure 2. All the height values of other points can be solved for in the same way.

II. To solve for the horizontal distance between point positions in the space as wanted; Double Projection Method is the incorporation of equatorial plane polar projection and equatorial plane plumb projection. Therefore, by using the latter, the horizontal distance between point positions in space can be obtained. The distance measured with the figure scale is then marked on figure 2.

III. To solve for the inclination element (i.e. inclination azimuth and dip angle) and the actual length of the edge lines of a sliding mass:

As described in the above two sections, the height value of each point position in the space and the horizontal distance between them are known, it is possible to solve for the dip angle and actual length of the edges by using the plumb projection method, as shown in figure 3.

For example, the height value of point B, i.e., point B' (100m) may be solved for as follows:

1. Connect point o, b, B on the equatorial plane (80m) into the line ObB. On the Wolfe's sphere, make line N_b O S_b intersects line ObB vertically. The former will intersect the Wolfe's sphere surface at points N_b, S_b.

2. Points b, S_b are connected into a line, its extension cuts Wolfe's sphere at point b'.

3. Connect points o, b' into a line and at point B, its extension intersects the straight line BB'. The latter has intersected the line ObB at point B vertically.
In addition, the inclination azimuth value of the edge lines may be measured from figure 1. These data are all shown in figure 4.

Figure 3  The dip angle and actual length of the incline lines.

Let us illustrate it with an example.

\[
\begin{align*}
&\text{G} \rightarrow \text{H} \\
&\text{N}77^\circ6\prime42.3'' - 40.9 \\
&\text{N}77^\circ59.7'' - 50.0
\end{align*}
\]

There are two edge lines \( \text{G}'\text{H}' \) and \( \text{A}'\text{B}' \) with the same inclination azimuth. The end point \( \text{G}' \) of the edge line \( \text{G}'\text{H}' \) is higher than its other end point \( \text{H}' \). The inclination element of the edge \( \text{G}'\text{H}' \) is \( N77^\circ52.3' \) and the actual length of it is 40.9 m. The edge line \( \text{A}'\text{B}' \) is just below the edge line \( \text{G}'\text{H}' \). Its end point \( \text{A}' \) is higher than its other end point \( \text{B}' \). Its inclination element value and the actual length are \( N77^\circ59.7' \) and 50.0m respectively.

If there is no denominator in the fraction, it means that there is no edge line in the lower part, even though there are two point positions but not connected into an edge line.

IV. To solve for the volume and weight of the sliding body on each slip plane:

1. According to the information shown in figures 2 and 4, the sliding body on each slip plane may be divided into several tetrahedrons. For example, the sliding body on the slip plane ABCD (No.1) is divided into six tetrahedrons. (see fig.5)

2. According to section 3 of the first paper—"Double Projection of a Plane", may solve the actual attitude and the area of every single plane of a tetrahedron in the space.

3. According to section 4—"Double-projections of a polyhedron", may solve for the volume of every tetrahedron and mark these data on figure 5. The sum of them is 10000 m³.

As shown in figure 1, the volume
density of the sliding body on the slip plane ABCD is 2.5 T/m³, so the total weight is 25000 T.

Using the same method, the total weight 25000 T of the sliding body on the main slip plane (No. 10) is obtained.

V. To solve for the attitude elements and area of each plane of the slip bed.

1. According to the inclination azi-

2. According to the inclination element value of two adjacent side lines of the slip plane and Wolfe's net made with the equatorial plane polar projection method, the attitude element value of this slip plane, N 0° 60° E is obtained (see fig. 6). The original information is thus restored.

Using the same method, all these data of another slip plane BBPCS and the inclination element of the intersecting line of two adjacent slip planes may be all solved for.

For the convenience of the mechanical calculation of the stability of slide, the above mentioned and restored information and the result of mechanical experiments are listed in figure 6.

VI. To derive the height value of any point position in the space substance, the inclination elements and actual length of any inclined line, the attitude elements and area of any inclined plane, the space attitude and volume of any polyhedron:

In the previous five sections, the space attitude of the point, line, plane and solid have been obtained, because the numerical relationship between both the
latter and the former, to obtain the answer from analysis of these.

For example, the centroid of slip plane BDFC (No:0) (see the latest figure 10) is laid at the middle point of the diagonal FB of the rectangle slip plane. With point position F, B and the horizontal distance between them, the height value 101.5 m is obtained. Similarly, other information can also be derived.

VII. To solve the maximum downward sliding force and its azimuth of the sliding body on the double slip plane ABCD and BDFC:

In view of the mechanical analysis process of this example by the numerical calculation method described in the Second Paper, this paper is aimed only to introduce the application of the Double Projection Method in this respect. Let us illustrate the application with the calculation of the maximum downward sliding force and the original data obtained in the Second Paper, and this may be used as a model to solve for other non-maximum downward sliding forces.

In the Second Paper, the maximum downward sliding force is 12031.9 T and its inclination elements is N 99.7° / 28.4°. Now let us use the graphic method to repeat the calculation of this value again.

Although the accuracy of the graphic method is inferior to that of the numerical calculation method, but it's saving in time. Moreover, the subject which the calculation in geology deals with is macro, provided the error is not great (0.18%), thus it is practical.

In author's Second Paper, in the calculation table of the downward sliding force, the value "γ" in the 10th to 19th column need not be calculated for, all the value in the 1st ~ 14th column may be expressed with the curves of Wolfe's stereographic net. Listed in figure 7, is the results of graphic method for other columns, as assumed γw = 17.7° (≈ 17.75° in the 1st column).

It is clear from figure 7 that inclination azimuths of these appointed component vectors of the downward sliding force are N 99.2° and N 194.7° (or 14.7°), respectively. In the following, the component force vectors of the sliding body (25000 T) in downward sliding along the slip plane i (i=1) is to be discussed with the graphic method as shown in figure 8.

1. To solve for the downward tangential forces Siu and Siz of the sliding body (25000 T) at the appointed azimuths:

(1) As shown in figure 8, the dip angle of the inclined line CB is 60°.

This line is the true dip line of the slip plane No:1.

(2) Use a plumb line AB to represent a gravity vector of 25000 T on this slip plane.

(3) By carrying out the resolution of the gravity vector with the fundamental mechanical theory of the slide, the downward tangential force S1 (CA) along the true dip direction of the slip plane and the positive pressure N1 (AC) Normal to the slip plane may be obtained.

(4) S1 may be resolved into S1w (CD) and S1z (CE) on this slip plane in these azimuths of Nw 99.2° and Nz 14.7°. These two values equal to the values of the first term at the right side of the formulas 25 and 26 in the Second Paper respectively.

2. To solve the value of Ni. f; with
Using two similar triangles $AOG$ and $HOC$, let $f = FC = 1$; $N_i = N_i'$ $f = \frac{AC}{AH}$, whereas on the line $FC$ take $GC = f_i = 0.325$ and from point $G$ make a straight line $GH$ parallel to line $FA$. Line $GH$ intersects line $AC$ at point $H$. On line $AC$ take $HC = f_i'N_i$.

3. To solve the value of $C_i'A_i'$ with graphic method:

Similarly, by using similar triangles $IGT$ and $LCG$, $\overrightarrow{GT} = C_i'A_i'$ may be solved for.

It will be seen from figure that,

$\overrightarrow{HT} = \overrightarrow{HC} + \overrightarrow{CT} = f_i'N_i + C_i'A_i'$.

They equal also to the value of the second term in the right side of formula 25 and 26 in the Second Paper respectively.

4. To solve for the downward sliding force $T_{i\mu}$ in the appointed azimuth $N_i\mu 99.2^\circ$:

According to principle of the formula 25, take

$\overrightarrow{CR} = \overrightarrow{HT} = f_i'N_i + C_i'A_i'$ on

$\overrightarrow{CB}$ ($S_{i\mu}$), whereas remainder $BD = T_{i\mu}$.

5. To solve for the downward sliding force at appointed azimuth $N_{14.7^\circ}$:

According to the principle of the formula 26, on the extension of $CB$ take

$\overrightarrow{CQ} = \overrightarrow{HC} = f_iN_i + C_i'A_i'$,

whereas $\overrightarrow{E} = - T_{i\zeta}$. It means that the component of the gravity vector of the sliding body cannot produce sliding at this azimuth.

As stated above, for the sliding body on the slip plane $i$, only $T_{i\mu}$ is involved with the mechanical calculation of the sliding body on the slip plane $O$, and the inclination element

Figure 8

The vector analysis of forces acting on the slide mass on the slip plane No. 1, along the given bearings.
value of $T_{iu}$ (RD) is $N\ 99.2^\circ \angle 59.7^\circ$.

In addition, the mechanical calculations of both the sliding body ($W_o=25000T$) on the main slip plane No.0 and $T_{iu}$ ($N99.2^\circ \angle 59.7^\circ$) in the appointed azimuth are to be discussed as follows:

1. In figure 9, the dip angle of the inclined line DB is $30^\circ$ and it represents the true dip line of the main slip plane.

2. On the plumb line AB, take $\overrightarrow{BC}$ as the gravity vector which is 25000 T.

3. According to the principle of formulae 27 and 28 in the Second Paper, through point A make line AX (with dip angle $59.7^\circ$), let $\overrightarrow{AX} = T_{iu}$.

4. Through point X make line CX (with dip angle of $28.3^\circ$—the apparent dip angle of the slip plane No. 0 at azimuth $N\ 99.2^\circ$); line CX intersects line AB at point C.

According to the triangle theorem about composition and resolution of the force vector $T_{iu}$ acting on the main slip plane can be resolved into both a plumb component force $G_{iou}$ ($\overrightarrow{AC}$) and another component force $S_{iou}$ ($\overrightarrow{CX}$) acting along the slip plane at azimuth $N\ 99.2^\circ$.

$G_{iou}$ is involved in the calculation of plumb force vectors on the slip plane No. 0 and $S_{iou}$ is involved in the calculation of the downward sliding forces on the slip plane No. 0 at azimuth $N\ 99.2^\circ$.

Thus, $\overrightarrow{AB} = \overrightarrow{AC} + \overrightarrow{CB} = G_{iou} + W_o$ are acting on the slip plane No. 0 (i.e. line DB) as the plumb forces.

5. According to the principle of formulae 42 and 43 in the Second Paper, with the graphic method similar to figure 8, it is found that $T_{ou}$ at the azimuth $N\ 194.7^\circ$ is negative. This means the sliding body is stable in this direction. At the azimuth $N\ 99.2^\circ$, the downward sliding force $T_{ou} = \overrightarrow{QR} = \overrightarrow{EF} + \overrightarrow{FK}$, where $\overrightarrow{EF} = S_{iou}$ is quoted from the line CX as shown in figure 9. With the figure scale, have been measured $\overrightarrow{QR} = T_{ou}=12010T$; the inclination elements of this force vector is $N\ 99.2^\circ \angle 28.3^\circ$. (compare it with the result obtained in the numerical calculation and listed on page III.188).

This is the whole process to solve the peak value with the graphic method; i.e. the whole process to solve $T_{ou}$ with the graphic method after $\omega_{ou} = 17.7^\circ$ is given. Similarly, when $\omega_{ou}=10^\circ, 15^\circ, \ldots$, value of $T_{ou}$ may be solved for. And the curve of $\omega_{ou}$ versus $T_{ou}$ is plotted. The peak value at this curve is the maximum of $T_{ou}$ and the value of $\omega_{ou}$ corresponds to former. Hence this paper is aimed at the illustration, so the maximum of $T_{ou}$ is taken from the numerical calculation me-
thod in the Second Paper directly. There is no need to determine it according to the curve of $\omega_{cu} \sim T_{cu}$.

If the result of graphic method is expressed with double projection, it will look like that shown in figure 10.

In this figure, the acting point of the downward sliding force vector is selected at the centroid $Q$ of the slip plane No. 0. And its height value and point position in space can be calculated with the information of survey in this paper and restored in figure 10.

Shown in figure 1 to 10 is the whole process described in this paper that begins with original data and ends with calculation result of graphic method.

Main Reference:
EVALUATION OF CREEP TESTS FOR ROCKS

L'ÉVALUATION D'ESSAI DE FLUAGE AUX ROCHES

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ABSTRACT

Creep properties of rocks are investigated as a part of rheology properties. A frame for rheology investigation with four possibilities for measuring has been built by the Department for Geology and Mineralogy at the Technical University of Budapest. This frame allows to determine the change in deformation in time by the means of uniaxial compression tests on constant stress level. The obtained creep curve should have a mathematical form to ensure its application in calculations. In this way rheological characteristics can be found for materials.

For equalizing the functions of the measured points method of minimum squares has been used applying functions which correspond to mechanical boundary conditions of the phenomenon in question. The paper presents the way of determination and interpretation of rheological characteristics in the function considered as the best for the purpose in case of rocks:

\[ e = e_0 + r \ln (1 + t) \]


ABSTRACT

Les propriétés de fluage peuvent être examinées comme un part des propriétés rhéologiques. La chaire de minéralogie et géologie de l’Université Technique à Budapest à construit un cadre d’essai rhéologique de 4 éprouvettes. La déformation différée peut être analysé au cours de l’essai de construction simple, en assurant un niveau constant de contraint. Le courbe de fluage est approché par une formule mathématique, approprié pour les calculs suivants.

Pour l’égualisation des points mesurés la méthode des moindres carrés était employée, en utilisant les fonctions le mieux correspondants aux conditions limites. Le rapport présent le mode de détermination de la fonction supposée d’être l’optimale:

\[ e = e_0 + r \ln (1 + t) \]

et son interpretation pour différentes roches.

RHEOLOGICAL PROPERTIES AND THE CREEP

The rheological properties take an important place among the mechanical properties of rocks. Different examination procedures have developed by the application of the theoretical mechanics. The simplest of them are the creep tests to be carried out by means of uniaxial compression tests. Making use of the rightful assumption that the majority of rocks — to a less or greater degree — behaves flexibly from creep tests we can define the material characteristics showing the physical relation between stresses and
deformations by using the Kelvin respectively Poynting–Thomson linearly flexible rheological material model (Asszonyi–Richter 1979.)

CREEP TEST EQUIPMENT

In 1978 a creep test equipment of oilhydraulic function with four measuring places was planned and set up by me in common with I. Kurth at the Department for Geology and Mineralogy of the Budapest Technical University, which was called REOS (figure 1).

![Fig. 1. Creep test equipment (REOS)](image)

On the four places parallel tests can be effected at the same stress level, but different stress levels also can be executed. The deformations are measured with high precision dial indicators.

By means of ROES — corresponding to the Hungarian standards — creep tests can be executed on cylindric test pieces of 3 ± 0.5 cm and 1:2 diameter height ratio. The creep process within the flexible range is generally observed for thousand hours, then the rheological material constants are counted from the definite creep curves.

MATHEMATICAL EVALUATION OF CREEP CURVES

By the help of the stress deviator tensor $\mathbf{T}$ and the deformation deviator tensor $\mathbf{E}$ respectively their speed tensor ($\dot{\mathbf{T}}; \dot{\mathbf{E}}$) as well as their sphere tensors ($T_0; E_0$) the tensor equation of the Kelvin model written down is:

$$\mathbf{T} = 2G\mathbf{E} + 2\eta\dot{\mathbf{E}}$$

and the tensor equation of the Poynting–thomson model:

$$\mathbf{T} = 2G\mathbf{E} + 2\eta\dot{\mathbf{E}} - \tau \dot{\mathbf{T}}$$

$$T_0 = 3KE_0 \quad \dot{T}_0 = 3K\dot{E}_0$$

where

- $G$ = tangential modulus of elasticity
- $K$ = modulus of compressibility
- $\eta$ = coefficient of viscosity
- $\tau$ = relaxation time

In case of both material models, by uniaxial load application at constant stress, the solution of the differential equation and thus the equation of the creep curve (figure 2) is:

$$\varepsilon = \frac{\alpha_0}{\lambda} \left( \frac{\alpha_0}{E} - \varepsilon_0 \right) e^{-\frac{E}{\lambda} t} = \varepsilon_\infty - (\varepsilon_\infty - \varepsilon_0) e^{-\frac{E}{\lambda} t}$$

where

$$\lambda = \text{linear viscosity factor}$$

![Fig. 2. Creep curves of the Kelvin and Poynting–Thomson model](image)

This relation, however, is not good for evaluating the test results as the value of $\varepsilon_\infty$ is not known. This is why for the description of the creep curve a function had to be searched after that expresses the physical contents of the phenomenon, is sitting on the measured
points, can be easily handled and by its help $\varepsilon_{\infty}$ can be well described in the following form:

$$\varepsilon = \varepsilon_0 + r \ln (1 + t)$$

where $r$ is the so-called rheological constant.

The logarithmic curve is the inverse of the exponential curve, and the function has the advantage that—in accordance with the planned duration of the structure—we ourselves can choose the deformation belonging to infinity by substitution of time $t_{\infty}$ chosen as infinite:

$$\varepsilon_{\infty} = \varepsilon_0 + r \ln (1 + t_{\infty})$$

where $t_{\infty} = 1$ year, 5 years, 10 years and so on.

The results of the creep test effected for determining the rheological constant $r$ is the curve $t - \varepsilon$ which is assumed at constant stress level by means of the mentioned cca one thousand hour = 40–50 day—test.

The measured values are naturally connected with errors of measuring and accidental mistakes. If $(t_1, \varepsilon_1; t_2, \varepsilon_2; \ldots; t_n, \varepsilon_n)$ mean conjugated points of measuring then the square of the „mistake“ is:

$$D = \frac{n}{\sum \varepsilon_i^2}$$

where $\varepsilon_i = \varepsilon(t_i)$ = the measured deformation

$\varepsilon(t) = \varepsilon(t) = \varepsilon(t) = \varepsilon(t)$

It is our task that the square sum should be the least, which means that the deviation of the approximate curve from points $(t_i, \varepsilon_i)$ is the least among all curves. It can be attained that the above square sum is minimalized

$$\frac{\partial D}{\partial r} = - 2 \sum_{i=1}^{n} [\varepsilon_i - r \ln (1 + t_i) - \varepsilon_0] \ln (1 + t_i) = 0$$

$$\frac{\partial D}{\partial \varepsilon_0} = - 2 \sum_{i=1}^{n} [\varepsilon_i - r \ln (1 + t_i) - \varepsilon_0] = 0$$

Solved according to the Cramer formula:

$$r = \frac{n \sum_{i=1}^{n} \varepsilon_i \ln (1 + t_i) - \sum_{i=1}^{n} \varepsilon_i \sum_{i=1}^{n} \ln (1 + t_i)}{n \sum_{i=1}^{n} \ln^2 (1 + t_i) - \left( \sum_{i=1}^{n} \ln (1 + t_i) \right)^2}$$

$$\varepsilon_{\infty} = \frac{n \sum_{i=1}^{n} \ln^2 (1 + t_i) - \left( \sum_{i=1}^{n} \ln (1 + t_i) \right)^2}{n \sum_{i=1}^{n} \ln (1 + t_i) - \sum_{i=1}^{n} \varepsilon_i \ln (1 + t_i)}$$

The 3, 4, 5 figures show some examples for using of this method.

![Fig. 3. Creep curve of Miocene rhyolite tuff](image-url)
Fig. 4. Creep curve of Oligocene clay marl

Fig. 5. Creep curve of Permian sandstone

REFERENCES

LANDSLIDE IN CUT SLOPE OF OVERCONSOLIDATED CLAY DERIVED FROM VOLCANIC ROCKS

GLISSEMENT DE TERRAINS AU TALUS DEBLAYE EN ALGILE SURCONSOLOIDEE AYANT SA SOURCE DE LA ROCHE VOLCANIQUE

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ABSTRACT

In this paper the origin and mechanism of the landslide are described and analysed from the standpoint of geotechnical engineering. Topographic and geological features and ground water condition in the sliding area became clear from the detailed geotechnical surveys. The landslide resulted from the cutting of hillside and the saturation of ground, and was classified as the rockslide of weathered materials derived from volcanic rock in a ridgy terrain. The condition of rainfall fully satisfied the rainfall level of threshold value of landslides.

ABSTRACT

Ce rapport donne l'explication sur la cause et le mécanisme dudit glissement de terrains au point de vue de la geotechnique, et effectue une analyse de stabilisation. Ainsi, l'édit phénomène est classifié comme un glissement des matériaux décomposés de roche volcanique dans un relief de terrains aristiforme, et, selon une confirmation par l'analyse de stabilisation au moyen basé sur les contraintes effectives, la cause dudit glissement de terrains réside en débrayage du talus et en saturation du sol. Étant une couche mince d'argile comprenant les montmorillonites, la surface glissante dudit glissement s'est composée le long d'une diaclasse dans un stuff subi d'une alteration hydrothermale.

INTRODUCTION

The survey on disasters gives the basic data for the environmental evaluation due to development and prediction of landslide. On the 24th June 1978 a landslide occurred at Shiraki, Ouchchi City located in the north of Kagoshima Prefecture, southern Kyushu in the Japanese Southwest; destroying two inhabitants and two at houses and causing damage to a road and a paddy field. The landslide occurred during the improvement-work of road and at the time of heavy rainfall, in the cutting slope of overconsolidated clays derived from altered volcanic rocks. In this paper the origin and mechanism of the landslide are described from the standpoint of geotechnical engineering. In order to investigate geological structure and groundwater condition in the area of landslide, electric prospecting, electric logging, drilling survey, groundwater tracing and interpretations of aerial photograph and of topographic map were performed. The dependence of the landslide on rainfall also was investigated. Physical and mechanical proper-
ties of clays were studied by means of laboratory soil tests. Stability analyses using the conventional slice method were performed on the slopes before and after the cutting.

**PROGRESS OF LANDSLIDE**

**Cutting and Excavation**

The landslide area is located within Shiraki district about 7 km from the urban area of Okuchi City along the local road of Izumi-Okuchi route. Accompanying the work of improving this road, the foot of a natural slope was cut at a gradient of 1:0.8, and moreover, the excavation for a retaining wall was carried out. The cutting-away work was performed from May 26 to June 10, and the excavation was finished on June 14. The landslide occurred at 10 minutes past 10 on June 24.

**Rainfall Condition**

It seems very important to study the influence of the amount of rainfall on the activation of landslide. The annual precipitation in Okuchi City is about 2500 mm. As for the rainfall which seemed to be responsible for this landslide, the total amount of the rainfall till the occurrence of the landslide was 332 mm, the rainfall during 24 hours just before the occurrence of the landslide was 174 mm, and the hourly rainfall was 26 mm as shown in Fig. 1. The return periods for the 24 hours rainfall and the hourly rainfall correspond to 3.4 years and 1.2 years, respectively, and these are not rare rainfall in view of the probability. Also excluding the rainfall of June 20 as a preceding rainfall, the effective accumulated rainfall for the occurrence of the landslide turned out to be 300 mm.

Plotting the rainfall condition with the effective rainfall of 300 mm and the hourly rainfall of 26 mm on the chart of rainfall-level of threshold value of landslides in the southern Kyushu (Haryama, 1980), this rainfall condition is clearly in the range of the occurrence of a heavy disaster as shown in Fig. 2.

**TOPOGRAPHIC AND GEOLOGICAL FEATURES**

The topographic map around the landslide area prepared from the aerial photographs is shown in Fig. 3. The sliding mass was at the margin of ridgy terrain, and the gradient of the original ground surface before the landslide was about 20 degrees at the steepest part and relatively gentle. When the topographic map around the landslide area was inspected, the microrelief of a typical type of ridge landslide was deciphered from the contour lines. Moreover, it was presumed according to the geomorphic lines shown with dotted lines that some potential slip surfaces might exist in the layer deeper than this slip surface. Such topography looks like being stable in natural state, but frequently causes the case in which the artificial change of the topographic environment such as the cutting-away of slope becomes the agent causing landslide.

In order to investigate the geological structure of the landslide area, electric prospecting tests along lines AA and BB, electric logging test in borehole Bo No. 1 and drilling surveys in Bo No. 1 and Bo No. 2 were performed as shown in Fig. 4. Electric prospecting consisted of vertical and horizontal surveys roughly distinguishes strata by the difference in specific

![Rainfall Graph](image_url)

**Fig. 1** Rainfall in June 20 to 24, 1978, in the landslide area
to be different according to the degree of alteration of strata, the distinction of strata based on the specific resistance value was carried out.

The surface of low specific resistance layer 1 corresponds to the slip surface of landslide. The boundary between low specific resistance layer 1 and high specific resistance layer II has a gentle slope and seems to be a potential slip surface. Also from the result of electric prospecting along line BB in Fig. 4, a complex structure accompanied with development of faults and fissures around the sliding area was estimated.

One of the geological profiles obtained from all-core drilling is shown in Fig. 6. In the profile of

Fig. 2 Rainfall-level of threshold value of landslide

Fig. 3 Microlief map around landslide area

resistance. Electric logging gives the deposit condition of strata from variation in specific resistance by using a borehole and gives more accurate geological structure.

As shown in Fig. 5, the geological structure clarified by electric prospecting along line AA in Fig. 4 was the alternate strata with high specific resistance layer and low specific resistance layer. Since the specific resistance value is considered

Fig. 4 Location of main geological surveys
Bo No.1, the strata down to the depth of 5.15 m (1st and 2nd layers) and the clay layer at 5.15 m (3rd layer) accurately corresponded to the sliding mass and the slip surface ascertained in the landslide outcrop, respectively. Furthermore, it was confirmed with the cores of drilling that the low specific resistance layer I mentioned before was the lava subjected to intense hydrothermal alteration and corresponded to almost 4th to 9th layers, while the high specific resistance layer II was also the lava subjected to weak alteration and corresponded to 10th and deeper layers. From the irregular change of specific resistance value in the electric logging diagram in Fig. 6, it was presumed that the degree of alteration was different in the thin layer form.

Geological sections obtained by the observation of outcrop in the landslide area are shown in Figs. 7 and 8. The gray altered lava in these figures is a high specific resistance layer I subjected to weak hydrothermal alteration. The maximum thickness of this altered lava was about 7 m, and the development of fissures, platy joints and columnar joints was conspicuous, which severed rocks into block form (Photo 1). The upper tuff was brown or reddish brown clay with the thickness of 1.0 to 3.0 m. On the surface of the lower tuff, the thin film of white clay with high water content and several mm in thickness had been formed.

From the X-ray diffraction patterns,
Fig. 7  Geological profile section by outcrop observation (Section CC in Fig. 4)

1. Landslide mass
2. Debris
3. Gray altered lava
4. Upper tuff
5. Lower tuff
6. Low specific resistance layer I consisted of intense altered lava

1 and 2; Sliding mass
2, 3 and 4; Sliding mass

Fig. 8  Geological cross section by outcrop observation (Section DD in Fig. 4)

it was found that the upper tuff contained halloysite and kaolinite and that the lower tuff contained montmorillonite and halloy-
site as main clay minerals.

The boundary of the upper tuff and the lower tuff was clear, and it was considered to be the joint. The landslide occurred
along this dipping joint with the clay seam contained montmorillonite between the upper and lower tuffs. The strike and the dip of
this joint were N to NNE and (10 to 35)°E, respectively. Therefore, the slip surface
was at a slope of 35 degrees near the scarp
at the head of landslide and became gradually gentle slope from the midslope to the
toe of slope. Also, as shown in Fig. 8, in the cross direction of slip surface, a
number of level differences caused by
fractures formed in the past geological age
were present, and a gentle concave terrain
was formed.

It is considered that the sliding mass
was restricted by a fault on the southeast
surface, by tension cracks from the opposite northwest surface to the scarp at the head
and by the dipping joint with clay seam on the bottom.

GROUND WATER

In the occurrence of landslide, the
influence of groundwater must be correctly
evaluated. As mentioned before, the develop-
ment of joints and fissures was conspicuous in these strata, therefore, the infil-
tration of surface water may be easy. Also
when the valleys in both sides of the land-
slide area where surface water always flows
and the flood-marks in the valleys are con-
sidered, the infiltration of groundwater
from surrounding area seems to be possible
at the time of heavy rainfall. In order to
confirm the state of infiltration of
ground water, the tracing test using salt
water as a tracer was carried out. The
injected hole of salt water was 0.7 m in
diameter and 0.5 m in depth and its loca-
tion, as shown in Fig. 4, was decided after
general consideration of the topography.

The tracing of salt water was performed
by the electric prospecting at a definite
time interval before and after pouring salt
water at the time of rainfall. Since the
specific resistance value in the tracing
of salt water scattered largely among the
measuring points, it was found that the groundwater did not infiltrate uniformly into ground, but it moved along the joints and fissures.

**PHYSICAL PROPERTIES OF CLAYS**

The lump of upper tuff consisted entirely of a overconsolidated intact clay and the lump of lower tuff was a overconsolidated stiff clay with microscopic fissures. Therefore, the upper tuff and the lower tuff were called the upper clay and the lower clay, respectively. The physical properties of clays are shown in Table 1.

**STRENGTH PROPERTIES OF CLAYS**

The specimens, 60 mm in diameter and 20 mm in thickness, were prepared by cutting out with a knife from the block sample of clays. Direct shear tests were carried out on the upper clay, lower clay and pre-cut specimens. The specimens were saturated by being immersed in water for 24 hours in shear box, then consolidated for 24 hours under a predetermined normal stress. Thereafter, they were sheared at the rate of 0.02 mm/min under the condition of constant volume. For determining the rate of shear, the condition under which the strain rate does not exert influence to shear strength was considered. As for the pre-cut specimens, the disks of 10 mm in thickness were cut out from the respective block samples of the upper and lower clays adjacent to the slip surface, and they were put together to make a specimen of 20 mm in thickness. The pre-cut plane was made to coincide with the shear plane.

**Residual Strength**

The residual strength of undisturbed clay is practically important, not only as

<table>
<thead>
<tr>
<th></th>
<th>Lower clay</th>
<th>Upper clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.70</td>
<td>2.79</td>
</tr>
<tr>
<td>Clay fraction (%)</td>
<td>35</td>
<td>20</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>103</td>
<td>112</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>69</td>
<td>86</td>
</tr>
<tr>
<td>Natural water content (%)</td>
<td>82</td>
<td>85</td>
</tr>
<tr>
<td>Degree of saturation (%)</td>
<td>88</td>
<td>92</td>
</tr>
<tr>
<td>Wet density (t/m³)</td>
<td>1.40</td>
<td>1.44</td>
</tr>
<tr>
<td>Dry density (t/m³)</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>Void ratio</td>
<td>2.50</td>
<td>2.58</td>
</tr>
<tr>
<td>Coefficient of permeability (cm/s)</td>
<td>$10^{-5}$</td>
<td>$10^{-6} - 10^{-7}$</td>
</tr>
</tbody>
</table>
the basic property of the clay but also in relation to the analysis of the long term stability of slope. Especially that is important in the study of the mechanism of causing progressive failure. The determination of residual strength requires the large deformation of the specimen, therefore, that is difficult by conventional direct shear box test and triaxial test, and that is carried out by ring shear test (Bishop et al, 1971). In this paper, as the convenient method for determining the residual strength by direct shear box, multiple reversal shear box tests were performed. After a pre-cut specimen was subjected to shear stress up to a certain displacement, upper and lower shear boxes were pushed back to the initial position again, and then shear stress was loaded. This process was repeated until the strength of clay had dropped to a steady value.

A typical relationship between shear stress $\tau$, effective normal stress $\sigma'$ and horizontal displacement $d$ for the pre-cut specimen in the reversal shear box test is shown in Fig. 9. In the first shear, shear stress decreased progressively with an increase in displacement and appeared to have a tendency finally to become constant after the peak was passed. In the second and third shears, shear stress was nearly constant.

The state in which shear stress and normal stress maintain constant value in the case of accompanying shear displacement under constant volume condition may be regarded as the state of residual strength. Similar tests on other pre-cut specimens, at various effective normal stresses, showed that the peak strength can be represented by the parameters $\phi' = 20$ degrees and $c' = 0$, while the residual strength fall along a line with a slope $\phi_r' = 14$ degrees and $c_r' = 0$ as shown in Fig. 10.

Shear Strength of Clays

Typical relationships between shear stress, effective normal stress and horizontal displacement for constant volume direct shear box tests on the upper clay and the lower clay are shown in Figs. 11 and 12, respectively. The shear planes after the direct shear test were very smooth and had formed extremely well-defined slickensides (Photo 3).

In Figs. 11 and 12, shear stress-displacement curves show the peak value and its peak is more clearly defined in the lower clay than in the upper clay. In order to maintain the constant volume of specimen, the effective normal stress during shear was required to change with an increase in displacement. Similar tests were performed on other specimens of clays at various effective normal stresses. Figs. 13 and 14 are stress path diagrams obtained from these tests on the upper clay and the lower clay, respectively.

In the stress paths of the upper clay, the scattering among the specimens was little, but in the lower clay, the scattering was large. This phenomenon is due to the fact that the lower clay was subjected
to uneven alteration as compared with upper clay subjected to relatively homogeneous alteration. Strength parameters, \( c' \) and \( \phi' \), were determined from the failure envelope connecting the peak points of the stress paths, and \( c_u \) and \( \phi_u \) were determined by plotting the peak strength of stress paths against the stress at the time of consolidation.

It is estimated that a preconsolidation stress was 50 tf/m² for the upper clay from Fig. 13 and was 65 tf/m² for the lower clay from Fig. 14. Considering the stress state in field, the overconsolidation ratio of these clays was 3.0 to 4.0.

**STABILITY ANALYSIS**

The sliding section CC shown in Fig. 4 was divided into slice as shown in Fig. 15. The stability analyses before and after the cutting, and at the times of normal and saturated states, were carried out by total stress method and effective stress method, using the conventional slice method. Since the development of joints and fissures was conspicuous, the slope-ground was in the unsaturated state of about 90% in the degree of saturation at the time of normal state, while the ground was considered to be saturated at the time of heavy rainfall. The unit weights of sliding mass decided from measured values for the stability analysis are shown in Table 2.

**Total Stress Analysis**

Total stress analysis is performed on the saturated state. As the landslide occurred along the dipping joint, the shear
strength of joint must be adapted for stability analysis. However, considering that landslides depend upon weak clay in the strength, the strength parameters of upper clay, $c_{cu} = 1.0 \text{ tf/m}^2$ and $\phi_{cu} = 17$ degrees in Fig. 13, were adapted. The calculated factors of safety are shown in Table 3. The decrease of the safety factor due to cutting of slope was confirmed, but the safety factor of post-cutting has not gone down below 1.0. However, in actuality, the landslide occurred after the cutting-away, therefore, it was concluded that the total stress analysis is unreasonable.

**Effective Stress Analysis Using Residual Strength**

The pre-cut specimen showed the residual strength parameters of $c_\gamma = 0$ and $\phi_\gamma = 14$ degrees as mentioned before. The safety factors obtained by using the above parameters are shown in Table 3. Since the natural slope before the cutting has maintained the stability for long term, the safety factor cannot go down below 1.0 in reality. Also the slope analysis by the residual strength is unreasonable in the same as total stress analysis mentioned above.

**Effective Stress Analysis Using Peak Strength**

The mobilized effective normal stress $\sigma'_m$ and the mobilized shear stress $\tau_m$ on slip surface are expressed as follows:

$$
\sigma'_m = \gamma h \cdot \cos^2 \alpha
$$

$$
\tau_m = \gamma h \cdot \cos \alpha \cdot \sin \alpha 
$$

$$
\sigma'_m = (\gamma' - \gamma_w) h \cdot \cos^2 \alpha 
$$

$$
\tau_m = \gamma' h \cdot \cos \alpha \cdot \sin \alpha 
$$

where $h$ = depth of slice in sliding mass
$\alpha$ = inclination of slip surface
$\gamma$ = unit weight of sliding mass

**Fig. 13** Stress paths for constant volume direct shear box tests on upper clay

**Fig. 14** Stress paths for constant volume direct shear box tests on lower clay

**Fig. 15** Division of sliding mass for stability analysis
\( \gamma' = \) saturated unit weight of sliding mass  \( \gamma_w = \) unit weight of water

Table 2  Unit weights of sliding mass for stability analysis

<table>
<thead>
<tr>
<th>Condition</th>
<th>Unsaturated</th>
<th>Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td>Altered lava (tf/m³)</td>
<td>2.23</td>
<td>2.25</td>
</tr>
<tr>
<td>Upper clay (tf/m³)</td>
<td>1.46</td>
<td>1.54</td>
</tr>
</tbody>
</table>

Table 3  Safety factors

<table>
<thead>
<tr>
<th>Condition</th>
<th>Total stress analysis</th>
<th>Effective stress analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Saturated</td>
<td>Residual strength</td>
</tr>
<tr>
<td></td>
<td>Normal</td>
<td>Saturated</td>
</tr>
<tr>
<td>Before cutting</td>
<td>1.25</td>
<td>0.77</td>
</tr>
<tr>
<td>After cutting</td>
<td>1.06</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Calculating \( \sigma_m' \) acting on the respective slices at the times of normal state and saturated state by using equations (1) and (2), the shearing strength \( s_f \) expected under the \( \sigma_m' \) was read from the failure envelope in Fig. 13. The peak strength parameters corresponding to the stress state of respective slice are \( c' = 0.5 \text{ tf/m}^2 \) and \( \phi' = 34 \) degrees. The safety factor is calculated from the ratio of total shearing strength estimated from the relation between \( s_f \) and \( \sigma_m' \) to total mobilized shear stress estimated from second formulas in the equations (1) and (2) along whole slip surface. The calculated values of safety factors are shown in the right column of Table 3. The effective stress analysis using the peak strength has explained well the decrease of the safety factor due to the cutting and the saturation.

In order to investigate the mechanism of landslide occurrence due to the cutting and the saturation of slope, \( \tau_m' \) and \( \sigma_m' \) of respective slices were plotted in the \( \tau - \sigma \) plane as shown in Fig. 16.

As shown in Fig. 16(a), the \( \tau_m - \sigma_m' \) plots at the time of normal state under unsaturated condition were on the safe side than the peak strength both before and after the cut regarding all the slices. At the time of heavy rainfall under the saturation of slope, as shown in Fig. 16 (b), \( \tau_m - \sigma_m' \) plots were on the dangerous side in the slice No. 11 to 22 both before and after the cut, and on the safe side in the slice No. 1 to 10 before the cut and in the slice No. 5 to 10 after the cut.

It is understood that the slice No. 1 to 10 achieved the role as the overburden load resisting the slip of the slope No. 11 to 22, thus the slope was stable as a whole. After all of the cross section of the slice No. 1 to 4 and a part of the slice No. 5 to 8 had been cut away, the safety factor along the slip surface had fallen owing to the decrease of resistance by the overburden load.

Skempton (1964) introduced the residual factor \( R \) by comparing the actual average shearing strength occurring at failure of slopes \( \bar{g} \) with the peak \( s_f \) and residual \( s_r \) shearing strength of specimens, and defined by the expression:

\[
R = \frac{s_f - \bar{g}}{s_f - s_r} \tag{3}
\]

In this clay, \( R < 0 \) ensued. Since the safety factor when landslide has occurred in reality equals to 1.0, this case should be considered as \( R = 0 \). This means that physically the peak strength was mobilized along the whole slip surface, and progressive failure did not occur. The fact that in spite of the landslide along the joint, \( R = 0 \) ensued, was able to be understood from that the time after the cutting till the occurrence of the landslide was very short.

Bishop et al (1971) proposed the brittleness index \( I_B \) to quantitatively express the reduction of strength in passing from peak to residual:

\[
I_B = \frac{s_f - s_r}{s_f} \tag{4}
\]
This index depends on the effective normal stress and generally decreases with the increasing normal stress. The $I_B$ value in this landslide, as shown in Fig. 17, became 1.00 in the case of $\sigma' = 0$, decreased rapidly up to about $\sigma' = 4$ tf/m², and approached the constant value of 0.64 in $\sigma' > 10$ tf/m². This clay has the somewhat higher $I_B$ value. As far as the clay was judged by the $I_B$ value, that was the clay in which the possibility of causing progressive failure cannot be denied.

CONCLUSION

This landslide is classified as the rockslide of weathered materials derived from the altered volcanic rocks in a ridgy terrain. The weathered strata consisting of lava and tuff with a great many of joints and fissures were formed by the hydrothermal alteration of volcanic rocks.

The observed slip surface was located in the dipping joint with montmorillonite seam between clayey tuffs. The sliding mass was restricted by fault, tension cracks and dipping joint.

The condition of rainfall with the effective accumulated amount of rainfall and the amount of hourly rainfall preceding the time of activation of the landslide fully satisfied the rainfall-level of threshold value of the landslides.

Physical and mechanical properties of clays are cleared from the laboratory tests.

Stability analysis using the conventional slice method based on the effective stress gave the decrease of safety factor due to the cutting of hillside and the saturation of ground.

REFERENCES


STABILITY ANALYSIS OF FISSURED CLAY SLOPES AND EXCAVATED SLOPES

ANALYSE DE STABILITE DE LA PENTE ET DU VERSANT D'ARGILE FISSUREE

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ABSTRACT

This paper deals with the problems on the stability analysis of fissured clays distributed in northwest China. It consists of three parts. The first part is concerned with the relation of clay minerals and their physico-chemical properties with the formation of fissures. The second part discusses the slope stability analysis and classification in terms of the local stability, clay fissuring, slope profiles and generation and development of landslides. The third part presents several principal problems in the stability analysis of excavated slopes: (1) the analysis and determination of slip planes; (2) the selection and determination of shear strength parameters; (3) the problem of how to take the value of the safety factor ($K_T$); (4) the analysis of the results of checking the stability in a geological cross-section; and (5) the classification of excavated slope stability. For these problems, the author elucidates the characteristics and nature of three main types of structural planes of weakness, indicates how to select shear strength parameters according to different structural planes of weakness, proposes the calculation formula $K_T = n \cdot p \cdot m$, raises the criteria for the classification of the state of stability, and makes a classification of excavated slope stability on the basis of the lithology, characteristics of weakness planes, ground-water and data on the calculation of stability in the slope segment.

ABSTRAIT

Le texte présente des considérations sur le problème de la stabilité du versant constitué par l'argile fissurée dans un secteur de la Chine Nord-
Ouest. L'exposé se présente en trois parties.

1. Considérations sur la relation entre des minéraux argileux, leur caractéristique physico-chimique et la formation des fissures.

2. Considérations sur l'analyse de la stabilité de la pente et sa classification à partir de la stabilité de la région, de la fissurité de l'argile, de la forme de pente, de l'apparition et du développement du glissement de versant.

3. Considérations sur des problèmes principaux surgis dans l'analyse de la stabilité du versant artificiel, à savoir:

   (a) Analyse et détermination de la surface de glissement, caractéristique et nature des trois types principaux du plan de faiblesses.

   (b) Choix et détermination de l'indice de la résistance au cisaillement, présentation de la méthode de la sélection de la valeur de résistance au cisaillement selon le plan de faiblesses.

   (c) Choix du coefficient de sécurité (Kp) et proposition de la formule de calcul Kp = Kp + m.

   (d) Analyse des résultats de calcul de la stabilité du profil géologique et proposition des critères pour la division des régimes de stabilité.

   (e) Classification de la stabilité du versant à partir de la nature lithologique de terrains dans le secteur de la pente, de la caractéristique du plan de faiblesses, de l'eau souterraine et des données de calcul de la stabilité.

The stability of natural slopes and excavated slopes is a problem of engineering geology frequently encountered during the construction in mountain areas, which is directly related to the utilization of construction sites and the safety of buildings.

This paper is dealt with analysis of stability of fissured-clay slopes and excavated slopes distributed in a certain district of North-west China.

I. Mineral Composition and Physico-chemical Properties of Fissured Clay

The fissured clay is of a special type of earth. The abundance of fissures in this clay is related to its mineral composition and physico-chemical properties. Based on the analytical data on clayey minerals and physico-chemical properties, it is known that in mineral composition, this clay contains grains (<2μ in size) composed mainly of a mixture of montmorillonite and illite (hydromica), which have a larger specific grain surface area and a higher index of cation exchange than those of in expandible clayey soil, and a relatively high content of free SiO2 and Fe2O3. The SiO2 and Fe2O3 can serve as a cement in the clay and may reduce the degree of expansion and contraction. The main indexes of expansibility of the clay are as follows: the average ratio of free expansion is > 50% and the average ratio of total expansion is < 2.0. Thus this clay may be of a weak expansible type. A series of primary and secondary fissures are formed in it under the influence of physico-chemical conditions. The primary fissures are a result of non-equilibrium expansion during the sedimentation or erosion of the clay, and the expansion in turn has led to the formation of columnar joints, of which the surfaces have a waxy luster. Generally, there may be two factors causing the secondary fissures: 1). During the late period of development of the clay, due to the exogenetic force (including structural and non-structural stresses) most fissures were formed by shearing stress produced along the primary fis-
sures. Therefore there are striae arranged in a certain direction on fissure surfaces. These fissures are continuous, and have a long extension and a regular attitude; 2) Irregular network fissures were formed by weathering force, which are narrow and dense. Because of the special nature of the fissured clay compared with the common clay, only after fully understanding the properties of the fissured clay can an analysis of slope stability be made correctly.

II. Stability Analysis of Fissured-clay slope

Natural slopes are geomorphological features formed by a long-term combined process of structural stress and weathering. The evolution and development of these slopes always followed a natural law of equilibrium-non-equilibrium-equilibrium. Practical work indicates that the following points have a direct influence on the stability of slopes:

1. Regional Stability

Regional stability refers mainly to the degree of influence on stability of a certain region by physico-geological process caused by tectonic activities. There are a lot of factors influencing the regional stability, among which the most important is earthquake. Earthquake can loose soil bodies and further expand fissures in them. On the other hand, it can reduce the shear strength of the weak textural surface and the contact surface between rock and soil, and weaken the stability of soil bodies, resulting in possible landslide. Based on related data, these physico-geological phenomena mostly occur in areas of intensity VII or higher except a few of them which may occur in areas of intensity VI. Particularly at the joining place of fault zones, slides are well developed on slopes. All this indicates that the stability of structures of a region has a direct influence on the stability of slopes.

2. Fissures of Fissured Clay

Fissured clay is characterised by well-developed fissures, which can be combined to form various weak textural surfaces. Therefore investigation and study of fissures should be paid special attention to in areas of fissured clay. Besides fissure type, occurrence, extension, roughness, water capacity, and composition of infilling material, etc., the combination of fissures and that of slopes should also be investigated, in order to determine the relationship between the distribution of fissures and the stability of slopes. It is clear from investigation that within 3.0-4.0m below the surface of fissured clay bed, weathered fissures are relatively dense, occurring as irregular networks and consisting of loose and soft soil, but at depth, fissures are sparse and consist of compact soil, and hence the fissured clay at the interface contains more water than the clay above and below, forming a weak surface in the weathered zone. According to analytical data, the water content of the clay at the weak surface is higher than that of liquid limit. Due to the presence of this weak surface, shallow-bed sliding of the fissured clay would readily take place. Another kind of fissures were formed under tenso-shearing stress. These fissures have certain attitude elements and combinations. Smooth fissure surface (shown by arrow, plate 1) is clearly found in exploratory pits, on which there is generally a viscous film formed by leaching. This film is wet and greasy, on which clear shearing striae can be found. Because of the good extension and connection nature, these fissures tend to be channelways for water infiltration, thus there is often water-seeping along the fissure surfaces. The smooth surface of secondary fissure is the main weak surface in the fissured clay, not only destructed the completeness of the clay, but also reduced the shear strength of it, thus decreased the stability of soil body of the slope.

3. Shape of slope

The shape of a slope results from a combination of various stresses and forces and is closely related to the nature of rocks and soil, the development of weak surface and the erosion by water. It is found from investigation that on the basis of shape, natural slopes in areas of fissured clay can be grouped
into the following three types: (1) Steep slope, which has an angle of $>25^\circ$. Most soil bodies of it are in an unstable state and there is relatively more surface-bed sliding; (2) Steep-gently slope, which is characterised by two sub-types, one being steep in the upper part and gentle in the lower, the other being gentle in the upper part and steep in the lower. The former is mostly formed by sliding and in a temporary stable state, and the latter is generally located on two sides of a combe, formed by flushing of surface water and in an unstable state; and (3) Gentle slope, which generally has an angle of $12^\circ-14^\circ$ and is in a stable state.

4. Occurrence and Development of Landslides

The occurrence and development of a slide is a result of the combination of internal and external factors and closely related to meteorological conditions. According to meteorological data of a district, the highest temperature is $39.3^\circ$C and the lowest $-12.3^\circ$C, and annual evaporation is greater than precipitation. Under these climatic conditions, in dry season fissures are further developed and in rainy season fissures become channelways for the infiltration of surface water, and may increase the shearing stress of soil bodies and decrease the shear strength of weak surfaces, which is the main factor causing the landslide. An engineering landslide would occur and develop through the following process: Digging below a slope results in an effective free surface; under tensile stress, fissures are further expanded and split; and after several successive days of heavy rains a landslide occurs. (Plate 2 has shown fissures about 1.0m wide in the lower part of a slope. Plate 3 has shown a series of fissures 10-20cm wide in the upper part of a slope.).

Plate 3

The occurrence and development of a landslide may destroy directly the stability of a slope. Therefore, the investigation and study of landslide, including the determination of its occurrence and development causes, its type and thickness, and the characteristics of sliding surface, is helpful in preventing the occurrence of landslide and maintaining the stability of slope.

5. Stability Classification of Natural Slopes

Stability classification of natural slopes is of significance in the understanding of slope stability and the utilization and the controlling of slopes to make them suitable for engineering construction. This classification is based on shape of slope, properties of rock and soil, underground water, and development of landslide, etc. Slopes can be roughly classified as three types:

(1) Stable slopes: Gentle, with an angle generally less than $14^\circ$, having compact soil and less fissures, and containing no readily sliding weak surface. Unfavourable physico-geological features are absent.

(2) Moderately stable slopes: Moderately steep, with an angle generally $>14^\circ$ but $<25^\circ$, and containing relatively more smooth fissure surfaces along which water-seepage is commonly found. There are relatively more landslides on these slopes.

(3) Unstable slopes: Steep, with an angle $>25^\circ$; having loose soil and readily sliding weak surface; and located at joining place of fault zones and strongly affected by earthquake. Landslides are well developed on these slopes.

III. Analysis of Stability of Excavated Slopes
The stability of fissured-clay excavated slopes is related not only to the coefficient of strength of soil body and the height of slope, but also to the development and shear strength of various weak surfaces present in the slope's soil body, and the altitude elements and the combination of side-slope lines. Generally, the analysis is made through the calculation of stability of weak surface using mathematical method. In the calculation, the following points should be paid special attention to:

1. Determination of Sliding Surface and Destructive Mechanism

Analysis of slope's stratigraphy shows that there are three major types of weak surfaces: 1) weak surface consisting of smooth fissure surface; 2) weathered-zone weak surface consisting of weathered fissures; and 3) weak contact surface located at the bottom of clay (Fig. 1).

![Fig. 1 Sketch of locations of major weak surfaces](image)

1.—Smooth fissure surface
2.—Weathered-Zone weak surface
3.—Weak contact surface

The weak contact surface at the bottom of clay bed was formed by the process of underground water at the interface, which can result in a water oversaturated state of the clay and the formation of a weak surface, and can directly influence the stability of the clay bed with the variation of slope of the top of underlying hard rock bed. Generally, if the slope of the top of this rock bed is steep, the overlying clay is in an unstable state. Based on the above analysis, three types of destruction mechanism can be presumed for the clay. The first is the shearing destruction of pushing type, occurred along smooth fissure surface; the second is the shearing destruction of dragging type, occurred along weathered-zone weak surface; and the third is the shearing destruction of pushing type, occurred along weak contact surface. From practical work, it is clear that the smooth fissure surface and the weathered-zone weak surface are highly subject to sliding, but whether the weak contact surface is a sliding one depends mainly on the slope of the top surface of the underlying hard rock bed. In general, the loss of stability of any side-slope soil bodies can occur only when boundary conditions of destruction exist. The main boundary condition is the determination of sliding surface, which is also an important requirement for the calculation of side-slope stability.

2. Selection and Determination of Indices of Shear Strength

Correct selection of indices of shear strength may directly influence the calculation of stability. Therefore, when selecting these indices, we should analyse comprehensively the data from indoor and outdoor tests and simultaneously consider the characteristics of fissured clay and the properties of weak surface. It is clear from test data that the value of residual shear strength of the remolded soil sample is basically consistent with that of shear strength of sliding surface, and that the data of water-saturated rapid shearing test for undisturbed earth are close to those of large-scale field shearing test, but slightly high compared with those of static triaxial shearing test. Theoretically, the static triaxial shearing test can reflect more accurately the actual situation of fissured clay, so the data from it is more reliable. The authors hold that it is more reliable to select the data of static triaxial shearing test for the C and \( \phi \) values of weak contact surface, and the data of plastic-earth residual shear strength test for those of smooth fissure surface and weathered-zone weak surface.

3. Determination of Safety Coefficient \( K_t \)

The coefficient of slope safety refers, in mechanics, to the ratio of strength leading to the destruction of soil body so that preventing the soil body from destruction, and is a criterion for the stability of soil body. Generally, when stability coefficient is 1.0, soil body is considered stable. However, for important construction site, the long-term stability of side slope is generally considered, with emphasis on the influence of this stability on the safety of buildings. Thus the safety coefficient is suggested. No formula has yet been proposed in current literature.
for the determination of safety coefficient, and hence subjective factors may have a relatively large effect on it. The authors consider that for permanent side slopes, the influence of earthquakes and the reliability of shear strength are the main factors that should be taken into account. Therefore it is suggested that the following formula be used: \( K_P = n_p + m \), where \( n_p \) is the coefficient of earthquake additional force, and \( m \) is determined by error of ultimate shear strength \( (\tau_u) \),

\[
\frac{n_p}{K_P} = \frac{\tan \phi_1 \sum Q_1 \cdot \cos \alpha_1 + C_1 E_1}{\sum Q_1 \cdot \sin \alpha_1}
\]

\[
K_P = \frac{\tan \phi_1 \cdot \sum Q_1 \cdot \cos \alpha_1 + C_1 E_1}{\sum Q_1 \cdot \sin \alpha_1 + P} \cdot P = K_0 Q,
\]

where \( P \) is the earthquake force, \( K_0 \) is the coefficient of earthquake and \( Q \) the weight of soil body, \( m = \tau_u V \).  

4. Discussion of Stability Calculation of Geological Section

Whether the stability calculation conforms to reality depends firstly on whether boundary conditions are really reflected, secondly on whether the parameters of mechanics for various boundary surfaces are reliable. The three types of weak surfaces in fissured clay represent the main boundary conditions of side-slope destruction. Therefore in the calculation of stability, emphasis should be laid on the calculation of stability of these weak surfaces (Fig.2).

![Fig. 2 Sketch of stability calculation](image)

1—Slope-cutting line  
2—No. of block  
3—Bottom surface excavated

Based on the analysis of calculated geological sections, when the ratio of the cut slope is greater than the average ratio of slope (1:2), slumping along smooth fissure surface (EF) and sliding along weatheredzone weak surface (CD), as well as sliding along weak contact surface (AB) if the top surface of underlying hard rock bed is steep, would occur. A classification of four types of stability state, based on the \( K \) produced by stability calculation, is suggested: (1) stable \( (K > K_P) \); (2) basically stable \((1.0 < K < K_P)\); (3) critical stable \((1.0 = K = the minimum value of (2))\); and (4) unstable \((K < 1)\).

5. Stability Classification of Excavated Slopes

The purpose of this classification is to provide valuable geological data for the designing of side-slopes and the formulating of operation scheme. Sideslopes can be generally classified as four types (see the accompanying table for details). We can get a systematic understanding of the stability state of the excavated slopes through this stability classification, and determine the ratio of the cut slope and take necessary measures of support and protection on the basis of different stability of side-slope segments, which is favorable for the speed-up of engineering construction, the guarantee of safe use of buildings and the better utilization of slopes in our country.
<table>
<thead>
<tr>
<th>No</th>
<th>Type</th>
<th>Sub-type</th>
<th>Geological and Lithological Features</th>
<th>Weak Surface Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Stable</td>
<td>I&lt;sub&gt;a&lt;/sub&gt; Rocky</td>
<td>Moderately-weakly weathered rocks; joints and fissures mostly closed; and complete rock body</td>
<td>Without weak surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I&lt;sub&gt;b&lt;/sub&gt; Earthy</td>
<td>Thin clay bed. Underlying bed is compact</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>Moderately Stable</td>
<td>II&lt;sub&gt;a&lt;/sub&gt; Rocky</td>
<td>Strongly-moderately weathered rocks. Beds are folded by compression. Joints and fissures are moderately developed</td>
<td>Most joints and fissures are not filled. Dip angle of the line of crosscutting of fissure combinations is less than 10°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>II&lt;sub&gt;b&lt;/sub&gt; Rocky and Earthy</td>
<td>Composed of several beds. Contact surfaces between beds are gentle</td>
<td>Without weak surface</td>
</tr>
<tr>
<td>III</td>
<td>Poorly Stable</td>
<td>III&lt;sub&gt;b&lt;/sub&gt; Earthy</td>
<td>Thick clay, relatively abundant fissures. Top of the underlying bed is steep</td>
<td>Mainly smooth fissure surface and weathered-zone weak surface. $K'&lt;1$, $K''&gt;1$</td>
</tr>
<tr>
<td>IV</td>
<td>Most Poorly Stable</td>
<td>IV&lt;sub&gt;b&lt;/sub&gt; Earthy</td>
<td>Thick clay, abundant fissures. Top of the underlying bed is steep</td>
<td>All three types of weak surfaces are present, among which smooth fissure surface is developed and wet. $K'&lt;1$, $K''&lt;1$</td>
</tr>
<tr>
<td>No</td>
<td>Topographical and geomorphological Features</td>
<td>Influence of Groundwater</td>
<td>Destruction Type of Rock and soil</td>
<td>Engineering Measures</td>
</tr>
<tr>
<td>----</td>
<td>-------------------------------------------</td>
<td>--------------------------</td>
<td>----------------------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>I_1</td>
<td>Gentle slope with an angle of less than 14°</td>
<td>No</td>
<td>Rock body is subject to slumping along fissure surface</td>
<td>Surface protection by laying paving blocks is needed</td>
</tr>
<tr>
<td>I_2</td>
<td>Trapezoidal platform</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II_a</td>
<td>Gentle slope with an angle of less than 14°</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II_b</td>
<td>Trapezoidal platform</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III_b</td>
<td>Moderately steep slope; slumping present</td>
<td>Seeping-out of water found at fissure surface and weak contact surface</td>
<td>Earth body is highly subject to slumping along smooth fissure surface</td>
<td>Support and draining-out of water are needed</td>
</tr>
<tr>
<td>IV_b</td>
<td>Steep slope; abundant slumping present</td>
<td>Flow of seeping-out water found along weak contact surface</td>
<td>Support and draining-out of water are needed. Ratio of cut slope should not be &gt;1:2</td>
<td></td>
</tr>
</tbody>
</table>

K' — Value of calculation along smooth fissure surface.
K'' — Value of calculation along weak contact surface.
ESTIMATION OF GEOLOGICAL FACTORS CAUSING EARTH FLOWS IN MOUNTAINOUS FOLDED AREAS (WITH THE EXAMPLE OF GEORGIA)

L'ÉVALUATION DES FACTEURS GÉOLOGIQUES DE L'ORIGINE DES FLEUVES DE BOUES DANS LES RÉGIONS MONTAGNEUSES (À L'EXEMPLE DE LA RSS DE GÉORGIE)

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ABSTRACT
The present paper deals with the primary geological factors of earth flow formation. These are: composition and intensity of lithification of rocks alimenting earth flows, mode and rate of their weathering, tectonic disturbance, climate and hydrodynamical factor. It was stated that earth flows are confined to certain rock types depending upon climatic conditions. Most affected by earth flows are areas composed by rocks of Early Terrigenous sandy-clayey, volcanogenous sedimentary and coarse molasse rocks high in clay mineral (of hydromica type) content. Each rock complex preserves peculiar crust of weathering the latter determining composition and pattern of earth flow. Subsequently mechanism of earth flow formation within different mudflow basins was established.

ABSTRACT
Il s'agit des facteurs géologiques essentiels de l'origine de fleuves de boues, particulièrement: de la composition et du degré de lithification des roches, du caractère et de la vitesse de leur altération, de leur intrusion tectonique et de leur rapport, de leur extension dans les zones climatériques. On a étudié que de fleuves de boues appartiennent d'une espèce des roches dans les divers conditions climatériques. Les plus répandus de cette espèce sont l'inférieur terrigène-grés argileux, Flischeux, pyroclastique et brut molassique roches dont la composition domine minérale d'argileux d'une espèce d'hydromicaçé. Chaque complexe de ces roches a sa couche particulier de l'altération qui détermine la composition et l'espèce de fleuves de boues. On a découvert la mécanisme de formation de fleuves de boues dans les divers bassins des fleuves.

Territory of the Georgian SSR abounds with earth flow basins which somehow damage national economy of the republic.
In the main earth flows occur within the range of geosynclinal and orogenic formations. Most affected by earth flows are regions composed by rocks of early terrigenous sandy-clayey formation dated back to Early and Middle Jurassic period, as well as
rocks of terrigenous-carbonaceous flysch dated back from Late Jurassic to Cretaceous periods. Then, earth flows are rather frequent within the range of rocks of molassa formation, dated back to Mioc-Pliocene, and volcanogenous-sedimentary formation of Neogene-Quaternary stage, making for overlapped topography (e.g. Kazbegi area). It should be also noted that earth flows are confined to Paleozoic metamorphic-magmatic formation both in the divide area of the Caucasus and in the Dzirula crystalline rock mass. Herein earth flows are rather frequent but incompetent. Earth flows occur in rocks of Silurian-Triassic metamorphic aspidite-shale formation.

In rocks of Baicos, Cretaceous and Paleogene volcanogenous-sedimentary formations and paleogene terrigenous-flysch formations earth flows are somewhat rarer. Rocks of the rest formations are characterized by ever more infrequent occurrence of earth flow basins. Herein modulus of earth flow abundance (N of earth flow basins at the area of 100 square kilometres) is less than 0.5.

In rocks of metamorphic-aspidite-shale, terrigenous-sandy-clayey, terrigenous-carbonated-flysch and volcanogenous-sedimentary earths of earth-flow-alimenting material formation are usually located above 2000-metres' absolute height mark, the latter involving a border line in between woodland and Alpine grassland zones. Earth flow seats are most abounding within subalpine, Alpine and subalpine zones and are located in the crest and near-crest regions of the Central and East Caucasus as well as in the Svaneti Ridge axis. Herein one should especially mention supremacy of physical, namely frost weathering, the latter making for discrete medium. Dimension and shape of crushed material depends upon degree of lithification, mode of occurrence, texture and partially structural peculiarities of rocks along with intensity of tectonic disturbance. Mode and degree of rock weathering determine granule content of earth-flow-forming material and hence type of earth flow.

Rocks of the metamorphic aspidite-shaly formation of the Dizi suite are highly lithified. As for sandstones and hornfelses herein a clumpy zone of weathering is developed, its thickness ranging from 20 to 30 metres. Very often coarse clastic crust of weathering resembles masonry without any filler. In clay shales and phyllitlike rocks one comes across rock debris - en bloc weathering, comprising a cover of 12 to 15 metres' thickness. Therefore weathering bears a selective nature this favouring manifestation of talus, collapse and slope subsidence phenomena. Colluvial masses which have been formed due to disintegration of the above cited rocks are usually composed by coarse fragmental en bloc - rock debris material, dispersion fraction of which is produced due to grinding and splintering of clayey rocks and is represented by hydromica and chlorite.

As for terrigenous sandy-clay formation of the Early and Middle Jurassic stage, herein thin-laminated and foliated structure of the composing it coaly-clay shales (and partly sandstones) along with eminent cleavage provide for extensive splintering of the above rock in the course of weathering. That is why thickness of rock debris belt of weathering is rather considerable ranging from 20 to 25 metres and being even as great as 100 to 150 metres at divide areas. Intensive weathering of coaly-clay shales is favoured by abundance and nearly vertical bedding of organical matter. In the upper part of the vertical layers one comes across a fan-shaped fissuring of separate bands of coaly-clay shales with subsequent rapid scree phenomena. If slope gradients do not exceed 30 to 35° rock debris does not slide down the slope, while greater gradients result in talus and slope subsidence.

Development of rock debris belt of weathering favours formation of thick rubble slopes. The latters are usually composed by
angular, oblong complanate rock debris - gruss material. Herein fine-dispersed product of weathering is formed as a result of splitting and grinding of rocks and resembles source rocks of sericite-hydroramic type. Sand partings undergo en bloc weathering and produce rubbly-pebbled material, the amount of the latter being not so great.

Rock of the terrigenous-carbonaceous flysch formation is characterized by alternate arrangement of badly lithified calcareous-arenaceous rocks and weakly lithified marls and argillites, this favouring development of a thick crust (from 15 to 20 metres) of selective weathering. Sandstones and limestones abound with en bloc weathering zomule, while marls and argillites make for rock debris and en bloc belt. The latter provides for downfall and talus phenomena and in case of selective weathering even landslides may occur. In the flysch deposits just on the slopes of river valleys deep landslides are rather frequent. Crust of the flysch rock weathering provides both coarse rubbly-pebbly and fine fragmental waste represented by hydromica and sometimes hydromica-montmorillonite with a considerable admixture of dog-tooth spar.

As for coarse and combined molasses herein mudflow hearths are located somewhat lower as compared with the foregoing rocks, just at the absolute height of 1700 to 1800 metres. That is in the border line in between the subalpine grasslands and woodlands with prevailing physical weathering. Temperature variation and atmospheric precipitation result in decrease of the intensity of cementation of the poorly lithified molasse rocks along with deconsolidation and loosening as well as soaking of the veneer of the crust (sandstones and gritstones with argillaceous cement get soaked in 40 minutes while conglomerates need from 15 to 20 minutes). Thus, all these provide for rapid talus phenomena, washout and sometimes even collapses. Crust of weathering is not thick ranging from 2 to 3 metres with development of gruss and poorly defined en bloc zomules.

Presence of deep erosion downcuttings (with steep /60 to 75°/ erosional slopes reaching from 50 to 200 metres in height) should be regarded as rather favourable for bad land areas. Herein ablation of fragmental product and talus are rather frequent this being mostly due to boulders with diameters ranging from 0.5 to 0.8 m, though usually predominates pebbly material with diameter of 100 to 200 mm.

Coarse waste composing rocks of the molasse formation includes rounded fragments of limestone, quartz-micaeous sandstone, marls, argillites, effusive and sometimes intrusive rocks. Filler is represented by gravelly-sandy-clayey material, finely dispersed phase involves hydromica, sometimes with admixture of montmorillonite and fine crystalline calcite.

Volcanogenous-sedimentary formations of the Neogene-Quaternary complex is composed by unevenly lithified rocks. Effusive sheets are rather strong with a pronounced jointing, whereas explosives are loose and readily get disintegrated resulting in thick hillside waste with heterogeneric granulometric composition.

Herein we present a brief account both of the regions which suffer from talu most of all and the mechanism of mudflow formation.

First of all we should mention range of Early Terrigenous badly metamorphosed clay shales combined with complex paleo- and recent structural environment. This area comprises the southward near-crest section of the Great Caucasus just from the Inguri basin up to the Azerbaijan SSR. Climate of the area is temperately humid with cold winter and short summer.

The rocks are mainly represented by highly metamorphosed and modified by tectonic processes coaly-clay shales. The latter may be subdivided to relatively compact black, partly keratinized clay shales (with thickness ranging from 300 to 400 metres) and subjacent coaly-clay shales. The latter become easily washed out and unconsolidated with
intensive development of close-joint cleavage. Surface of these rocks has not been stripped.

Depth of erosional dissection is as large as 0.7 to 1.0 km, its density ranging from 2.5 to 5 km/square km. Slopes are steep with benches at their near-brow sections. Herein denudation of the mudflow hearths takes place.

Vast accumulation of unconsolidated material alimenting mudflows has been stated to be favored by below factors:

a) environment - involving variable lithological composition of rocks composing high and steep slopes, tectogeneic and gravitational tension and presence of primary discontinuous medium with subsequent softening due to weathering processes, Alpine climatic zone with insignificant precipitation;

b) processes including abounding hearths of Early Pleistocene and recent talus, collapses and tremendous gravitational and tectogene-gravitational landslides. All the above processes provide for thick sheet of colluvial unconsolidated accumulations to be deposited upon comparatively gentle slopes of the Durudji and other rivers' basin.

The 'standstill' stage of the unconsolidated deposits ranging from 10 to 15 years is succeeded by alteration of their composition to form hydrophilic minerals of hydromica-montmorillonite series. Then, structure and texture of loose accumulations become rather intricate along the section. As for snow cover it happens to be buried in between yearly unconsolidated accumulation cycles to result in banded structure. 'Burried' snow bands contribute to further inundation of the embedding loose accumulations. Herein mud- and earth flows are formed both after heavy showers and in fine winter, this being due to structure and texture peculiarities of the mudflow-forming material, alteration of the composition of accumulated clay partings, presence of concentrated (mostly buried) seats of inundation.

This accumulation along with the above peculiarities involves a main factor of earth flow formation. Undoubtedly functional interrelation between the thickness of unconsolidated accumulations and angle of gradient of the underlying original slope may provide prognosis of the earth flow, but, of course, it is a matter of further thorough investigations.

Genetical characters of the distinctive mudflow basin of the Durudji allowed differentiation of 5 types of slopes. These are:

a) tectogenous - landslide slopes with thick potential landslides;

b) tectogenous-erosional slopes with continuous seats of mudflows with M_e.f. as high as 1;

c) tectogenous-erosional slopes with separate thick seats of mudflows (M_e.f. ranging from 0.5 to 0.1);

d) tectogenous-erosional slopes with rare seats of mudflows (M_e.f. as small as 0.1);

e) proluvial accumulative slopes at debris cones.

Accumulation of the unconsolidated material forming mudflows proceeds recurrently in the upper part of the basin just at the bottom of erosional downcuttings. Thus, estimation of mudflow seats as well as prognosis of their occurrence should be expeditiously accomplished according to genetical classification of the slopes (drawn up by G.Zolotarev, I.Popov, G.Areshidze et al) with indices of M_e.f. affected by recent exogenetic processes producing unconsolidated mudflow forming accumulations.

The range of poly lithogenic rocks comprises a vast area just from the divide and the Great Caucasus Southern Slope with the Aragvi and the Terek basins (temperate humid subtropical climate) up to Dusheti and Kazbegi regions with perennial snow and glaciers (Alpine climate). The area is characterized by rather involved geological structure. Rocks are dated back from Early Paleozoic up to Pleistocene stages and include their derivative cover of unconsolidated and loosely consolidated soils. Most frequent are granitoids, crystalline shales, gneisses, Paleozoic quartzites and
metamorphic clay shales, Mesozoic terrigenous-carbonaceous flysch, as well as the whole range of Quaternary Effusive rock related to andesitic basaltic lava, volcanic tuff and ash accumulations.

All the above rocks are badly weathered, thickness of the crust of selective weathering varying from 5 to 30 metres. Metamorphic rocks are characterized by cleavage providing surfaces of weakness within the mass, while magmatic rocks are notable for dense jointing.

Cover unconsolidated and loosely consolidated deposits involve eluvial, glacial-morainic, talus, collapse, inrush, landslide, fluvioglacial, alluvial, proluvial and Anthropogenic formations. Thickness of the above genetic species of loose accumulations varies from several to hundred metres. Sometimes the ridge make an impression of being plunged into unconsolidated formations. This is especially true for the Kazbegi-Kelskoye Quaternary volcanogenous upland, composed by lava flows alternating with readily weathered volcanogenous explosives. Herein intensive frost weathering easily disintegrates polylithogenic rocks and slopes having no glacial or snow cover. As a result appear various positive and negative topographic forms, which favour formation and development of seats of significant collapses, talus and landslides.

Deglaciation and retreat of perennial snow border leave behind flattened beds of spurs with steep slopes. Those beds are covered by unconsolidated glacial-morainic deposits, while the slopes involve hearths of gravitational processes. Unconsolidated deposits produced by the above processes are superimposed upon glacial-morainic and fluvioglacial ones forming thick seats of unconsolidated accumulations. These seats provide mudflows as soon as the 'burried' snow partings begin to melt.

In effusive rocks usually prevail loose coarse clastic accumulations with low percentage (about 5%) of powdered clay particles, that is why herein predominate earthflows of water-and-

stone composition.

Clay shales and slates produce earthflows of geotectonic type, with the provision of proper hydrodynamical factor, the latter soon after shift initiation are converted into turbulent flow of mud-and-stone composition.

Unlike the rest regions herein along with other genetic'slope types one should recognize englacial-morainic slope varieties. The latter are covered by loose mudflow-forming accumulations of poly-genetical type, produced as a result of glacial, fluvioglacial and slope gravitational processes.

To judge by intensity of mudflows herein we may distinguish extinct or stabilized, active and potential mudflow basins. Extinct basins involve those ones wherein all mudflows have been already arrested. In such basins one comes across an environment rather unfavourable for intensive accumulation of loose material, the slopes being flattened and covered by turf. It is noteworthy that in some spots fastening of unconsolidated material is due to traverse being deposited by mineral springs in excess (the Truso canyon, Georgian Military Road, etc.).

Active mudflow basins comprise earthflows which proceed just nowadays. These flows may be subdivided to rather active (with annual repetition), average active (repeated in every 2 to 5 years) and just active (with a recurrence in 10 to 15 years).

The term 'potential basins' stands for areas abounding with loose accumulations. The parameters suffer annual increase, but so far no mudflow-favouring factors have been observed, though they are to make their appearance beyond all shadow of doubt.

The region composed by rocks of molasse formation comprises troughs, situated in between the geosyncline and subplatform zones. This area is noted for presence of brachyfolded structures and faults. Herein mudflows have been investigated just on the Tsiivi-Gombori Ridge, which was proposed as a key area for mudflow manifestation in molasse deposits within
duplication overlap. Set forth below are the most significant mudflow basins: the Telavis-Rike, the Vardibus-Khevi, the Kisis-Khevi, etc.

The basic exogenous mudflow-forming processes involve talus and minor landslides. The former are confined to "bad lands", and the latter - to common slopes. In the zone of stratigraphic contact between Pliocene and flysch rocks one comes across bad landslides. All the above exogenous processes proceed within the crust of rock weathering. Herein mudflows are usually provided for just by heavy rain or rush of backwaters, impounded by tributary debris cones and landslides.

Detrital, well rounded Miocene rocks, low in clay and powdered particles, form earthflows. This may be observed in the Telavis-Rike basin and in the adjacent streams. Debris cones, having been formed and grown in the junction parts, afterwards in the major river outfalls pile up like earth dams. Provided the water level is normal the latter do not hamper discharge of the river waters on account of the latter being engulfed by the subsurface drainage. As for the surface run-off it accounts for only 3 to 5 per cent by the normal river run-off. At flooding of this region, caused by down-pours and snow melting, debris cones formed near junctions function as cfferdams blocking flood waters. Under definite conditions water breaks these cfferdams and acquires washing-out and destroying property. For no other reason than the debris cones being formed at every junction these cfferdams are not broken at a time this providing for pulsating regime of earthflows. The same is true for those landslides which for the present are in potential state.

Thus it may be inferred that the area of the Georglian SSR is characterized by abundance of various earth flow basins. According to the intensity the earthflows may be subdivided to stabilized, active and potential ones. Composition of earth flows allows distinguishing of mudflows, mud- and stone and water-and-stone flows, this being directly correlated with composition and degree of lithification of mudflow-alimenting rocks along with the value of hydrodynamical factor. To judge by the structure of earthflows we may recognize structural, turbulent and combined mudflows. As for the earth flow basins it is rather reasonable to classify them just by the amount of loose accumulations potentially involved in mudflows. Set forth below is such classification of earth flow basins:

very small - those involving about 4000 cubic metres of accumulations;
small - those involving from 1000 to 10 000 cubic metres of accumulations;
average - those involving from 10 000 to 100 000 cubic metres of accumulations;
large - those involving from 100 000 to 1 000 000 cubic metres of accumulations; and
very large - those involving above 1 000 000 cubic metres of accumulations.

As a peculiarity of mudflow accumulations present within mudflow basin one should note clay minerals with poor hydrophyllic features. Most frequent is hydromica (with liquid limit ranging from 30 to 40, and limit of plasticity varying from 13 to 18, number of plasticity reaching 17). Provided montmorillonite is present its hydrophilic properties are decreased by admixture of fine crystallized spar.

Herein set forth below are some measures regarded as holding promise at predicting earthflows. This is statement of the degree and rate of weathering of loose accumulations, this favouring determination of the depth of illuvial process, making for deposition of clay-colloidal particles upon slopes. Then, retention of 'burried' snow partings usually involving surfaces of weakness, the latter presenting potential sliding surfaces for loose accumulations commencing structural mudflows.
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MASS MOVEMENTS IN THE GREEK TERRITORY: A CRITICAL FACTOR FOR ENVIRONMENTAL EVALUATION AND DEVELOPMENT

MOUVEMENT DE GLISSEMENT DE TERRE DANS LE DOMAINE HELLENIC: FACTEUR CRITIQUE POUR L'ÉVALUATION ET LE DÉVELOPPEMENT D'ENVIRONNEMENT

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ABSTRACT

The Greek territory is characterized by a complicate geological structure and intense tectonic fatigue of the formations, as a result of its geotectonic evolution. These conditions dictate the frequent manifestation of landslide phenomena, the extent and frequency of which vary in different places, but they appear with a distinct intensity in Western and Central Greece.

The geomechanical characteristics of the various formations are described, the geological characters of landslides are studied and the problems arising for the technical works, urban-regional planning and development and the national economy, due to these engineering geological conditions prevailing, are pointed out.

INTRODUCTION

The present configuration of the Greek territory is due to its geotectonic evolution, through orogenic movements accompanied with intense folding, the older of which have their share, while, the alpine orogeny, has put the final seal to
it. Particularly the last phases of this orogeny, with the intense fracturing and the vertical quaternary movements, have resulted in a multifarious geomorphological picture.

These evolutionary tectonic processes explain the complex geological structure and the peculiar engineering geological conditions and the recent endogenic processes which contribute for their formation.

The above geotectonic evolution of the Greek territory dictates its differentiation in unique units, geotectonic zones, which are characterized by special geological features (Maratos 1974, Mariolakos 1977).

These conditions reflect the frequency, intensity and distribution of the geological catastrophic phenomena, such as landslides and earthquakes, which consist, as far as the geological environment is concerned, the basic determinants for the planning and development of the country. More generally, based on these factors and the broad geological features, we could divide the country in three parts, Western, Central and Eastern (Fig. 1).

The climatic conditions which prevail today in the Greek area do not allow a wide activation of masses and the frequent manifestation of serious landslide phenomena. So, most of the landslides are located in zones activated in the past (unstable geological areas) and are mainly triggered by human activities, the seismic impact and sometimes, by the amount and intensity of atmospheric precipitations.

Moreover, the problem with landslides is becoming very acute, if we take into consideration that most of the 9000 villages are mountainous, founded on steep slopes and formations prone to sliding, conditions which tend to deteriorate with human interference and the high seismic activity (Koukis 1980). As a result of these conditions, the removal has taken place in the last two decades, of over 500 villages from unstable foundation conditions to other geologically stable places. An effort for continuous improvement of these conditions for the rest of the villages has been made, by taking protective measures against sliding, erosion, subsidence, undermining etc.

**GEOREGIONAL SETTING AND TECTONICS**

In Western Greece the strong morphological relief is characterized by superposed elongated mountain ranges and valleys, with a N-S direction. The formations participating in the struct-

![Fig. 1 Indicative map of the broad division of the country on engineering geological terms.](image)

Carte générale indicative de Grèce, basée sur de critères de géologie de l'ingénieur.
ture and consisting the geological base-
ment are limestones, dolomites and do-
omitic limestones, schists and cherts of
Triassic—Jurassic age, Cretaceous lime-
stones with radiolarites or chert nodules
and the flysch. The pre-alpine basement
appears only at places in the southern part, being formed by Paleozoic semi-meta-
morphic rocks (schists, phyllites, sipo-
lines).

However, in the frame of each geotec-
tonic zone the above stratigraphic series
is represented partially or as whole, and
the various formations appear with dif-
erent characters but in general they are
discerned by intense and multifarious
fracturing, the calcareous rocks being
also karstic. Flysch is mainly consisted
of marls, sandstones, siltstones and con-
glomerates to a limited extent. The post-
alpine sediments are represented by the
Neogene and Quaternary deposits.

The tectonics is characterized by
synclinal and anticlinal folds of exten-
sive length and with an axis directed N-S
and intersected by two main fault systems,
of NW-SE and NE-SW directions. In the
southern part predominate faults and dis-
placements are in N-S and E-W directions,
while the thrusts are generally directed
N-S. This tectonic picture is completed
by the post-alpine gravity faults.

In Central Greece the
strong mountainous relief prevails. The
pre-alpine basement, represented by schists,
appears at a limited extent, while the al-
pine sediments are composed, in the lower
Triassic-Jurassic horizons, by limestones,
dolomites, cherts alternating or not with
argillaceous schists. The Cretaceous sedi-
ments consist of limestones, marls, sand-
stones, breccia, thin platy limestones. The
deposition of flysch, with alternating
layers of sandstones, siltstones and con-
glomerates is following as well as the
post-alpine deposits.

The tectonics is represented by in-
ternal upthrusts, with a folding axis N-S,
as well as by overthrusting movements from
E to W, while the faults belong to two sys-
tems, with main directions NW-SE and NE-SW
with the predominance of the first. This
tectonic fatigue is later completed with
the effect of stresses applied due to
younger vertical movements of the Plio-
Pleistocene, especially in the coastal
areas.

In Eastern Greece, which
is of a moutainous-semimountainous relief,
the most characteristic feature is the
presence of the pre-alpine basement con-
sisting of the metamorphic system (mar-
bles, gneisses, schists) as well as semi-
metamorphic formations. The alpine sedi-
mentation is distinguished by a variety
of sediments, such as limestones, dolomi-
tes, schist-sandstones, schist-cherts and
ophiolites of Triassic-Jurassic age, lime-
stones mainly of the Cretaceous and fin-
ally the formations of flysch, the mo-
lassic sediments and the Neogene and Qua-
ternary deposits.

The geotectonic evolution of this
part dictates in the formations of the
solid pre-alpine basement mixed tectonics,
or folding in wide anticlines and syncli-
nes, fractured by big faults, while the
alpine formations appear much more folded
and intensively fractured with upthrusts
at places.

The intense neotectonic activity, in
general, with the continuous geodynamic
evolution of the various tectonic grabens,
must be considered of special interest
for the Greek area. This results in the activation of faulting zones and the manifestation of strong earthquakes, a fact which intensifies the already unstable geological conditions and particularly on the steep coastal slopes of these grabens. A recent example is the earthquakes of February-March 1981 in the Eastern Corinthian Gulf, with catastrophic consequences in three districts among which was Athens (Andronopoulos et al. 1981).

LANDSLIDE PHENOMENA AND THEIR GEOLOGICAL CHARACTERS

The above geological structure justifies, basically, the non-uniform distribution of landslides in the Greek territory mainly manifested in the Western and Central parts (Fig. 2). The lithological setting, composition and tectonics therefore are mainly responsible for the existence of sliding stresses and are connected with some other factors, such as human activities, earthquakes etc., for the manifestation of the phenomena. On the contrary in Eastern Greece the limited activity of the tectonic forces during the alpine cycle, the solid pre-alpine basement and the low rainfall are the factors which inhibit the occurrence of landslides.

In Western and Central Greece the landslides occur especially in the flysch, the neogene, the

Fig. 2

Synoptic map of the distribution of landslides in the Greek territory (compiled by Ch. Angelidis).

Carte synoptique de la répartition de glissements de terre dans le territoire grec (rédigée par Ch. Angelidis).
formations consisting the upthrusts and
the loose quaternary deposits (Figs 3, 4, 5). Of the landslides recorded in these parts
the higher percentage (60-70%) is referred

Fig. 3 Slump slides and rockfalls in neogene deposits, particularly favoured along the
contact of the two phases (conglomerates and clay-silts).
Glissements de rotation et chutes rocheuses dans les dépôts du Néogène particu-
lièrement favorisées le long du contact de deux phases (conglomerats-argiles li-
monieux).

Fig. 4 Landslides of considerable vertical and lateral displacement, in the loose Qua-
ternary deposits, lying on the Neogene.
Glissements de déplacement considérable de sens horizontal et vertical dans les
dépôts quaternaires superposés sur le Néogène.
to the flysch formations. The phenomena are of small extent and in the form of sheet (slab) slides, earth-flows and rock falls. In the Neogene rock slides, falls and slumps occur depending on the existing phase. In the formations consisting the upthrust blocks rock slides and falls are quite frequent. The movements in the Paleozoic and Mesozoic formations are more rare and mainly referred as slides and falls of rocks. These sometimes become serious, creating such problems the confrontation of which is difficult and very costly.

In Eastern Greece the landslides are rare, mainly in the formations of flysch, the Neogene and the quaternary deposits, while in the rock formations they are of considerably small number, but sometimes become quite serious (Figs 6, 7). More specifically of the landslides recorded in this part a percentage of 80-85 is located in the Neogene sediments.

GEOMECHANICAL CHARACTERISTICS OF THE MAIN LITHOLOGICAL UNITS

After the consideration of the geological conditions in the Greek territory, and the different divisions, a general appreciation of the broad geomechanical characters could be useful for the main

Fig. 5

Slump slides manifested in the embankment of a road, due to subsidence and sliding of the flysch weathering mantle.

Glissements de rotation manifestés dans la digue routière, due à la subsidence et au glissement de manteau d'altération du flysch.
Fig. 6 Volcanic rocks, with intense and multifarious fracturing, prone to sliding and falling. In populated areas the risk is imminent and the protection measures to be taken are difficult and costly but inevitable.
Roches volcaniques intesément fracturées portés aux glissements et aux écroulements. Dans les régions urbaines le danger est immédiat et les mesures de sécurité sont indispensables malgré ses difficultés.

Fig. 7
National road of Athens to Yugoslavia. Rock slides and falls in the highly fractured crystalline limestones, and mainly along discontinuity planes with steep slope, occur very often.
Autoroute nationale Athènes-Yugoslavie, glissements et chutes des roches dans les calcaires cristallins intesément fracturés, principalement le long de surfaces de discontinuité qui sont presque parallèles à la pente.
units composing it which serves as a general guide, from the geotechnical point of view, for the regional planning and development (Koukis 1980). On the basis of these units the engineering geological map of the country was also compiled (Fig. 8).

QUATERNARY DEPOSITS

They occupy a considerable part and are mainly consisted of alluvial deposits, weathering mantle and scree, cemented or not, with an obvious predominance of the first, which are generally loose, of variable composition and usually mixed phases, with frequent horizontal and vertical alternation and high water table. They create serious disturbances in the various technical works.

NEogene SEDiments

They are also of considerable extent.

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Fig. 8

Engineering geological map of Greece, based mainly on litho-stratigraphical criteria.

Carte de géologie de l'ingénieur de Grèce basée sur des critères lithostratigraphiques.
and characterized by the low density and strength and this depends on the degree of saturation and the way of loading. The seismic risk in these and the Quaternary deposits depends on their thickness, composition and physical—mechanical properties. The velocity of the Vp (longitudinal) seismic waves ranges from 300 to 2150 m/sec, the lower values corresponding to the recent alluvial deposits.

FLYSC

It favours the manifestation of landslides, due to its composition, the intense tectonic fatigue, the morphology and the climatic conditions. The Vp values range from 1400-4700 m/sec, while the construction of serious technical works in these formations, such as dams and highways, requires a fair knowledge of their special characters and behaviour.

LIMESTONES

As far as seismicity is concerned, they are considered, as of good static and dynamic stability and satisfactory geomechanical characteristics. So, intact specimens tested in the laboratory have given values for the unconfined compression strength between 500-1000 Kg/cm². The modulus of elasticity is over 1 x 10⁶ Kg/cm² and Poisson's ratio 0.25 – 0.30. But in places where they are highly fractured and form steep slopes they might create unstable conditions and especially if some other factors contribute, such as seismicity, compressional—extensional stresses developed with ice, weathering—erosional processes, unfavourable orientation of the planes of weakness (fractures, bedding planes), undermining of the slopes etc.

SCHIST—CHERT FORMATIONS

These, though they show satisfactory geomechanical features under certain conditions they create such an instability as above mentioned (in limestones).

METAMORPHIC ROCKS

They are distinguished by high density, unconfined compression strength usually over 1000 Kg/cm², and tensile strength which is as well high. The modulus of elasticity is usually ranging between 3 x 10⁵—5 x 10⁵ Kg/cm² and the residual deformations deriving, in these rocks, by dynamic loading due to earthquakes (Vp= 4600-6250 m/sec) are not considerable.

IGNEOUS ROCKS

Their geomechanical behaviour is very satisfactory, since they are of high density, Vp 4600—8600 m/sec, and have the same strength values as the metamorphic rocks and in some cases higher; Meanwhile in these rocks and the metamorphic as well, the phenomena of rock slides and falls occur in some places quite often.

THE IMPORTANCE AND CONSEQUENCES OF THE ENGINEERING GEOLOGICAL CONDITIONS IN PLANNING AND DEVELOPMENT

On the basis of the above analysis, relative to the geological structure, the tectonic evolution and the engineering geological conditions formed through these, with particular emphasis to the landslide phenomena, we would make the following remarks, as far as the planning and development of the country is concerned.

In Western Greece the geological structure and lithological composition dictate unstable conditions, especially in the areas occupied by loose
Quaternary deposits, semicohesive neogene sediments and flysch. The phenomena, of differential settlement, subsidence, hydrostatic pressure and sliding are very common in these formations. Here it must be pointed out that most of the villages are founded on the recent loose deposits (mainly scree), due to the existence of water, so they suffer because of problems of sliding and mainly where these deposits are of small thickness and subjected to seismic loading.

Slope stability problems occur also in the upthrust hard and of small thickness rocks on the more plastic flysch formations. Finally, rockfalls in this part are favoured. So, for the various technical works such as roads, dams etc. this peculiar geological conditions and the geometry of the slopes must be taken seriously into account.

The suitable protective measures, in most of the cases, consisted of a well designed drainage, after the necessary hydrogeological study, planting, restraint of the erosional agents, away from steep slopes and areas with probable rockfalls and obvious neotectonic faults, foundation on the solid basement and not on the weathering mantle or the loose scree. Especially for the foundation of serious technical works, the geotectonic evolution and structure of the greater area, with emphasis on neotectonics and seismic risk, must not be disregarded.

In Central Greece the problems are of the same nature as in the western part, but occur with higher intensity due to the strong relief and the tectonic history. Generally the lithological composition, the multifolded structure, the intense morphology and the accumulation of stresses in the rocks render wide areas problematic and create serious problems in the technical works.

It is even emphasized that this part is mainly connected with the Corinthian gulf graben, which in under geodynamic evolution and represents the epicenter of many strong earthquakes. Especially sensitive are considered the formations of flysch, the Upper Cretaceous transition to the flysch series, the Neogene and the loose Quaternary deposits (particularly scree which are of great extent along the thrust lines). Landslide movements of great extent have occurred in this part recently and mainly along the steep coastal slopes of the Corinthian gulf graben.

The above conditions must be taken into account in planning and construction of the various projects. Especially the installation of nuclear power stations in this area must be excluded, mainly because of high seismicity and no satisfactory respond of the basement formations to the seismic impact.

The protective measures are the same as in Western Greece, with special emphasis on the aseismic design. Also the probable subsidence of the coasts and even in considerable extent after strong earthquakes must not be disregarded.

In Eastern Greece the geotechnical conditions are considered much better, because, in its greater extent, it is occupied by compact and cohesive rocks of great thickness and with uniform distribution in their physico-mechanical characters.

Landslide phenomena are as a rule, observed in the loose formations such as Neogene, flysch and Quaternary deposits and in lower frequency in the rock forma-
tions, in which when they occur cause serious problems.

Special attention must be paid, during the planning of technical works, on the steep slopes, the orientation of planes of weakness, the erosional-weathering processes, the subsidence—sliding of the loose deposits and the weathering mantle and the hydrogeological conditions. The design of the necessary drainage, the avoidance of unfavourable tectonic planes, the study of the respond of the formations and their probable deformation in the seismic loading, are among the measures which have to be taken and the parameters to be investigated. This is emphasized because many villages or towns and important technical works are founded at the base of steep rock slopes or along obvious neotectonic faults.

CONCLUSIONS

Greece is a country of peculiar geological and tectonic structure, which implies unfavourable engineering geological conditions for large areas and especially in the zones where the manifestation of landslides is favoured or the response of the formations to a probable seismic impact is not satisfactory.

The higher percentage of landslides is recorded in Cretaceous to Quaternary sediments in the Western-Central parts of the country, this being due to the geological conditions existing, the neotectonic activity, the morphology and the rainfall. On the contrary, in Eastern Greece the solid and with uniform geomechanical properties pre-alpine basement and the rainfalls of much lower intensity dictate more favourable stability conditions.

So, the various parameters of the geological environment must consist of basic determinants for the right planning and development, at urban and regional scale. Among these parameters the landslide phenomena, the neotectonic fracturing and the seismicity are particularly emphasized, within the framework of the assessment of the foundations conditions at various populated or considered for development areas and of important technical works.

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THE METHOD OF PROSPECTING AND ANALYSIS ON COMPLEX LANDSLIDE REGIONS AND GROUPS

LA METHODE DE PROSPECTION ET D'ANALYSE SUR DES REGIONS COMPLEXES D'ÉBOULEMENT DE TERRE ET DES GROUPES

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ABSTRACT

This paper has been written based on the experience gained over many years in landslide prevention and control. The authors have developed a series of methods for surveying and analysing the complex landslide regions and for classification and have outlined the principles governing these methods. These include different conditions of geomorphology, structural aspects, geological surveying and investigation and observation of landslide indications to find out relations between the rock mass structure and the boundary of the landslide mass, the factors acting on the landslide, and the movement of the landslide groups etc.

We had come across or examined many large scale complex landslide regions and groups in the course of advising on their prevention and control. There were 1-3 landslides per kilometer in a landslide region which was along the railway line or one side of a river. They extended from several kilometers to several tens of kilometers. The area of a landslide group often reached 0.5-5 km², in which, there were many big and small landslides developed at different levels. Landslide regions and groups are all produced under certain geological conditions. We could usually find many more landslides on one side of a river, but nothing or rarely any on its opposite side, such as, the accumulative soil slides in Xipo, Tanjiazhuang, Baishuijiang, Dingjiaha, Taba of Bao-Cheng Railway, and k163, k602 of Ying-Xia Railway; the large-scale loess slides at k1357, k1358, Wolongshi, Doujital and Caijiapo of Long-Hai Railway, and the districts of Lantian in Shaanxi Province; and a great deal of clayey soil slides laid out in the red coloured basin sections where the Cheng-Kun, Jiáo-zhi, Yang-An, Xiang-Yu Railways had been passed through. Besides, there were the landslides in Xigeda stratum along Cheng-Kun Railway, and the landslides in the Tertiary system stratum (or Q₃) in Hongya and Nanwang districts of Tai-Jiao Railway and the big type of the Quaternary system semidiagenetic sand-stone and mud stone slides along the Lanyangxia Valley of the Yellow River, the broken stone slides in Putaoyuan of Long-Hai Railway, Niujingping, Zaiye of Tai-Jiao Railway, and Naituo of Cheng-Kun Railway, etc. The consequent and insequent landslide of complete rock are almost distributed along the railway line of all mountainous regions. Every kind of landslides was measured to ascertain its own properties because of the difference of its lithological characters, texture and structural conditions.

* The French version of the abstract appears at the end of the paper.
How to prospect fast and distinguish exactly the common geological conditions, properties, stabilities, and develop trends of various slides in every landslide region or group is the key for preventing and controlling landslides. Being based on the texture and structural conditions of rock-mass, we utilized in a composite form the theory and method of geological mechanics, engineering geology, rock and soil mechanics in the analysis of the complex landslide regions and groups, and aimed at the practical solutions.

The steps of this method and the principle by which it is assessed are as follows:

1. From the geomorphic form, decide first the structural pattern of mountain-mass and the relation between the structural pattern and the strip or lump of landslide.

Geomorphic form is the result of the natural forces acting on the lithological Character the textural and structural aspects of the mountain-mass for a long time. Hence, from the geomorphic form we could decide first the structural pattern of the mountain-mass, the properties of the textural form and lithological character distribution of every rock-mass. The gravity fault produced by the stress slack in the tectonic movement period may be compared to the gravity deformation produced by the downward cutting of a valley in present day on the view of mechanism. Therefore, starting from geomorphic form, if we understand fully the gravity structural pattern in the tectonic movement period, we should decide the model of gravity deformation produced in the valley side during its growth or development.

(1) Observe the mountain features. From the geomorphic form of the mountain-mass of landslide regions or groups and near them, find out at first the tectonic order it might possibly receive, the properties of every acting force and the direction of the main stress, etc. For example, in the cuesta shape of the mountain chains trending straight but parallelly arranged, if their ridges showed continual linear shape and natural slope surface showed a plane shape, they were often formed by the monodirectional tectonic compressive stress; if on the other hand, the linear ridge consists of a chain of arches, it received possibly the action of different directional or monodirectional tectonic compressive stress twice; if the mountain chains were arranged in wild goose shape, the trend of their ridges were in S or in reverse S shape, and if the natural slope surface in curved shape, it would be the result of having suffered a couple; and if the whole area of the mountain chain showed turbine shape distribution and the trend of their ridges showed curved shape, it would be the result of having suffered rotational force action.

(2) Observe the strikes, dip angles and their cutting relations of all triangular planes of the mountain slope. The triangular slope surface usually develop along the tensile and shear twisting structural crack planes. Thus, the main stress field can be decided from the mountain feature and from both combined, we could decide first the structural pattern of the mountain-mass.

(3) Observe the mountain pass and the indication of the mountain slope between two passes. The mountain slope passes on the geomorphic form, except a few of them, were the marginal position of the bad rock stratum and the majority of them are almost in the position where the tectonic line passed through. When the two ends of this tectonic line were in the same rock stratum, the strike of the tectonic line must be decided by the linking line of the triangular plane of mountain slope situated between two passes and the back edge line of the collapse may be either straight or not. According to the colour tone of the rock surface exposed by collapse, we could foresee how the passes were formed, either by the tectonic brakes or by the difference in the lithology. The geomorphic shapes provide the main clue for the inspection of the scene of the structural indication.

(4) The step-form mountain slope and gravity structure: when we observe the distribution, strike, occurrence and the distance of displacement of scarp on the step-form mountain slopes and their relations to the occurrence of the rock stratum, we could decide first the structural pattern of the mountain-mass. The mountain slope was often divided into several lumps by a series of scarps paralleled to each other, their strikes were in the same direction or inclined, intersecting the mountain slope strike. These scarps were formed along the tension or shear twisting tectonic plane due to gravity action in the tectonic movement period or the disintegration period. As shown in Fig. 1., the step-form mountain slope with its scarp was usually in
tensile bend fissuring plane, under the mountain-mass, because there were a set of the reverse faults running along the slope direction, a series of the "fault-fall" or the landslides transformed by the "fault-fall", formed easily along the scarps while the river was cutting down. Similarly, the excavation of slope-foot could cause the rock-mass to produce the same character of deformation along this series of fissure planes and along the other fissuring planes parallel to the reversed fault. The range and stability of the rock-mass deformation could be decided from the shape of various deformed bodies that ever existed with the similar conditions in those districts. When the soft rock stratum under the mountain slope were kneaded, it could produce the shape of the similar phenomena as described above. The step-form mountain slope with stratum plane and scarp all inclined outwards, could reflect the shape of the consequent landslides. Its scarp was in tensional fissured plane. Its slipping plane was the fault zone between the rock strata. In the step-form mountain slope formed by the interstratified soft and hard rock beds, there are usually a great deal of colluvial deposit, slope wash and diluvium slides on the terrace formed by soft rock. In general, the shape of slipping beds were determined by denudation surface of soft rock, but the valleys still developed along the tectonic line. The protuberant position of the top surface of slide-mass was usually the hollow point of the slipping beds. Therefore the flat slide-mass in shape generally reflects the slipping bed in plane form. For example, in the mountain slope accumulative soil slide, the water in the slipping band is distributed in band form; and in slide accumulated at the mouth of a gully where the slide-mass is thick at the original natural gully with convex ground surface and concave slipping bed, the water in slipping band is distributed linearly, and the ridge between the two gulleys is the limit line of the slide.

(5) The terrace and the exit of landslide: The river bed undercutting and the lateral stress release caused the rock-mass slackening, deformation and fissure, further more, leading the surface water to percolate down and the ground water to collect towards the erosion base. These happen in the devolutions and landslides. Therefore, in the terrace forming period, the mountain slope deformations grow up as the undercutting is decreased and the lateral erosion strengthened. If the erosion base at that time was the soft, weak and water-resisting layer, the exit of landslide usually coincided with the elevation of the river terrace. In the edge of the multi-benched loess terrace, the papoda-form of multi-stage slides with exits at various benches could be found. From the geomorphology, we could find marshes connected with the standing water areas and a chain of undulating hills along the elevation of the terrace. These happened to be the landscape at the front edge of the landslide region very often.

In the absence of bench sections, the position of the changed point of the multi-stepped terrace of the valley could be utilized as indicating the mark of the terrace.

(6) The valley formed earlier and later to the landslide: To find out whether natural valleys of intersecting slopes, were recent valleys or ancient buried valleys, we must distinguish the relation of formation between the valley and landslide as to which was earlier. The bottoms of valley formed by the eroded plane in various stages were generally the slipping bed of the accumulative soil slide slipped along the valley. The deep cut old valleys were almost the lateral margin of the accumulative soil slide of the mountain slope. The valley was formed before landslides may often, as reflected from their deep and straight shape in geomorphology, and cutting into the integral bedrock. The valley trend ran either along the soft rock or along the fault zone. At the valley bank, we could excavate out the slipping band. When the valleys were formed later than the landslides, they were produced by the surface water which eroded the porous soil and caused the earth shearing failure to the difference in the velocity rate of slipping between two slide-masses or the slide-mass and the stable rock-mass at its both sides. Their reflections in the geomorphology were that they are shallow, and with more accumulation in general were connected with the crescent depression situated at the upper part of the landslide.

2. Analysis of the structural generation, determine the pattern of the mountain-mass and the relation between this pattern and development of the landslide groups:

Although the pattern of the mountain-mass had been foreseen mostly in geomorphology, the same geomorphology had not been formed by the only one type of phenomena. Therefore, we must engage in the
inspection of the scene of the geological structure to check this observed result
and find out the relation between the geomorphology and the structure.

(1) Investigate and survey the indication of structure. On the slide-mass and
the stable rock-mass around it, survey the occurrence of rock strata, the occurrence
of the crack plane, the number of groups, their mechanical properties, and the
scratches and direction of the crack plane; observe the shape, the rate of course of
the crack plane, whether they have or haven't any filling materials and what is
their states; survey actually the relative condition between the crack planes
and the relation of cutting each other. According to the measured result (don't
adopt the statistical Figures) fill these information in the corresponding position
of the engineering geology map with 1/500 or 1/1000 scale, as the basic informations
to decide the structural generation.

(2) Decide the structural generation:

1) To analyse and determine the main tectonic line. Here what is called the first structure is that structure which
moulded the fodder shape of the recent mountain-mass. The structure in later
period (except in respect of the couple action) could have only produced some new
set of crack planes and remoulded the crack plane of the structure of the earlier period. If the mountain slope,
bedding plane and crack plane were shown basically in bend shape, the main stress
of the first structure was a pair of couple action, the press-twisting plane and tensile-twisting plane appeared at the same
time, but the latter had been cut off very often. If the mountain slope, bedding
plane and crack plane were flat and straight, the main stress of the first structure is a single directional stress
which developed only one set of shearing plane. No matter what main stress pro-
duced the first structure, its trend is often in the same direction with the basic
occurrence of the rock stratum. The present occurrence is the result of a lot of
structural actions. Specially in the later period, the rock stratum had under-
gone the turn-twisting remoulding, the occurrence of above and under the fault
and the difference of lithology between the two strata were often in difference,
and with the lean-moving to each other. If the angle between the strike of bedding
plane and crack plane is a constant, the formed set of crack planes belongs to the
same generation, otherwise, it would be in

the another one.

As for the main tectonic line of the later period structure we should do many
constitutional analyses according to the peculiarities of the syncline, anticline,
reversed fault, line of the brachy-axis structure, the relation of cutted crack
plane and the scratch trend on the crack plane, and so on respectively. This conclu-
sion was established during the identical provement is obtained from the practi-
cal investigation.

2) The result of formed set analyses must be identical to the spot surveyed
informations. When the informations of the crack planes of the spot surveyed were
considered to be dependable through checking repeatedly, each set of crack plane
of formed set analyses must be identical to the following items: (a) mechanical
properties; (b) intersecting angle between the occurrence of crack plane and main
stress according to the relation of formed set; (c) later crack plane cutting the
earlier crack plane.

3) Compare with the regional geologi-
cal information; the regional geological
information and the spot surveyed inform-
ation in landsliide region should be in
identical. If not, first of all, check the
investigation informations step by step,
then analyse the regional position of this
site and the reason of producing the partial
deformation and at least, correct the
regional informations. But don't negate
the spot surveyed result with the regional
informations.

(3) Determine the relation between
the structural pattern of the mountain-
mass and the strip or lump of the land-
slide:

When analysing the relation between
the structural pattern of mountain-mass
and the strip or lump of landslide, we
should pay more attention to those well-
penetrated later crack planes, because
their controlling action was more in
strengthening the pattern of slides. At
the same time, we should analyse also
the plane trend which faces the valley.

1) The division of strip or lump of
rock slide is guided by the structural
pattern of rock-mass. The round margin of
the slide is usually developed in attaching
to one or two sets of tectonic planes
with straight or zig form. The slipping
bands generally were a certain soft rock
stratum or fault zone and sometimes a
hidden crack plane. When there were mul-
ple soft bed rock or fault zones in a rock-
mass, it could form multiple bed or multi-stage landslide. The shape of the slipping surface is decided by the shape of tectonic plane too.

The relations between the position of the major-axis or minor-axis of anticline or syncline and the strip or lump of landslide are as follows: the anticline or syncline parallel to the slope trend controls the classification of landslides, the anticlinal axis is the marginal position of the upper and lower landsides and the anticline or syncline is perpendicular to the mountain slope that controls the strips of landslide. The anticlinal axis is always the ridge between the two landslides.

2) The pattern of soil slides a more controlled by the texture of the soil-mass. For example, the distributed position of a certain bad soil stratum and the shape of every erosional surface, specially the shape of the top surface of bedrock, decided the position of slipping band, and the strips or lumps and stages of landslides. But they are closely related to the structural pattern of rock-mass too. For example, the form and properties of certain erosional surfaces were controlled by the structure. The natural valleys being the margin of landslides are formed along the tectonic surface in majority and the colluvial deposit and slope wash, being the slide-mass always developed along the fault or broken zone. Some fault zones also supplied water to promote the landslides.

3. From necessary explorations, tests, observations and experiment find out the constructions, acting factors and moving peculiarities of landslides.

From the geomorphic observation and the tectonic analysis we could basically determine the boundary, scope, the possible position of the slipping band and the basic properties of the landslide. The explorations, tests, observations, etc. are all aimed at: 1) verifying the structural pattern analysis; 2) verify the divided position of strips or lumps of landslides; 3) verifying the judgements of the position of slipping band and the occurrence of the crack plane for forming slip surface, and the shape, lithology and strength of the slipping band; 4) finding out the layer sequence, distributions, supplements and drainage conditions of ground water; 5) Finding out the acting factors and moving state of landslides.

Since, the sequences of process and methods had been introduced in relative books, we won't repeat them here. It is however, worthy to point out the following:

(1) The landslides exploration must be aimed at assessment of the properties and formed mechanism connections with different type of landslides, and with due emphasis on the position, shape, lithology of slipping band and the exploration of the ground water which aids the slipping. The following are some of examples to illustrate: (1) The lake deposited clayey soil at the mound area in the red basin slipped along the layer containing abundant ascanite and with extraordinary strength of expansibility, and dosely related to the infiltrating of water from pond at the mound. Thus in the exploration we should emphatically find out the distributed position of this layer, the condition of ground water and the relations between them and the plane facing the valley, and test the physical and mechanical properties of this layer. 2) A type of loess slide in which the loess covered on the erosional plane of the steep dipping bedrock was involved. Although the ground water is not in abundance, due to the strong intensity of the earthquake in the region, ground water could infiltrate down along the earthquake cracks and saturate a soft band, and then produce the usual developmental slide; another type is that new loess slipping along intersecting plane with the old loess or the loess slipping along the top of Tertiary System clay stratum laid down below where plenty of groundwater exists. In the terrace or the edge of the terrace of the loess, the accumulated loess at the mouth of valley easily formed the landslide because of large amount of ground water in the high terrace which flowed down gradually to form gullies. In the exploration, we should find out carefully the boundary of each stratum, and the conditions of water-bearing bed and relations to each other. 3) Some of the slipping surfaces of accumulative soil slides are soft bedrock plane underneath. Some others are the drifted plane or the intersecting plane between the colluvial deposit and slope wash in different periods. Some more are the eluvium of dinaceous granite, where with the increase of the excavated depth and due to infiltration of ground water, the depth of slipping band would be increased correspondingly and reach to more than 10 meters through eluvium. In such an exploration, we must find out clearly the top surface of bedrock and the shape, lithology of the intermediate layers of the accumulation, the thickness in eluvium, and the
buried condition of ground water. 4) The filling-soil slides may be formed due to bad quality of filling materials or soil, or produce dispersion as the filling materials rapidly weather and get water saturated later; or may be formed because the base was treated unsuitably. The focal point of such exploration is to find out the old ground surface and the shape, lithology of the top surface of bedrock and to test the bad soil lithology. 5) In rock slides, (except the tectonic fault zone mentioned above) the mud stone of New Tertiary System (or Q3), the mud stone, shale and their metamorphic rock of Jurassic, Dyas, Trias System, the mud stone, shale and marlrite of Devonian System, the clays of Anti-Sinian System etc., are the type of strata to form the slipping band easily. As regards slipping mechanism, besides the slipping along the top of soft bed due to the slipping band loosing its strength slipping also occurred due to the lower soft rock getting kneaded under the high compressive action of the huge thick upper bedrock and also due to the lower rock getting eroded and upper part subsiding, thus causing the mountain-mass lose its support. As regards role of ground water in rock slides where the water is need in the vein shaped openings resulting from the tectonic activity, and which is not so easy to decipher through normal exploration, we must make analyses to evaluate the condition governed by the properties of crack planes.

(2) The arrangement of prospect points is based on the strips or lumps of various landslides. A suitable number of drillholes and test pits are arranged in cross-sections of the main axis for each strip or lump. The prospect points are densified at the divisional position of every strip or lump, and the arrangements according to the area is not necessary.

(3) For the test on soil strength of slipping band one must choose different test methods according to the position, composition and state of slipping band, and the different stages of slipping. For example, in the case of landslides which had slipped many times, adopt the shearing method a number of times; for the resisting section which is creeping but has not formed slipping surface yet, adopt the normal test method; again, for the very thin slipping band of rock slides adopt the most common type of direct shearing test, etc. In short, the test methods should try to initiate the actual bearing force and moving state as fully as possible.

(4) The surveying, observing and analyzing of the landslide acting factors: There are many factors acting on the landslides. The same factors acting on different type and scope of landslides, give different results. Therefore, for different types of landslides, we should survey and analyse the change of every factor responsible for the forming and development of the landslide. Only when we find out its main acting factors, we could get the correct result in conformity with the real objective.

Ground water is usually the main acting factor for the landslide. In the exploration, we should divide carefully the position, water level, discharge, quality and temperature of various water-bearing bed. Through test and surveying find out the mutual connections of the various water-bearing bed, the effect on the variation of state and properties of slipping band, and the relation between their variation and slide moving state.

4. Proceed with the comprehensive analysis and evaluate landslide regions and groups, so that for every landslide the basis for the constructions of preventive and control measures can be worked out.

Through the preceding works as shown above, the pattern, texture, main acting factors and main cause of production and moving state of landslide could be found and, we could evaluate the development stage and stability of landslide regions and groups, and pre-estimate the developing tendencies of each one of landslide. About the development stage and peculiarity of landslides and the evaluation methods of the stability of landslide, the authors had expounded all the details in the Reference (1,3). However, they would like to emphasise the following points:

(1) To evaluate the stability of landslide in view of different type and slipping mechanism. For example, is the slipping along the existing soft weak plane caused by losing lateral support for excavation, or is it caused by the soft rock in base getting kneaded and squeezed out under high pressure formed due to loss of lateral load, or is it caused by the strength attenuation of bedrock of the main slipping band.

(2) To evaluate the stability of landslide taking into account the main reason for forming slide and the variation of this factor. For example, the landslide is mainly caused by excavation or ground water increment, or earthquake. What is the marginal extent of these fac-
tors that endanger the slide? Evaluate the influence of these factors on the development of landslide (according to their probability of appearance and variable extent in future).

(3) Whether there exists the resisting slipping section or not and whether the passive resisting force is big or small in the resisted slide section are to the known to evaluate the stability and thrust extent of the landslide.

(4) Analysing the slip sequence of all upper and lower classes of landslides and the mutual effect between them in landslide regions and groups. To evaluate the integral stability of the landslide regions and groups and their developing tendencies according to the developments of every slide in future.

(5) When to use the formula of mechanical calculation to evaluate the slide's stability and calculate the slide's thrust, we should choose different calculated parameter for different section of slipping band in view of concrete condition and moving state of each slide. These parameters could be found out in field with the comparison method of engineering geology.

REFERENCES


ABSTRACT

Cet article a été écrit fondant sur les expériences gagnées pendant des années dans l'empêchement et maîtrise de l'éboulement de terre. Les auteurs ont développé une série de méthodes pour évaluer et analyser des régions complexes d'éboulement de terre et aussi pour la classification, et ont dessiné les principes qui gouvernent ces méthodes. Tout cela comporte des conditions différentes de géomorphologie, des aspects structuraux, des inspections géologiques et l'investigation et l'observation des indications d'éboulement de terre pour trouver des relations entre la structure de masse de roche et la masse de frontière d'éboulement de terre, des facteurs fonctionnant sur l'éboulement de terre et le mouvement des groupes de l'éboulement de terre.
LANDSLIDES ON SLOPES OF DANUBE AND SAVA RIVERS NEAR BELGRADE

GLISEMENTS SUR LES PENTES DES RIVIERES SAVA ET DANUBE, BELGRADE

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ABSTRACT

Lower parts of the terrain of wider Belgrade area - slopes of the Sava and the Danube rivers are built up by Tertiary sediments: clays, marls and sands. Along the rivers occur alluvial sediments. Ruptures bordering the Pannonian depression of pre-Tertiary age are neotectonically active. Because of the erosional activity of rivers, migrating to the depression boundary, it is being intensified. Specific lithological composition and morphological evolution of the terrain represents suitable condition for development of landslides. The Sava and the Danube slopes, having been intensively urbanized, many active landslides are registered. Mass movements could appear not only in surficial, weathered material but also, periodically, in bedrock. Due to its progressive shear failure and great depth, these are difficult to be remedied.

Two characteristic examples are described: landslide "Duboko" on the Sava river and landslide "Karaburma" on the Danube.

ABSTRACT

Les pentes des rivières Sava et Danube aux environs de Belgrade sont composées des sédiments tertiaires: argiles, marnes et sables. Le long de ces deux rivières on trouve des sédiments alluviaux. Les ruptures périphériques, entourant la depression pannonienne d'âge pré-tertiaire, sont renouvelées grâce au phénomène néotectonique. Pour cette raison, l'activité érosive des rivières, qui avancent vers le sud, est devenue plus intensive. La composition lithogénétique et l'évolution morphologique du terrain sont favorables pour le développement des glissements. Sur les pentes des rivières Sava & Danube, faisant objet d'une urbanisation croissante, on a enregistré un grand nombre de glissements. Ces glissements peuvent engendrer non seulement des masses surfacicales désintégrées, mais aussi, périodiquement, le bedrock. A cause des glissements progressifs et des grandes profondeurs, ceux derniers sont très difficiles à être protégés.

Les deux exemples caractéristiques y sont décrits en détail: le glissement "Duboko" sur la rivière Sava et "Karaburma" sur le Danube.

INTRODUCTION

The area of Belgrade, capital city of the Socialist Federal Republic of Yugoslavia is located on the confluence of the two largest Yugoslav rivers Sava and the Danube. This area between the Sava and the Danube rivers marks the end of a hilly relief. Further to the north, on the left side of the rivers Sava and Danube, begins the large Pannonian plain.
Relief of Belgrade surroundings has been formed during Tertiary regression of the Pannonian sea to the north. Large marine terraces formed on that occasion, have been dissected by a subsequent activity of smaller rivers, formed as tributaries of the Sava and Danube rivers.

Essential characteristics of geological composition of wider Belgrade surroundings are also influenced by regression of Pannonian sea. Central and the highest part of the terrain is a solid ground. It has been built up by Mesozoic rocks. Jurassic diabase-chert formation and serpentinites appear to a lesser extent. Sediments of Cretaceous age, limestones, dolomites and sandstones, and Upper Cretaceous flysch sediments are extensively exposed.

Lower parts of the terrain, with slopes toward the Sava and the Danube, are made up of Miopliocene sediments of the Pannonian sea. In general, here two lithological media occur. The first one is represented by a series of limestones, marls and sandstones, and in the second one clays are dominant in addition to the sands and marls.

During Miocene, volcanic activities occurred. In several localities at the surface of the terrain appear andesites and tuffs.

Along the Sava and the Danube as well as in valleys of smaller rivers cut into marine terraces, occur Quaternary alluvial sediments - sands, gravels, muds and sandy clays.

The larger area around Belgrade is cut by large ruptures. These ruptures of pre-Tertiary age show renewed activity even in the youngest periods of geological history. It is characteristic that the marginal ruptures toward Pannonian depression, located along the Sava and the Danube rivers, show a neotectonic activity. Pannonian depression has recently subsided, whereas the hilly area of Belgrade surroundings show relative upheaval. Neotectonic activity controls the morphological evolution of the rivers. Both, the Sava and the Danube, are migrating to the south, towards the depression edges. Their erosive activity on the slopes is in that way strengthened.

Geological characteristics of Belgrade wider surroundings are schematized on the geological map, Fig. 1.

Specific geological and morphological evolution of the terrain created favourable conditions for the formation and development of landslides. Many landslides are located on the slopes, representing right banks of the Sava and the Danube. These slopes are now a days an area of an intensive urbanization. Belgrade city and its surroundings are being developed for construction. Therefore, stability researches of these slopes are of great importance.

While investigating stability of the terrain on the slopes of the Sava and Danube, complex of methods have been applied. Investigation started with remote sensing, geomorphological and Neotectonic analyses on a wider area, and concluded with the detailed investigation of several slopes, borings and laboratory analyses. Results of such investigations of two localities are given: the first one is the landslide "Duboko" (Fig. 1), the Sava right bank, and the second one is "Kara-burma" (Fig. 2) on the Danube right bank.

LANDSLIDE "DUBOKO"

The landslide "Duboko" is located on the right bank of the Sava river, near Umka. It extends over north-west parts of slopes of the local hill in length of about 1000 m (Fig. 2). The height of the terrain, from the top to the river bed, is 150 m. On the slope, whose inclination locally amounts to 25°, a deep and large landslide has been formed.

The slope is dissected by deep streams, in general perpendicular to the river Sava course. The terrain surface in the zone of sliding is very uneven; intersected by scars and gullies - different in dimension and directions, local elevations (hummocks) and depressions are often swampy (Fig. 3).

As it could be seen from the enclosed map, the landslide "Duboko" represents a unique regional block, with three noticeable sub-blocks: A, B and C. Along the well expressed neotectonic rupture (direction NE-SW), this block upheaves with relation to the "Sava block", located more to the north. Long ago strong erosion of the Sava river current has cut the south block, maintaining permanently uneven stress state, in its upper parts. Smaller sub-blocks (A, B, C) differ in dimensions and intensity of developing process. So, the sub-block A has the highest main scar (30 m), the central sub-block B reaches the greatest distance up to the hill (330 m), and the sub-block C is relatively the widest one.

Terrain investigated is built up by
Fig. 2 LANDSLIDE "DUBOKO": 1. STABLE AREA: ELUVIAL AND DELUVIAL CLAY 2. LANDSLIDE BODY. A, B, C: SUBBLOCS 3. LANDSLIDE SCARS 4. GULLY 5. MARSH 6. HIGHWAY IN DESIGN
Fig. 3  LANDSLIDE "DUBOKO"-CROSSECTION: 1. COLLUVIUM: YELLOW AND GRAY CLAY  2. COLLUVIUM: GRAY RARELY YELLOW CLAY AND MARL  3. ELUVIAL AND DELUVIAL CLAY, SAND  4. GRAY OVERCONSOLIDATED CLAY AND MARL  5. SLIDING SURFACES: ① OBSERVED, ② INFERRED  6. JOINTS
Fig. 4  IDENTIFICATION PROPERTIES AND SHEAR STRENGTH OF COLLUVIAL AND IN SITU CLAY AND MARL
coherent Tertiary and Quaternary sediments. Miocene overconsolidated grey clays and marls (cl+mr) are dominant, whereas Pleistocene less consolidated brown sandy clays and sands (cl+s) overlying them, are relatively subordinate (Fig. 3). Surficial parts of these sediments are bedded in layers, gently dipping towards NE, angle of dip being approximately 5°. According to the present results of investigations, rock masses reveal the following properties. Weathering zone is intensively fissured (in fragments of cm and mm dimensions), oxidised (yellow-brownish), porous, liable to the changes of volume - depending on water content and, when it rises, to the plastic stage, it softens (ED-cl). The thickness of the oxidised zone varies: from 20 m in the hinterland to 28 m in the body of disturbed - removed masses. Lower boundary of weathering zone determines the depth of active slides, and it is confirmed by the investigations (Fig.3).

However, in the deeper parts of bedrock, relatively fresh masses of Miocene gray clays and marls - are rarely and unequally intersected into large blocks (m and cm dimensions). The presence of joints, from geological point of view, runs certainly a risk of reactivation, i.e. of acceleration and enlarging of mass movements. It seems that the appearance of these joints should be related primarily with the neotectonic, and to a lesser degree with the shearing and sliding.

Due to the position in the structure of the terrain, and the presence of capillary and super capillary porosity, rock masses behave in two ways. Disturbed rock masses, particularly surficial (Co-cl) are rather permeable and, therefore, form aquifers. Because of heterogeneity and variable permeability of moved masses, groundwater drains slowly, at a high gradient, generally towards the Sava river. Depending on the river level, the gradient locally, from time to time, exceeds 25°. These conditions provoke permanent changes of consistency and mechanical properties of clayey masses (Fig. 4) already being under alterable seepage pressure. In this way the process of instability is temporarily renewed.

Grey clays and marls (cl+mr) forming bedrock, with hygroscopic water and, eventually, a small percent of capillary water, are practically hydrogeological isolator (aquiclude). Up to the investigated depth (15 m), no ground water body was found. After a careful analysis of all the investigation data, it was concluded that the landslide "Duboko" was ancient, most probably dated back to Pleistocene.

One of the basic reasons for the formation of landslides in this area is undoubtedly the lithological composition of the terrain: coherent rock masses liable to weathering processes. Neotectonic movements as well as an intensive erosion on the surface of relief have decisive influence on the formation of landslides. Raising of hipsometrically higher and steeper bank of the south block (in relation to the northern one) and undercutting of its toe in the river, as well as dredging of materials, are the main causes for reactivation of sliding processes. Main sliding mechanism in "Duboko" is a slow movement of blocks at the depths up to 25 m. It accelerates periodically, because of an intensive river erosion or seismic processes.

LANDSLIDE "KARABURNA"

The locality of landslides "Karaburna" is situated on the right bank of the Danube river, north of Belgrade. It embraces the whole slope up to the watershed, on which the last scars up to 25 m are visible. The surface of landslide is about 1.5 km², uneven - with many hummocks and depressions. The bottom of the slope continues in an alluvial plain which is about 300 m wide. During the Quaternary, the river moved its main current toward the left bank, so that it does not any more undercut the toe of the slope (Fig. 5).

In the investigated area, two complexes are to be distinguished: one representing disturbed - removed rock masses and the other undisturbed - in situ rock masses.

The first one, very heterogeneous, has different thickness: from 5 - 34 m. It consists of sandy clays, marly clays, loess & sands in mutually chaotic relations. Their structure and physico-mechanical properties are therefore very variable. The clays in this complex are fissured: with many sliding surfaces, having smooth and shining walls and striations. Due to the sliding, sand layers continuity was interrupted so that they appear now in "piles" (in m dimension), and are often saturated.

The complex of undisturbed rock masses, in the hinterland, is built up by loess sediments (1, Fig. 6) of Pleistocene age, a series of Miocene thinbedded clays and sands (cl+s) and, finally,
Fig. 5 LANDSLIDE "KARABURMA": 1. ALLUVIAL SEDIMENTS: GRAVEL 2. COLLUVIUM: CLAY, MARL AND SAND 3. LOESS 4. LANDSLIDE SCAR 5. MARSH
Fig. 6 LANDSLIDE "KARABURMA" CROSSECTION: 1. ALLUVIAL SEDIMENTS: GRAVEL 2. COLLUVIUM: YELLOW CLAY, MARL, SAND 3. LOESS 4. GRAY, OVERCONSOLIDATED CLAY AND MARL 5. CLAY AND SAND 6. SLIDING SURFACE
clays and marls (cl + mr). The latter ones, in thicker layers, oriented gently opposite to the slope (about 50°) form the bedrock of the upper, disturbed complex. Clay fraction from clay-sand series contains mineral montmorillonite this being very hygroscopic and susceptible to the volume changes: swelling and shrinkage.

According to its porosity, this series represents a multilayered hydrogeological collector, whereas clays and marls beneath them, oxidized, are hydrogeological isolators.

The slope of the right bank of the river Danube is neotectonically active. Along a large rupture, parallel to the bank, the slopes upheaves in relation to the depression in its foothill. Exogenic processes, that influences the formation of the slope morphological shape, were rather complex: the first falling and sliding were caused by undercutting of the river Danube's former current. This process was renewed many times. The depth of slides depended on the depth of the river bed. The deepest one passes downwards 30 m - to the contact of the two afore-mentioned Miocene series: thin layered clays and sands, and the underlying thick bedded clays and marls.

The next, not less important process was a progressive lowering of shearing strength in cohesive deposits, caused by weathering, i.e. wetting and drying processes, dissolution and seepage destruction.

By the dislocation of the river current to the north, the sliding processes did not stop. The most recent movements occurred in different parts of the terrain, mainly in the steeper ones, at various times.

Movements were slow, producing considerable material damages. A series of unfavourable circumstances provoked these phenomena. Among them, the most important were: unsuitable lithological composition and hydrogeological condition, as well as human influences. Namely, an uncontrolled and chaotic urbanization in the past years did not respect natural conditions of the terrain. Surficial waste and rain water very often spread down collecting in clayey depressions for a long time or infiltrating into sand "lenses", saturating them entirely. Owing to that, in sandy scars, water seeps each spring or after abundant rainfalls (or water conduit damage). Already disturbed surficial complex, thus being more or less saturated, often slides down the slope carrying with it smaller buildings, parts of roads or damaging them only if they lie in their direction of movement.

CONCLUSION

Landslides in Belgrade area along the Sava and the Danube rivers are conditioned by specific characteristics of the terrain structure - history of its formation and neotectonic processes.

Results of complex investigations made so far on few active landslides, built up mainly by clays and marls, rarely combined by sands, of Miocenic age, revealed two types of sliding: the first, a shallow one, which embraces only the weathered zone i.e. its smaller or greater part (5 - 8 m most frequently), and the second, a deep one (over 20-25 m), which embraces unchanged masses of clays and marls (bedrock) locally divided in large blocks.

The first type of sliding in disturbed, moistened or saturated clayey masses of reduced strength (\(C = 0, \phi = \phi_{res}\)), occurs more frequently in spring or autumn, locally - apparently "suddenly", owing to the intensive construction of suburban or week-end settlements.

The second type of deep sliding, represents a slow, translatory movement of large grey marl blocks, along the existing mechanical discontinuities. These movements are provoked by undercutting of slopes, that regionally upheave, thus enabling the river stream migration towards the border of the Pannonian depression. Due to the river depth and earthquake, this type of movement was activated in spring 1981 in the landslide "Duboko".

A rational protection of this type of landsliding is at present hardly possible.

REFERENCES


METHODS FOR DETERMINATION OF OPTIMUM SCOPE OF ENGINEERING GEOLOGY INVESTIGATIONS AT SITES OF HYDRAULIC STRUCTURES

DETERMINATION DES METHODES D'ETENDUE OPTIMUM DE TECHNIQUE GEOLOGIE DES INVESTIGATIONS AUX EMPLOACEMENTS DES STRUCTURES HYDRAULIQUES

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ABSTRACT

The tightening of requirements for substantiation of construction of the hydraulic structures and the development of new design methods necessitate the application of new methods for planning the investigations which enable the needed data to be obtained at the least cost, i.e. to optimize the investigation. One of the most effective methods of optimizing the investigations is the quantitative analysis of effect of geological factors on the design solutions and, on the basis of such analysis, identifying the most important and significant factors on which emphasis should be placed during investigations. Given in this paper is a planning model for engineering geology investigations which is based on the analysis of significance of the geological factors, and the examples of such analysis.

ABSTRAIT

Les exigences accrues pour la justification des ouvrages hydrauliques et le développement de nouvelles méthodes de calcul de ces derniers, imposent l'emploi de nouvelles méthodes de planification des études géologiques permettant d'obtenir l'information nécessaire pour un coût minimum des travaux, c'est-à-dire, d'optimiser les études. L'une des méthodes les plus efficaces pour l'optimisation des études consiste à réaliser une analyse quantitative (calcul) de l'influence des facteurs géologiques sur les solutions de projet, et à établir à base de cette analyse les facteurs les plus importants et significatifs, auxquels on doit attacher l'attention primordiale lors des études. Dans le présent rapport, est donné un schéma de principe pour l'optimisation des études géologiques, fondé sur les méthodes d'analyse de l'importance des facteurs géologiques, et sont donnés des exemples concrets de cette analyse.

Effectiveness of site investigations, in a great measure, depends on proper determination of extent scope and techniques to be used. But, it should be acknowledged that often site investigations are not conducted in the most optimum way. At the present time the traditional approach to the planning of
investigations is not sufficient. It is necessary not only to maintain close contacts between design and investigation activities but to arrange their mutual overlapping, i.e., employment of the so-called feedback principle. Exactly at the junction of these processes the most rational ways should be found to make investigations more task-oriented and effective. Besides, one of the most important requirements concerning the optimization of investigations consists in passing from the qualitative feedback principle to the quantitative analysis requiring the corresponding computational mechanism and methods for its applications.

Over a number of years the "Hydroproject" Institute is developing the model for optimization of the engineering geology investigations based on the quantitative analysis of the geological factors affecting the design and the following assessment of their significance.

First we shall give the examples of such analysis and then shall discuss the general investigation optimization model.

We shall begin with the analysis of the deformation modulus $E$ — the rock mass characteristic used for the tunnel lining design. The example of the analysis is given in Fig. 1. Shown here is the relationship between the cost of pressure tunnel of the Zarawagh hydroelectric station on the Ardon River (Caucasus) and the value $E$. As it is seen from the curve, the deformation modulus influences the tunnel cost only within the range 0–1200 MPa. When the value $E$ is greater, the tunnel cost remains unchanged irrespective of possible variations in $E$. Therefore if the deformation modulus is greater than 1200 MPa, its absolute value and the exactness of its determination are of no importance. Only its lower limit is of consequence. For example, the deformation modulus at the Zarawagh hydroelectric station for the sound zone represented by sedimentary rocks, namely, schists, argilites, sandstones was determined by the approximate method to be within 1500–3000 MPa. Therefore, in this case special field tests to finalize the deformation modulus were not required.

Another example of the design analysis is given in Fig. 2, where

Fig. 1. Analysis of the relationship between the modulus of deformation and the cost of the Zarawagh hydroelectric station tunnel (shaded part shows $E$ value in the sound rock).

Fig. 2. The relationship between the opening of cracks in the tunnel lining and the modulus of deformation (Tashlyk pumped storage station): I - zone of sound rock; II - zone of tectonic crushing; 1 - concrete lining; 2 - reinforced concrete lining; $a_{lim}$ - locally allowable crack opening in the lining taking into account ground water attack.

the relationship between the width of the crack opening in the concrete - the value which is critical for the computation of lining
parameters, and the deformation modulus of the rock mass composed of strong Archean granites is shown for one of the pressure tunnels of the Tashlyk pumped storage station. The following possible limits of variation were established by indirect methods: in the undisturbed part of the rock mass of various preservation - \( E = 6000-7500 \) MPa, in the crushed zones - \( E = 300-500 \) MPa. As it is seen from Fig. 2, the deformation modulus value for the undisturbed rock mass is within the range where its influence on the crack opening in the lining is negligible and is beyond the value \( E \) corresponding to the allowable crack opening (0.15 mm) for the conditions of the Tashlyk pumped storage station. On the contrary, in the crushed zones \( E \) influences considerably the crack opening. It was concluded that the further investigations should be concentrated solely in the crushed zones which are encountered at the construction site but have not yet been revealed along the tunnel route. Thus, by simple preliminary calculations the investigations can be aimed at finding solutions to rather a well defined task.

However, the potentialities of the analysis, concerning the deformation modulus influence on the tunnel construction of this hydroelectric scheme have not yet been exhausted. As the next step, it was decided to make the economic analysis. In case the crushed zone is not thick it may be more economically effective to provide additional reinforcement for the lining, proceeding from the most "cautious" characteristics of the deformation modulus, instead of performance of special field studies to have the modulus finalized, which would be, as a rule, time and cost intensive. Here is the example of such analysis.

The Tashlyk pumped storage station comprises three tunnels of 10 m in diameter and 6 tunnels of 7 m in diameter, 800 m long each and with a maximum head of 35 m. According to the rock mass model from 2 to 4 crushed zones 1-3 m thick may come across the tunnel route. The lining design is affected mainly by crushed zones of the thickness not less than 3 m. Based on the most unfavourable combination of factors the total thickness of crushed zones along the tunnel route would be 106 m. The calculations showed that in this case the difference between the cost of lining reinforcement, with the maximum value of \( E \) being 600 MPa and the minimum one - 200 MPa would amount to half of the cost of field studies for determination of the rebound factor, which should have been carried out at least in two chambers plus the cost of the exploratory adit for the revealing of crushed zones along the tunnel route, the time of the work execution being 6 months. Thus, the additional analysis showed that the exploration and investigations of crushed zones along the tunnel route to finalize the thickness of crushed zones and deformation properties proved to be unreasonable.

There is another example when the analysis is used for the optimization of the shearing resistance \( \tan \varphi \) and \( C \) studies. The relationship between the concrete dam stability and shear strength was considered for the Konstantinovskaya hydroelectric station on the South Bug (the Ukraine). The dam foundation is composed of strong Archean granites. Before, at the previous stage of designing \( \tan \varphi \) and \( C \) values were taken by analogues with the following limits of their probable deviations with respect to their average value for the contact concrete - rock \( \tan \varphi = 0.7-0.9 \) and \( C = 0.20-0.25 \) MPa, for large fractures and crushed zones \( \tan \varphi = 0.55-0.75 \) and \( C = 0.04-0.09 \) MPa. The results of the analysis are given in Fig. 3. As it is seen from Fig. 3, the shaded parts, corresponding to stability factors \( K \) for various combinations of shearing resistance values, are above the standard value of \( K \). The intake dam section is the exception due to the presence of a large subhorizontal fracture or crushed rock zone. Thus, for the adopted parameters of the dam which were determined mainly by the structural considerations design, the concrete dam shear strength at the dam base is provided in all cases thus making the finalizing of \( \tan \varphi \) and \( C \) for the concrete - rock contact and
tigations to be properly channeled and it plays a role of the screen, which sorts out the required investigations and rejects those of no importance. Such analysis, apart from placing the emphasis on the actually needed investigations, makes it possible to avoid geological surprises which stem not from a geologist's "error" but rather from failure to identify during design studies the natural factors critical for the structure, which should be given special care by the geologists during investigations.

The evaluation of significance of geological factors makes it possible not only to establish the necessary extent and identify kinds of investigations but also specify the accuracy of these investigations. It helps to choose the required technique and determine the number of tests to be performed.

For example, should it be necessary to make tests for the determination of the deformation modulus according to the tectonic pattern of the Tashlyk pumped storage station, then, as it is seen from Fig. 2, this parameter is to be determined with an accuracy of ±100 MPA. Specialists know that such an accuracy can be achieved only by large-scale in-situ tests, e.g., by loading the circular workings of 2-2.5 m in diameter by radial jacks. Labour and cost intensity of such tests was rather high and this fact has predetermined the rejection of E investigations in the tectonic zones of the Tashlyk pumped storage station taking into account the economic considerations.

The situation may arise when the required accuracy of the rock mass design characteristics cannot be achieved by the existing methods of studies. Then, the investigations become useless. This still narrows down the area of these characteristics.

The typical example is the results of the stability analysis of the Maryn River canyon flanks at the Toktogul dam site in Kirghizia. At the early stage of designing, the site investigations revealed several large rock masses separated by fractures in the canyon slopes. The stability of these rock masses was to be determined. The calculations of the relationship between

the performance of special in-situ jacking tests unnecessary. At the same time the results of the analysis point to the necessity of more detailed study of the subhorizontal planes of weakening in the rock mass, which endanger stability of the intake dam section. Therefore, the purpose of the further studies is to identify the above faults and, if any, to conduct in-situ shear tests on the sound rock blocks, along the fractures.

Here, as in the case of the study of rock mass deformation properties, the investigations were task-oriented. Thus, the method of analyzing the significance of geological factors allows the inver...
is also the limiting technically possible anchor load which is 80000 t/1.m of the slope length. As it is seen from the figure, the deviation within \( \pm 1° \) and that of \( \tan \phi \) within \( \pm 0.03 \) may lead either to the rock mass stability \( (A=0) \) or to the situation when it is impossible to stabilize it by anchoring \( (A > 80000 \) t/1.m). It is known that the accuracy of \( \alpha \) and \( \tan \phi \) determination is below the required one. The example is given in Fig. 4c showing the profile of the flat fracture along which a costly 122 m long cross cut rarely used in the investigation practice was made to finalize the angle. Nevertheless, the required accuracy of angle \( \alpha \) determination was not achieved. As it is seen from Fig. 4b, the method of drawing the line of sliding determines the dip angle varying within 10°-16° (naturally, the determination of angle \( \alpha \) only by its measuring on the slope surface would be still more inaccurate). Proceeding from the results of the analysis it was found reasonable to give up the further investigations concerning the fracture parameters for the slope stability of the Toktogul dam and to find out other ways of solving the stability problem, e.g. the method of natural analogues.

In addition to the above cited there are examples of the analysis of relationship between the deformation modulus of the rock mass and the stressed state of arch dams and between fracture parameters and slope stability, etc. All of them have shown great effectiveness of the analysis of geological factor significance in optimization of extent and methods of investigations. At present, a number of nomographs and charts based on such analysis is made for the evaluation of the significance of various engineering-geological factors during the designing of main hydraulic structures, this will help the design engineers to work out the investigation program and the surveying engineers to determine the approximate extent and accuracy, required for the investigation of rock foundation properties and structure.

the stability factor of the above rock masses and the original parameters showed that with the volumes of rock mass being so big, the gravity forces have such a great effect (the maximum one is 700·10^3 m^3), that the slightest change in original parameters in either direction leads to a sharp change of the stability factor and adopted design solutions. Fig. 4a gives the relationships between anchor load required for the rock mass stabilization, the angle of flat fracture gradient \( \alpha \) undercutting the rock mass, and the shear strength value \( \tan \phi \) along this fracture. There

Fig. 4. The analysis of the relationship between the stability and slope protection at the Toktogul hydroelectric station: a) the relationship between the anchor load, the dip angle of the flat fracture \( (\alpha) \) and the shearing resistance \( (\tan \phi) \); \( A_{lim} \) - limiting anchor load; \( \beta \) - design of the rock mass anchorage; 1 - rock mass; 2 - flat fracture \# 550; C - the cross cut along flat fracture \# 550.
the certain accuracy limits which should meet the objectives specified. The work of the first stage is the prerogative of engineering geologists.

At the second stage, the analysis establishing the relationship between design solutions and natural factors varying within wide (but physically possible) limits is made. Then the approximate values adopted at the first stage are compared with the analysis results. If these values are within the range where their possible variations have no or little effect on the cost and other parameters of the structure and vice versa, if they are within the range, where the required accuracy of design values is beyond the possibility of the existing investigation methods then the conclusion, concerning the adequacy of the preliminary established values for the design substantiation, is made (this is shown in Fig. 5 by the arrow). If the above said values are within the range, where their effect is considerable the decision is adopted to carry out the required engineering-geological and geomechanical investigation (III-d step). The work of the second step is the prerogative of design engineers, employing the calculation and design methods and geologists for staged revision of the design and extent of investigations.

But the decision about additional investigations should be taken only after the economic analysis has been done. Its task is to establish whether additional field and laboratory studies can improve the design characteristics and thus reduce the structure cost or vice versa these investigations are unable to justify the expenditure. The analysis is not easy to fulfil but in some cases, as it is said above, it is quite possible.

After the overall assessment we can pass to the III step - the more detailed field and laboratory studies. At this stage, the optimum investigation methods, which should meet the requirements of the parameter accuracy, established at the previous stage, are selected and the optimum volumes of work are determined. The work of
the III-d stage is completely the
task of engineering geologists.

The described optimization
system of investigations is the
most applicable to the engineering
and final design stage, when the
main types of structures and their
parameters are already finalized.
Besides, it is employed mainly for
the well-known natural phenomena
and foundation properties and ap-
proved design methods. However it
should be noted that the optimum
analysis method can be used some-
times at the earlier design stages,
but for rough and approximate es-
timates corresponding to the de-
tails of changes at this stage. As
calculations show, the presence of
only one parameter, such as the
head, is already sufficient for
the rough estimate of the signifi-
cance limits of the certain geolo-
gical factors and their characte-
ristics. A set of nomographs will
make it possible to sort out the
alternatives of various structures
and identify the governing (or eli-
minate insignificant) natural fac-
tors.

In conclusion it should be
emphasized that the analysis of the
geological factor significance is
not the only way of the investiga-
tion optimization. Besides, the
quantitative analysis cannot be al-
ways used for optimizing the ex-
tent of investigations. Apart from
calculations, any design requires
the thorough geological analysis,
experience, intuition and common
sense. Only this combination can
provide the required information
obtained by the minimum volume of
work, i.e. the complete optimiza-
tion of investigations. Neverthe-
less, the theses given in this pa-
per can be utilized even to day
for the solution of a number of
problems concerning rock foundation
properties and as the gaining of
the experience and data proceeds,
can gradually embrace still greater
area of field engineering investi-
gations.
TESTING FOR ROCK PARAMETERS IN A FOUNDATION DESIGN

ESSAI POUR DES PARAMETRES DE ROCHE AU PLAN DE FONDEMENT

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ABSTRACT

Nature of rocks at a project site can be understood by conducting suitable geotechnical investigation programme. The rock parameters required for safe and economic design of foundations need to be obtained through in-situ and/or laboratory tests. The paper discusses a detailed geotechnical investigation programme carried out for a cement factory. Deep borings, in-situ horizontal plate load tests, block shear tests have been carried out. Laboratory tests included - sonic wave velocity, uniaxial compressive strength and other tests on gouge. The data from various tests is analysed to obtain design parameters taking into account various rock layers met with. As the site lies close to quarry area blast tests have also been carried out to recommend blast spectra. Also earthquake response spectra for the design of structures has been recommended.

ABSTRACT

On a étudié les comportement des roches par un programme d'investigation géotechnique qui a été organisé. On besoin d'obtenir les paramètres des roches - pour la sécurité de la foundations et aussi de faire une dessins economique par les tests dans un laboratoire on a la exterior (in situ) en place. On a discute' dans cet article un programme d'investigations géotechnique qui furent faites pour une fabriqué de ciment. On a fait le sondage à grande profondeur, les assais des 'treintes late'rales (deslames) et les assais de eisailement(bloc). Les tests sur la vitesse des onde sonore, sur la résistance à la compression uniaxiale et les autres (on gouge) sont fait dans un laboratoire. Les data de tests différent sont analyse' pour obtenir les parametres de dessins prenant en consideration les couches des roches qui on a rencontre'. Comme le lieu de project est pre de la proie, on a aussi fait les tests avec charge d'explosif (blast tests) pour recommander de spectre de blast (blast spectra). Les spectres de réponse (response spectra) produit par un tremblement de terre sont aussi recommandé pour les dessins des fontes different.

INTRODUCTION

A 1200 tonnes per day cement factory is proposed in the Southern part of India. The site is gently sloping southwards without undulation, having a height of 449 m above M.S.L. in the northern part and about 444 m in southern part. Most of the part of site is covered by black cotton soil with occasional outcrops of limestone. The entire area had been under cultivation. The soil cover is followed by jointed/ massive limestone in the factory area, whereas by shale in the adjacent area marked for colony. The level of colony
area is in general lower than the factory area.

As per seismic zoning map of India (IS:1893-1975), the site lies in the seismic zone. The quarry is also adjacent to the factory site.

A cement factory normally has typical structures (Ranjana et. al.1977) e.g. heavily loaded structures such as storage and blending silos, tall structures such as pre-heater tower and machines such as rotary kiln. Besides that as the factory is located adjoining to the factory, the structures have to be designed to avoid any damage due to anticipated quarry blast.

The paper discusses the detailed geotechnical investigations carried out at the site and in laboratory to estimate realistic design parameters required for design of foundations. The test data is analysed and recommendations for design parameters have been made.

GEOLGY OF THE AREA

The bed rocks at the factory site are formed of light grey flaggy limestone, with interbeds of purple shales towards the bottom and massive limestone in the upper part, yielding slabs of 5 to 25 cm thickness. The northern part is formed of bluish grey massive limestone.

The bed rock limestone at the factory site is light dark grey and buff in colour, hard, compact and fine grained; occasionally cherty and with variable minor content of aggradaceous micaceous and ferrogensious impurities. The various outcrops and exposures of the limestone and shale near the adjoining village indicate a NW-SE strike with nearly horizontal disposition of the beds to very gentle dips (20 to 30°) towards NE. The limestone is highly jointed having the following sets, spaced at 0.5 to 2 m interval:

1 E-W Vertical to steeply dipping (65° due N)
2 N-S Vertical
3 ENE-WSW Vertical
4 SSE-NNW Vertical
5 ESE-WNN Vertical

Of these, the E-W and N-S joints are the prominent ones, having well developed solution cavities. While the southern part of the area formed of the flaggy limestone is generally devoid of solution cavities, the northern part of the area formed of massive limestone contains thin solution cavities. The cavities developed along the joints are irregular in width, varying from 5 cm to 15 cm. At shallow depths the cavities are filled with soil matrix, while in deeper zones (3 m) the cavities remain open partly, although their width tends to decrease significantly. The soil profile has an average thickness of 1 m. It does not show any stratification although the lower part is relatively coarser with good amount of Kankar and concretionary silica and iron oxide. The change from soil to the bed rock is rather sharp. The soil in general is predominantly clayey with little sand grade material. The soil horizon contains boulders of limestone.

GROUND WATER

In the northern part of the site, there was a well from which the water table was found to be 11-12 m deep in January 1978 below ground surface. During the detailed investigations in May/June 1979 water table was not met within any of the test pits varying in depth from 3-6 metres. A number of bore-holes were drilled through the rock strata in the factory area upto the depth of 20 m. No water was met within these drill holes.

GEOTECHNICAL INVESTIGATIONS AND TEST DATA

Examination of pits at different locations in the factory and colony area indicated a thin black cotton soil cover varying up to about 80 cm in thickness underlain by jointed (horizontal joints) limestone and by massive limestone. In colony area the soil cover was followed by shale. The joints in limestone region were observed to be fairly thick with gougé material. A detailed investigation programme was thus drawn out which included the following tests.

Borings - Ten bore-holes (each 20 m deep and N x size) were made to cover the complete factory area including one in the adjoining colony site. Continuous samples/rock cores were obtained for laboratory tests. The spacing of bore-holes was about 200 m.

Field Tests - The field tests included

(a) Horizontal plate load tests (4 nos) covering the factory and colony site.
These tests were carried out using 30 cm x 30 cm size steel plates. Figure 1 shows the test set-up. The depth of centre of plate below ground surface varied from 1.6 m to 2.5 m. Figure 2 shows the typical plot obtained from one of these tests. The data was utilised to obtain safe bearing pressure and horizontal modulus of subgrade reaction.

(b) Block shear test - Three in-situ block shear tests (block size varying from 20 cm x 16 cm and 27 cm x 21 cm) with depth of centre of block varying from 2.4 m to 3.2 m were carried out. Typical plot obtained is shown in Figure 3. These tests served as means for finding the crushing strength of rock and thus the allowable bearing pressure.

![Fig.3 Load displacement plot from block shear test](image)

(c) Blast tests - Blast tests (using 80% gelatine) were carried out to determine the value of peak ground acceleration at certain distances away from the blast point. Intensity of blast used varied from 23 kg to 100 kg. Peak ground accelerations using acceleration pickups buried in the ground at a depth of about 30 cm were recorded at distances varying from 50 m to 150 m from blast points.

Laboratory Tests

The laboratory tests were carried out on gouge material and also on the rock cores obtained during drilling. The tests carried out were:

(a) Wet sieve analysis, specific gravity, free swell, liquid limit and plastic limit tests were conducted on gouge material from the samples obtained from various pits. The tests were conducted as per procedures laid down by Indian Standards. The tests were aimed to assess the nature of gouge material. Table 1 summarises the test results.

(b) The laboratory tests on rock cores were carried out to assess the rock quality and its engineering classification. Then using the well-known correlations between mass modulus and rock quality, the moduli of deformation for the fractured rock mass at the site
## Table 1 - Analyses of gouge samples

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth (m)</th>
<th>Specific gravity</th>
<th>( w_L ) (%)</th>
<th>( w_p ) (%)</th>
<th>( I_p ) (%)</th>
<th>Free swell (%)</th>
<th>Fraction finer than 75 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pit No.1</td>
<td>2.0</td>
<td>2.50</td>
<td>67.5</td>
<td>19.2</td>
<td>48.3</td>
<td>37.5</td>
<td>86</td>
</tr>
<tr>
<td>HPL - 1</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>85</td>
</tr>
<tr>
<td>Pit No.2</td>
<td>0.5</td>
<td>2.90</td>
<td>50.0</td>
<td>22.0</td>
<td>28.0</td>
<td>50.0</td>
<td>92</td>
</tr>
<tr>
<td>HPL - 2</td>
<td>2.5</td>
<td>2.70</td>
<td>72.0</td>
<td>19.0</td>
<td>53.0</td>
<td>75.0</td>
<td>68</td>
</tr>
<tr>
<td>Pit No.3</td>
<td>1.0</td>
<td>2.80</td>
<td>70.0</td>
<td>33.0</td>
<td>37.0</td>
<td>-</td>
<td>50.0</td>
</tr>
<tr>
<td>HPL - 3</td>
<td>2.1</td>
<td>2.65</td>
<td>68.0</td>
<td>28.0</td>
<td>40.0</td>
<td>-</td>
<td>70.0</td>
</tr>
<tr>
<td>Pit No.4</td>
<td>0.5</td>
<td>2.90</td>
<td>50.0</td>
<td>17.0</td>
<td>33.0</td>
<td>50.0</td>
<td>91</td>
</tr>
<tr>
<td>Block shear 1</td>
<td>2.0</td>
<td>2.80</td>
<td>38.0</td>
<td>18.0</td>
<td>20.0</td>
<td>100.0</td>
<td>65</td>
</tr>
<tr>
<td>Pit No.5</td>
<td>1.0</td>
<td>2.70</td>
<td>57.0</td>
<td>20.0</td>
<td>37.0</td>
<td>55.0</td>
<td>89</td>
</tr>
<tr>
<td>Block shear 2</td>
<td>2.7</td>
<td>2.62</td>
<td>70.0</td>
<td>19.0</td>
<td>51.0</td>
<td>110.0</td>
<td>62</td>
</tr>
<tr>
<td>Pit No.6</td>
<td>0.5</td>
<td>2.63</td>
<td>61.0</td>
<td>17.0</td>
<td>44.0</td>
<td>100.0</td>
<td>91</td>
</tr>
<tr>
<td>Block shear 3</td>
<td>2.4</td>
<td>2.70</td>
<td>36.0</td>
<td>16.0</td>
<td>20.0</td>
<td>100.0</td>
<td>60</td>
</tr>
<tr>
<td>Pit No.7</td>
<td>0</td>
<td>2.70</td>
<td>36.0</td>
<td>16.0</td>
<td>20.0</td>
<td>100.0</td>
<td>72</td>
</tr>
<tr>
<td>HPL - 4</td>
<td>0.3</td>
<td>2.66</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>2.80</td>
<td>38.0</td>
<td>13.0</td>
<td>25.0</td>
<td>77.0</td>
<td>4</td>
</tr>
</tbody>
</table>

The cores were also examined to estimate the spacing and nature of natural bedding planes. All natural fractures were noted to be nearly horizontal, slightly rough and sometimes stained by calcium carbonate. The average gouge thickness was determined by Equation 2.

\[
\text{Average gouge thickness} = \frac{\text{(length of drillings)} - \text{(total length of cores)}}{\text{(number of natural cores)}} \quad \ldots \ldots (2)
\]

### iv) Determination of Sonic-velocity,

\[ V_{r} \text{ of rock cores using ultra-conic test using Equation 3} \]

\[
V_{r} = \frac{L}{t} \times 10^4 \text{ (m/sec)} \quad \ldots \ldots (3)
\]

where \( V_{r} \) = longitudinal wave velocity of rock material in metre per second.

\( L \) = length of the specimen in cm

\( t \) = time of travel of pulse in micro-seconds

The dynamic elastic modulus of rock material is calculated from Equation 4

\[
E_{r} = \frac{V_{r}^2}{g} \times \frac{1}{10} \text{ (kg/cm}^2) \quad \ldots \ldots (4)
\]

where \( E_{r} \) = dynamic elastic modulus of rock material in kg/cm^2

\( V_{r} \) = unit weight of rock material, g/cc

\( V_{r} \) = longitudinal wave velocity in m/sec from Equation 3

\( g \) = gravity constant 9.81 m/sec^2.

It may be mentioned that the static elastic modulus of rock material is generally slightly less than dynamic modulus and is, however, taken as equal to \( E_{r} \).

The uniaxial compressive strength is about 22 times the point load strength index (IS: 8764-1968). However, such a correlation is not considered suitable in case of soft rocks like shale. In view of this the uniaxial compressive strength, \( q_c' \) is determined using Equation 5.

\[
q_c' = \frac{Q}{A} \text{ (kg/cm}^2\text{)} \quad \ldots (5)
\]

where \( q_c' \) = uniaxial compressive strength in kg/cm\(^2\)

\( Q \) = peak load at the instant of fracture in kg

\( A \) = area of the specimen in cm\(^2\)

\( = \pi D^2/4 \)

\( D \) = diameter of the specimen in cm

Since the length and diameter of specimens were different. A correction was applied to estimate the uniaxial strength of unit slenderness (L/D) as per Equation 6 (Obert and Duvall, 1962).

\[
q_c = \frac{q_c'}{0.778+0.222 \frac{D}{L}} \text{ (kg/cm}^2\text{)} \quad \ldots (6)
\]

where \( q_c \) = corrected uniaxial compressive strength of rock specimen of unit slenderness ratio in kg/cm\(^2\)

\( q_c' \) = uniaxial compressive strength obtained from test in kg/cm\(^2\)

\( L \) = length of specimen in cm

\( D \) = diameter of specimen in cm

Table 2 gives the values of uniaxial strength of shale samples for the ten bore-holes. The corresponding Atterberg limits were determined on powered shale as per IS:2720(Pt. V) - 1970.

vii) Tests for slaking tendency for swelling potential for shales.

A few shale samples showed a tendency to split diametrically along bedding planes during air drying. Hence it was thought necessary to confirm this tendency by placing cores in a tray full of water for several days. However, it was seen that cores did not split and there was no sign of pieces of shale

<table>
<thead>
<tr>
<th>No.**</th>
<th>Saturation (%)</th>
<th>Liquid plasticity index</th>
<th>Uniaxial strength in saturated condition (kg/cm(^2))</th>
<th>Uniaxial strength in drained condition (kg/cm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>2.71</td>
<td>21.0</td>
<td>15.5</td>
<td>638</td>
</tr>
<tr>
<td>A-2</td>
<td>2.71</td>
<td>17.0</td>
<td>15.5</td>
<td>621</td>
</tr>
<tr>
<td>A-3</td>
<td>2.70</td>
<td>19.0</td>
<td>11.2</td>
<td>552</td>
</tr>
<tr>
<td>A-4</td>
<td>2.68</td>
<td>19.6</td>
<td>12.1</td>
<td>421</td>
</tr>
<tr>
<td>A-5</td>
<td>2.66</td>
<td>26.1</td>
<td>13.5</td>
<td>329</td>
</tr>
<tr>
<td>B-1</td>
<td>2.75</td>
<td>19.2</td>
<td>12.4</td>
<td>912</td>
</tr>
<tr>
<td>B-2</td>
<td>2.71</td>
<td>19.7</td>
<td>13.6</td>
<td>831</td>
</tr>
<tr>
<td>B-3</td>
<td>2.76</td>
<td>19.0</td>
<td>12.4</td>
<td>503</td>
</tr>
<tr>
<td>B-4</td>
<td>2.68</td>
<td>20.0</td>
<td>12.9</td>
<td>441</td>
</tr>
<tr>
<td>B-5</td>
<td>2.72</td>
<td>19.0</td>
<td>13.9</td>
<td>663</td>
</tr>
</tbody>
</table>

* Shaking tendency and swelling potential were noted to be insignificant.
** Depth of each bore-hole 19 m

Table 2 Characteristics of Shales

Typical results on rock cores from bore-hole A-1 are shown in Table 3. Similar data is obtained on rock cores from other bore-holes.

CLASSIFICATION OF ROCK MASS

Bieniawski (1975) proposed a simple and reliable geomechanical classification based on extensive field data from hundreds of projects and several case histories. The geomechanical classification rating (known as G.C. rating) is quantitative and accounts for several important parameters. The range of parameters from cores obtained from ten bore-holes have been tabulated in Table 4. The rating is given just below the corresponding parameter according to tables of ratings of Bieniawski (1975). Since, the observed range of parameters do not fit in the prescribed range of Bieniawski's parameters, a range of rating is listed in the Table 4. The sum of the rating has also been included along with average value of G.C. rating. The rock quality is also indicated. Thus massive limestone is a 'fair rock' but other rocks are 'poor rock.'
<table>
<thead>
<tr>
<th>Bore Hole No. Al</th>
<th>Rock Type</th>
<th>Length of cores more than 5 cm</th>
<th>Core recovery (%)</th>
<th>Density of air dry sample (gm/cc)</th>
<th>Sonic velocity of core (m/sec)</th>
<th>Point load strength index (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.48</td>
<td>Black cotton soil</td>
<td>12.6, 12</td>
<td></td>
<td></td>
<td></td>
<td>42.5, 50</td>
</tr>
<tr>
<td>0.83</td>
<td></td>
<td>15.7</td>
<td>97</td>
<td>2.74</td>
<td>6450</td>
<td>35</td>
</tr>
<tr>
<td>1.23</td>
<td></td>
<td>8.4, 5.7, 11</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.68</td>
<td></td>
<td>8.7, 13, 36, 8.3, 7, 7, 14.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.63</td>
<td>Flaggy lime stone dark grey, fine grained</td>
<td>29, 21, 8, 6.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.25</td>
<td></td>
<td>5.6, 10.2, 12.7, 6, 6.5, 9.1, 5.5, 6.5, 11.7, 11.4, 12.5, 11.1, 6.4, 8.4, 11.1, 10.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.25</td>
<td></td>
<td>6.6</td>
<td></td>
<td></td>
<td></td>
<td>98</td>
</tr>
<tr>
<td>5.45</td>
<td></td>
<td>6.9</td>
<td></td>
<td></td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>5.65</td>
<td></td>
<td>6.8, 5.7, 7.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.55</td>
<td></td>
<td>5.8, 9.3, 100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.70</td>
<td>Shaly limestone</td>
<td>12.5, 7, 8.5, 10.9, 5, 6.7, 13.7, 2, 9.2, 5.5, 5.8, 5.4, 10.6, 8.5</td>
<td></td>
<td></td>
<td>2.71</td>
<td>5353</td>
</tr>
<tr>
<td>9.60</td>
<td></td>
<td>5.7, 2, 5.7, 6.8, 5.4, 4, 5.5, 100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.85</td>
<td></td>
<td>12.7, 6.6, 6.6, 10.6, 7</td>
<td></td>
<td></td>
<td>2.68</td>
<td>2446</td>
</tr>
<tr>
<td>11.10</td>
<td></td>
<td>6.18, 71</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.69</td>
<td></td>
<td>7.7, 14.2, 11.9, 9.1, 8.1, 17.7, 6.5, 13.3, 8.7, 6.3, 8.7, 10, 12.9, 20.4</td>
<td></td>
<td></td>
<td>2.68</td>
<td>3</td>
</tr>
<tr>
<td>15.7</td>
<td>Pink shale</td>
<td>6.5, 7, 5.2, 11.7, 6.8, 5, 8.12, 8.17, 2, 17.2, 5.7, 5.5</td>
<td></td>
<td></td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>17.2</td>
<td></td>
<td>10, 10.5, 5, 5.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17.67</td>
<td></td>
<td>6.7, 9.5, 10, 5.5, 5.5, 9.5, 5.3, 5.2, 6.5, 5.2, 8, 5.2, 5.8</td>
<td></td>
<td></td>
<td>100</td>
<td>5</td>
</tr>
<tr>
<td>20.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bieniawski classification</td>
<td>Massive limestone</td>
<td>Flaggy limestone</td>
<td>Shaly limestone</td>
<td>Pink shale</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------------------</td>
<td>------------------</td>
<td>-----------------</td>
<td>----------------</td>
<td>-----------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parameters</td>
<td>$I_s = 22 - 66$</td>
<td>$I_s = 22 - 62$</td>
<td>$I_s = 10.5-51.5$</td>
<td>$q^*_{sc} = 329 - 912$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength of rock material (kg/cm²)</td>
<td>7-12**</td>
<td>7-12</td>
<td>4-12</td>
<td>4-7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RQD Percentage (cm)</td>
<td>28 - 16</td>
<td>13 - 70</td>
<td>14 - 40</td>
<td>0 - 33</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-13**</td>
<td>3-13</td>
<td>3-8</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing of joints (cm)</td>
<td>Horiz. Vert. 4-29</td>
<td>Horiz. Vert. 0.5-25</td>
<td>Horiz. Vert. 0.5-13</td>
<td>Horiz. Vert. 0.5-17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10 - 25**</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition of joints</td>
<td>Horiz. Vert. 2.5 cm rough</td>
<td>Horiz. Vert. 2.15 cm tight</td>
<td>Horiz. Vert. sli- rough &amp; tight</td>
<td>Horiz. Vert. sli- rough &amp; tight</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 25**</td>
<td>0 - 25</td>
<td>12 - 25</td>
<td>12 - 25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ground water flow</td>
<td>No 10**</td>
<td>No 10</td>
<td>No 10</td>
<td>Moist 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adjustment for joint orientation</td>
<td>$\text{dip} = 0^\circ$</td>
<td>$\text{dip} = 0^\circ$</td>
<td>$\text{dip} = 0^\circ$</td>
<td>$\text{dip} = 0^\circ$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G.C. rating</td>
<td>35 - 85**</td>
<td>25 - 70</td>
<td>34 - 65</td>
<td>39 - 52</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average = 60</td>
<td>Average = 47.5</td>
<td>Average = 49.5</td>
<td>Average = 45.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Point load strength index in kg/cm²

** Classification rating (all denominators indicate a range of rating corresponding to the parameter on the numerator)

*** Uniaxial compressive strength in kg/cm²

Table continued........
Table 4 continued

<table>
<thead>
<tr>
<th>Bieniawski classification</th>
<th>Massive limestone</th>
<th>Flaggy limestone</th>
<th>Shaly limestone</th>
<th>Pink shale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock quality</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Poor rock</td>
<td>Poor rock</td>
</tr>
<tr>
<td>Elastic modulus(^{*})</td>
<td>6.76 - 10.79</td>
<td>1.32 - 10.83</td>
<td>0.48 - 3.02</td>
<td>0.14 - 1.77</td>
</tr>
<tr>
<td>of core (x 10^5)</td>
<td>(\text{kg/cm}^2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus reaction factor(^{**})</td>
<td>0.21</td>
<td>0.17</td>
<td>0.17</td>
<td>0.16</td>
</tr>
<tr>
<td>Elastic modulus of rock-mass (10^4 \text{kg/cm}^2)</td>
<td>14 - 23</td>
<td>2 - 18</td>
<td>0.8 - 5</td>
<td>0.2 - 3</td>
</tr>
</tbody>
</table>

\(^{*}\) Values obtained from ultra-sonic tests on rock cores

\(^{**}\) Defined as the ratio between elastic modulus of the jointed rock mass and the elastic modulus of the rock core. Values as per the correlations (Bieniawski, 1975)
DESIGN PARAMETERS

The test data obtained from various tests is analysed to provide the range of design parameters for foundation design.

Nature of Deposit

As indicated earlier, the analysis of rock cores and other data reveals that there is a black cotton soil cover, of about 80 cm thickness followed by four layers of rocks classified as massive limestone, flaggy limestone, shaly limestone and pink shale. The typical thicknesses of these layers are shown in Table 5. Some of the boreholes exhibited no massive limestone whereas in a few boreholes flaggy limestone was found to be missing. Only one bore-hole exhibited reverse order of deposit.

Table 5 Values of Elastic Constants and Allowable Bearing Pressure from Laboratory Tests on Rock Cores

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Typical thickness of layer, ( t_i )</th>
<th>Elastic modulus of layer, ( E_i )</th>
<th>Effective modulus of the foundation at mid-height of the layer, ( E_i )</th>
<th>( C_u ) for 10 sq.m.</th>
<th>( C_u ) for 10 sq.m.</th>
<th>Bearing pressure for 40 sq.m. settlement (kg/cm²)</th>
<th>Bearing pressure for 1.25 cm settlement (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black cotton soil</td>
<td>0.75</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Massive limestone</td>
<td>1.50</td>
<td>14 x 10⁴</td>
<td>0.7 x 10⁴</td>
<td>25.5</td>
<td>12.7</td>
<td>160</td>
<td></td>
</tr>
<tr>
<td>Flaggy limestone</td>
<td>7.00</td>
<td>2.0 x 10⁴</td>
<td>0.33x 10⁴</td>
<td>12.0</td>
<td>6.0</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>Shaly limestone</td>
<td>2.00</td>
<td>0.8 x 10⁴</td>
<td>0.21x 10⁴</td>
<td>7.6</td>
<td>3.8</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>Pink shale</td>
<td>9.00</td>
<td>0.2 x 10⁴</td>
<td>0.20x 10⁴</td>
<td>7.3</td>
<td>3.6</td>
<td>46</td>
<td></td>
</tr>
</tbody>
</table>

Allowable Bearing Pressure and Elastic Constants

Safety against shear failure and settlement within permissible limits are the two basic requirements for satisfactory action of foundation. Rock strata, even if they are partially weathered are sufficiently strong to rule out the possibility of shear failure. Thus the settlement consideration becomes important. A permissible settlement of 12.5 m is recommended (Peck et. al. 1974). The corresponding settlement of the test plate can be worked out using Equations 7 and 8.

\[
\frac{S_P}{S_f} = \left[ \frac{B_P}{B_f} \frac{(B_f + 30)}{(B_f + 30)} \right]^2
\]

for laminated rocks

\[
\frac{S}{S_f} = \frac{B_P}{B_f}
\]

and \( \frac{S}{S_f} = \frac{B_P}{B_f} \) for massive rocks

where \( S_P \) = settlement in plate in mm

\( S_f \) = settlement of footing in mm

permit of value of 12.5 mm adopted in the present case

\( B_P \) = width of test plate in cm

\( B_f \) = width of footing in cm

Field Tests

The permissible settlement values for the test plates are worked out corresponding to assumed footing widths \( B_f \) of 200 cm and 600 cm the two typical footings dimensions. Beyond the footing width of 600 cm the width may not have any significant effect on the settlement. The allowable bearing pressure is taken from the load settlement curve obtained from horizontal plate load test. The computations have been made and results summarised (Table 6). The results of the block shear tests conducted in the factory area are given in Table 7. The values of crushing strength, \( q_u \) along with allowable bearing pressures, based on 0.2 \( q_u \) are given in Table 7. The minimum value of allowable bearing pressure, \( q_{as} \) is noted to be 125 t/m² which (taking into account submergence) is reduced to 62.5 t/m².
Table 6 Allowable Bearing Pressure from HPL Tests

<table>
<thead>
<tr>
<th>Pit No</th>
<th>Test location</th>
<th>Depth(cm)</th>
<th>Assumed width of footing (cm)</th>
<th>Corresponding settlement of plate* bearing pressure(t/m²)</th>
<th>Allowable pressure(t/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Laminate sound rock (Eq.7)</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Factory area</td>
<td>230</td>
<td>200</td>
<td>4.50</td>
<td>1300</td>
</tr>
<tr>
<td></td>
<td>(flaggy limestone)</td>
<td></td>
<td>200</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>1.87</td>
<td>155</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>3.45</td>
<td>810</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>-</td>
<td>105</td>
</tr>
<tr>
<td>2</td>
<td>Factory area</td>
<td>250</td>
<td>200</td>
<td>4.50</td>
<td>188</td>
</tr>
<tr>
<td></td>
<td>(Flaggy limestone)</td>
<td></td>
<td>200</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>1.87</td>
<td>152</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>3.45</td>
<td>175</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>-</td>
<td>120</td>
</tr>
<tr>
<td>3</td>
<td>Factory area</td>
<td>220</td>
<td>200</td>
<td>4.50</td>
<td>&gt;1300</td>
</tr>
<tr>
<td></td>
<td>(Flaggy limestone)</td>
<td></td>
<td>200</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>1.87</td>
<td>620</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>3.45</td>
<td>&gt;1300</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>-</td>
<td>310</td>
</tr>
<tr>
<td>7</td>
<td>Colony area</td>
<td>160</td>
<td>200</td>
<td>4.50</td>
<td>&gt;1300</td>
</tr>
<tr>
<td></td>
<td>(Shale)</td>
<td></td>
<td>200</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>1.87</td>
<td>855, 895</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>3.45</td>
<td>&gt;1300</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>600</td>
<td>-</td>
<td>395, 555</td>
</tr>
</tbody>
</table>

* For permissible settlement of footing of 12.5 cm

**Size of plate used 30 cm x 30 cm

Table 7 Allowable Bearing Pressure from Block Shear Tests

<table>
<thead>
<tr>
<th>Pit No</th>
<th>Test location</th>
<th>Block face (cm)x(cm)</th>
<th>Depth (cm)</th>
<th>Crushing strength $q_u$(t/m²)</th>
<th>Allowable bearing pressure* $q_a=0.2q_u$</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Factory site</td>
<td>24 x 21</td>
<td>240</td>
<td>625</td>
<td>125</td>
<td>As per observations the cracking started at 750 t/m²</td>
</tr>
<tr>
<td></td>
<td>(Massive limestone)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Factory site</td>
<td>27 x 21</td>
<td>320</td>
<td>1500</td>
<td>300</td>
<td>No failure observed till 1800 t/m²</td>
</tr>
<tr>
<td></td>
<td>(Massive limestone)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Factory site</td>
<td>20 x 16</td>
<td>240</td>
<td>1825</td>
<td>365</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(Massive limestone)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Values will be reduced to half on account of submergence of rocks due to rise of water table

Laboratory Tests

The allowable bearing pressure can be obtained from the values of elastic modulus also.

If a typical foundation width of 600 cm is assumed the pressure bulb below the foundation may extend up to 1200 cm encompassing various rock layers. The effective modulus of the layered rock mass within the pressure bulb of 1200 cm could be estimated from Equation 9.

$$\frac{E_i}{E_i} = \frac{0.5t_i}{E_i} + \frac{t(i+1)}{E(i+1)} + \ldots + \frac{t}{E_n} \ldots (9)$$

where $t = \text{depth of pressure bulb (} = 1200 \text{ cm)}$
\[ E_i = \text{effective elastic modulus of the foundation at the mid-height of the } i\text{th rock layer} \]
\[ t_i = \text{thickness of } i\text{th rock layer} \]
\[ t_n = \text{remaining thickness of the deepest layer within the pressure bulb (shale at this site)} \]
\[ t = t(0.5t(1)^{i+1}t(1+1)^{i+1}) \]
\[ E_i = \text{elastic modulus of the } i\text{th layer} \]
\[ E_n = \text{elastic modulus of the deepest layer i.e. shale} \]

The above analysis is conservative because variation in stress is neglected.

Consider for example, flaggy limestone which is rock layer No.2 its effective modulus calculated from Equation 9 is

\[ E_2 = \frac{3.5\times 10^4 + 2\times 0.8\times 10^4 + 6.5\times 0.2\times 10^4}{2\times 10^4} = 1.75 + 2.5 + 32.5 = 0.33 \times 10^4 \]

The same is listed in col.4 (Table 5) against flaggy limestone. The values of effective modulus for other rock layers are similarly computed and listed in column 4.

The coefficients of elastic uniform compression is determined using equation 10 (Barkan, 1962)

\[ C_u = 1.13\left(\frac{\bar{E}}{1-\nu^2}\right) \frac{1}{\sqrt{A}} \text{ (kg/cm}^2) \]

where \( C_u \) = coefficient of elastic uniform compression
\[ \bar{E} = \text{effective elastic modulus of the layered rock mass computed as indicated above, col. 4(Table 5)} \]
\[ \nu = \text{Poisson's ratio} = 0.25 \text{ (assumed)} \]
\[ A = \text{area of foundation in sq.m. (taken as 10 sq.m.)} \]

The values of \( C_u \) computed as above are listed in column 5 of Table 5. The coefficients of uniform shear, \( C_F \) is taken half of \( C_u \). The same are listed in column 6 (Table 5).

The bearing pressure of rock mass is determined from permissible settlement of 1.25 cm for a foundation area of 40 sq.m. using Equation 11.

\[ q = C_u \times 1.25 \times \frac{10}{40} \]

The value of \( q \) is calculated from Equation 11 considering area, \( A \) equal to 40 sq.m. The bearing pressures are shown in column 7(Table 5).

Table 8 summarises the recommended values of allowable bearing pressures and elastic constants for rocks at different depths from various considerations. The effect of gouge on these

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Bearing pressure from field tests (t/m²)</th>
<th>Bearing pressure from classification (t/m²)</th>
<th>Recommended bearing pressure (t/m²)</th>
<th>Recommended ( C_u ) (kg/cm²)</th>
<th>Recommended ( C_F ) (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive limestone</td>
<td>62.5</td>
<td>160</td>
<td>60</td>
<td>12</td>
<td>6</td>
</tr>
<tr>
<td>Flaggy limestone</td>
<td>52.5</td>
<td>75</td>
<td>50</td>
<td>9</td>
<td>4.5</td>
</tr>
<tr>
<td>Shaly limestone</td>
<td>-</td>
<td>48</td>
<td>45</td>
<td>7</td>
<td>3.5</td>
</tr>
<tr>
<td>Pink shale</td>
<td>197.5</td>
<td>46</td>
<td>40</td>
<td>6</td>
<td>3</td>
</tr>
</tbody>
</table>
parameters has been indirectly considered in field tests and also in classification of rock mass. No need is felt for further reduction of bearing pressure and elastic constants on this account.

Peak Ground Acceleration and Acceleration Spectra

In each blast, the horizontal component of ground acceleration was measured at various distances from blast holes. Table 9 also gives the values of recorded acceleration and P-wave velocities observed during these tests.

Table 9 Blast Test Details

<table>
<thead>
<tr>
<th>Intensity of blast used (kg)</th>
<th>Distance Peak ground acceleration of pick-up from explosion (cm/sec²)</th>
<th>Peak wave velocity (m/sec)</th>
<th>Point (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>23</td>
<td>56</td>
<td>208.46</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>122</td>
<td>61.66</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>50</td>
<td>564.07</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>100</td>
<td>35.67</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>50</td>
<td>603.69</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>128</td>
<td>134.69</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>100</td>
<td>3335.04</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>150</td>
<td>350.35</td>
<td>3300</td>
</tr>
</tbody>
</table>

From the observed values of peak ground acceleration, and compression wave velocity, Vc, the weight of the charge, W per delay and the distance, R from the centre of the blast a plot is made between dimensionless parameters (a²W/L³/Vc²) and R/(W¹/³) (Fig.4). The trend of variation these quantities on a log-log plot is linear and is represented by Equation 12.

\[
\frac{a^2W^{1/3}}{V_c^2} = 0.239 \left( \frac{-R}{W^{1/3}} \right)^{-2.42} \quad \ldots (12)
\]

Weight W can be expressed in terms of volume as \( Q \times \text{RBS/V_e} \) where \( Q \) is the weight of the explosive, R.B.S. is the relative weight strength expressed as ratio and \( V_e \) the density of explosive. In the present investigation 80% gelignite was used having R.B.S.=0.78 and density, \( V_e = 1.5 \text{ g/cc} \)

Thus \( a = 7.28 \times 10^6 \left( \frac{Q}{0.47} \right) \left( \frac{R}{2.42} \right) \ldots (13) \)

where \( a \) = peak ground acceleration in cm/sec²

Q = intensity of blast charge per delay in kg
R = distance of the structure from blast point in metres

From the recorded accelerograms of the different blast tests, spectral accelerations for structures of different periods and damping are worked out. Using these values, the design acceleration spectra has been obtained (Figure 5).
Earthquake Response Spectra

The proposed site lies in seismic zone I according to the Seismic Zoning Map of India incorporated in the Indian Standard Criteria for Earthquake Resistant Design of Structures (Third Revision) (IS:1893-1975). The centres of seismic activity lie far away from the project site.

Many other minor shocks have occurred but those are unimportant as far as the present project site is concerned. The other large earthquakes of the Peninsular Shield of India have larger epicentral distances from the project site. The probable peak ground acceleration at the project site produced by the three recorded earthquakes are all less than 0.005 g. Assuming a uniform focal depth of 33 km for each of the earthquake and using Equation 14 for peak ground acceleration applicable over hard rock sites.

\[
a_{\text{max}} = \frac{0.066 \times 10^{0.4M}}{R^{1.39}}
\]

where \( M \) is magnitude and \( R \) is focal distance in km from the site. The peak acceleration are thus quite small. Also there is no important seismic lineament near the site. The contact between the areas of unclassified crystalline rocks of Archean age and Deccan Trap Basalts cannot be considered seismically active on the basis of present evidence alone. Hence the project site can be safely considered to lie in a relatively stabler portion of the Deccan shield.

Under these circumstances, it will be advisable to adopt the response spectra as given in the code IS:1893-1975(Third Revision) for earthquake resistant design of structures. These spectra incorporate in them the seismic zone factor \( F = 0.05 \) for seismic zone I). These are shown in Fig. 6. The seismic coefficient calculated from Fig.6 has to be multiplied by the coefficient depending upon the soil foundation system (Table 10) and a coefficient I depending upon the importance of the structure (Table 11). The value of \( I \) may be taken as unity as the rock is available at site at shallow depths. The coefficient I may be chosen suitably by the designer depending on the importance based on economy, strategy and other considerations(IS:1893-1975).

Fig.6 Recommended design spectra for earthquake

If ductile deformations and an increase of 33 percent in allowable stresses are permitted, the structures designed based upon the spectra shown in Fig.5 will withstand a peak ground acceleration of 3.36 g for \( I = 1 \) and \( \beta = 1 \). When \( \beta I > I \) the peak ground acceleration which could be withstood will be \( \beta I \) times the above value viz. 3.36\( \beta I \). The above calculation assumes a ductility ratio of 4 and a load factor of 1.4.

CONCLUSIONS

A properly chalked out geotechnical investigation programme is essential in estimating various rock parameters for design of foundations of a cement factory. In-situ testing was necessary in view of the complex nature of the deposit. Laboratory tests on rock cores provide a useful method to determine rock characteristics and parameters for design. Blast tests have been used to obtain blast spectra. In view of location of site, earthquake response spectra is needed for design of structures. Thus the detailed geotechnical investigation programme has proved successful in estimating various rock parameters for design and also spectra for design of structures.
Table 10 - Values of $\beta$ for Different Soil-Foundation Systems

<table>
<thead>
<tr>
<th>Type of soil mainly constituting the foundation</th>
<th>Values of $\beta$ for</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile passing through any soil, resisting on soil type I</td>
<td>Pile not covered under any soil, resisting on soil type I</td>
</tr>
<tr>
<td>Type I Rock or Hard soils</td>
<td>1.0</td>
</tr>
<tr>
<td>Type II Medium soils</td>
<td>1.0</td>
</tr>
<tr>
<td>Type III Soft soils</td>
<td>1.0</td>
</tr>
</tbody>
</table>

NOTE: The value of $\beta$ for dams shall be taken as 1.0

Table 11 - Values of Importance Factor, $I$

<table>
<thead>
<tr>
<th>Structure</th>
<th>Values of Importance factor, $I$ (see note)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Containment structures of atomic power reactors for preliminary design</td>
<td>6.0</td>
</tr>
<tr>
<td>Dams (all types)</td>
<td>2.0</td>
</tr>
<tr>
<td>Containers of inflammable or poisonous gases or liquids</td>
<td>2.0</td>
</tr>
<tr>
<td>Important service and community structures, such as hospitals; water towers and tanks; schools; important bridges; important power houses; monumental structures; emergency buildings like telephone exchange and fire brigade; large assembly structures like cinema, assembly halls and subway stations</td>
<td>1.5</td>
</tr>
<tr>
<td>All others</td>
<td>1.0</td>
</tr>
</tbody>
</table>

NOTE: The values of importance factor, $I$ given in this table are for guidance. A designer may choose suitable values depending on the importance based on economy, strategy and other considerations.
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