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VOLUME IV
THEME 2

Engineering geological problems of tunnelling and excavation of cavities.

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INDE—1982

VOLUME IV
THEME 2
Engineering geological problems of tunnelling and excavation of cavities
Les problèmes de géologie de l’ingénieur du percement et de l’excavation des cavités

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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 5000 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 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
## CONTENTS

Theme 2. Engineering Geological Problems of Tunnelling and Excavation of Cavities

2.1. Prediction of Hazards and Problems of Tunnelling and Excavation of Cavities

<table>
<thead>
<tr>
<th>No.</th>
<th>Title</th>
<th>Authors</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>GEOTECHNICAL CONSIDERATIONS FOR RAPID TUNNELLING TECHNIQUES—</td>
<td>Anthony R. Umney, Ian McFate-Smith and Victor D. Turner</td>
<td>IV. 1</td>
</tr>
<tr>
<td>2.</td>
<td>UMBRELLA ARCH METHOD FOR TUNNELLING IN DIFFICULT CONDITIONS—</td>
<td>Giampiero Barisone, Sebastiano Pelizza and Bruno Pigorini</td>
<td>IV. 15</td>
</tr>
<tr>
<td>3.</td>
<td>STATE OF ART ON TUNNELLING THROUGH SOFT SOILS USING PIPE JACKING TECHNIQUE—</td>
<td>Narendra Singh and O.K. Singhal</td>
<td>IV. 29</td>
</tr>
<tr>
<td>4.</td>
<td>ROCK BREAKING BY MICROWAVE RADIATION—</td>
<td>R. Okamoto, H. Sugahara and I. Harano</td>
<td>IV. 43</td>
</tr>
<tr>
<td>5.</td>
<td>A SHALLOW TUNNEL IN A HETEROGENEOUS GROUND IN ROME (ITALY)—</td>
<td>Beniamino D’Elia, Giovanni Federico and Liborio Rivera</td>
<td>IV. 58</td>
</tr>
<tr>
<td>6.</td>
<td>SOME GEOTECHNICAL CONSIDERATIONS ON TUNNELLING IN HIMALAYAS—</td>
<td>Pratap Singh and J.L. Mahajan</td>
<td>IV. 65</td>
</tr>
<tr>
<td>7.</td>
<td>GEOTECHNICAL PROBLEMS IN TUNNELLING AND CONSTRUCTION OF UNDERGROUND POWER HOUSE AT THE PENCH HYDRO-ELECTRIC PROJECT, TOTLADOH IN MAHARASHTRA STATE, INDIA—A.R. Patil</td>
<td></td>
<td>IV. 75</td>
</tr>
<tr>
<td>8.</td>
<td>TUNNELLING EXPERIENCE AT TEHRI Dictates the Choice and Support Methodology of Underground Power House—G. Pant, Vinai Kumar and U.P. Gupta</td>
<td></td>
<td>IV. 83</td>
</tr>
<tr>
<td>9.</td>
<td>GEOLOGICAL SET-UP AND PREDICTION OF TUNNELLING PROBLEMS ALONG TUNNEL ALIGNMENT, LOKTAK HYDEL PROJECT, MANIPUR, INDIA—Apurba Kanta Choudhury and Balaram Chattopadhyay</td>
<td></td>
<td>IV. 98</td>
</tr>
<tr>
<td>11.</td>
<td>ENGINEERING GEOLOGICAL FUNDAMENTAL STUDY ON STABILITY OF SURROUNDING ROCK IN MINING UNDERGROUND EXCAVATION—Xu Bing and Li Yurui</td>
<td></td>
<td>IV. 111</td>
</tr>
<tr>
<td>12.</td>
<td>ENGINEERING GEOLOGICAL PROBLEMS RELATED TO TUNNELING IN THE OLOMOS-PINDOS ZONE OF WESTERN GREECE—G. Koukis and G. Tsiambaos</td>
<td></td>
<td>IV. 117</td>
</tr>
<tr>
<td>13.</td>
<td>MAKKAH INNER RING ROAD PROJECT—ENGINEERING GEOLOGICAL SITE INVESTIGATIONS FOR TUNNEL MIT-4—Vedat Doyuran</td>
<td></td>
<td>IV. 127</td>
</tr>
<tr>
<td>14.</td>
<td>EXCAVATION AND TREATMENT OF THE PRINCIPAL FAULTS IN THE TAILRACE TUNNEL OF THE RIO GRANDE I HYDROELECTRIC COMPLEX, ARGENTINA—Raúl Eduardo Sarra Pistone and Juan Carlos Del Rio</td>
<td></td>
<td>IV. 199</td>
</tr>
</tbody>
</table>
15. DISCUSSION ON ENGINEERING GEOLOGICAL PROBLEMS AROUND NANLING TUNNEL IN KARSTIC ROCK—Shi Wenhui IV. 153
16. COST AND BENEFIT OF SITE INVESTIGATIONS FOR TUNNELLING IN PORTUGAL—J.A. Rodrigues Carvalho and Ricardo Oliveira IV. 167
17. RISK ASSESSMENT AND CONTROL ON TUNNELLING CONTRACTS IN THE UNITED KINGDOM—P.B. Attewell, C.R. Clark and W.B. Norgrove IV. 175

2.2. Quantitative Appraisal of Hazards and Problems of Tunnelling and Excavation of Cavities

18. RESISTIVITY ELECTRICAL PROSPECTING AND THE STUDIES OF TUNNELS—Angel Garcia Yagüe IV. 183
19. ANALYSE DES FACTEURS QUI CONDITIONNENT LES PREVISIONS DES ALEAS EN TUNNEL—Marc Durand IV. 195
20. IMPORTANCE OF DETAILED MORPHOTECTONIC AND GEOLOGICAL STUDIES AS A MEANS OF PREDICTING POTENTIAL HAZARDS AND PROBLEMS IN TUNNELLING AND SITE INVESTIGATIONS—Piyadasa Wijeratne Vitanage IV. 203
22. PROBLEMS DURING CONSTRUCTION OF THE VÅRÐ Tunnel—a 2.6 km LONG SUBMARINE ROAD TUNNEL—Ariid Palmström IV. 231
23. AN ASSESSMENT OF THEORETICAL MODELS FOR THE DESIGN OF MINE TUNNELS—Raghu N. Singh, Barrie T. Wells and Ahmed M.H. Zadeh IV. 245
24. LA SIMULATION ARTIFICIELLE DU PROCESSUS D'ALTERNATION DES ROCHES EN VUE DE LA CONSTRUCTION DES TUNNELS ET DES EXCAVATIONS PROFONDES—L. Jorg IV. 253
25. SEISMIC EVALUATION OF ROCK ELASTICITY IN TUNNEL MEDIA—V.N. Srivastava IV. 263
26. SEISMOLOGICAL STUDY OF THE ROCK BURSTS AT THE KOLAR GOLD FIELD, INDIA—S.K. Guha IV. 271
27. A PRELIMINARY STUDY ON THE PHENOMENON OF CORE DISCING—Tao Zhen-Yu IV. 281
GEOTECHNICAL CONSIDERATIONS FOR RAPID TUNNELLING TECHNIQUES

CONSIDERATION GEOTECHNIQUES ASSOCIEES AUX MOYENS RAPIDES D'EXCAVATION DE TUNNEL

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ABSTRACT

Comprehensive site investigations are essential for the accurate design and planning of tunnels and for selecting the most suitable methods of construction. This paper outlines important aspects of designing site investigations for tunnels and reviews the full range of mechanised methods of excavation available relative to the ground conditions for which they are most suited. Comment is also made on the selection of tunnel support systems and methods of ground control for both soft ground and rock tunnelling.

INTRODUCTION

For industrial development in Asia the need to improve transportation, water and power resources is creating a strong demand for tunnels. The advantages of tunnels to obtain short, direct routes at acceptable gradients and free of adverse climatic conditions are considerable.

In the West the inherent disadvantages of conventional methods of excavation together with increasing costs, improved technology and the need for rapid tunnelling methods have led to the development and widespread use of machines. Technology and advanced techniques such as this are now available for tunnelling in Asia, and it is to our advantage to make full use of these.

The influence of geological conditions on performance using modern tunnelling methods cannot be too highly stressed. This controls the progress rates gained and has a major bearing on the costing and planning of tunnel projects.

For example the geology of the tunnel route is the key factor controlling
the economic application of tunnelling machines. In the past machine design has followed one of two trends namely, the soft ground shield used for support purposes, and the hard rock tunnel boring machine (TBM) - employed primarily to cut rock. However within the last 20 years a considerable wealth of experience has been gained on the design and application of intermediate machines that can be successfully employed in more varied ground conditions. These machines are now capable of cutting some of the hardest rock and excavating in the most arduous support conditions to be encountered in tunnelling. Although extensive geotechnical data is not a pre-requisite for their installation a poorly conducted investigation will limit the methods employed to the more conventional techniques.

Clearly, as each tunnel is unique it must be judged on its own merits. The initial cost and time required for the installation modern techniques such as machines must be balanced against the very much higher progress rates gained and reduced running, overbreak and support costs.

SITE INVESTIGATION

A well executed site investigation programme is essential for the design of the tunnel works, for the contractors estimating and for the selection of the most suitable form of contract. The cost of such a programme, being an integral and essential part of the project, must be related primarily to the geology of the tunnel route as opposed to the total project cost. Site investigation programmes should be phased and tailored to suit the particular geological conditions encountered and the expected timing of the tunnel contracts. One possible approach is outlined as follows:

1. Feasibility Stage:
   a. Existing data survey
   b. Aerial and surface mapping
   c. Initial boreholes - vertical

2. Major Investigation Stage:
   a. Final surface mapping in key areas
   b. Main borehole investigation
   c. In situ testing

3. Additional Investigation:
   a. Laboratory rock testing
   b. Additional boreholes in key areas if required

The site investigation should be carried out by the engineer, or at least fall within his responsibility. This will allow him to direct the investigations according to his design requirements. Particularly as the engineer, as opposed to the contractor, has more time, facilities and finances available to carry out a well directed comprehensive study.

Investigation Techniques

Borehole drilling is the most acceptable method of investigation for tunnelling and full advantage should be taken of every borehole drilled in terms of accurate logging to accepted standards (GEOLOGICAL SOC. OF LONDON - 1970), in situ testing and more varied use of orientated boreholes. The logging of borehole cores should be undertaken by trained engineering geologists. Preferably those who are familiar with the design of the tunnel and difficulties of the construction method. The geologist is also the person most suited to interpretate the condition of the rock mass from borehole data, and therefore to write the geotechnical appraisal to be submitted to tendering contractors.

An initial geophysical or aerial photographic survey can provide an overall assessment of the geological structure of the tunnel route which is not easily obtained by other means. It will also help to locate the position of zones of adverse conditions but not their nature or extent which must be examined by other means. For rock tunnelling additional orientated boreholes i.e. horizontal or inclined boreholes can be used most profitably for this purpose and to obtain data on jointing and other rock mass features not always obtained from vertical boreholes as outlined by COATES, CARTER and McFEAT-SMITH (1977).

The use of experimental tunnels may be acceptable for large projects to examine ground behaviour by intensive monitoring programmes (WARD, COATES and TEDD - 1976). However the use of pilot tunnels driven in advance of the main tunnel would not be recommended except in the most extreme circumstances due to the time, effort and cost involved.

Borehole testing for soils consist largely of standard penetration, piezometer and permeability tests. Stress-strain analysis can be carried out on site by the Menard pressuremeter test although this is more commonly carried out by laboratory tri-axial tests. Borehole testing in rock consists principally of field permeability assessment. Laboratory testing for rock
tunnelling is mainly carried out to assess rock drillability and machineability. For rock tunnelling by machine the precise tests required will depend upon the machines to be used (McFEAT-SMITH and TARKOY - 1980A).

Geotechnical Reports

Site investigation reports should be written by the Engineer using local expertise where necessary. Reports structured from the investigation data normally come under the following headings:

1. General Geology, including detailed sections of the tunnel geology.

2. Engineering Geology
   a. Borehole logs
   b. Summary of Engineering Geology classifications
   c. Joint & fault survey
   d. Hydrogeology

3. Tunnel Driving Conditions

4. Rock and Soil Machineability

5. Rock and Soil Testing Data

These reports would be issued as part of the tender documents. They can then be used as a satisfactory basis for measurement if the type of contract employed warrants this.

Role of Site Investigation Data in Contracts

Of the number of types of contract available the measurement type where the owner accepts the geological risk, offers most advantages to both owner and contractor. Here the former can expect a lower tender price, whilst the contractor can expect to receive additional payment when unexpected geological conditions are encountered. When working at its best the method of making 'Claims' for these payments (ICF - 1973) provides a satisfactory solution to both parties. However, an increasing frequency of disputes have arisen in recent years, (CIRIA - 1978) and this is leading to the development and use of geological classification systems for this purpose.

TUNNEL EXCAVATION

Methods adopted for excavating tunnels are dependent on the ground conditions and can conveniently be divided into those appropriate for rock and for soft ground. These merge in the case of soft or friable rocks. Many of the methods described are applicable to both types of ground. The aim of continuing development especially in mechanical excavation methods is to make them applicable to as wide a range of soil and rock types as possible.

Tunnels in any material have traditionally been excavated by hand. Today this tradition continues although the pick and shovel of former times have given way to pneumatic tools and this development alone has made more consistent rates of advance possible. The method of excavation selected for any tunnel requires consideration based on a number of factors including:

1. The ground conditions, especially where fully mechanised systems are contemplated.
2. The length of drive.
3. Methods of preventing the ingress of water in permeable materials situated below the water table.
4. The cost and availability of labour.

Some of these factors are inter-related and in the description of specific methods which follow, their relative importance to each method is emphasised.

SOFT GROUND TUNNELLING

For short drives of less than 0.3km where construction can take place in free air, for example through stiff clays, the traditional method of excavation by hand and the erection of a segmental lining is normally best. However this depends on the diameter. For large diameters or where ground conditions are poor a shield will be required regardless of length of drive. As the length of drive increases the level of mechanisation may be increased. For drives from 0.3 to 2.0km shield erector and spoil removal conveyor can be used to advantage. Finally for drives in excess of 2km a fullface machine may be the most favourable solution. Table 1 shows a general comparison between the various methods of excavation available. An extensive range of shields is currently available for different geological conditions. Figure 1 shows one type ideal for excavation of clayey sands and silts.

This type of machine provides the capability for fairly rapid advance rates at reasonable initial cost. It is therefore most suited for drives of the order of 1km in length. Face support, usually of timber, must be decided upon as the face advances, face rams being provided for this purpose.
<table>
<thead>
<tr>
<th>Item</th>
<th>Manual excavation</th>
<th>Shield-manual or backhoe or boom excavation</th>
<th>Full face machine</th>
<th>Full face slurry/bentonite machine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Costs</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial cost</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
<td>Very high</td>
</tr>
<tr>
<td>running cost</td>
<td>High</td>
<td>Medium</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>Rate of Advance m/100hr wk</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Favourable ground</td>
<td>30</td>
<td>100</td>
<td>400</td>
<td>75</td>
</tr>
<tr>
<td>Mixed ground</td>
<td>10-15</td>
<td>10-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>New equipment delivery time</td>
<td>Available</td>
<td>6-9 months</td>
<td>12 months</td>
<td>12 months</td>
</tr>
<tr>
<td>Installation time</td>
<td>Nil</td>
<td>4 weeks</td>
<td>8-10 weeks</td>
<td>15-20 weeks</td>
</tr>
<tr>
<td>Space required</td>
<td>No restriction</td>
<td>Shaft or chamber 1.2 x tunnel diameter</td>
<td>Shaft or chamber 1.2 x tunnel diameter</td>
<td></td>
</tr>
<tr>
<td>Application</td>
<td>Traditional method for short drives</td>
<td>Mixed ground conditions</td>
<td>Homogeneous ground conditions</td>
<td>Water bearing Homogeneous ground</td>
</tr>
<tr>
<td>General access for operations and maintenance</td>
<td>Good</td>
<td>Good</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td>Tunnel size, m.</td>
<td>Any</td>
<td>Any</td>
<td>Any</td>
<td>2.5 min</td>
</tr>
<tr>
<td>Support/lining type</td>
<td>In-situ, segmental</td>
<td>Segmental</td>
<td>Segmental</td>
<td>Segmental</td>
</tr>
</tbody>
</table>

**Table 1** Comparison of different methods of soft ground tunnelling

### Tunnelling Machines

A range of machines are available for excavation of soils and soft rock although it is the fullface type that offers greatest scope for rapid continuous tunnelling. An example of a machine for soft rock is the Priestley 5.3m diameter TBM shown on Figure 2. This was designed to excavate the English Channel tunnel (TRRL-1977). The tunnel route was in Lower Chalk, which is soft, competent material, ideal for the installation of this type of machine. For weak rocks like chalk, progress rates in the order of 400m per week are now possible. This type of average advance rate assumes the use of disc cutting tools and requires smooth continuity of materials and planned maintenance together with systematic installation of the final tunnel lining.

Modern machine technology is now able to offer a variety of solutions to the problem of supporting a water-logged or unstable face without the use of compressed air whilst continuing to advance the drive. The slurry machines shown on Figure 3 are now the most popular method of doing this and the system is most highly developed in Japan where at present over 30 of these machines are at work. Production of this type of machine with external diameters up to eight metres is presently in hand. In their present state of development they have provided one of the most acceptable solutions in this type of ground, however the presence of large cobbles and boulders would limit their use.

IV.4
Figure 1 Tunnelling shield with bucket excavator

Figure 2 Shielded TBM for soft rock tunnelling

Figure 3 Mitsubishi slurry shields
In 'confined soil machines' suitable for soft clays and silts, excavated material plugs the cutter chamber to create earth pressure during excavation. The excavation material is removed by a screw conveyor and this by its consolidation action prevents any ingress of water from the face. A bulkhead completely isolates the face and offers complete protection to the tunnelling workers. This type of machine may also be used in the confined plenum process. With this system only the area between the bulkhead and the face is pressurised and the rest of the tunnel is at normal atmospheric pressure. Further modern methods of excavation available for soft ground are the mini tunnel system and pipe jacking techniques (LENEY - 1979).

<table>
<thead>
<tr>
<th>Item</th>
<th>Drill &amp; Blast</th>
<th>Roadheader</th>
<th>Full face machine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost 1978 Great Britain</td>
<td>Low/medium High</td>
<td>Low/medium Low</td>
<td>High Medium</td>
</tr>
<tr>
<td>Typical progress m/100hr. wk.</td>
<td>50</td>
<td>70</td>
<td>180</td>
</tr>
<tr>
<td>Favourable ground</td>
<td>30</td>
<td>35</td>
<td>20 - 180</td>
</tr>
<tr>
<td>Faulted ground</td>
<td>1 month</td>
<td>2 months</td>
<td>12 months</td>
</tr>
<tr>
<td>Installation</td>
<td>2 weeks *</td>
<td>2 weeks *</td>
<td>4-6 weeks</td>
</tr>
<tr>
<td>Tunnel shape</td>
<td>Any</td>
<td>Most</td>
<td>Circular</td>
</tr>
<tr>
<td>Application</td>
<td>Most rock types</td>
<td>Dependent upon rock hardness and abrasivity</td>
<td>Most rock types</td>
</tr>
<tr>
<td>Overbreak</td>
<td>High</td>
<td>Medium</td>
<td>Low</td>
</tr>
<tr>
<td>General access</td>
<td>Good</td>
<td>Good</td>
<td>Fair</td>
</tr>
<tr>
<td>Temporary support</td>
<td>Face</td>
<td>Face</td>
<td>3-12 metres behind face</td>
</tr>
<tr>
<td>Position installed</td>
<td>Any</td>
<td>Any</td>
<td>Shutting difficult</td>
</tr>
<tr>
<td>Type</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter range (metres)</td>
<td>Any</td>
<td>2 - 6</td>
<td>1.8 - 10.7</td>
</tr>
</tbody>
</table>

* depends on access

Table 2 Comparison of different methods of rock tunnelling

ROCK EXCAVATION

Drill and blast excavation remains the conventional method excavation in rock tunnelling due to the ability to cope with different tunnel dimensions and with varied ground conditions. Recent developments include the use of larger drilling rigs, the implementation of hydraulic rotary-percussive drills and improved mucking facilities.

In spite of these improvements the advance rates to be gained remain low in comparison with fully mechanised tunnelling. Table 2 shows a general comparison between drill and blast excavation and that of several types of machine. Once initial cost and installation problems are overcome machine excavation gives substantially higher advance rates with reduced running, support and overbreak costs, and savings from the proportion of skilled labour employed.

Machine excavation avoids excessive vibrations from blasting giving little or no effect on buildings, improved safety conditions and leaving a perfect circular section that is statically more stable than rough blast sections. The differences in overbreak can be so large that savings obtained after 2 kms of tunnelling can cover the additional cost of a roadheader, and for a hard rock machine after 7kms of tunnelling. Often in argillites, and other such rocks adversely affected by blasting the high speed excavation and minimum ground disturbance achieved by machines will prevent difficult support situations occurring.

IV.6
Roadheader Tunnelling Machines

These are the most widely used type of tunnelling machine in use today. They employ a flexible cutting, mucking and tracking system as shown on Figure 4. This flexibility together with their relatively low purchase price and running costs, has been their main attraction for the Civil Engineering Industry. Within the last 7 years there has been a demand to develop bigger and more robust roadheaders with higher excavation capacities and to cut harder rock. Machines such as this have achieved advance rates of over 130m/week and recently a machine averaged 70m/week in a 1:4 down gradient tunnel through beds of hard sandstone (ANON-1978). Other machine types include slim line machines for narrow tunnels, machines for low roof tunnels, shielded machines used for the installation of segmental lining (Figure 5) and twin boom machines for high production mining.

Figure 4 Heavy-weight roadheader

Figure 5 Shielded roadheader
From Table 2 it can be seen that the roadheader system retains many of the advantages of drill and blast excavation as it is easily integrated with tunnelling operations. Installation of all types of temporary support can be made immediately at the face where it is most effective in preventing loss of ground. It is not prone to major delays when broken or faulted ground is encountered. When required, blasting, grouting and advance probing can be accomplished with a machine mounted drill.

Good access is available for planned maintenance and repairs to most working parts of the machines. As a result consistently high utilization rates of 40-60% can therefore be achieved. Cutting rates vary more directly with rock conditions and are therefore the key feature in the prediction of performance. Estimation of progress is made for each rock unit to be encountered in a tunnel from the product of the predicted cutting rates and utilization rates. Experience has shown that these can be very high. To avoid unnecessary risks and to obtain accurate prediction data the correlations given on Figure 6 have been developed from detailed field and laboratory measurements (McFEEAT-SMITH and FOWELL - 1979).

Using the appropriate laboratory rock testing we can therefore accurately assess the cutting performance of roadheaders for the widest range of rock types and ground conditions. This assessment is made in terms of field cutting rates which are used in conjunction with system analysis data to predict overall advance rates, and cutting tool costs. Further charts are available to check the overall applicability of utilizing the roadheader cutting system relative to their limits of application. This type of approach is important to avoid unnecessary risk and ensure the successful installation of machines.

Tunnel Boring Machines (TBM's) for Hard Rock

Rapid development of this type of machine has been made during the last decade in the United States and Europe. As a result, considerable controversy has taken place over the most suitable elements of machine design for different geological conditions. A selection of machine designs is available for this purpose and for different tunnel diameters (SUGDEN - 1975). The minimum economic tunnel diameter is 1.8 metres whilst the maximum diameter machines produced to date are 10.8m. Figure 7 shows one of several large diameter machines employed successfully on the TARP tunnels in Chicago, USA.

For rock tunnels where a substantial proportion of the tunnel is expected to be in adverse support conditions hard rock machines with full shields are now available. These 'Hybrid' machines allow the erection of primary segmental lining in adverse ground yet retain their hard rock cutting abilities and flexibility for thrusting and steering under less arduous conditions (GRANDONI - 1976).
For the more common type of 'open' machine shown on Figure 8 temporary support used in conjunction with insitu concrete provides the most economic form of excavation. Table 3 shows a simplified example of system analysis data obtained from this TBM. This was recorded in a tunnel driven in a highly variable sequence of sedimentary rocks containing many fault gouge zones and dolerite dykes. As shown on the table progress rates in the order of 120–150m/week were achieved in a number of sections. This gave highly economic tunnelling rates. When initially used on the project the TBM had previously excavated four tunnels. With the modern equivalent of this machine we would expect an increase of 30–100% in the advance rates quoted.

When the installation of these machines is contemplated a brief knowledge of the principal design features that affect the contractor's ability to carry out his tunnelling operation is essential. Design features which are advantageous (McFEAT-SMITH and TARKOY - 1980A) are summarised as follows:--

![Figure 7 Large diameter TBM](image)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Sandstone</th>
<th>Limestone</th>
<th>Sandstone</th>
<th>Mudstone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist. from portal (km)</td>
<td>0.5</td>
<td>3.0</td>
<td>5.5</td>
<td>7.0</td>
</tr>
<tr>
<td>Test length (m)</td>
<td>200</td>
<td>560</td>
<td>750</td>
<td>210</td>
</tr>
<tr>
<td>ANCIENT WORK</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maintenance %</td>
<td>10.5</td>
<td>22.3</td>
<td>14.2</td>
<td>6.5</td>
</tr>
<tr>
<td>Support/Invert %</td>
<td>1.2</td>
<td>3.9</td>
<td>9.3</td>
<td>18.4</td>
</tr>
<tr>
<td>Services %</td>
<td>19.5</td>
<td>20.4</td>
<td>19.4</td>
<td>14.2</td>
</tr>
<tr>
<td>DELAYS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mucking/water %</td>
<td>26.8</td>
<td>2.2</td>
<td>5.7</td>
<td>7.4</td>
</tr>
<tr>
<td>Machine E&amp;M %</td>
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<td>2.0</td>
<td>2.6</td>
<td>3.1</td>
</tr>
<tr>
<td>Miscellaneous %</td>
<td>9.2</td>
<td>10.1</td>
<td>13.8</td>
<td>14.4</td>
</tr>
<tr>
<td>Utilization %</td>
<td>28.1</td>
<td>39.1</td>
<td>35.0</td>
<td>36.0</td>
</tr>
<tr>
<td>Progress m/wk</td>
<td>118.0</td>
<td>140.8</td>
<td>155.4</td>
<td>125.3</td>
</tr>
</tbody>
</table>

Table 3 Shift analysis data for 3.5m diameter TBM.
1. Lightweight rigid cutting structures employing single disc cutters on a well balanced counterhead.

2. One set of gripper pads at axis level.

3. Easy access for ancillary operations. For tunnels likely to intersect major gouge zones additional facilities for forepoling above the cutter-head and conventional tunnelling at the face are essential. A short, wide access route to the face is particularly desirable.

In the last decade significant developments have taken place in the ability of disc cutters and their bearings to sustain continuously high loadings and to absorb much higher shock loads in blocky rocks. This has given reduced cutter costs and greatly increased penetration rates in the harder rock types. Figure 9 shows a comparison of the penetration rates obtained from different cutting tool arrangements and different rock hardnesses. This illustrates that for all rock hardnesses the single disc on machine A gave a much better performance than the multiple tool arrangements on machine B. A study of cutter costs gave the same trend. The results clearly demonstrate that this was due to the most favourable application of the best cutting tool, and that the single disc cutter now sets the limits of economic cutting. For prediction of TBM cutting performance a number of rock strength or hardness tests can be used in conjunction with system analysis data to predict overall progress and costs (McPheat-Smith and Tarkoy 1980B).

Figure 8 Open, 3.5m diameter TBM

Figure 9 Chart for predicting TBM penetration rates after McPheat-Smith and Tarkoy 1979.
Shaft Excavation

Raised drills have been in use for a number of years now. They achieved notable success in even the hardest, most abrasive rock types such as the South African Gold Reef quartzites. A more recent development is the blind shaft borer shown on Figure 10. Machines of this type are applicable in the full range of rock types and have effectively minimised the time required for shaft construction.

TUNNEL SUPPORT SYSTEMS

The tunnel support system is an integral part of the excavation process and both systems are therefore inter-dependent. Consideration for the selection of different methods of support are their overall effectiveness relative to the prevailing geological conditions, safety, time required for installation and the cost of equipment and materials. For rapid excavation the combination of temporary support and concrete cast in situ generally gives the most economic system. Alternatively, where poor quality ground is encountered segmental lining can be used to give a stable excavation.

Temporary Support

Whilst liner plates or arch ribs and lagging may be used in certain favourable soil conditions it is more normal to erect a permanent segmental lining immediately. Temporary support is not generally used in tunnels in soil. For rock a range of types of temporary supports are available to cope with the wide range of conditions encountered. These include rock bolts combinations of rock bolts and mesh, shotcrete, reinforced shotcrete, steel arches, arches and lagging and arches with lagging and shotcrete.

Temporary support systems are generally favoured for machine excavation in rock, where the open type of machine is used. Shotcrete can be used for this purpose in tunnels driven by roadheaders and the larger diameter TBMs. Rockbolts and mesh systems have been developed to take advantage of the relatively smooth bore provided and lightweight circular steel arches can be placed in direct contact with the rock without the use of timber packing.

Final Lining Design

The major problem facing the tunnel engineer is to accurately model the ground and lining interaction. Normally this is carried out using a plane strain analysis. The simplest way is to set up a plane frame analysis using a standard computer programme and to represent the ground by a series of elastic restraints at discrete nodes. This method ignores the fact that the ground offers continuous support and that the elastic stiffness of the ground is variable depending on relative displacements.

MORGAN (1971) has proposed methods of linear elastic analysis so that simple solutions can be easily obtained. The only ground properties required for such an analysis are elastic constants. In order to establish a more accurate ground model elastoplastic constants for both the ground and the movement of joints in the lining are required. Many theoretical studies are at present being undertaken to establish a simplified theory which can provide a useful design tool. The application of both empirical rules and some of the newer methods, such as boundary element analysis, levered with a fair degree of engineering judgement, should lead the designer to an acceptable solution.
PREVENTING WATER INGRESS

Excessive inflows of water through almost any soil type will result in a collapse. Permeability, grading and shear strength should be established. The extent of any soil type relative to the alignment is critical to the choice of tunnelling method. A thick and continuous layer of stiff clay, for example, will indicate that a mechanical fullface tunnelling machine could be used to advantage. Continuous layers of silt and sand with permeabilities in the order of 1 x 10^{-3} cm/sec would lead to the choice of a slurry or bentonite tunnelling machine. Where permeabilities greater than about 1 x 10^{-2} cm/sec are found then the use of ground treatment as an additional aid must be considered. Soils in this range are likely to dry out and slip under the influence of compressed air. There can also be difficulties in maintaining a stable face even with a bentonite shield.

There is now a wide range of grouting methods available to tunnel engineers. These vary from ground fracturing methods using cement-based grouts to chemical gels which fully permeate and consolidate the ground under controlled conditions. The normal approach is to assess where ground treatment is required and to carry out all necessary ground treatment from the surface.

Excepting tunnelling machines fitted with bulkheads there are four major methods of preventing water ingress which may be summarised as: compressed air, which remains the best alternative for short drive; injection of a grout for non-fines soils; dewatering in permeable soils where settlement would not be a problem; and ground freezing for very severe or unusual conditions.

GEOTECHNICAL WORK CARRIED OUT ON SITE

On-site testing is carried out in rock tunnelling to monitor rock strength and hardness. The Schmidt hammer point-load test and cone indenter are the most useful instruments available for this purpose. For soft ground tunnels, apart from instrumentation used to measure ground and lining movements, insitu testing is not common. Where dispute occurs large block samples are normally collected for testing in more established laboratories. During excavation of tunnels it is important that continuous engineering geological records are kept of the conditions encountered. A comprehensive system utilizing tunnel inspectors and geologists logs have been developed by MCFEAT-SMITH (1982) for this purpose.

CONCLUSIONS

The state of the art of defining the engineering geological properties of soils and rock masses is now at a relatively high level. Similarly techniques available for site investigation and tunnelling are more than adequate for our purposes. However, standards of site investigations, and the use made of geotechnical data in the financial structure of tunnel contracts and for assessing the performance of modern techniques, vary substantially from project to project. Clearly it is the implementation of the art that requires improving.

In certain geological terrains a poor standard of site investigation may discourage tunnelling contractors using some of the advanced techniques discussed even although they will be more economic than the more conventional alternatives. Where conditions are not adequately known contractors electing to use techniques such as machines must ensure that:

1. They have chosen the best equipment according to the known geological conditions.

2. Methods of ground control are available to give sufficient versatility to the system.

3. That the tunnelling crews are familiar with this equipment before, and not after, adverse conditions are encountered. These situations are often man-made and can be avoided by the use of the appropriate techniques.

Work presented in this paper has demonstrated that there is now available a wide range of modern equipment to deal with most geological conditions encountered in tunnelling. The writers' appreciation of these comes from the knowledge that, when some effort is made to optimise the system of tunnelling operations and level of planned maintenance, then record breaking results are gained.
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UMBRELLA ARCH METHOD FOR TUNNELLING IN DIFFICULT CONDITIONS—ANALYSIS OF ITALIAN CASES

LA METHODE DU PARAPLUIE DE TUBES POUR L'EXCAVATION DE TUNNELS DANS DES CONDITIONS DIFFICILES—ANALYSE DES CAS ITALIENS

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ABSTRACT

Several examples of tunnels driven for different purposes (roads, railroads, etc.) in grounds with stability problems, and sometimes with surface damage limitation problems too, are considered.

The tunnels considered fall in two groups, with somewhat different problems: tunnels whose portals are excavated in detritus from old or recent slides, and tunnels in sound rock that suddenly cross some band of intensely fractured by tectonic stresses rock, mylonitized rock or loose rock. In both cases the problems bear essentially on the stability of the excavation; in the first case, an additional difficulty comes from the anomalous thrust of the ground interested by the slide.

In all the cases that have been examined it has been possible to run regularly the excavation operations, without large modifications of the working place organization and with fairly good advance rates, by means of a preliminary ground consolidation with umbrella arch method.

The method relies on the construction of a support system composed of tubes that form an arch in the ground outside of the excavation profile, having the function of an in situ support placed in advance before the excavation.

The umbrella, that is built on lengths of about ten meters (8 - 16 m), is an arch of perforated steel tubes, with a diameter ranging between 120 and 180 mm, forming a truncated cone; cement grout is then injected at low pressure in the tubes.

A bearing structure is so obtained, that protects the vault of the excavation. If needed the protection can be extended on the flanks, and founded on ground pillars that have been beforehand consolidated by grouting.

With that sistem, that is comparatively simple and not too much expensive, no troubles have been found in the several works examined.

A special advantage resides in the fact that the excavation runs safely and without interruptions; moreover the loads are well distributed on the steel arches that are subsequently placed, and it allows to build the lining according the established time table, also in difficult ground.
ABSTRAIT

La communication examine plusieurs exemples de tunnels excavés dans des buts différents (routes, chemins de fer, etc.) dans des terrains caractérisés par des problèmes de stabilité et parfois aussi par des problèmes de limitation des affaissements en surface.

On peut classer ces tunnels en deux groupes, c'est à dire: tunnels dont les entrées sont excavés dans le détritus d'éboulements anciens ou récents, et tunnels excavés dans des roches saines et cohérentes dans lesquelles on trouve subitement des bandes de roches fracturées, milonitisées ou incohérentes.

Dans les deux cas, le problème principal est représenté par la stabilité de l'excavation; dans le premier cas une difficulté additionnelle est causée par la poussée anormale des terrains intéressés par l'éboulement.

Dans tous les cas examinés, il a été possible une exécution régulière des opérations d'excavation, sans modifier l'organisation des travaux et avec une bonne vitesse d'avancement au moyen d'une consolidation préliminaire du terrain réalisée avec la méthode du parapluie de tubes.

Cette méthode est basée sur la construction d'un système de tubes en forme d'arc (parapluie) dans le terrain; cet arc, exécuté avant l'excavation, a la fonction d'assurer la protection de la voute du tunnel pour un temps suffisant à la mise en place des cintrés. Plus en détail le parapluie, qui est construit sur une longueur de environ 10 mètres, est constitué par un système de tubes perforés de diamètre compris entre 120 et 180 mm, disposés en forme de cône; ensuite on injecte a basse pression, du mortier de ciment dans les tubes.

De cette façon on obtient une structure plus au moins continue qui protège le ciel de l'excavation; le cas échéant, la protection peut être construite aussi sur les côtés de l'excavation et reliée à la base à de pieux réalisés en consolidant le terrain avec injection de ciment.

Cette méthode, simple et pas trop coûteuse, a permis d'exécuter sans problème tous les tunnels examinés. De plus, le travail d'excavation est sûr et n'est pas soumis à des interruptions imprévues; les charges sont bien réparties sur les cintrés en acier qui sont successivement mises en place, et on peut bâtir le revêtement avec le délai qu'on desire, même en terrains difficiles.

1. Foreword

The umbrella arch method has been often employed in Italy and abroad to solve the difficult excavation problems that can be met in particular tracts of a tunnel.

We think therefore it could be interesting to review the method, through an analysis of several recent Italian examples; it is noteworthy that, thanks to technological improvement and simplifications in the construction stage, it has been possible to apply systematically the method in the excavation of long tracts of tunnel, under difficult conditions.

2. The umbrella arch

What we mean by "umbrella arch" is usually obtained by means of a series of holes, whose diameter ranges between 100 and 180 mm, that are drilled, evenly spaced, along the contour of the tunnel vault in such a way to form an half cone whose generatrices lie as much as possible parallel to the tunnel axis; similar holes can be drilled, if needed, also along the contour of the sides of the tunnel cross section.

In the holes are then placed steel pipes of suitable diameter, perforated along the lower side; grouting agents (usually water-cement grouts) are pressure injected in
the pipes.

The umbrella combines the advantages of a modern forepoling system and of a series of grouting injections; it allows the excavation to progress safely (and at comparatively high speed), with a full control of subsidences, through heterogeneous rocks having poor geo-mechanical properties, as alluvial deposits, moraines, mylonites. A limit to the useful application of the method is posed only by groundwater under high pressure.

3. Design elements

The load acting on the tunnel vault (and the horizontal thrust acting on the tunnel sides, if the umbrella is intended to protect also the sides) are first calculated with the usual procedures; the pipes composing the umbrella are then designed as almost horizontal, fixed end beams.

Static stability is usually checked by calculating the bending and shear stresses arising in the pipes from the anticipated loads, and taking into account only the steel section of the pipes as resisting section (a reductive factor is applied to account for the pipe perforations). It must be noticed that the umbrella forms, all around the tunnel vault, a shell of grouted and reinforced rock that, thanks to the arched shape, could sustain a part of the...
rock load; it is wise, however, to disregard in the stability calculations the help such a structure can give, because its effectiveness depends upon the perfect accomplishment of grouting and can not be warranted in advance (it can be checked only during the excavation work, too late to correct defects if any).

The maximum allowable span between umbrella supporting structures is calculated on the basis of the above mentioned shear and bending stress calculations; bending stresses pose the most severe limit to the span, under the usual load conditions. Stresses do not depend much upon the changes of hole spacing (that must be chosen according to the ground properties: grain size, permeability etc.); on the contrary, they are strongly influenced by the type of end constraints of the beams, therefore the correct construction of an umbrella arch requires a careful check of the grout injection through the pipes and an effective support-rock coupling. It can be safely supposed that the loads acting on the umbrella are evenly transmitted to the support, and steel arches must be chosen as support structure, because rock bolts do not represent a reliable and continuous support (however, rock bolts with large plates of rolled formed steel can be satisfactory); on the other side, rock bolts usually do not hold firmly in the rocks whose poor geomechanical properties dictated the umbrella arch support method. (Fig. 1, 2).
4. Construction procedure

The working cycle comprises the following steps:
- drilling
- placement of the pipes in the holes
- injection
- excavation and support placement

4.1. Drilling is performed by large (100-180 mm) drills, and requires usually from 3 to 5 lining up of the drilling machine (to obtain a 15-20 holes umbrella).

The most critical problem is represented by hole divergence from the tunnel axis, that should be as small as possible, never exceeding 15°. Actually some overlap between two subsequent umbrella arches must be warranted, so that the end of the first umbrella can provide stability to the excavation face and support the tunnel vault when the next umbrella must be built, thus allowing safe working conditions.

It is apparent that, as far as excavation economy is concerned, single umbrella arches span must be maximized. Unavoidable hole divergence however poses a limit to the span, because the influence radii of grouting of adjacent holes must overlap, and, moreover, unconsolidated rock must not remain between the umbrella arch pipes and the tunnel vault; it is not advisable to exceed the distance of 2 m between the tunnel vault and the umbrella arch pipes.

4.2. Placement of the pipes in the holes can be made difficult by caving of the hole wall, or by hole shrinking. A very effective method to overcome such difficulties
is to adopt an eccentric bit drill: in that case the pipe can follow the drilling bit, as drilling progresses.

Pipes usually employed are commercial steel pipes, in lengths of 3 or 6 m, that can be welded together in the contractor's workshop, or at the excavation face (when an eccentric bit drill is used), to obtain the desired length. Outside diameter of pipes commonly ranges between 80 and 140 mm, and wall thickness between 3 and 10 mm. Perforations in the pipe wall, through which the grouting mixture is injected in the rock, are preferably made only in the two thirds (away from the hole mouth) of the pipe length; excessive flow back of grout in the tunnel through the annulus between hole and pipe is so avoided.

It can be seen in Fig. 3 that the part of the umbrella arch close to the hole mouth must be destroyed as excavation resumes; in that part standard steel pipe is therefore advantageously replaced by a length of thin walled pipe (whose purpose is only to convey the grout) that can be easily cut during the excavation.

4.3 Grouting mixture injection does not require sleeve pipes, because a large diameter pipe allows to effect the injection along the full pipe length, without appreciable pressure drop.

Pressure injection can vary, being a function of cover thickness and of rock permeability; usually ranges between 3 and 10 bar.

In a number of cases we designed and inspected it has not been found necessary to resort on special grouting mixtures; cement-water grout proved always to be satisfactory, with a cement to water ratio ranging between 1/1 and 2/1 (sometimes it proved to be useful to increase the grout fluidity by adding a small

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**FIG. 4. POSITION OF TUNNELS IN A GEOGRAPHIC MAP OF NORTHERN ITALY**

1 - Fiumelatte tunnel
2 - Merone tunnel
3 - Delle Tanze tunnel
4 - Cernobbio tunnel
5 - Serre la Voue tun.
6 - Pietratalaglia tun.
7 - Spallanzani tunnel
<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Span /m/</th>
<th>Rock</th>
<th>Driving section</th>
<th>Drilling diameter /mm/</th>
<th>Holes inclination related to tunnel axis /°/</th>
<th>Holes length /m/</th>
<th>Holes spacing (horizontal projection) /m/</th>
<th>Nº pipes in one umbrella</th>
<th>External diameter of pipes /mm/</th>
<th>Pipes thickness /mm/</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiumelatte</td>
<td>10,5</td>
<td>Debris (limestone)</td>
<td>Half</td>
<td>95</td>
<td>15°</td>
<td>12</td>
<td>0,7</td>
<td>12</td>
<td>84</td>
<td>4,5</td>
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<td>8</td>
<td>Landslide (marl)</td>
<td>Full</td>
<td>150</td>
<td>15°</td>
<td>18</td>
<td>0,6</td>
<td>17</td>
<td>148</td>
<td>6</td>
</tr>
<tr>
<td>Delle Tanze</td>
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<td>Mylonite (limestone)</td>
<td>Full</td>
<td>95</td>
<td>15°</td>
<td>12</td>
<td>0,6-0,8</td>
<td>12-9</td>
<td>84</td>
<td>4,5</td>
</tr>
<tr>
<td>Cernobbio</td>
<td>10,5</td>
<td>Alluvium (gravel-cobbles)</td>
<td>Full</td>
<td>150</td>
<td>15°</td>
<td>18</td>
<td>0,5</td>
<td>18</td>
<td>148</td>
<td>6</td>
</tr>
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<td>Serre la Voute</td>
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<td>Landslide (gneiss)</td>
<td>Half</td>
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<td>15°</td>
<td>14</td>
<td>0,75</td>
<td>18</td>
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<td>10</td>
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<td>Pietratagliata</td>
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<td>Landslide (clay)</td>
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<td>4°</td>
<td>18</td>
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<td>10</td>
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<td>Spallanzani</td>
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<td>15</td>
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<td>22</td>
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### SYNTHESES TABLE OF ITALIAN CASES OF EXCAVATION WITH UMBRELLA ARCH METHOD

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Ribs type</th>
<th>Ribs spacing /m/</th>
<th>Spritz-beton thickness /m/</th>
<th>Injection grout (cement/water)</th>
<th>Injection pressure /bar/</th>
<th>Grout absorption of each tube /t/</th>
<th>Total time for execution of each umbrella /h/</th>
<th>Umbrella overlap /m/</th>
<th>Total length of intervention /m/</th>
<th>Excavation speed net /m/d/</th>
<th>Excavation speed gross /m/d/</th>
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<tbody>
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<td>0.3</td>
<td>2/1</td>
<td>12</td>
<td>2.5</td>
<td>24</td>
<td>4</td>
<td>640</td>
<td>4</td>
<td>3</td>
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<td>Merone</td>
<td>NP160 X2</td>
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<td>2/1</td>
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<td>185</td>
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<td>1-1.2</td>
<td>0.2-0.3</td>
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<td>3</td>
<td>1630</td>
<td>4</td>
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<td>2/1</td>
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<td>72</td>
<td>5</td>
<td>240</td>
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<td>2/1</td>
<td>3</td>
<td>2</td>
<td>60</td>
<td>4</td>
<td>160</td>
<td>3</td>
<td>2</td>
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<td>Pietratagliata</td>
<td>NP160 X2</td>
<td>0.5</td>
<td>0.2</td>
<td>2/1</td>
<td>10</td>
<td>2</td>
<td>80</td>
<td>4</td>
<td>32</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Spallanzani</td>
<td>NP140 X2</td>
<td>1</td>
<td>0.15</td>
<td>1/1</td>
<td>3</td>
<td>1.6</td>
<td>140</td>
<td>4</td>
<td>18</td>
<td>4</td>
<td>0.8</td>
</tr>
</tbody>
</table>
amount (5-10%) of bentonite.

4.4. As the umbrella arch is set, excavation can progress along the useful length of the umbrella without noteworthy cautions; care must be taken however to place very carefully the supports (as mentioned, support is usually provided by steel arches), so that the umbrella is immediately supported. Moreover, the span of the pipes must never exceed the designed length.

An hazardous working zone is the part immediately close to the face, where the pipes rest on the last arch, on one side, and on the partly consolidated core of the face on the other side; according to the character of the rock it can be necessary, at each advance step (that coincides with the interval between the steel arches, usually 0.5-1.5 m), partly support and consolidate the face with sprayed concrete or with injections of grout.

5. Examples of results obtained in Italian tunnels

The umbrella arch method in the last few years has been applied in Italy in about 15 cases; in the table enclosed the most relevant data of seven cases we investigated are collected. The examples have been chosen in order to show the full range of rock properties and excavation cross sections the umbrella arch method can cover; their geographic position is shown in Fig. 5.

5.1. The Fiumelatte tunnel, on the Lecco-Trivio di Fuentes road, is a twin bore road tunnel; each tunnel is 10.5 m wide, and the excavated section is about 100 m².

Both tunnels cross, in the first 600 m stretch, talus material with large limestone blocks, and some clay filling between the blocks; water circulation is practically absent.

Due to the large size of the blocks and to the moderate thickness of the cover, it has been possible to allow a broad spacing of the umbrella pipes; injection has been made with a rather thick grout (cement to water ratio 2/1), and grout absorption was, as a whole, moderate.

The average gross advance rate, before umbrella arch method adoption, working half a face at a time, was about 1 m/d; with the umbrella arch method it has been improved to 3 m/d, and no special troubles have been met.

5.2. The Merone (Como) tunnel is an haulageway linking a cement rock quarry to the cement factory; a conveyor belt has been installed in the tunnel, to carry the raw rock to the plant. Tunnel section is horseshoe shaped, with a width of 8 m (excavation section is about 70 m²).

Approximately 200 m of poor ground (chaotic landslide accumulations of marl fragments embedded in a mass of highly plastic clay), exerting a considerable thrust on the supports, had to be crossed; excavation started according to conventional methods, but had to be stopped at once, due to a serious cave-in who broke through the full thickness of the
cover.

The umbrella arch method was therefore adopted, resorting on pipes of great length injected with cement-water grout under high pressure; the umbrella arch allowed to resume the excavation with a standard full face procedure, and to obtain an average gross advance rate greater than 2 m/d.

5.3. The "Dalle Tanze" tunnel, on the Turin-Modane railroad, is
a single track tunnel, 6.5 m wide, with an excavation section of about 60 m²; the overall length of the tunnel is about 6 km.

Several bands of mylonite had to be crossed, totaling over 1600 m; mylonites were the result of tectonic stresses on limestone rocks, and had the appearance of very fine, incompetent sands, sometimes clay rich, including blocks of limestone and metamorphic rocks (calcareous schist, greenstone); also large size blocks have been found.

A cement-water grout has been employed; grout absorption was rather slight, due to the high compaction of the ground; it has been possible

Fig. 10 - . Serre la Voute tunnel.
The east entrance of one tunnel with his first umbrella arch.

Fig. 11 - Pietratagliata tunnel.
The entrance; on the upper right side the umbrella arch.

Fig. 12 - Spallanzani tunnel.
The cave in close to an entrance; on the lower side the pipes of the umbrella arch.
Fig. 13 - Spallanzani tunnel. An external view of the cave in, with the umbrella arch pipes (before filling and injection).

To drive the tunnel with full face excavation, at an average gross advance rate greater than 3 m/d.

5.4. The tunnel of Cernobbio (Como) is a road tunnel, 10.5 m wide (excavation section about 100 m²). About 200 m of bad ground (moraine and fluvio-glacial deposits) had to be crossed, composed of sands, gravels, large cobbles and blocks, mostly calcareous. A particular problem was posed by the comparatively thin cover and densely inhabited land, imposing an excavation procedure excluding the slightest subsidence phenomena.

The umbrella arch method has been adopted, with large diameter, closely spaced steel pipes; cement grout has been pressure injected, to obtain a shell of consolidated and reinforced rock in the form of an arch encompassing the excavation section.

It has been possible to safely excavate the full section of the tunnel, at an average gross advance rate of about 2 m/d.

No trace of subsidence effects has been observed.

5.5. The tunnel of Serre la Voute is a twin bore road tunnel of the motorway (still under construction) Turin-Frejus tunnel. Both tunnels are 12 m wide, with an excavation section of over 100 m².

At both entrances, on the west side, a deposit of slide material with large rock blocks and a small amount of clay matrix had to be crossed; moreover the tunnel entrances underpass the Monginevro State road, whose deck lies, at that place, only 5-6 m above the tunnel vault.

It was mandatory not to stop the traffic on the State road during the excavation of the tunnel, and it has been obtained thanks to an umbrella arch made with large diameter pipes. Grout injection has been made under low pressure, due to the small thickness of the cover; a water-cement grout has been employed, with a cement to water ratio of 2/1, and considerable grout absorption has been observed because the mass of slide material contained an high percentage of voids. However a satisfactory result has been obtained: it has been possible to excavate half section of the tunnel at a rate of about 2 m/d. After excavation of the breast and curbs and concrete lining casting, the maximum subsidence observed of the road deck was lower than 1 cm.

5.6. The Pietratagliata tunnel, on the Udine-Tarvisio motorway (still under construction) is a twin bore tunnel. Both tunnels are 12.5 m wide, with an excavation section over 100 m². At the entrances, on the south side, 30 m of tunnel had to be driven through a deposit of slide material, mostly clay with some limestone blocks.

Ground load was not great, so an umbrella with widely spaced pipes was designed; some difficulties however arose in the excavation work, because clay material was poorly penetrated by the injected grout, leaving unconsolidated openings.
between the pipes that allowed the clay to flow. The trouble did not hinder too much the excavation work, that progressed (on half section) at a rate greater than 3 m/d.

5.7. The Spallanzani tunnel, still under construction at La Spezia, is a road tunnel with an excavation section approaching 100 m²; the tunnel underpasses an urban area, the rock to be crossed is a marl-sandstone flysch, locally disturbed by an old (and presently caved) air raid shelter whose axis is almost parallel to the tunnel axis.

A huge caving happened close to an entrance of the tunnel, and required the placement of an umbrella of steel pipes: pipes were parallel to the tunnel axis, being installed from the outside of the tunnel. The caved volume was filled with aggregates of proper grain size, and cement-water grout (with cement to water ratio of 1/1) was then injected under low pressure.

It has been possible to resume the excavation on half section, and to cross the caved tract (about 20 m) in 18 worked days, that is at a rate of about 0,8 m/d.

6. Conclusions

Based on the experience gathered in the last years on the umbrella arch method, we can briefly conclude it is fully effective for tunnel excavation in a number of difficult conditions: tunnels driven through rocks with short self supporting time, tunnel entrances under particular restraints (i.e. when incompetent rock is excavated and subsidence must be strictly controlled), tunnel stretches crossing geologically disturbed rocks or cavings.

Care must be taken, obviously, in strictly following the design elements (correct hole location, careful grout injection, timely placement of supports under the established procedures and having the designed span); when the umbrella arch is correctly placed, excavation can progress without troubles and at a fully satisfactory average advance rate.

REFERENCES


STATE OF ART ON TUNNELLING THROUGH SOFT SOILS USING PIPE JACKING TECHNIQUE

ETAT D'ART SUR CONSTRUCTION DE TUNNEL A TRAVERS DES SOLS TENDRES EMPLOYANT LA TECHNIQUE DE TUYAU TOURNE-BROCHE

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ABSTRACT*

Pipe jacking technique, though not new to the international community of Civil Engineers, could not be widely practised in our country due to financial constraints, lack of experience & the tendency to follow the conventional method of construction. Pipe jacking is the method of construction of underground pipe lines or conduits by forcing a pipe line or 'sleeve' through the ground, the material at the leading edge of the pipe line being excavated by a miner as the work proceeds. It can be used to provide pipe lines, conduits or access ways below ground in a wide variety of cases e.g. for negotiating obstacles such as roads, railways, canals & buildings etc. A very recent development of the technique for the construction of abutments & piers is rather unique and finds application mainly for building road underbridges or subways through existing embankments. Pipe jacking has the inherent advantage of being an extremely safe & fast method of working and the nature and speed of driving allows the cover to be greatly reduced without disturbance to the surface. There is a lot of scope for development of this technique according to Indian conditions and it would get upper hand over the conventional cut & cover method of construction in near future. In the following paragraphs, the authors have endeavoured to exercise their best efforts to review the diversified literature on pipe jacking technique with a view to promote & popularise this unique technique among Civil Engineers in India.

*The French version of the abstract appears at the end of the paper.
INTRODUCTION

Pipe jacking is the method of construction of underground pipe lines or conduits by forcing a pipe line or sleeve through the ground, the material at the leading edge of the pipe line being excavated by a miner as the work proceeds. The technique consists in placing prefabricated pipes in the ground by driving them from a pit prepared before hand, in situations where deep installations are necessary & where conventional tunnelling or cut and cover method will be difficult. The method of pushing pipes or tubes into position under ground is rather an old one, and there are many examples of steel pipes in the past being jacked through rail or road embankments. The main drawback of the old system were that there was no means of properly controlling the alignment, and the jacking could be carried out only for a short length.

The largest mileage of pipe forcing operations has been carried out in deep sewer installations, either as primary or secondary lining. Pipe forcing has almost entirely replaced timber headings for the construction of new sewers and with improved economics resulting from advanced developments and improved techniques has taken a proportion of what has been generally accepted as open cut work, particularly where surface disruptions is undesirable or expensive.

The first prefabricated concrete pipes were jacked in place by the North Pacific Rail road in the U.S.A. some time between 1896 & 1900. The original concept has since then been gradually developed and greatly improved upon. The use of sophisticated equipment for the close control of accuracy of alignment has made the system economical & attractive as an alternative to the more traditional forms of construction in heading, tunnelling or open cut.

USES OF PIPE JACKING TECHNIQUE

For constructing any pipe line under embankment the most common practice in our country is to install relieving girders supported on sleeper cribs or by laying a diversion line and construct the permanent structure by open excavation. This necessarily imposes restrictions to the railway board traffic not only during the construction period but also during the subsequent period of construction of the backfill material.

The pipe jacking method can be used to provide pipe lines, conduits or access ways below ground in a wide variety of cases. For example, it is applicable for negotiating obstacles such as roads, railways, canals & buildings and can also be used in multiple lengths for producing long pipe lines.
2.1 Sewers:

Apart from the general use of the method in circular sewer construction greater than 900m diameter, it can be adopted economically where smaller diameters are needed.

2.1.1 Gas & Water Mains:

Gas and water mains may be accommodated in jacked concrete pipes. The mains can be threaded through & the annular space filled with a suitable grout.

2.1.2 Electricity and Telephone Cables:

The pipe jacked sleeve provides an ideal and economic conduit in which to place these cables in urban areas where it is essential to avoid disruption. Once again easy access for maintenance and replacement is a great advantage.

2.1.3 Relining Sewers & Culverts:

A new lining for old and weak sewers can be installed by jacking concrete pipes along the sewers and grouting the annular void between the new & the old.

2.1.4 Rectangular Sections:

Rectangular concrete sections can also be jacked using this technique.

2.1.5 Abutment & Piers:

A very recent development of the technique for the construction of abutments & piers is rather unique and finds application mainly for building road underbridges or subway through existing embankments.

3 DETAILS OF THE TECHNIQUE

3.1 General:

The pipes which are made either of steel or concrete, are jacked forward by a hydraulic system suitably mounted in the driven pit. The reaction at the rear face is derived from any suitable method of thrust foundation. The pipe alignment is controlled by a specially designed steel head known as the shield which forms the working face and from which the soil is excavated and taken away. The pipe may be of any suitable shape but generally these are circular or rectangular. They are prefabricated in small lengths according to the handling facilities available, with suitable design for joints. Since conventional jacking procedure requires access by workman through the pipe to the heading, 900mm diameter is generally the smallest practical cable size of pipe used for jacking, there being no theoretical limit on maximum size although this will generally be 4 m.

3.2 Jacking Equipment:

3.2.1 Jacking rig:

The rig consists of essentially a means whereby the jacking force reacting on vertical earth face or jacking frame can be distributed on the face of the pipe being driven. Its simplest form is a group of hydraulic rams fitted in suitable holders which pushes a series of packing pieces bearing on the pipe. The packing pieces called spacers are extended as the pipes are pushed forward.
The ram holders are mounted on suitable beams fabricated from steel sections, according to the shape and size of the pipe being pushed. The thrust from the ram is transmitted through the beams to the jacking wall at the rear of the drive pit.

3.2.2 Hydraulic Jacks:

Various types of large capacity jacks are used for this kind of work but the most common ones are the double acting hydraulic jacks having a minimum 30 cm stroke. For speedy operations, large stroke rams are useful, and rams up to 120 cm stroke have been used. In order that the rams and equipment may be kept in a form which can conveniently be handled, it is necessary to work with smaller diameter rams of high jacking capacity operated by high hydraulic pressure.

3.2.3 The Shield:

In all concrete pipe jacking work there will generally be a steel cutting shield in front fitted into a recess on the outside of the first pipe. The shield is much lighter and less complex than the conventional tunnelling shield. A set of hydraulic jacks, called steering jacks, are inserted between the steel ring welded to the shield and the concrete pipe. By differential operation of the jacks, it is possible to change the inclination and direction of the shield and thereby control the alignment of the pipe line.

The shield has a very important role in pipe jacking. The type of ground determines the length of the shield. The design of the shield is very much dependent on the nature of subsoil through which jacking is to be carried out. Its design is a matter of the application of the principles of Soil Mechanics in combination with experience gained from actual field practice.

3.2.4 Guide Rails:

There are steel or timber rails set firmly to give directional control of the pipes for the drive and for accurate locations of the pipe joints whilst in the drive pit.

3.2.5 Intermediate jacking Station:

When it is required to drive very long lengths, the load on the jacking rig & thrust wall increases with the increase of load transfer to the surrounding soil. This jacking load can be reduced by the use of Intermediate jacking stations which consist of a sliding steel collar fitting into the recess in two specially made pipes (Fig. 1). An equal number of jacks of the same capacity as in the jacking rig is inserted between the faces of the two pipes inside the steel collar, and the pipes in front and at the back of the jacks are moved forward by operating the hydraulic system in the intermediate jacking station and the jacking rig in turn.

3.2.6 Reception Pit:

Reception pit is an excavated shaft at the end of the jacked
section of the pipe line from which the jacking shield is recovered.

3.3 Sequence of Operations:

A drive pit and a reception pit are made on either side of the embankment. The jacking is to be carried out from the drive pit. A thrust wall of concrete or other material is built at the rear face of the drive pit and the jacking rig fixed to the thrust wall at the correct alignment. Hydraulic rams fitted into the jacking rig are connected to a hydraulic pump (Fig 2(a)). The front shield together with a set of jacks and concrete lead pipes is lowered into the drive pit and is fitted into the recesses provided in the lead pipe. The whole set up is currently aligned with the jacking rig. The rams fitted in the rig are operated by the hydraulic pump pushing the shield and the lead pipe forward. The rams are retracted and a spacer is added between the rams & pipe. The rams are operated again and the shield & lead pipe are pushed forward. The process is repeated adding a spacer every time until the pipe has been pushed forward sufficiently to allow room for placing the next section of the pipe. In between two jacking operations the soil is excavated and removed from inside the shield.

The jacking operation is continued and jacking load increases with increase in the length of pipe line jacked inside the soil mass. When the jacking load reaches its design limit, an intermediate jacking station is added in the pipe line (Fig 2(b)). The pipes in front of the intermediate jacking station are pushed forward and the pipes behind are jacked by the jacking system installed in the drive pit; thus the total jacking force is divided between the jacking rig & the intermediate jacking station. This operation goes on and the intermediate jacking station moves forward with the pipes (Fig 2(c)). The work progresses in this manner until the shield comes out in the reception pit. The jacks and fittings of the intermediate jacking station are removed. The pipes behind are pushed forward by the jacking rig so as to close the gap left by the intermediate jacking station. Next the shield is removed from the reception pit and then the jacking rig, spacers, fittings etc. from the drive pit.

3.40 Checking of Alignment during jacking operation:

The head end of the pipe must be checked frequently for line and grade. The device shown in fig(3) has been successfully used for checking. The frame is unfolded and then held in position with one arm horizontal and the other vertical. The miner then holds a light behind the slots while the foreman checks...
the position by sighting accross two nails driven in the timbering of the jacking pit. If the pipe is off line, the drift is corrected by widening the excavation a few inches in the desired direction. More sophisticated control can be obtained by using an electric gas laser. Vertical & horizontal alignments of the pipe line are also controlled by the steering jacks provided in the front shield.

4 GROUND CONDITION:

From an engineering point of view pipe jacking is ideal as it replaces the ground and becomes an integral part of the soil mass, occupying the same space as the material excavated and removed. Like any other tunnelling, the method of excavation is dependent on the types of ground encountered. In stiff clays and cemented or cohesive granular materials the heading can be advanced a metre or more without any danger or collapse. In revelling ground, where the materials above the tunnel, or in the upper part of the working face tend to flake off and fall into the heading, the excavation is carried out within the shield by first jacking the shield inside the soil mass. In cohesionless materials such as dry sand or clean loose gravel, the materials will run from any unsupported lateral face until a stable pile is built up at the angle of repose. Pipe jacking in such materials does not prevent much of a problem except that the front shield has a hood of sufficient length to prevent the material falling from the sloping surface from burying the shield jacks.

In revelling or running ground, if seepage pressures are permitted to develop towards the working face, the soil may be transferred into a flowing mass, and like a thick liquid, may advance into the heading. It may thus bury the shield and jacking through such materials can be accomplished only with the greater difficulty and risk unless the soil condition is improved by predrainage or by the use of air pressure or at times even by the injection of chemical grouts.

The pipe jacking contractor should be provided with information on the characteristics of the soil likely to be encountered together with details of the water table.

4.1 Frictional Resistance:

During the pipe jacking operation, friction forces build up around the pipe line as the line of pipes is advanced behind the shield. The frictional forces depend on the type of soil, depth of overburden, length and diameter of the pipe being jacked and the time of operation. It is difficult accurately to assess these forces but after years of experience, pipe jacking contractors have derived
empirical values. As an approximate guide, frictional forces fall between 0.50 and 2.50 tonnes per m² of external circumferential area, dependent upon site conditions.

Frictional forces on the pipeline may be reduced by applying a suitable lubricant such as bentonite slurry under pressure i.e. forcing around the pipe by means of an injection pump through holes provided in the pipe and in the shield. Soap solution or grease can be applied on the outside of the pipes with good effect while they are being driven from the drive pit.

5 TYPES OF PIPES FOR JACKING OPERATION:

It is to be realised that a thrust pipe is installed in such a way as to give loading conditions which are almost ideal for sustaining of ground pressure and where it made of flexible material e.g. steel, would maintain a condition of compressional stress in the wall. Many service duct crossings are constructed using thin steel sleeve, however, these are obviously unsuitable for sewer installations because of corrosion. A concrete pipe is reinforced and is designed to bear predominantly vertical loads as a circular continuous beam and does not allow a effective redistribution of pressure into the ground due to deformation of the lining.

Steel and concrete pipes are the two main types used in pipe jacking. Steel is more expensive than concrete. In certain types of ground there is problem of corrosion in the case of steel pipes, and driving such pipes with external sheathing can present difficulties. However the use of steel pipes in specific situations can not be wholly ruled out. The most widely used material for pipe jacking is concrete. High grade concrete is always preferred as it reduces the weight of the pipe.

6 ACTUAL EXAMPLES OF THE TECHNIQUE (CASE HISTORIES):

Pipe jacking technique has been recognised as one of the standard methods for the construction of sewer lines and other soft ground tunnelling. A large number of works have been completed satisfactorily using this technique and some interesting examples are cited below:

(i) A sewer 1714m long and 1m to 1.20m in diameter, was laid by this technique for the wester Valley Trunk sewer improvement scheme in Monmouthshire, U.K. Although a major portion of the pipe line went through the thickly populated areas of the city, it was possible to complete the work without disturbing the surface amenities.

(ii) Under the river Neckar in Mannheim in Germany, twin sewers, one of 1m and other of 1.37 m diameter, and both 130m long were
jacked side by side from one thrust pit. The ground consisted of coarse sand and pneumatic pressure was used to stabilize the face of the working.

(iii) A concrete pipe, 3.12 m internal diameter was jacked under an existing railway embankment at Richmond in the U.K. to construct a 16.76m long pedestrian subway with cover of 1.50m below the rail level. The work was carried out by first driving a line of steel sheet piles at the far face of the embankment with a minimum penetration of 3m below ground level. The steel jacking frame was then tied with the sheet piles by means of four sets of high tensile bars which were augered through the embankment. The reaction of the jack was thereby transferred to the sheet piles which were bearing on the far face of the embankment.

(iv) An innovation in the pipe jacking technique developed and patented by Tube Headings limited of U.K. was employed for constructing abutments and piers for a road underbridge below the existing rail track at Wandsworth in the southern region of the British Railways. The railway embankment carrying seven tracks was 6m high and 32m wide; it came in the way of a newly constructed highway.

The local authorities originally intended to build a bridge consisting of two spans of 16.76m each but Tube Headings limited in collaboration with the British Railways and the consulting engineers (M/s. Trevor Crocker and partners) evolved a scheme which in essence involved the use of "pipe jacking" to form the bridge abutments and the central pier from precast concrete sections, obviating the need for track possession & interference with the rail traffic except while the superstructure was being installed. The entire jacking operation for both abutments and the central pier took only six weeks. Speed restriction for the railway traffic was required only for a short period in the last stages of the jacking.

(v) A 20 storey office/apartment building at Riverdale, New York, using the jacking system was erected in 15 months only. If conventional building methods were employed, the construction time would have been 20 to 24 months and the estimate price tag on this high-rise structure would have been closer to $9 million instead of $8 million (Ref.11)

(vi) The 3m diameter trunk sewer line of proposed sewerage system of Howrah city was to cross the main south eastern railway line at a point about 5.10Km. from Howrah Railway station. The invert level of the sewer line near the embankment was about 7m below the railway level. Pipe jacking technique was employed to construct the sewer line below the embankment. A length of 35m of the sewer line beneath the embankment
was installed by the pipe jacking technique. This was probably the first time that the technique had been employed in India and found to be highly competitive, much faster compared to the conventional method and causing minimum disturbance of rail bed with no consequent settlement (Fig-4) shows the cross section of the embankment at the line of crossing.

Disturbed & undisturbed soil samples were collected, tested and soil data for the design purpose was established. From that subsoil information, the total jacking force was estimated to be of the order of 1000 tonnes. The driving system was then designed for 1200 tonnes based on two I.J. S. and the drive pit each having a hydraulic system to generate a safe force of 400 tonnes.

The pipes were of precast each 1.20m long suitable to withstand all loading including jacking load during driving. M 400 concrete was used and the thickness of pipe was 22.50cm with steel deformed bars as reinforcement. The shield was made from M.S. plate and was 2.50m long.

Alignment and level were checked in every shift with a theodolite and levelling instrument & whenever required adjustments were made by operating the shield jack. The 35M of jacking took 18 days. (Ref-15).

7 ASPECTS OF PIPE JACKING:

Pipe jacking has the inherent advantage of being an extremely safe & fast method of working. The man working at the face is at all times within the shield or pipe. The nature and speed of driving allows the cover to be greatly reduced, without disturbance to the surface and pipe has been driven under normal running rail tracks with as little as about 1m cover. Obviously the nature of the work and the ground conditions determines the safe working cover.

On long drives, it may be necessary for the work to be on a continuous basis, 24 hours a day for a week. The main reason for this is that if a jacking job is left for any length of time, the ground has a tendency to grip the pipe and over the pipe done. The amount of grip will vary with the nature of ground but a stoppage for as short a time as eight hours cause a considerable increase in the jacking force needed to restart the drive.

Essentially it is the speed of cutting, the muck at the face and its removal along the pipe that determines the speed of the jacking operation.

8 ECONOMICS

A guide indication of current values for pipe jacking in the U.K. as compared to other tunnelling methods is given in (Figure-5). Values only take into account stable subsoil condition and do not allow for geotechnical costs or rock excavation. Prices are
based on an average assessment over the industry. It can be seen that pipe jacking is a viable economical alternative in the range 600mm-1800mm to other tunnelling methods and competes well with traditional heading prices even at the smaller diameters. Above 1800mm, segmental tunnel construction starts gaining due to savings in the tunnel linings and the cost of additional jacking equipment necessary to overcome the higher friction forces. The technique must also be challenging traditional deep open cuts methods in congested urban conditions where the cost of reinstatement is avoided, apart from the unmeasurable cost of disruption to traffic and the general public.

9. CONCLUSIONS

Pipe jacking technique, though not new to the international community of civil engineers, could not be widely practised in our country because of financial constraints, lack of experience and the tendency to follow the conventional methods of construction. Installation of pipe jacking technique is safe & fast.

There is a lot of scope for development of this technique according to Indian conditions and to utilize the same though it may be costly in the beginning even taking into consideration the indirect benefits. If the indirect benefits are taken into consideration, this technique would get upper hand over the conventional methods in the near future. Some firms in India have made a head way in this connection, the name of cementation company Ltd. Calcutta can be mentioned in this connection.

As the market grows, more research is needed to find new materials for the jacked tunnel lining. At present reinforced concrete pipes are the only suitable lining both to withstand the jacking force and external sub-soil pressure and the corrosive effect of the effluents, where used for sewage construction. Asbestos cement or some other fibre reinforced material is a possibility. If the weight of the pipe could be reduced and the strength increased, economics could be improved. The economy of pipe jacking technique depends on various factors involved in the operation but it is no doubt that this new technique if carefully employed can serve to a great extent public in-convenience.

As the shortage of the skilled miners bites into the tunnel industry, pipe jacking will become a more attractive method to automate than conventional segmental tunnelling.

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and thus acknowledge the valuable contribution referred to while carrying out this literature survey.

REFERENCES:
ABSTRAIT:

La technique de tuyau tourné-broche quoique n'est pas nouvelle à la communauté internationale des génies-civils, on n'a pas pu largement pratiquer dans notre pays à cause des contraintes financières, manque d'expérience et la tendance pour suivre la méthode conventionnelle de construction. Tuyau tourné-broche est une méthode de construction des canalisations souterraines ou des conduits en forçant une canalisation ou une manche à travers la terre, le matériel à un bord menant d'une canalisation état excavée par un mineur comme le travail continue. On peut l'utiliser à fournir des canalisations, des conduits ou des chemin d'accès sous la terre dans les cas de grandes variétés, c'est à dire pour négocier des obstacles comme des rues, des chemins de fer, des canaux et des bâtiments etc. Un développement très recent de la technique pour la construction des aboutements et des jetées dans plutôt unique et trouve application principalement pour construire des rues, des sous-ponts et des passages souterrains à travers des envasements qui existent. La technique de tuyau tourné-broche à un avantage inhérent d'être une méthode de travail extrêmement sûr et vite et la nature et vitesse d'action de conduire permet la couverture d'être réduit grandement sans aucun trouble à la surface. Il y a beaucoup d'étendue pour le développement de cette technique selon les conditions indiennes et il sera beaucoup mieux en comparaison de la coupe conventionnelle et la méthode de couverture de construction en futur proche. Dans les paragraphes suivants, les auteurs a essayé a exercer leurs meilleurs efforts à revier la littérature diversifiée sur la technique de tuyau tourné-broche en view d'avancer et populariser cette unique technique entre les ingénieurs civils en Inde.

FIG.1 INTERMEDIATE JACKING STATION
FIG. 2 (a)

- HYDRAULIC PUMP
- JACKING SHOE
- DRIVE PIT
- SPACER
- RECEPTION PIT
- JACKING RIG
- SHIELD
- SHIELD JACKS
- CONCRETE LEAD PIPE
- PROPOSED ALIGNMENT OF PIPE LINE
- HYDRAULIC RAMS FITTED IN SUITABLE RAM-HOLDERS

FIG. 2 (b)

- HYDRAULIC PUMP
- INTERMEDIATE JACKING STATION
- RECEPTION PIT
- THRUST WALL
- JACKING RIG
- HYDRAULIC RAMS
- SHIELD
- PROPOSED ALIGNMENT OF PIPE LINE

FIG. 2 (c)

- HYDRAULIC PUMP
- RECEPTION PIT
- INTERMEDIATE JACKING STATION
- JACKING RIG
- HYDRAULIC RAMS
- CONCRETE PIPES
- THRUST WALL

SKETCH SHOWING THE SEQUENCE OF OPERATION FOR THE PIPE JACKING
FIG. 3  SIGHTING SQUARE FOR CHECKING AT ALIGNMENT OF PIPE IN JACKING PROGRESS.

FIG. 4. CROSS SECTION THROUGH EMBANKMENT.

FIG. 5  COMPARATIVE VALUES OF TUNNELLING METHOD IN U.K.
ROCK BREAKING BY MICROWAVE RADIATION

RUPTURE DE ROCHE PAR RADIATION D’ONDE ULTRACOURTE

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ABSTRACT

To examine the applicability of the method of breaking rocks by microwave radiation, preliminary tests were conducted with the use of a 100kW microwave generator. Test specimens were prepared for four kinds of rocks; granite, slate, sandstone and pumice-tuff, and while radiating microwave onto them varying output of radiation and moisture contents of test specimens, measurements were made of the temperature rise due to dielectric heating. The thermal and breaking properties were also observed.

1. Degree of temperature rise varies from rock to rock species. It is greater in the sequence of slate, sandstone, pumice-tuff and granite.

2. Temperature rise varies proportionally with moisture contents of specimens.

3. The type of temperature distributions in the specimens are classified into two groups. One is granite type which temperature rise is broad deeply from the surface of radiation. The other includes slate, sandstone and pumice-tuff. Temperature rise is limited to the shallow part from the surface of radiation.

4. Two patterns of breaking rocks are found. One includes granite, slate and sandstone. Cracks are produced at nearly right angle to the plane of radiation. The other is pumice-tuff type which produces a small dent of explosion crater.

ABSTRAIT

Pour examiner l'applicabilité de la méthode de cassure des roches par radiations micro-ondes, on a procédé à des tests préliminaires en utilisant un générateur de micro-ondes de 100kW. On a préparé des spécimens de test pour quatre catégories de roches différentes: le granite, l'ardoise, le grès et la pierre ponce volcanique. Pendant que l'on irradiait des micro-ondes et variait la production des radiations et le degré d'humidité des spécimens de test, on a mesuré l'élévation de la température dûe au chauffage diélectrique. On a observé également les propriétés de cassure et les propriété thermiques.
(1) L'élévation de la température varie selon les catégories de roches. Elle va croissant respectivement pour l'aidoise, le grès, la pierre ponce volcanique et ce granit.

(2) L'élévation de la température varie proportionnellement avec le degré d'humidité des spécimens.

(3) Les types de distributions de température dans les spécimens, sont classés en deux groupes. Le premier est le type granite dont l'élévation de température se fait sur une large zone et en profondeur, à partir de la surface de radiation, l'autre inclut l'ardoise, le grès et la pierre ponce volcanique. L'élévation de température est limitée à la partie creuse à partir de la surface de radiation.

(4) On trouve deux schémas de cassure de roches. L'un comprend le granite, l'ardoise et le grès. Les fissures apparaissent presque à angle droit par rapport au plan de radiation, l'autre est le type pierre ponce volcanique, dans laquelle se produit une petite explosion qui forme un cratère.

INTRODUCTION

Explosive blasting is generally used for the excavation of hard rocks of tunnels and rock slopes. But the blasting is accompanied by noise and vibration which tend to lead to social nuisance problems, while there is a shortage of engineers and skilled laborers for tunnels. Thus, development of a new excavation method correspond to prevent public hazards and to save labor is urgently called for.

Recently, as one of the means capable of coping with these problems, a method of radiating microwaves onto rocks and concrete and thus breaking them has been attracting attention.¹

To examine the applicability of the method of breaking rocks by microwave radiation, the authors conducted preliminary tests with a maximum capacity of 100kW generator. The results of tests are reported here.

PRINCIPLE OF BREAKING

When a rock is heated by the dielectric heating effect of microwave radiation, it undergoes the following changes:

(1) Thermal stress caused by local radiation.

(2) Decrease of strength due to structural and chemical changes of the rock forming minerals; and

(3) Generation of superheated vapor pressure within the rock.

For (1), when the adjacent areas of the heated portion are confined, the smaller heat conductivity and the greater elastic modulus, thermal expansion and temperature gradient, the greater thermal stress produced.

As an example of the structural and chemical changes of the minerals in (2) above, the volumetric change of quartz by phase transition from α- to β-type may be cited. The superheated vapor pressure of (3) is generated by which the moisture in the rock is heated, and exerts a pressure to the surrounding to produce an effect of breaking or explosion. Of these, the authors believe that the thermal stress generated as stated in (1) plays an important role in breaking.

APPARATUS

The rock breaking test apparatus is comprised of a microwave generator (Fig.1),

Fig. 1 100kW microwave generator

Fig. 1 Générateur de micro-ondes de 100kW
a radiator, a shield room and a cooling water circulator. Its specifications are:
Oscillating frequency: 915±25MHz;
Oscillator output: 100kW;
Power: AC 420V, 3 phases, 50 Hz, 180kVA.

A schematic diagram of the rock breaking test apparatus is shown in Fig. 2. Microwaves generated are introduced to a waveguide. The radiator is fitted to the extreme end of the waveguide to radiate the microwaves onto the specimen. The shield room is of duplex construction comprising two rooms, outer one and inner one. This construction aims to prevent adverse effects through a leakage of microwaves on the human body, communications and measurements.

PROCEDURE

1. General

Test specimens of four species of rocks — granite, slate (Triassic), sandstone (Cretaceous) and pumice-tuff (Neogene) — were used. The physical and mechanical properties of the test specimens are shown in Table 1. The dimensions of the test specimens were 50cm long, 50cm wide and 30cm thick. With microwaves radiated on these specimens, temperature rises were measured, and the breaking conditions including cracks and explosion craters produced in the specimens were observed.

2. Radiation

Each specimen was set 4cm apart from the radiator (Fig. 3) and received microwave according to the radiating pattern shown in Table 2. The work cycle time until the subsequent radiation was 50-60 minutes on intermittent radiation.

The continuous radiations aim to make clear the temperature rise, distribution of temperature under fixed microwave radiation.

---

Table 1

<table>
<thead>
<tr>
<th></th>
<th>Granite</th>
<th>Slate</th>
<th>Sandstone</th>
<th>Pumice-tuff</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compressive strength (kg/cm²)</td>
<td>1.713</td>
<td>1.420</td>
<td>630</td>
<td>60</td>
</tr>
<tr>
<td>Tensile strength (Brazilian test) (kg/cm²)</td>
<td>79.0</td>
<td>112.5</td>
<td>61.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Coefficient of static elasticity (kg/cm²)</td>
<td>4.13x10⁶</td>
<td>1.86x10⁴</td>
<td>7.21x10⁴</td>
<td>1.48x10⁴</td>
</tr>
<tr>
<td>Effective porosity (%)</td>
<td>0.77</td>
<td>0.5</td>
<td>10.8</td>
<td>35.1</td>
</tr>
<tr>
<td>Moisture content (%)</td>
<td>0.318</td>
<td>0.2</td>
<td>2.1</td>
<td>10.5</td>
</tr>
<tr>
<td>Room temperature (°C)</td>
<td>5.5</td>
<td>4.5</td>
<td>5.8</td>
<td>6.2</td>
</tr>
</tbody>
</table>

Fig. 2 Schematic diagram of rock breaking test apparatus

Fig. 2 Diagramme schématique du montage des appareils utilisé pour le test sur la cassure des roches

IV.45
3. Temperature Measurement

For the specimens which received microwave radiation, both inside and outside temperature were measured with thermocouples inserted in holes (25cm ~ 2.5cm) drilled beforehand (Fig. 4).

The purpose of intermittent radiations is to gain the progress of breaking conditions by microwave radiation.

Table 2 Outputs pattern of microwave radiations
Tableau 2 Schéma de la production de radiations micro-ondes

<table>
<thead>
<tr>
<th>Continuous radiation</th>
<th>A</th>
<th>B-1</th>
<th>B-2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10kW·1min + 15kW·1min + 20kW·1min + 25kW·1min + 30kW·1min +</td>
<td>40kW·1min + 40kW·1min + 40kW·1min</td>
<td>70kW·1min + 70kW·1min + 70kW·3mins</td>
</tr>
<tr>
<td></td>
<td>35kW·1min + 40kW·1min + 45kW·1min + 50kW·1min + 45kW·1min</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3 Test specimen set 4cm apart from the radiator

Fig. 3 Appareil de test pour spécimens situé à 4cm du radiateur

Fig. 4 Temperature measurements by thermocouples
Fig. 4 Mesures de la température par thermo-couples

Fig. 4 Internal temperature (°C) distribution of rock specimens by 20kW·2mins microwave radiation
Fig. 4 Distribution de la température (°C) interne des spécimens de roches par une radiation de 20kW pendant 2 minutes

(a) Granite  (b) Slate  (c) Sandstone  (d) Pumice-tuff

IV.46
RESULTS

1. Granite

The temperature distribution of granite by 20kW·2mins microwave radiation is shown in Fig. 4 (a). Temperature rise is broad deeply from the surface of radiation. Maximum temperature was 18.1°C. Temperature rise of granite is the least of the four rock species (Fig. 5).

![Graph showing temperature distribution of granite, slate, sandstone, and pumice-tuff.]

Fig. 5 Maximum temperature of the specimens by 20kW·2mins microwave radiation

By continuous radiation for 24 minutes at about 50kW, cracks were produced at nearly right angles to the plane of radiation and in directions nearly equal to two diagonal lines on the plane of radiation (Fig. 6(a)). Three specimens produced similar cracks at similar output and radiation time.

2. Slate

The temperature distribution of slate by 20kW·2mins microwave radiation is shown in Fig. 4(b). Temperature rise is limited to the shallow part from the radiated surface in comparison with granite. Maximum temperature was 132.0°C, the highest of the four rock species (Fig. 5).

By continuous radiation for about 5~10 minutes at about 30~50kW, a number of cracks were produced, mainly in the radiated part. The radiated part became red hot. It swelled and protruded, nearly coming into contact with the radiator. In its inside, fusion occurred.

In the pattern A radiation, crack generation or extension was observed at 3, 5, 7 and 8 cycles. At stages of smaller output, cracks were produced, as shown in Fig. 7 (a), along the slaty cleavage. With increased output, small extension cracks were produced near the radiated part.

![Images of granite, slate, sandstone, and pumice-tuff specimens.]

Fig. 6 Fracture of specimens by microwave radiation

Fig. 6 Cassure des spécimens par radiations micro-ondes
With pattern B, crack generation and extension were observed at 2 and 3 cycles in B-1, and 1, 2 and 3 cycles in B-2. Cracks tended to spread radially about the radiated part. In the second and subsequent radiation, cracks which were produced up to the preceding cycle tended to widen, while short cracks were produced in the radiated part and in its vicinity (Fig. 7 (a)).

At 2.5 minutes of the third radiation (63kW) under B-2, a red heated melted section protruded from the 2.5cm deep temperature measurement hole at the center of the radiation. When cooled, this melted section turned into a dark green glassy substance. It contains numerous gas holes and is similar to slag (Fig. 6 (b)).

The temperature measured at this time was 529°C at 5cm inside the surface. Further, in the radiated part and its vicinity, there were discolorations of pale brown and brown, and these correspond to those portions which are surrounded by an isothermal line of 400°C and that of 200°C on the planar temperature distribution estimated from the results of temperature measurement.

Fig. 9 (a) shows the length of all cracks* produced as a function of the energy applied in the slate specimens. In general, the greater the energy applied, the total crack length is the greater. Further, continuous radiation clearly produces a greater value for total crack length than intermittent radiation, apparently because continuous radiation has a constant rate of temperature increase so that the temperature gradient increases with time, while intermittent radiation involves a cooling period so that the temperature gradient is smaller due to heat conduction. In general, the total crack length increases as increasing amounts of energy are applied, but a higher temperature gradient can be expected by increasing the power of the radiation and also the rate of elevation of the temperature. For example, the specimen M-A had a crack produced at the third radiation. The value of energy applied up to this point was 43kW·min, while the energy applied for the first radiation of M-B-1 was 38kW·min. From Fig. 7 (a), M-B-1 can be seen to have greater cracks produced. Further, M-B-2 having received a greater power of radiation had many more cracks produced in it.

These show that it is advantageous for breaking a rock such as slate or producing cracks therein to increase the temperature gradient.

3. Sandstone

Temperature distribution of sandstone by 20kW·2mins microwave radiation is shown in Fig. 4 (c). Temperature rise limited to the shallow part from the radiated surface is similar to slate. Maximum temperature was 121.6°C (Fig. 5).

By continuous radiation for 10 minutes at about 10-50kW, 1-2 cracks were produced at right angles to the plane of radiation and in a horizontal or vertical direction on the plane of radiation (Fig. 6 (c)).

In the case of intermittent radiation, a crack appeared first along the line of temperature measuring holes on the upper surface of the specimen (Fig. 7 (b)). With increasing cycles of radiation, it grew from the upper part of the specimen toward the radiated part, but there was no increase noted in the number of cracks, and the crack developed into one similar to that produced in the vertical direction in the case of continuous radiation. Further, the crack gap tended to spread with increasing cycles of radiation.

Fig. 8 (b) shows the total crack length as a function of the energy applied. Unlike with slate, the trend of increasing crack length with increasing energy apply is not clearly appreciable. Comparing continuous radiation and intermittent radiation with each other, continuous radiation is greater for total crack length.

With intermittent radiation, the number of cycles at which a crack appears for the first time is the 8th with pattern A, 2nd with B-1 and 1st with B-2. That is, the crack is produced earlier when there is greater power of radiation and its extension is longer.

In the case of sandstone, the total crack length is not proportional to the energy apply. This is because it is believed that compared with slate, sandstone has a slower temperature rising speed and greater heat conductivity so that when the radiation output is small, a temperature gradient necessary for producing cracks is rarely attained. That sandstone is slow in temperature elevation (calorific value per unit time) is

* Quantitatively, it will be necessary to determine the cracks in terms of a plane, but it is difficult to measure the plane in the course of a test so that the length is substituted for the plane.
Cracks produced at 3rd radiation
--- Cracks produced at 5th radiation
----- Cracks produced at 7th radiation
--------- Cracks produced at 8th radiation

M-A

Cracks produced at 1st radiation
--- Cracks produced at 2nd radiation
----- Cracks produced at 3rd radiation

M-B-1

Cracks produced at 1st radiation

M-B-2

Produced at 8th radiation

S-A

Produced at 2nd radiation

S-B-1

Production at 1st radiation

S-B-2

Explosion crater by 2nd radiation (2.1)
--- Explosion crater by 3rd radiation (3.5)
----- Explosion crater by 4th radiation (5.0)

T-A

Explosion crater by 1st radiation (4.0)

T-B-1

Explosion crater by 1st and 2nd radiation (3.5)

T-B-2

Parenthesized figures show the depth of the central dent (cm)

M: Slate  S: Sandstone  T: Pumice-tuff  A,B-1, B-2: Radiation pattern

Fig. 7 Fracture of specimens by intermittent microwave radiations

Fig. 7 Cassure des spécimens par radiations micro-ondes intermittentes

IV.49
Fig. 8 Amount of applied microwave energy and breaking condition of specimens

Fig. 8 Quantité de l'énergie de micro-ondes appliquée et conditions de cassure des spécimens

deduced from the result of temperature measurement (Fig. 10).

4. Pumice-tuff

Temperature distribution of 6.35% moisture content pumice-tuff by 20kW-2mins microwave radiation is shown in Fig. 4 (3). Temperature rise limited to shallow part from the radiated surface is similar to slate and sandstone. Maximum temperature was 75.4°C (Fig. 5).

Fig. 9 shows the relationship between the moisture content of specimens and maximum temperature by 20kW-2mins microwave radiation. With the increase of moisture content, temperature rises higher proportionally.

Pumice-tuff causes a small explosion with a pop in a relatively short time under either continuous or intermittent radiation and produces a small dent (explosion crater) in the radiated part. Cracking is not observed. The volume of the explosion crater is generally proper-

IV.50
Distance from center of radiated surface (cm)

O: Surface temperature

In: Initial temperature (internal temperature of specimen before radiation)

1st: 1st temperature (temperature measured on specimen taken out of shield room after radiation)

2nd: 2nd temperature (temperature of specimen allowed to cool for some time after measurement of 1st temperature)

--- Slate, --- Sandstone, --- Pumice-tuff

<table>
<thead>
<tr>
<th></th>
<th>Rocks</th>
<th>Slate</th>
<th>Sandstone</th>
<th>Pumice-tuff</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radiation</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>Power (kW)</td>
<td>38</td>
<td>36</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>Time Interval (min)</td>
<td>7</td>
<td>7</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Radiation end</td>
<td>33</td>
<td>35</td>
<td>50</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 10 Internal temperature of specimens after 1st radiation under B-1 pattern of 40kw/1min

Fig. 10 Température interne des spécimens après la première radiation du schéma B-1 de 40kw pendant 1 minute

APPLICABILITY TO ROCK BREAKING

It was confirmed that microwave radiation is a promising method for breaking rocks since it produces a number of cracks and explosions, although short of an instantaneous breaking effect similar to blasting. However, its applicability has some problems which should be resolved.

(1) Heat generation and its distribution in a rock is governed by the dielectric properties, which vary with microwave frequency and moisture content. But measurement of these properties is hardly practicable by conventional methods because of the complexity of the physical properties of rock, and it is necessary to classify rock by a simpler identification method (1), into those which are highly adapted for rock breaking by microwave radiation and those which are not.

(2) Breaking and temperature gradient are closely related to each other. Fig. 10 shows the curves of temperature distribution in rocks. As can be seen, the greatest temperature gradient occurs between the surface and the point of highest temperature 5cm inside the surface. A sharp drop of temperature near the surface is considered to be due to an escape of heat into the atmosphere because of radiation or a conduction of heat from the heated surface during or after microwave radiation (1), (5). Therefore, by cooling the surface,
for example by blowing and thus maintaining a necessary temperature gradient, breaking will be carried out effectively.

(3) For rocks which explode under microwave radiation, continuous excavation will be possible if an adequate muck haulage is also employed. In the case of pumice-tuff in this test, the volume of breaking per unit time is about 0.5m$^3$ an hour. On the present condition, the method of breaking rocks by microwave radiation may be inferior to the blasting method in point of construction cost and term. It is necessary to examine the radiation test further to increase the volume of breaking. For rocks producing cracks in which no explosion crater is produced, a method of loosening the rockmass by microwave radiation and then breaking with a raker or cutting with a tunnel excavator, is considered$^{(5)}$.

CONCLUSION

Four kinds of rocks; granite, slate, sandstone and pumice-tuff were subjected to continuous and/or intermittent microwave radiation, and their breaking conditions were observed and temperatures measured.

(1) Degree of temperature rise varies from rock to rock species, it is greater in the sequence of slate, sandstone, pumice-tuff and granite.

(2) Temperature rise proportionally varies with moisture content of specimen.

(3) The type of temperature distributions in the specimens are classified into two groups, one is granite type which temperature rise is broad deeply from the surface of radiation. The other includes slate, sandstone and pumice-tuff. Temperature rise is limited almost to the shallow part from the surface of radiation.

(4) Two patterns of breaking are found, one includes granite, slate and sandstone. Cracks are produced at nearly right angle to the plane of radiation of the specimen. The other is pumice-tuff type which produces a small dent of explosion crater.

For field applications, the method involves many points which should be examined, including the improvement of equipment so that it is smaller in size and lighter in weight and prevents microwave leakage. Rock breaking by microwave radiation should be developed as a new rock breaking method which can comply with the need to prevent public hazards and save labor.

ACKNOWLEDGEMENTS

The authors are grateful for Mr. Shigeo Kanno, Hatabara Dam Office, Chugoku Regional Bureau, Ministry of Construction, and Mr. Satoru Jimbo, Public Works Research Institute, for their executing the tests.

REFERENCES


A SHALLOW TUNNEL IN A HETEROGENEOUS GROUND IN ROME (ITALY)

UN TUNNEL SUPERFICIEL DANS UN TERRAIN HETEROGENE A ROME (ITALIE)

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ABSTRACT
This paper highlights the problems related to the driving of a surface tunnel in an urban area. A main sewer tunnel (diameter: 3.8 m; depth: 5-20 m; length: 2100 m) driven with a shield in an area of Rome is described. The ground is highly heterogeneous and the geotechnical and hydraulic conditions along the tunnel axis are complex. The tunnel excavation was preceded by a very accurate in situ and laboratory investigation. A description is given of the behaviour of the various soils and of the surface settlements during the tunnelling operations. The values of the surface settlements are compared with those evaluated through theoretical and empirical relations.

ABSTRACT
Ce rapport expose les problèmes posés par le creusement d'un tunnel superficiel dans un quartier urbain. Il décrit un tunnel d'égout principal (diamètre: 3.8 m; profondeur: 5-20 m; longueur: 2100 m) creusé à l'aide d'un bouclier dans un quartier de Rome. Le terrain est particulièrement hétérogène et les conditions géotechniques sont complexes le long de l'axe du tunnel. L'excavation du tunnel a été précédé d'un analyse très soigné sur place et en laboratoire. On décrit ensuite le comportement des différents terrains et les tassements de la surface pendant les travaux d'exca-vation. Les valeurs de tassement de la surface sont comparées avec celles qui ont été évaluées par des relations théoriques et empiriques.

INTRODUCTION
The main problem encountered when excavating shallow urban tunnels is the need to limit ground deformations, since no undue damage whatever should be caused to any surrounding and overlying structures and services (PECK, 1969; WARD, 1981).

In the urban area of Rome this requirement takes on a particular shade of complexity. Indeed, the historical development and events have deeply affected the very structure of the subsoil because over the centuries a great number of excavations and fills, measuring up to 20 m in thickness (VENTRIGLIA, 1971), have brought about topo-
graphic changes in many parts of the town. Furthermore, it is well known that there are a great many archaeological findings of considerable importance that must be imperatively preserved, and great care must be taken not to disturb in any way the outstanding buildings and monuments that are of artistic value.

And besides, problems of a geologic nature come to add their weight to these environmental and historical problems. The ground here consists of soils having diverse origins (deltaic, alluvial and marine sediments and volcaniclastic deposits are present) varying widely over space.

The subject matter of this paper deals with the case of a surface tunnel driven to house a main sewer in a portion of the town that flanks the historical centre. The tunnel, being circular, has an outer diameter of 3.80 m and is about 2 km long; its depth varies between 5 and 20 m; it underlies a main road (Via Gregorio VII) in a densely-built area. Being this thoroughfare essential to the road traffic, it was imperative that the road remained undisturbed to the greatest extent possible, and that the underground activities did not interfere with the normal surface activities.

On the basis of the investigations carried out for the tunnel design, and here-in referred to, a contribution is meant to be given to the general knowledge of a typical subsoil of the city of Rome.

An analysis of the behaviour of the soils crossed by the excavations provides the chance to discuss the complex geologic and geotechnical aspects of the ground itself in relation to the construction of surface tunnels.

GROUND CONDITIONS

Engineering Geology

The tunnel runs along a valley where an ancient righthand tributary to the River Tiber formerly flowed. Urbanization in this area, West of Rome, took place about 40 years ago, and on that very occasion the topography of the valley bed was altered by heightening it with fills up to about 10 m.

The subsoil shows evidence of the typical soils present in the Rome area (VENTRI-GLIA,1971): the bedrock consists of Pliocene marine sediments and the overlying soils (from the Calabrian to the present age) are of marine and continental origin.

The stratigraphic column in Fig.1 shows the sequence of the soils along the tunnel. With reference to the typical succession of the soils in the Rome area, some upper units are not found here, but they outcrop in the valley slopes. More exhaustive information can be found in the paper presented by SCIOTTI (1882) at this very meeting.

Starting from the oldest ones through to the more recent ones, the soil units present are the following:

- Marine sediments (Pliocene, pl): clayey silts with layers of silty sands ("Argille Azzurre" Formation);
- Fluvo-deltaic sediments (Sicilian, si): layers and lenses of sandy and silty clays, sands and sandy gravels ("Ponte Galeria" Formation), the thickness ranging up to about 20 m;
- Volcaniclastic deposits (Tyrrhenian-Sicilian, ta) more or less altered tuffs with pumice layers ("Tufi Antichi" Formation), the thickness ranging up to about 10 m;
- Alluvial sediments (Holocene, al): sandy and clayey silts with lenses and layers of silty sands, the thickness ranging up to about 15 m;
- Fills (f): heterogeneous materials made of reworked clays, silts, sands and gravels of various origin, the thickness ranging up to about 10 m.

A succession of sedimentation and erosion processes have brought about the present stratigraphic sequence, starting from the first emersion of the clayey substratum, so that all the separation surfaces between units differing in age and origin are erosion surfaces, including the separation surfaces between the fills and the underlying alluvial and volcanic deposits.

Site investigation and subsoil profile

A campaign of rotary borings drilled at 200-m intervals and of in-situ and laboratory tests preceded the excavation of the tunnel. Casagrande piezometers were installed at different depths. Near or inside the
Fig. 1 - Engineering geologic profile
### TABLE I: Geotechnical properties of the soils

<table>
<thead>
<tr>
<th>TYPE OF SOIL</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$I_c$</th>
<th>$C_u$ (kPa)</th>
<th>$C'$ (kPa)</th>
<th>$\phi'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILLS</td>
<td>$f_1$ 15 - 18 *</td>
<td>-</td>
<td>-</td>
<td>~ 0 *</td>
<td>25° - 30° *</td>
</tr>
<tr>
<td></td>
<td>$f_2$ 16 - 18 *</td>
<td>-</td>
<td>-</td>
<td>~ 0 *</td>
<td>30° *</td>
</tr>
<tr>
<td></td>
<td>$f_3$ 14 - 17 *</td>
<td>-</td>
<td>-</td>
<td>~ 0 *</td>
<td>30° *</td>
</tr>
<tr>
<td>ALLUVIAL SEDIMENTS</td>
<td>$a_1$ 18 - 21</td>
<td>~ 0 - 1.4</td>
<td>20 - 140</td>
<td>10 - 30</td>
<td>20° - 30°</td>
</tr>
<tr>
<td></td>
<td>$a_2$ Silty Clays</td>
<td>18 - 21</td>
<td>~ 0 - 1.4</td>
<td>20 - 140</td>
<td>10 - 30</td>
</tr>
<tr>
<td></td>
<td>Silty Sands</td>
<td>~ 20 *</td>
<td>-</td>
<td>-</td>
<td>~ 0 *</td>
</tr>
<tr>
<td></td>
<td>$a_3$ 17 - 20</td>
<td>0.3 - 1.4</td>
<td>60 - 180</td>
<td>10 - 30</td>
<td>20° - 30°</td>
</tr>
<tr>
<td>VOLCANICLASTIC DEPOSITS</td>
<td>ta 14 - 17</td>
<td>-</td>
<td>-</td>
<td>20 - 30</td>
<td>* 30°</td>
</tr>
<tr>
<td>FLUVIO - DELTAIC</td>
<td>$s_1$ Silty Clays</td>
<td>20</td>
<td>-</td>
<td>20 - 240</td>
<td>-</td>
</tr>
<tr>
<td>SEDIMENTS</td>
<td>Sands</td>
<td>21</td>
<td>-</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>MARINE SEDIMENTS</td>
<td>$p_1$ 20.0-20.5</td>
<td>0.8 - 1.3</td>
<td>80 - 310</td>
<td>30 - 40</td>
<td>22°</td>
</tr>
<tr>
<td>(ALTERED ZONE)</td>
<td>Silty Sands</td>
<td>~ 20 *</td>
<td>-</td>
<td>~ 0 *</td>
<td>30° *</td>
</tr>
</tbody>
</table>

*Estimated values on the basis of the in situ tests and through comparisons with similar soils.

Boreholes both static (C.P.T.) and dynamic (S.P.T.) penetrometer tests were carried out. Vane tests were conducted in one of the boreholes (Fig. 1).

The geologic profile worked out on the basis of the investigation is shown in Fig. 1. The data collected during the works fully confirmed the expected profile except for a 70-80-metre stretch, at about 700 m from the beginning of the tunnel, where the Pliocene substratum was encountered at about 8 m above the expected height.

**Ground Water Conditions**

The water table depth ranges between 4 and 14 m (Fig.1). This surface almost coincides with the separation surface between the fills and the alluvial sediments from the beginning of the tunnel through to the centre, and with the contact surface between the tuffs and the underlying soils towards the end of the tunnel. This situation refers to the conditions prior to the excavation, which were affected both by the boundary hydraulic conditions and by the drainage due to the old sewer that was in a rather bad state of maintenance.

**Physical and Mechanical Properties of the Soils**

This paragraph provides a summary of the characteristics of each soil; Table I summarizes their geotechnical properties.

Marine sediments (pl)  
Layers of overconsolidated clayey silts alternate with thin layers of fine silty sands. Fig. 2 provides the grain-size distribution for the two types of materials.

In depth, underneath the altered zone, the clayey silts are stiff and the silty sands are dense. Static penetrometer tests have shown that the thickness of the alter-
ed zone ranges from 2 m up to a maximum of 5 m. The excavation did cross the altered zone. Here, the geotechnical properties improve gradually with depth (Fig. 3).

Fluvio-deltaic sediments (ai)
Here layers and lenses of silty-sandy clays alternate with uniform sands and sandy gravels (the grain-size distribution of the first two soils is shown in Fig. 4).

The sands account for the highest percentage (58%), the silty-sandy clays (34%) are found mostly in the upper part where they come into contact with the volcanic deposits; the sandy gravels (8%) are found in the lower part of the formation where their thickness does not exceed 2 m. These values were worked out on the basis of the boring logs. The sands are dense ($N_{SPT} \geq 70$) and the silty-sandy clays are of medium consistency.

Volcaniclastic deposits (ta)
These consist of pyroclastic materials, argillised to a greater or lesser extent, ranging from cohesionless to semilithoid materials, with thin pumice layers. Being located above the groundwater level, these materials are not saturated. The penetrometer tests to which they were submitted in the lower part of the deposit showed a cone penetration resistance of between 3 and over 12 MPa.

Alluvial sediments (ai)
These were divided into the following three types:
- ai1 - sandy-clayey silts and silty clays;
- ai2 - more or less silty sands alternating with sandy-clayey silts with sublevels of silty-sandy gravels;
- ai3 - clayey-sandy silts containing pyroclastic materials and organic matter, with layers of sands at the base.

The distribution of the three types of alluvial materials along the tunnel is shown in Fig. 1.

The ai1 type alluvial sediments are rather heterogeneous soils as can be inferred from the broad grain-size range of the finer materials (Fig. 5) and from the variation intervals of the Atterberg limits and of the undrained shear strength (Fig. 6).

The ai2 type alluvial sediments differ from the ai1 in that large quantities of more or less silty sands are present, that alternate with silty-clayey layers having geotechnical characteristics that come very close to those of the ai1 type. Consequently...
in some sections perpendicular to the axis and the latter were detected by means of the piezometers installed during the investigations.

RESPONSE OF THE SOILS TO TUNNELLING

Where the tunnel crossed cohesive soils (alluvial and marine sediments, clayey soils of fluvio-deltaic origin), the shield could advance without particular problems. The use of a shield machine capable of partially supporting the face avoided the difficulties that would have been encountered especially where, because of the low shear strength values of the cohesive soils, the stability ratio $N = \frac{\sigma_v}{c_u}$ ($\sigma_v =$ total vertical pressure; $c_u =$ undrained shear strength) was higher than that corresponding to the limit stability conditions ($N = 5-6$) (BRONN and BENNERMARK, 1967; PECK, 1969; DAVIS et al., 1980). This condition referred mainly to the beginning of the tunnel where all sediments with very low minimum strength values are present (Fig. 6).

Independently from the depth of the excavation, the rate of advancing of the shield in the clayey soils was on average of the order of 1.0 m/h.

The excavation in the cohesionless soils below ground water level did entail some difficulties. Indeed, in these conditions, even if the shield increases the factors of safety against the complete collapse of the heading, it does necessarily lead to the prevention of flows even if a small portion of the face is exposed.

During the excavation the cohesionless alluvial and fluvio-deltaic sediments showed different responses. In the former, there were no flows; this could be attributed to the well-graded grain-size distribution of this material, that also includes a silty fraction between 10 and 50% (Fig. 5); it could also be ascribed to the probable lack of hydraulic continuity between and among the lenses of permeable soils. In the latter, for a short stretch at the end of the tunnel, where the full face of the shield cut into uniform sands below the ground water level, flows towards the excavation occurred, thus slowing down the advance rate since it was necessary to proceed with the shield almost blind.

DRAINAGE OF THE GROUND

In the time interval between the excavation and the setting up of the permanent lining, the piezometers generally showed small and temporary variations in the water level.

The inflows towards the excavation were generally modest, taking the form of water percolations with a very low discharge in between the precast segments.

Greater inflows occurred with the flow of sands from the excavation face in the zone of fluvio-deltaic sediments. Here, the inflow of water, before laying the permanent lining, was kept in check by using special low-permeability sheets, placed between the temporary lining and the excavation perimeter, which avoided flows of materials.

With the setting up of the permanent lining the original level of the water table was restored.

SURFACE SETTLEMENTS

Surface settlements induced by shield-driven tunnelling are generally caused by the loss of ground through the face and by loss of ground due to overcutting and to the closure of the annulus between lining and ground at the tail of the shield.
Fig. 10 - a) Surface settlements as a normal probability curve; b) measured and evaluated maximum surface settlements related to depth (symbols inside fields refer to excavated soils)

While the settlements caused by loss of ground through the face are difficult to evaluate, especially for cohesionless soils, since they depend both on the properties of the soils and on construction factors, subsidence due to radial deformations around the excavation can be assessed with a good approximation.

For the case of the shield adopted, the loss of ground was essentially located at the face; indeed, the use of expanding ribs greatly cut down the loss of ground that would occur upon closing the annular cavity around the lining.

As to the cohesionless soils below the water table, the loss of ground at the face was cut down by keeping the doors of the machine almost shut; a contribution to this was also probably given by the lowering of the water table through controlled drainage beyond the tail of the shield by using the special low permeability sheets.

Fig. 10 shows, as a function of the excavation depth, the field of values of the surface settlements corresponding to the axis of the tunnel, grouped according to type of soil. The settlements measured vary between less than 1 cm to about 10 cm and they appear to vary greatly for the various soils and do not appear to be clearly related to the depth of the excavation.

The width of the surface settlement trough, independently from the type of soil, proved to be generally small in relation to the excavation depth and to the diameter of the tunnel.

In order to discuss the behaviour of the ground involved in the tunnel excavation with regard to the surface effects, the measured settlements are compared with the values inferred from theories and empirical criteria regarding tunnels driven in homogeneous soils. An evaluation of the settlements was made by using Peck's diagram (1969), where the width of the settlement profile, assumed as a normal probability curve, is a function of the depth and diameter of the tunnel and of the type of soil.

In particular, according to Kanji (1979), the width of the surface settlement profile was expressed as a function of angle \( \beta \) (Fig. 9) which depends on the characteristics of the soils (sands below WL: \( \beta = 50^\circ \); soft to stiff clays: \( \beta = 50^\circ - 33^\circ \); hard clays and sands above WL: \( \beta = 33^\circ - 11^\circ \)).

The maximum surface settlement was evaluated with the \( d_{\text{max}} = V_s / 4 \sqrt{\pi} \) relationship, where \( V_s \) is the volume of the settlement trough per unit advance (Fig. 10a). \( V_s \) is estimated as a function of the nature and characteristics of the soil and of the section area of the tunnel face (Atterwill, 1978; Kanji, 1979).

For \( \beta = 33^\circ \) and \( \beta = 50^\circ \) the diagram in Fig. 10b shows the settlement \( (d_{\text{max}}) - \text{depth} \) curves for the case in which \( V_s = 0.05 A_T \) (loose cohesionless soils and cohesive soils where \( d<4\mu<6 \)) and for the case in which \( V_s = 0.02 A_T \) (dense cohesionless soils and cohesive soils where \( N>4 \)).

A comparison between the evaluated and measured settlements shows that:

- for the case in which the excavation at
a depth of 5-6 m, affected at the same time the alluvial sediments and the overlying fills, the measured settlements correspond to the values obtained with \( V_s = 0.05 A_m \) and \( \beta = 33^\circ \); such results can be accounted for by the rather poor mechanical properties of the excavated soils and by the small cover (3-4 m);

- for the case in which the excavation went through the alluvial sediments (12-15 m depth), the measured settlements still correspond to the values obtained for \( V_s = 0.05 A_m \) and \( \beta = 33^\circ \); such results do agree with the characteristics of the subsoil; the low shear strength of the excavated soils justify the high \( V_s \) value, whereas the presence between the excavation and the surface of granular fills above water level justifies the low \( \beta \) value;

- for the case of alluvial sediments (7-13 m depth) for \( \beta = 33^\circ \), the settlements measured are included between those relating to \( V_s = 0.02 A_m \) and \( V_s = 0.05 A_m \) (in these cases the fills are between 2 and 5 m thick); the smaller loss of ground in these sediments can be accounted for the smaller thickness of the cover;

- for the case in which the mainly clays fluvio-deltaic sediments were involved in the excavation, the measured settlements were lower than the minimal val-

ue evaluated for \( V_s = 0.02 A_m \), even for the greatest depths reached (20 m); this can be explained by the higher strength of the excavated clayey soils \((c_u = 80 - 240 \text{ kPa})\) and by the presence above these of volcanic deposits having good mechanical properties.

In the sections where the excavation involved soft clayey soils, the settlements measured at the surface, immediately after the shield had passed, were 65% of the final ones; in the sandy and clayey fluvio-deltaic sediments the settlements, that were not very large, stopped immediately after the shield had passed.

In order to complete the analysis of the subsidence that occurred above the tunnel, the values of the measured settlements are compared with those provided by Peck (1969) and by Attewells (1978), relating to shield driven tunnels in mainly homogeneous soils. The maximum settlements, scaled with respect to the diameter, are shown as a function of the depth-diameter ratio in Fig. 11. The data relating to the examined tunnel, even if rather scattered, are of the same order of magnitude as those used in the comparison.

**CONCLUSIONS**

The investigation reported in this paper have led in particular to the geotechnical characterization of recent alluvial deposits that, together with the other typical sediments of the Rome area, are present in the ground crossed by the tunnel. The thickness of the former sediments reach about 20 m, and they are mainly soft-to-medium silty-clayey soils containing, in some places, lenses and layers of loose silty sands. The shear strength of the soils forming these sediments varies irregularly in space.

An accurate site investigation, even if in a heterogeneous and geotechnically complex subsoil, made it possible to ascertain the ground conditions, on the basis of which it was possible to predict the problems most likely to be encountered during excavation and thus choose the most appropriate method of excavation. Indeed, the digger shield used was built to meet the specific requirements of this subsoil and thus the tunnel could be driven without any particu-
lar drawbacks and without causing too much disturbance to surface activities and structures.

The maximum surface settlements caused by the excavation and the width of the settled zone can be justified by the methods of evaluation proposed for homogeneous subsoils, if due account is kept of the diverse characteristics of the soils directly involved in the excavation and of those present between the excavation and the surface.

ACKNOWLEDGEMENTS

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REFERENCES


IV.63
SOME GEOTECHNICAL CONSIDERATIONS ON TUNNELLING IN HIMALAYAS

QUELQUES CONSIDERATIONS GEOTECHNIQUES A PROPOS DE PERCEMENTS DANS L'HIMALAYA

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ABSTRACT

The tremendous latent potential of Himalayan rivers for power generation has, of late, drawn attention to the necessity of more and larger hydro-electric projects, in the interior of this mountain systems. The experiences in exploration and construction of tunnels in Himalayan and sub-Himalayan ranges, have been unique, and have taught many new and exotic geotechniques, apart from the usual ones, in exploring, forecasting and constructing tunnels through different parts of Himalayas.

The paper describes the tunnelling difficulties in Himalayas and subdivides Himalayas into six geotechnical zones, each having its own kind of tunnelling problems. This zoning may be useful while planning explorations for any new tunnelling venture in Himalayas.

ABSTRAIT

Le potentiel extraordinaire des rivières de l'Himalaya pour engendrer l'électricité a récemment attiré l'attention sur la nécessité d'avoir de grands et nombreux projects hydro-électriques dans l'interieur du système Himalayen. L'expérience dans l'exploration et la construction de tunnels dans les chaînes Himalayennes et sub-Himalayennes ont été uniques, et ont enseigné bien des méthodes géo-techniques nouvelles pour l'exploration, la prevision et la construction de tunnels à travers les différentes parties de l'Himalaya.

Cet article décrit les difficultés de perçement dans les Himalayas et subdivise la chaîne Himalayenne en six zones géo-techniques, chacune d'elles ayant ses problèmes particuliers de perçement. Cette division en zones peut être utile dans la prévision des explorations qui conduiraient à de nouvelles entreprises dans l'Himalaya.

The great Himalayan mountain ranges, which contain rocks of different ages, rose to the present elevations through five or more stupendous orogenic movements, in the Tertiary and Quaternary eras. These movements uplifted and reworked the ancient cores and massifs, gave rise to many thrusts, faults and folded chains extending with the general Himalayan trend and eliminated 'Tethys', the earlier marine zone within this region. Krishan (1956) sub-divides the Himalayas into the following seven zones from south to north:

1. Main boundary fault of the Siwalik zone;
2. Imbricate thrusts of the Himalayan border;
3. Thrust of the Lesser Himalayan border;
4. Central Himalayan thrust;
5. Thrust of the 'Tethys' Himalayan zone;
6. Thrust of the flysch and exotic zone, and
7. Counter thrust of the Darchen zone.

The 'Tectonic Map of India' (1963) sub-divides Himalayas into five regions:

1. Upper structural stage;
2. Foredeeps;
3. Himalayan granitoid occurring within ancient cores and reworked massifs;
4. Basic and ultrabasic intrusions, and
5. Paleozoic-Mesozoic volcanic effusions.

Lithologically, sedimentary rocks like sandstone, shale, limestone; metamorphic rocks like gneiss, schist, phyllite, slate and rocks having an igneous origin like basalt, granite, amphibolite and pegmatite constitute the major bulk of the outer shell of the Himalayan belt. The general strike in Himalayas is northwest-southeast, east of Kashmir Syntaxis, gradually swerving to east-west, and then to east of northeast-west of southeast, west of Assam Syntaxis, and finally taking a north-south trend in Burma.

The Himalayas, therefore, represent a mixed lithology, tectonics and structure with geotechnical side effects.

Of late, a number of tunnels have been, or are being driven in different parts of Himalayas. The authors have attempted to correlate the Himalayan geological and tectonic set up with the geotechnical evaluations that have been made with regards to the tunnelling in the Himalayas. Different tunnels for which such an evaluation has been made, are given in table 1.

It is seen in the table that tunnelling hazards in the Himalayas may be of two types:

1. Usual tunnelling difficulties, like rock pressures, occurrence of clay pockets, water seepages etc., as experienced in any terrain;
2. Hazards associated with Himalayan tectonics, specially like montane-stresses, abnormal rock loads, geothermal and seismic conditions etc. The paper confines to the hazards of the latter type only and the same have been described in detail below:

1. Montane-stresses and abnormal rock loads

The concept of absolute bridging action above Terzaghi, or any other, load line, i.e., the line below which the rocks constitute the load requiring support, irrespective of the total rock or soil cover above, probably holds good upto a hundred meters or so only. Experiences in deep mines in India, viz. Kolar Gold Mines which go down to about 3000 m., amply show that rock loads, as well as, stresses start building up again after a certain rock cover and one has to cope with abnormal rock loads, the magnitude of which fringe with the rock stresses created at that level.

At Pandoh-Baghi tunnel of Beas Sutlej Link Project, twisting and buckling of ribs was recorded after the rock cover exceeded about 1400 metres. Similar phenomenon is observed in Giri Tunnel, where twisting, buckling and shearing of ribs started at places exceeding rock cover of about 250 metres. It has been seen that the closure in the tunnel was proportional to the rock cover. However, the generalisation of this kind, at this stage, would be premature. Loktak Tunnel has faced hazards similar to those of Giri Tunnel.

The closure bolt observations in Giri Tunnel with respect to time, have shown a sharp rise for the first 20 days or so, with a flexure proceeded by a slow closure (Fig. 1). The three parts of the Closure/Time graph represent two obviously different closure rates, with a fringe zone in between. An attempt at solving the closure problem in Giri Tunnel revealed that if the initial fast closures be yielded to, the latter ones can be safely supported to, without the disconcerting tearing of ribs. The authors somehow feel a link between montane-stresses, residual to tectonic/orogenic movements of Himalayas, with the initial fast closures, which have been found difficult to support and are better yielded to. The concept that 'earlier the support, lesser the rock loads' probably holds good in general, but in the cases illustrated as above, where the closure rates are high with probable link with the latest orogenic movements (Verison/Hercynian), this concept does not hold good, as is evidenced by uncontrollable distortions in ribs at Giri. The latter part of the closure rate curves probably indicate a rock load remanant of the Terzaghi loads, at that place, at that time.

2. Tunnel closure

A certain amount of overcutting is made, in the initial tunnel diameters to
allow for supports and lining, to get a finished planned diameter, normally. Closure phenomenon observed in Himalayan tunnels, such as Giri and Loktak, show that while planning a finished tunnel diameter, the extent of closures has also to be kept in mind.

3. Geothermal Gradients

An average 0.039°C/m geothermal gradient (or a 33 metres/°C geothermal step) is found to hold good, in general, in Himalayas too (Pratap Singh & S. Kumar, 1973-78) with modifications due to topography (Szechy). However, manifestation of hot springs, as at Manikaran (H.P.), Puga and Chumathang (J&K) and other places indicate the presence of chemically active hot fluids existing and circulating through subterranean conduits. These have modified the natural geothermal gradients with the added discomfort due to the possibility of surprises by sudden release of hot fluids and gases in the tunnel. Such a possibility has been investigated upon for Nathpa-Jhakri Project (H.P.), where at least a six-kilometre long part of head race tunnel passes through zones suspected to give surprises in any reach.

4. Heavy inflows of water

The topographic relief contrast is, since, more in Himalayas than in plains, high hydrostatic or piezometric heads of groundwater are inevitable. Such sudden on-rush of water, has been found in Sundernagar-Slater Tunnel of Beas Sutlej Link Project, Baira-Siul Tunnel, Chibro-Kodri

Figure 1

IV. 67
<table>
<thead>
<tr>
<th>Name of project, name of tunnel, length &amp; cross-section</th>
<th>Rock types</th>
<th>Tunnelling problems</th>
<th>Remedial measures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yamuna Project: Ichari-Chibro tunnel, 6.2 km, 7.0 m dia</td>
<td>Quartzite, slate and limestone</td>
<td>Poor stand up time, high overbreak, semi solid flowing conditions</td>
<td>Shotcreting, pre-reinforcement, pregrouthing and heading and benching</td>
</tr>
<tr>
<td>Chibro-Khodri tunnel, 5.6 km, 7.5 m dia</td>
<td>Quartzite, slate, limestone, shale, sandstone and clays</td>
<td>High overbreak, tunnel closures, abnormal rock loads</td>
<td>Shotcreting, perfo-bolting, forepoling flexible lining, heading and benching, and multi drift method</td>
</tr>
<tr>
<td>Beas Sutlej Link Project: Pandoh-Baggi tunnel, 13.12 km, 7.62 m dia</td>
<td>Granite with schistose bands and kaolinitised pockets and phyllite</td>
<td>Overbreak, cavity formation, flowing ground conditions, squeezing ground with abnormal load, twisting of ribs, heavy water inflows</td>
<td>Forepoling, destress by drilling advance holes at heading, heading and benching, draining rock from behind the heading</td>
</tr>
<tr>
<td>Sundarnagar-Slapper tunnel, 12.23 km, 8.5 m dia</td>
<td>Limestone, dolomite</td>
<td>Overbreak, cavity formation, flowing ground, heavy water inflow</td>
<td>Forepoling, draining rock from behind the heading, changing tunnel alignment</td>
</tr>
<tr>
<td>Giri Project: Giri tunnel, 7.0 km, 3.66 m dia</td>
<td>Slates with boulder beds, phyllite, shale, clay, sandstone</td>
<td>Overbreak, tunnel closures with abnormal rock loads, twisting of ribs, occurrence of gases</td>
<td>Shotcreting with perfo-bolting, flexible lining, excavating larger dia. tunnel to allow closure before supporting, use of gas detectors</td>
</tr>
<tr>
<td>Chineni: six tunnels, aggregate length 6.5 km 2.1 m dia</td>
<td>Sandstone, clay</td>
<td>Overbreak</td>
<td>Controlled blasting</td>
</tr>
<tr>
<td>Baira-Suir: Headrace tunnel 7.6 km, 4.5 m dia</td>
<td>Carb. phyllite, pebbly slates, phyllite</td>
<td>Overbreak, cavity formation, flowing ground, occasional heavy water inflows</td>
<td>Forepoling</td>
</tr>
<tr>
<td>Baira-Baledh feeder tunnel, 7.9 km, 2.5 m dia</td>
<td>Phyllite</td>
<td>Overbreak, occasional cavity formation</td>
<td>Forepoling</td>
</tr>
<tr>
<td>Mândini: twin traffic tunnel, 118 &amp; 122 m, 5.5 m dia</td>
<td>Sandstone &amp; clay</td>
<td>No major problem</td>
<td>-</td>
</tr>
<tr>
<td>Area</td>
<td>Rocks/Conditions</td>
<td>Problems/Issues</td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>-------------------------------------------------------</td>
<td>---------------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>Upper Sindh: water</td>
<td>Slate</td>
<td>Overbreak, cavity formation and heavy water inflows</td>
<td></td>
</tr>
<tr>
<td>conductor, 484 m,</td>
<td>-do-</td>
<td>Forepoling and draining of rocks before drive</td>
<td></td>
</tr>
<tr>
<td>4.1 m dia</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maneri Bhali: two</td>
<td>Quartzite, basic rocks and limestone</td>
<td>Overbreak, cavity formation and heavy water inflows</td>
<td></td>
</tr>
<tr>
<td>tunnels, 16.8 km,</td>
<td>-do-</td>
<td>Forepoling and draining of rocks before drive</td>
<td></td>
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<td>Salsal: twin tail</td>
<td>Limestone &amp; dolomite</td>
<td>Occasional seepage</td>
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<td>race tunnels, 2.0 km,</td>
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<td>Nathpa Jhakri: head</td>
<td>Gneiss, schist, amphibolite and granite</td>
<td>Flowing gougy zones, geothermal hazards, abnormal rock loads and heavy water</td>
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<td>race tunnel, 30.0 km,</td>
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<td>Sanjay (Bhaba): head</td>
<td>Granite, gneiss, amphibolite and schist</td>
<td>Gougy flowing ground</td>
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<td>Dhauli Ganga: head</td>
<td>Quartzite, granite and gneiss</td>
<td>Montane-stresses, abnormal rock loads, chlorite/sericite/kaolinised pockets,</td>
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<td>race tunnel, 29.0 km,</td>
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<td>heavy water inflows</td>
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<td>Gori Ganga: head</td>
<td>Phyllite, quartzite, granite, gneiss and limestone</td>
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<td>race tunnel, 19.0 km,</td>
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<td>5.0 m dia</td>
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<td>Banhal: twin traffic</td>
<td>Basic flows &amp; limestone</td>
<td>No major problem</td>
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<td>tunnels, 2.25 km,</td>
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<td>5.5 m dia, each</td>
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<td>Khara Project: head</td>
<td>Sandstone, Clay and conglomerate</td>
<td>Overbreak and flowing ground conditions where saturated with water</td>
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<td>race tunnel, 10.2 km</td>
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<td>Pabar-Tons Project</td>
<td>Limestone</td>
<td>Solution channels</td>
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<td>head race tunnel,</td>
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<td>Kopli Project: head</td>
<td>Limestone, shale and sandstone</td>
<td>Solution channels, overbreak and flowing ground conditions where surcharged with</td>
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<td>race tunnel</td>
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<td>water</td>
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<td>Uniam Project: link</td>
<td>Granite and gneiss</td>
<td>Montane-stresses, Chlorite/sericite/kaolinised pockets and heavy water inflows</td>
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<td>tunnel, 2.8 km</td>
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<tr>
<td>Tenga Project: two tunnels, 16.5 km, 5.0 m dia</td>
<td>Granite, gneiss, schist, phyllite, quartzite, sandstone, slate and shale</td>
<td>Heavy water inflow, flowing ground conditions and gases</td>
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<tr>
<td>Jaldhaka Project: head race tunnel, 4.4 km, 3.0 m dia</td>
<td>Schist, amphibolite, and gneiss</td>
<td>Heavy overbreak, Forepoling and cavity formation</td>
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<tr>
<td>Loktak Project: head race tunnel, 6.25 km, 3.65 m dia</td>
<td>Sandstone and shale</td>
<td>Squeezing ground, abnormal rock, loads and gases</td>
<td>Perforbolling, shotcreting and use of NATM</td>
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</table>

Tunnel and Maneri-Bhali Tunnel, to name a few. Possibilities of heavy water inflows are more at places where the rock is shattered or cavernous, and the piezometric pressures are high.

5. Active faults

As the Himalayan orogeny is not so old geologically, some of its parts are still seeking adjustment within (isostatic or otherwise). This results in active faults which are still moving, or are subject to rejuvenation any time. The existence of large thrust sheets (Krishnan, 1959), e.g., Main Boundary Fault, with its offshoots and homologous members and other thrusts like Murree, Krol, Girli, Chali, Jutogh, Panjal, Naga etc. make the geotechnical picture more complex as the faulted and thrust sheets create many hazards to tunnelling.

It is, though, not conclusively decided, yet, the existence of a 'Suture Line', joining the Indian and the Eurasian plates, with stresses and movements presumably still active, is widely believed. Any tunnel in the vicinity of 'Indus-Suture' may have to be designed keeping this tectonic phenomenon well in view.

6. Seismicity

Earthquakes occur in regions of marked instability of the crust such as, mountain belts of geologically recent date (Krishnan, 1956). The Alpine-Himalayan belts fall within the major earthquake regions of the earth, and the earthquakes in the Himalayan region are mostly believed to be due to sudden release of energy stored as stresses, in any part of the crust, which may be of shallow, intermediate, or of deep seated origin. The northwestern Kashmir and the northeast corner of Assam, where the geological formations show sharp changes in strike, due to the presence of 'Peninsular-wedges' underneath, had a tendency to be seismically more active during the past. The Kangra (1905) and Kinnaur (1975) earthquakes bring seismicity all along Lesser Himalayas and Foothills. Seismicity, therefore, appears to be an important hazard in Himalayas, and may adversely affect the tunnels too.

GEOTECHNICAL CONSIDERATIONS

Himalayas, owing to their unique tectonic history, present a complex heterogeneous mass, tunnelling through which may be a smooth drive or a difficult slogging—it all depends upon how much it is planned in advance. A geotechnical approach in Himalayas, though includes the various parameters usually, nevertheless, at times much larger geotechnical aspects have been, incidently, obscured by the details worked out later. The authors have attempted to decipher geotechnical zones in Himalayas relevant to tunnelling. This has the specific objective of offering a preliminary information while planning tunnels in Himalayas.

Considering the tectonic history of Himalayas, the authors have divided it into the following six geotechnical zones (Figs. 2 & 3):

1. Calcareous formations;
2. Himalayan core granite;
3. Gneiss and schist with minor variants;
4. Basic and ultrabasic intrusives;
5. Younger sedimentary rocks, and
6. Tectonically disturbed zone.

The tunnelling hazards experienced and expected to be experienced, are as follows:
I. Calcareous formations

Limestones and dolomites of Permian, Triassic, Eocene and other ages occur in Himalayan folded sequence. These rocks have either shown banding with argillaceous matter (e.g. Eocene) or shearing and shattering together with solution channels, cavities and caverns. The authors have seen large solution tunnels and cavities in 'Sirban Limestone' of J&K State and 'Krol Limestone' of H.P. Many of these solution channels conduct large amounts of water, and any of these, if encountered while tunnelling, can prove to be a sudden surprise with lingering disappointments. To delineate and treat a subsurface solution channel has always been, more or less, evasive, though most of the cavities or solution channels have been found to be trending along the bedding or master shear strike directions.

2. Himalayan granite

The granite in Himalayas is Pre-Cambrian or Tertiary in age and is pink, purple or grey in colour. It is massive, yet contains three or more sets of joints along which secondary kaolinisation or mineralisation has made the rock blocky. Wherever there has been an enrichment and subsequent alteration of quartz-felspathic material, randomly oriented and located clay or gouge pockets may be expected. The granite, thus, offers a terrain where the 'block size' and kaolinised pockets are important considerations while planning a tunnel.

3. Gneiss and schist etc.

In the central part of Himalayas, not away from the core, Pre-Cambrian gneiss and schist are exposed. These are intruded upon by basic and acidic bodies with resultant contact metamorphism. A disconcerting rock type, in this zone, is chlorite-sericite-schist, which is a poor rock for most of civil constructions, including tunnelling.

4. Basic and Ultrabasic intrusives

Basic rocks, like Panjal Traps, are good tunnelling media (e.g. Banhal Tunnel), except where thrusted or faulted. The thrusts and faults, nevertheless, are mostly past and dead tectonically.

Amphibolitic intrusive bodies of smaller size, which are mostly concordant, have posed special problems while tunnelling. A variant member of this rock group, is again chlorite schist, which is intercalated in thin bands. These, as well as, quartz-sericite-schist, which is found in bands and pockets, are almost as lubricating as graphite. These rocks form a very poor tunnelling media. It has been seen that sand derived from sericitic or chloritic rocks tends to turn into quicksand on the slightest saturation. These rocks, are very soft with little shear resistance.

Smaller basic intrusives, hence, are more hazardous, since these offer a poor tunnelling media, interspersed with water surcharged flows of schistose rocks. The larger basic bodies, as such, are more or less a good tunnelling media.

5. Younger sedimentary rocks

The younger rocks in Himalayas are mostly Tertiary in age, and are shale, clay and sandstone with minor and local variations. Rocks in Siwalik Group are not much indurated. The older Tertiaries, too, contain clay or clay with conglomerate zones which are quite plastic when exposed and saturated with water. The rocks on the whole offer moderate tunnelling conditions with chances of occasional encounter with 'flowing grounds' conditions, or 'loose rock strata'.

Tertiaries and younger rocks have been the source and reservoir rocks for oil and gas, as is very well known. The presence of gaseous pockets in tunnel courses may not be surprising. In fact, all the tunnels in Tertiaries, preferably, should have a gas testing facility in advance. Tunnels driving through older rocks, but located in the vicinity of gaseous Tertiaries, may also require the same precautions with regard to gases.

6. Tectonically disturbed zone

The Himalayas rose to the present height through five or so stupendous orogenic movements, is a well accepted fact. The major stress axes lay across the Himalayan trend. As a result of orogenies, certain parts of Himalayas were subjected to tight folding, thrusting and faulting, the trends of all these structures running parallel to general Himalayan strike. A number of Himalayan thrusts and faults, like Naga, Panjal, Murree, Krol, Giri, Chail, Jutogh thrusts and Main Boundary Fault etc. lie along a zone, which shows a concentration of tectonic activity.

The junction between the Indian Plate and the Eurasian Plate, geologically lies along river Indus and is called the 'Indus Suture Zone'. It is evident that this zone too, will be tectonically quite disturbed.
TUNNELS IN EASTERN INDIAN HIMALAYAS

INDEX

- Himalayan core granite
- Gneiss and schist with minor variants
- Basic and ultra basic intrusions
- Younger sedimentary rocks
- Tectonically disturbed zones

NOTE: THE ENTIRE AREA FALLS UNDER SEISMIC ZONE - V

PROJECTS

18. Kopili Tunnel
19. Umiami Tunnel
20. Loktak Project
21. Tenga Project

Figure - 3
All such zones, including those which manifest an otherwise concentration of orogenic energy through repeated folding, have also been put in tectonically disturbed zone.

Tunnels in this zone, may have to cut across, or run in vicinity of major faults and thrusts. The attendant hazards may be excessive water seepages in saturated zones, flowing ground conditions, squeezing and development of abnormal rock loads.

In addition, the orogenic stresses, may still be active in certain reaches, making it obligatory to account for and adjust for the same in course of a tunnel drive. The Giri, Yamuna and Loktak tunnels are examples of this phenomenon.

PRE-DESTRESSING

Experiences in tunnels under 'Montane-Stress' have shown that the initial fast closures have better to be yielded to. Provision of temporary yielding supports, for the initial fast closure periods, to be ultimately replaced by permanent supports and lining, has successfully been tried.

The authors suggest, alternatively, that a destressing prior to final tunnel face advance, may be preferable. A part of the total rock to be excavated through tunnelling, has to be removed through drifts, to destress the rocks proportionally, for a period determinable in advance, before the final face advances. The cycle of operation, may be adjusted in such a way that the foreprobing and destressing face is always a certain number of days ahead of the main tunnel face. Temporary supports, however, in the foredrifts are to be provided in such a fashion so that these provide a safe working facility, as well as, allow the rocks to destress. The marginal apparent increase in cost, in such an operation is more than offset by the advantage derived through advance probing, lesser consumption of steel and concrete and smoother operations, resulting in better progress at lesser cost ultimately.

CONCLUSIONS

Himalayas represent a complex lithology comprising ancient core granite, gneiss and schist, sedimentary rocks, calcareous rocks and a semi-consolidated admixture of arenaceous and argillaceous sediments. These have been acted upon by five orogenic movements in the Tertiary Era and later, resulting in the development of thrusted and folded belts where the 'Montane-Stress' have shown a clear manifestation in some of the latest tunneling projects.

Considering the lithology and orogenic history of Himalayas, in the light of up-to-date tunnelling experiences, the authors have divided the Indian Himalayas into six zones, wherein, the anticipated and actual tunnelling difficulties have been listed. Out of these, five zones are based on lithology and age group of rocks and the sixth one is related to the orogenic history.

It is hoped that an overall estimation of tunnelling conditions, as per above zoning, initially, would save time and money later, and would evolve a better and comprehensive investigational strategy.

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GEOTECHNICAL PROBLEMS IN TUNNELLING AND CONSTRUCTION OF UNDERGROUND POWER HOUSE AT THE PENCH HYDRO-ELECTRIC PROJECT, TOTLADOH IN MAHARASHTRA STATE, INDIA

PROBLEMES GEOTECHNIQUES DU PERCEMENT DE TUNNEL ET DE LA CONSTRUCTION D'UNE CENTRALE ELECTRIQUE SOUTERRAINE AU CHANTIER DE L'INSTALLATION HYDROELECTRIQUE DE PENCH A TOTLADOH, ETAT DE MAHARASHTRA (INDE)

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ABSTRACT

The construction of an underground power house and its auxiliary structures at the Pench Hydro-Electric Project in the Maharashtra State, India, posed severe problems of tunnelling which necessitated detailed geotechnical evaluation and modifications in planning and design layouts. The project is located in an intensely folded and deformed Sausar metasedimentary rocks of the Archaean age. The rocks include numerous bands of cavernous marble, cutting across the various tunnel alignments which connect the main power house cavity at about 100 m below the surface. The Sausar rocks comprise the banded granulites, the pink and white calcitic marble and the foliated gneisses and schists, and these have been subjected to repeated folding and faulting of a complex nature. These complexities, together with the cavernous nature of marble and the highly kaolinised veins of intrusive granite and pegmatite, add significantly to the hazards of tunnelling and construction of the underground power house at the Pench Hydel Project.

The 75 m high gravity dam under construction on the Pench river and the main power house complex at Totladoh lie within a tightly folded and overturned syncline with its trough being occupied by the soft schistose rocks, while the marble and the Utkatka granulites are repeated on the either limb. The subsurface power house cavity will be excavated entirely within the banded granulites and outside the influence of cavernous marble and the major faults and shear zones. Excavation of the 857 m long approach tunnel posed numerous problems, including stability of the portal, rock-falls and caving of the schistose strata and flooding of the tunnel due to diversion of the Pench river through the cavernous marble. The sinking of the 95 m bus shaft well through an extensively sheared and kaolinised pegmatite required heavy supports and adequate lining of the steining walls.

The geotechnical problems and the hazards of underground excavation involved were predicted and the remedial measures were outlined on the basis of the detailed geological studies carried out during the various stages of planning, design and construction of the subsurface power house.

ABSTRAIT

Au chantier de l'installation hydroélectrique de Pench, située sur le fleuve Pench à Totladoh, Etat de Maharashtra (Inde), la construction d'une centrale électrique souterraine et ses annexes avait pose de graves problèmes de percement de tunnel et de creusement, qui exigeaient des estimations géotechniques détaillées ainsi que des modifications dans le planning et les tracés types. Le projet se situe dans des roches métasédimentaires de Sausar d'âge Archaen fortement plissées et deformées. Elles s'associent avec de nom-
breuses bandes de marbre crystallin passant en travers aux différents alignements de tunnel conduisant à la cavité de la centrale électrique située à peu près 100 m sous la surface. Les roches de Sausar comprenant des rubans de granulite, du marbre de calcite de couleur rose et blanche et de gneiss ont été touchées par des plissemens renouvelés et de dérangement de nature très compliquée. Celles-ci associées avec son aspect cavernieux de marbre et de veines d'intrusion granitique et pegmatitique, s'ajoutentent a risques de percement de tunnel et de creusement souterraine de la centrale électrique au chantier de l'installation hydroélectrique de Pench;

Le barrage poids de 75 m haut sur le fleuve Pench et le complexe de la centrale électrique à Totladoh s'étendent sur un synclinal fortement plissé et renversé dont la charnière synclinaire présente des roches dounées et schistuses du niveau Utekata sont redoublées des deux cotés de la branche. La cavité principale de la centrale électrique se situe au sein des rubans granulitiques et hors de l'influence du marbre karstique et de nombreuses zones de faille et de cisaillement. Cependant, le creusement d'un tunnel d'approche de 857 m de long a pose des problèmes graves de stabilité de l'entrée, de chutes de roches et d'effondrement des rouches schistues, détournement du fleuve Pench au tunnel de creusement à travers le marbre cavernieux. Une zone fortement cisailée et kaolissée a également rendu difficile le creusement d'une ligne barre de 95 m et qui demandait de fortes armature et le revêtement du mur de support.

On avait bien auguré les différents problèmes géotechniques en jeu et les risques du creusement souterrain en des terrain difficiles et on avait aussi exposé à grands trais les mesures réparatrices en se basant sur les études géologiques et détaillées entreprises au courant de planning, de la conception du dessin et de la constructin de centrale électrique souterraine.

1. GEOTECHNICAL PROBLEMS AT THE PENCH HYDRO-ELECTRICAL PROJECT

The Pench Hydro-Electric Project at Totladoh, located in the eastern part of the Maharashtra State, India envisages construction of a 75 m high gravity dam across the Pench river, an underground power house and a 8057 m long tail race tunnel, for generating 160 MW electricity. The main power house complex is now under construction on the left bank of the river at a depth of about 100 m below the surface. The power house discharge will be further utilised at the already constructed Kamthikheri Dam, located on the Pench river 30 km downstream of Totladoh, for augmenting irrigation, for supplying cooling waters to the Superthermal Power Station at Kordih and will also cater to needs of industrial and domestic water supply to the Nagpur Metropolitan.

The project was taken up for construction in 1974 as a joint venture between the states of Maharashtra and Madhya Pradesh for harnessing 12,000 m³/sec discharge of the Pench river at the finally selected site at Totladoh, after carrying out detailed pre-construction geological investigations by the author between 1971 and 1974.

The Pench Hydel Project is located in an area of great geologic complexity, being underlain by the intensely folded and faulted metasedimentary rocks belonging to the Sausar Supergroup of Archean age. The experience of construction at the Kamthikheri Dam, located on the Pench river about 30 km downstream of Totladoh in a similar geological setting, was taken into consideration in planning the detailed geo-technical investigation of the Pench Hydel Project. During construction of the 32 m high Kathikheri Dam in 1971, major surprises like occurrence of cavernous marble, a major transverse fault and a deep burried channel, which escaped detection during the earlier geological and geophysical investigations, came to light at an advanced stage and necessitated major changes in design and adoption of unique and unconventional remedial measures to ensure stability and watertightness of the dam (Patil, 1971). The occurrence of a cavernous marble and a deep and extensive pool of water at the site of the Totladoh Dam were reported during the preliminary geological investigations of Pench Hydel Project scheme. A detailed geo-technical evaluation of the project was therefore taken up by the author to establish the feasibility of the site and to effective changes in planning and modifications in design layouts of the various component structures, for avoiding major surprises during construction (Patil, 1973, 1974).

A fresh programme of detailed geological mapping and subsurface exploration by drilling was undertaken at the power house site and along the different tunnel align-
ments, to evaluate the significant geological features, such as: details of the geologic structure, the depth of weathering along joint planes, the extent and depth of cavitation within the marble occurring at different appurtenant structures, effects of the various faults, shear zones, clay-filled seams and kaolised veins of pegmatite on the stability and construction of the underground power house and its ancillary structures, etc. This paper incorporates the results of pre-construction and construction stage geological investigations carried out by the author during the period extending from March, 1971 to June, 1977, by which time the major planning and design aspects of the Pench Hydroelectric Scheme had been finalised by the project engineers.

2. GEOLOGICAL SETTING OF THE PROJECT

The Pench H.E. Project consist of a 75 m high masonry-concrete gravity dam with a 201 m long central spillway, an underground power house with an intake and the twin pressure shafts, for diverting 120 m³/sec discharge of the Pench river into the two Francis-type turbines, a 95 m deep bus shaft connecting the power house cavity with the surface switch yard, a 857 m long approach tunnel leading to the main power house chamber and a 8057 m long tail race tunnel having a finished diameter of 8.1 m and gradient of 1:1586 for leading the power house discharge back into the Pench river at its outfall end at Gawalighat. The project is located in an area of complex geological set-up represented by the intensely deformed rocks of Saurashtra metasedimentary group belonging to the Archaean age. The Saurashtra Supergroup at the project site consists of the banded calc-granulites and calc-gneisses, the foliated schists and gneisses and thin bands of calcitic marble, which have been repeatedly folded into plunging and overturned synclines and anticlines. The Saurashtra rocks were supposed to have been deposited on the basement of ancient crystalline complex (Pascoe, 1950) and have been exposed along the narrow river basins of the Pench and Kanhan rivers after denudation of the overlying Deccan Traps. The Deccan basalts from an extensive plateau on the western bank of the Pench river and its remnants are occasionally seen also on the eastern bank. Coal-bearing Gondwana rocks are exposed on the northern and southern fringes of the Saurashtra basin, which extends over a length of 100 km in an east-west direction, encompassing the valleys of the Waihganga, the Pench and the Kanhan rivers in the Godavari basin.

2.1. Geologic Structure at Totladoh Site

The Saurashtra metasedimentary rocks exhibit a general east-west strike of foliation and moderate to steep downstream dips in the southerly direction. The trend corresponds approximately with the alignment of the main dam across the river Penna and is parallel to the longer axis of the sub-surface power house, located on the left abutment ridge, formed of the banded granulites (see Fig. 1). The main gravity dam and the underground power house complex at Totladoh are located within a narrow, tightly folded synformal structure involving a thick sequence of basal granulites of the Utakata Stage, which are overlain in turn by the thin bands of calcitic marble belonging to the Lohangi Stage and the soft schistose rocks of the Manusar Stage, the last group occupying the trough of the 'Totladoh Syncline'. The marble and the banded granulite have been repeated on the either limbs of the overturned syncline, which plunges at a low angle of 10° towards the west. The Saurashtra rocks have been subjected to severe folding and faulting of a very complex nature and have been profusely invaded by veins and dykes of granite and pegmatite along the planes of weakness, during the various stages of deformation (Pascoe, 1950).

The Saurashtra metasediments, namely the Manasa schists and the Lohangi marbles are usually associated with the manganiferous ore deposits of the Pench-Kanhan Valley in the Central India and have been described in detail by Shri G.V. Rao (1970). The rocks of the higher group in the Saurashtra sequence occur to the south of the project area in the form of thrust sheets around the Kamthikheri Dam. Dr. W.D. West (Pascoe, 1950) has also described an occurrence of a 'nappe' structure around Deolapar to the east of the area straddling the topographic divide between the Pench and the Bawanathi river basins. Continuation of several bands of cavernous marble along the strike, from the former to the latter, has already been established. This has given rise to the problems of leakage and inter-basin transfer of storage waters, needing adequate geotechnical appraisal (Patil, 1973).

2.2. Revision of the Seismic Factor

The project area has been considered a part of the stable shield in which all tectonic activity had ceased at the end of the Archaean era, followed by release of stresses along the numerous faults and fissures which resulted in close and multiple fracturing of the rocks. The project
Fig. 1. Geological map of the Pench Hydro-Electric Project, Totladoh with locations of the main gravity dam, underground power house and appurtenant structures.

falls in isoseismal II of the 1969 Bhadra-chalam Earthquake and a horizontal seismic coefficient of 0.02G has been provided in the design of the Totladoh Dam. However, following a series of tremors of moderate intensity in the Pench Valley in 1957 and also in the adjoining Tawa and Wainganga basins between 1957 and 1969 (Patil, 1975), it became necessary to revise the seismic status of the project. After consultations with the UNESCO expert at Koyanagar during March, 1975, an enhancement of seismic factor from 0.02G to 0.05G has been recommended.

2.3. Changes in Design Layouts of the Dam

On the basis of detailed geotechnical evaluation—the central spillway section, located in the main gorge of the Pench river, was shifted upstream by 60 m and the stilling basin was reduced by about 40 m to accommodate the main gravity structure on the fresh, hard granulite foundations (see Fig. 1). A 20 m wide horizontal apron has been provided to arrest the downstream scour of the cavernous bed-rock and retrogression of the soft, schistose strata along the spill channel. This, however, involved curvature of dam on the abutments and concentration of stresses on the end blocks with attendant problems of stability and additional keying in the firm bed-rock (Patil, 1977). The left abutment of the masonry dam is traversed by a 3 m wide transverse fault and closely spaced shears which extend into the approach tunnel and power house excavations (Patil, 1976). These problems were dealt on the basis of detailed geological studies of the rocks exposed in the foundation excavation of the dam and along the approach tunnel, for adopting necessary remedial measures.

2.4. Selection of the Power House Site

The principal problem in selection of the power house site was the concern on the part of the engineers to locate the main power house cavern free from any major ad-
verse geologic feature, which would pose serious problems of design and construction. It was desirable that the subsurface excavation for the powerhouse should be as far away from the cavernous marble as was practicable, for avoiding the possibility of diversion of the river and the consequent flooding of the cavity. The powerhouse site was also required to be free from any major zone of faulting or kaolinnisation, which was likely to affect its stability.

The originally considered site of the powerhouse, PH-1 (see Fig. 1), which was located on the steep hill-slope was eminently suitable from the above considerations, besides the availability of adequate rock cover above the powerhouse arch. The project engineers, however, took a decision to shift the site by about 50 m downstream on some considerations of design of the intake and pressure shaft structures and also on account of deep weathering of the rock. The change in location of the powerhouse involved corresponding changes in the layouts and design of the appurtenant structures, such as the approach tunnel, bus shaft, collection gallery, surge chamber, etc., and the possibility of approaching the marble band at the designed levels. It was, therefore, imperative to carry out detailed geotechnical evaluation of the powerhouse site by means of large scale precision mapping and subsurface explorations by drilling and trenching. The problems of cavernous marble and the possibility of flooding of the underground excavations became matters of crucial importance at the Pench Hydel Project (Patil, 1975).

3. GEOTECHNICAL PROBLEMS OF TUNNELLING

The total tunnelling work at the Pench H.E. Project, excluding that of the main powerhouse complex on the left bank, exceeds 11,200 m. This includes the excavation for the 857 m long approach tunnel, the twin pressure shafts with a combined length of 522 m, the 8057 m long tail race tunnel and of its two access adits at Amankori and Hathigota with lengths of 996 m and 773 m, respectively. The 6.1 m wide D-shaped approach tunnel with an average gradient of 1:9.76 and the 8.1 m high horseshoe shaped tail race tunnel having a slope of 1:1586 have been excavated across the foliation of rocks and cut across the entire sequence of Sausar rocks, including schists, granulites and the several bands of marble, repeated on the limbs of isoclinal folds. The two access adits in the T.R.T. complex have been driven nearly parallel or slightly askew to the foliation of granulites through which they are passing for a larger part of their lengths. The twin pressure shafts will be driven from the intake face mainly through the banded granulites across their strike. All the tunnels are beset with problems of deep weathering and instability of excavation at their inlet portals, the occurrence of major strike and transverse faults and the deeply sheared and kaolinnised pegmatite veins, apart from the hazards of excavation through the numerous bands of calcitic marble (Patil, 1974).

3.1. Excavation of the Approach Tunnel

Construction of the 857 m long approach tunnel through the various rock units of the 'Totladoh Syncline' posed difficult and often hazardous problems of tunnelling. These resulted from the deeply weathered and kaolinnised strata occurring at its portal, diversion of the Pench river through an interconnected solution channel in the marble and the frequent caving of jointed and kaolinnised blocks of pegmatite along a 50 m stretch of the fault zone, extending from the left abutment of the masonry dam (see Fig. 1). Another problem was of 'popping' of the soft, schistose rock while negotiating through the highly deformed zone lying within the trough of the syncline (Patil, 1976). Some of the geotechnical problems and hazards of excavation including the diversion of river through the cavernous marble were predicted with precision during the pre-construction stage investigations of the tunnel (Patil, 1974).

The exploratory drilling carried out along the alignment of the tunnel indicated the occurrence of a 'structural terrace' on the southerly limb of the syncline which resulted in an increased width of about 120 m of the marble band due to flattening of dip along the tunnel grade.

Permanent supports, rockbolting and thick concrete lining were provided, together with erection of the 'false portal', to stabilise the portal of the approach tunnel. Flooding of a 90 m section of the tunnel took place during April, 1976 consequent on taking a full-face blast by the engineers despite an evidence of increasing amount of seepage through the probe holes drilled in the soft and altered marble. The source of seepage was revealed to be an existence of a narrow solution channel developed along a strike joint and the scarp zone which marked the contact between marble and an intrusive vein of pegmatite. The seepage of the order of 3,000 l/min was controlled by means of reducer pipes and
employing large capacity turbine pumps, and after adequately supporting the affected zone, the seepage through the marble was successfully diverted by pressure-grouting the contact between marble and the intrusive pegmatite (Patil, 1976).

3.2. Excavation of the Pressure Shafts

The twin pressure shafts with a finished diameter of 4.11 m and separated from one another by a 6 m barrier of weathered and jointed rock, were excavated entirely through the banded granulites from an intake face of nearly 26 m height. The intake site is traversed by a 10 m wide strike fault which has been occupied by an intensely crushed and kaolinised vein of pegmatite (see Fig. 1). Excavation at the intake face was prone to major rock-slides which frequently blocked the inlet portals of the pressure shafts. After detailed evaluation of the geological factors and the criteria for design supports, an artificial or a 'false' portal was erected outside each portal of the pressure shafts and concrete wedges were provided on the crown to reduce the hazards of sliding during construction of the first 50 m horizontal sections of the tunnels.

3.3. Excavation of the Tail Race Tunnel

The 8057 m long and 8.5 m wide horseshoe shaped tail race tunnel cuts across all the formations of the Sausar rocks which are folded into assymetrical synclines and anticlines of very complex nature. Along the tunnel alignment, calcitic marble of the Lohangi Stage has been repeated at five places on each limb of the folded structure. Subsurface exploration has been carried out to locate these marble bands along the designed tunnel grade and to evaluate their karstic or non-karstic nature at depths varying from 90 to 150 m from the surface. The marble has a limited width due to its steep dip and is largely free from any significant solution activity to cause problems of seepage and diversion during the tunnelling operations (Patil, 1974).

The tunnelling operations have been relatively smooth and regular in an unaltered schistose and granulitic strata involving minimum overbreaks and supports during the excavation. However, the occurrence of faults and shear zones in certain reaches of the tunnel and kaolinised contacts of the pegmatite bodies, disposed horizontally on the tunnel crown, resulted in collapses at the Amakhori and Hathigota adit sites and cessation of work over long periods. These problems were tackled by providing adequate reinforcements and with provision of telescopic shield supports during excavation through the difficult reaches (Patil, 1977).

4. GEOTECHNICAL PROBLEMS OF THE POWER HOUSE COMPLEX

The main power house cavity, 50 m x 19 m x 20 m size has been located at the foot of the left abutment ridge and will be excavated entirely within the banded granulites of the Utekata Stage. The final site of the underground power house has been selected on the basis of results obtained from 12 nos. of deep exploratory drill holes, which have helped in locating the power house complex free from any major structural plane of weakness. The rock has, however, been traversed by the closely spaced strike and transverse fractures developed sympathetically to the faults and shear zones occurring on the western and the northern peripheries of the power house (see Fig. 1).

4.1. Field and Laboratory Tests

Flat Jack tests along and across the foliation strike of granulites were carried out in an exploratory drift which was excavated from the approach tunnel, aligned parallel with the longer axis of the subsurface power house. These tests indicate development of negligible stresses on the cavity walls likely to cause deformation. Laboratory tests were also performed on the core specimens of rocks obtained from a 12 m zone of poor core recovery occurring above the power house arch (see Fig. 2).

Specific gravity of the banded granulites varies from 2.92 to 3.02 and the compressive strength of rock ranges between 1014 and 1571 kg/cm². The modulus of elasticity of the rock is of the order of 10.4 x 10⁵ kg/cm² at a stress level of 314.26 kg/cm². On an analysis of the data from the modulus of elasticity tests on several core specimens, it was noted that the 'E' value of granulite increases gradually with the corresponding increase in stress levels while the marble cores are seen to develop relatively higher strain as compared to the granulite for the identical stresses (Vaidyanath, 1978). This indicated that deformation in the granulite is much less at higher stresses compared to that in the marble; this consideration has led to the idea of locating the subsurface power house at Totladoh entirely within the granulite and away from the influence of marble, apart from the problem of cavitation within the latter at the power house grade.
From this, the applied horizontal stress can be calculated as follows:

\[ S_h = M \times S_v = \frac{0.25 \times 29}{1-0.25} = 9.7 \text{ kg/cm}^2 \]

where \( M \) is the constant equal to \((r/1-r)\), \( r \) being the Poisson's ratio.

Taking into consideration the width to height ratio of the cavity and using these values in the graphs prepared by Obert, DuVall and Merill (Verma and Tiwari, 1973) the critical values of the compressive and tensile stresses acting on the cavity can be calculated. The safety factors are the ratio of compressive strength to compressive stress and the ratio of tensile strength to the maximum tensile stress. The safety factors calculated from the above figures in case of the rocks at the Pench Hydel Project are nearly five times the average values of 4 to 8 assumed for the stability of an underground power house cavity (Patil, 1975).

The actual tunnelling conditions are, however, modified by complexity of geologic structure and occurrence of the planes of structural weakness, such as faults, shears and kaolinised seams whose orientation with respect to the tunnel face actually governs the pattern of excavation and stability of an underground cavity. Opening of the 95 m deep bus shaft from surface to the power house arch has facilitated the geotechnical evaluation of the rock strata occurring at the power house site at Totladoh.

4.3. The Bus Shaft Excavation

The 6 m x 6 m size bus shaft has been located above the power house cavern in a sheared and deeply kaolinised rock which indicated requirements of heavy supports and lining down to a depth of 40 m from the surface (see Fig. 2). The transverse shear zone with an average width of 10 m and steeply dipping towards the west has caused complete kaolinisation of the 17 m thick vein of pegmatite occurring above the roof of the power house, down to R.L. 400 m as indicated by results of an exploratory drill hole P-9 drilled along the bus shaft. This shear zone appeared to be extending to the centre of the power house arch, where its width is expected to be narrowed down to about 5 m. This zone is likely to extend further downwards below the floor level of the power house.

The excavation for bus shaft was carried out by the 'well-sinking' technique which is normally adopted in loose alluvial strata.
Excavation in the kaolinised rock by this method created problems of stability due to differential settlement, development of earth pressures and large scale caving of the decomposed material into the shaft. The bus shaft excavation was required to be adequately supported for nearly half the depth and special efforts were necessary to prevent tilting of the staining walls (Patil, 1977).

The bus shaft excavation has revealed the critical nature of geotechnical problems facing the construction of subsurface power house at the Pench Hydel Project. A persistent zone of low core recovery met with along the exploratory drill holes between R.L.s 360 and 372 m above the crown has resulted due to the steeply dipping shears developed sympathetically to a major transverse fault, delineated on the western side of the power house chamber and created only the problems of local overbreaks rather than those of a horizontally extending zone of weakness lying immediately above the power house arch.

The critical geotechnical problems connected with the underground excavation for the power house and its ancillary structures at the Pench Hydro-Electric Project at Trotlahor were evaluated on the basis of detailed geological studies and adequate remedial measures were planned in advance to ensure the stability of the subsurface structures.

5. ACKNOWLEDGEMENT

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6. REFERENCES


TUNNELLING EXPERIENCE AT TEHRI DICATES THE CHOICE OF LAYOUT AND SUPPORT METHODOLOGY OF UNDERGROUND POWER HOUSE

L'EXPERIENCE DE PERCEMENT A TEHRI COMMANDE AU CHOIX DE TRACE ET METHODOLOGIE SOUTIENNE D'UNE USINE GENERATRICE SOUTERRAINE

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ABSTRACT

Tehri dam project envisages the construction of a 250 m long, 22.5 m wide and 50 m high underground power house on the left bank of river Bhagirathi, 0.5 km downstream of the proposed 260.5 m high dam. The project also includes four division tunnels--two on each bank of the river.

The rocks encompassing the project area are phyllites of Tehri formation which have been differentiated in three grades on the basis of physical appearance and overall engineering properties. The geomechanical analysis of the structural discontinuities in these rocks have revealed that the foliation planes and foliation joints are the major discontinuity surfaces, thus emphasising the need to orient the cavity at right angle to foliation. The remarkably large differences in the elastic properties of the different grades of phyllites have largely been instrumental in locating the cavity in anyone single unit, as far as practicable, to avoid differential load development.

The experiences of tunnelling and the results of instrumentation of the heading reaches of the already constructed right bank diversion tunnels through similar grades of phyllites have been discussed. These have in turn been utilised for arriving at the methodology of excavation and support requirement of the power house cavity. In addition, the experience, of already constructed cavity at Chibbro power house, having rocks of similar engineering parameters and tectonic set up, have been taken as a guide line for the proposed Tehri underground power house.

ABSTRAIT

Le project de barrage à Tehri envisage la construction d'une usine génératrice souterraine de 250 m long, 22.5 m large et 50 m haut au bord gauche de la rivière Bhagirathi; 0.5 km en aval du barrage proposé de 260.5 m haut. Le projet aussi comporte quatre tunnels divertissements--deux à chaque bord de la rivière.

Les roches se composant de la surface de projet sont phyllites de la formation Tehri qui ont été différenciée en trois grades sur la base de l'aspect physique et des propriétés totales d'ingénierie. L'analyse géomécanique des discontinuités structurales dans ces roches ont révélé quelles planes de foliation et des joints de foliation sont des surfaces de discontinuités majeures, accentuant ainsi la nécessité d'orienter la cavité normale à foliation. Les différences remarquable-ment larges dans
propriétés élastiques des degrés différents de phyllithes a été largement instrumental en décidant à déterminer la place de cavité dans une unité particulière quoique ce soit, autant que practicable, pour éviter un développement différentiel de charge.

L'expérience de percement et les résultats d'instrumentation de l'atteinte avancée des tunnels à diversion bord droit déjà construits par des dégrés pareils de phyllithes ont été discutés. Tout cela, en proposit, ont été utilisé pour arriver à la méthodologie d'excavation et nourrit l'exigence de la cavité d'usine génératrice. En plus l'expérience, de cavité déjà construite à usine génératrice à Chibbro, ayant des roches des paramètres d'ingénieries pareils et préparation tectonique, ont été pris comme un exemple pour l'usine génératrice souterraine proposée à Tehri.

INTRODUCTION

For the geotechnical evaluation and analysis of any larger underground opening, apart from the study of geological and tectonic set-up, in the current practice, test sections are utilised for stimulation of load development and evolve excavation procedure. The proposed 260.5 m high Tehri dam having 2000 MW installed capacity, an underground power house having a machine hall 250 m long, 22.5 m wide and 50 m high, the 1298 m and 1429 m long and 13 m diameter diversion tunnels, excavated on the right bank of the river (Plate I) through similar geological formation as is expected in case of power house cavity have been simulated as a test section for extrapolating the excavation sequence and support requirements.

The rock encompassing the project area belongs to phyllites of Chandpur series (Audem). The phyllites have been classified broadly in three categories mainly on lithologic and physical characteristics. The most competent and the best quality of phyllite on engineering consideration is termed as phyllite grade-I and corresponds to phyllitic quartzite, whereas the poorest type is designated as grade III phyllite which is very thinly foliated, sheared and shattered in nature and corresponds to schistose phyllite. The intermediate one is termed as phyllite grade-II and is the most predominantly exposed in the project area. The machine hall cavity, which is the single largest cavity, is expected to encounter phyllite grade-I, grade-II and grade-III in 60%, 35% and 5% respectively.

The compressive strength of grade-I phyllite is generally of the order of 1300 kg/cm² and that of grade-II phyllite is of the order of 700 kg/cm². The modulus of deformation as determined by plate bearing tests for grade-I, grade-II and grade-III phyllites are of the order of 0.113 x 10⁵ kg/cm², 0.048 x 10⁵ kg/cm² and 0.018 x 10⁵ kg/cm² in horizontal direction and 0.06 x 10⁵ kg/cm², 0.055 x 10⁵ kg/cm² and 0.05 x 10⁵ kg/cm² in vertical direction, i.e. the ratio of the values of D for grade-I and II in horizontal and vertical direction is 1:2 and 1:6 respectively.

The foliation trend of phyllites exposed in the project area varies in general from N55°W-S55°E to N80°W-S80°E with a dip of 35° to 55° in southwesterly (downstream) direction. These exposures represent the southerly limb of an anticline plunging in southeasterly direction. Several minor folds in the form of board warping or tight puckering are responsible for local variation in strike and dip of foliation.

The structural discontinuities are present in the form of joints and shear zones. Such structural discontinuities recorded in the 336 m long exploratory drift to power house site, have been plotted in equal area net (Plate 2). From a perusal of the data of the following three sets of joints are inferred.

<table>
<thead>
<tr>
<th>Trend</th>
<th>Dip</th>
<th>Nature</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. N63°W-S63°E</td>
<td>52°/827°E</td>
<td>Foliation</td>
</tr>
<tr>
<td>2. N71°W-S71°E</td>
<td>52°/N19°E</td>
<td>Joint, tight</td>
</tr>
<tr>
<td>3. N62°E-S62°W</td>
<td>45°/N28°W</td>
<td>Joint, tight</td>
</tr>
</tbody>
</table>

IV. 84
These joints are invariably tight, with continuity along strike ranging from a few cms to 5 m. In addition to these joints, a few zones showing shearing and having gouge/rock flour of thickness varying from a few cm to 40 cm have also been recorded in the drift and are classified below in order of prominence:

1. N63°W-S63°E 52°S27°W Foliation shear with gouge of varying thickness from few cm to 10 cm.

2. N62°E-S62°W 45°N 20°W Shear zones with gouge of varying thickness from few cms to 10 cm.

3. N80°E-S80°W 43°S10°E Shear zones with maximum 30 cm gouge.

4. N55°W-S55°E 55°S35°W Shear zones with maximum 40 cm thick gouge.

5. N-S 30°N -do-

Relation of these eight (3 joints and 5 shear zones) discontinuity surfaces vis-a-vis the orientation of the cavity are depicted on plate-3.

2. EXPERIENCE OF DIVERSION TUNNELS

2.1. General

In order to divert about 9000 cumec's full monsoon discharge of river Bhagirathi, 4 numbers of 13 m excavated diameter diversion tunnels have been proposed (Plate-1). These diversion tunnels are being excavated by conventional drilling and blasting and in view of the constructional constraints and not very good geological conditions of the tunneling media, the excavation is being carried out by heading and benching.

The right bank diversion tunnels (T3 & T4) where the excavation for heading reaches are complete are 1298 m and 1429 m long and have encountered phyllite grade-I, grade-II and grade-III in about 30%, 60% and 10% respectively. These diversion tunnels are aligned in N6°W-S6°E direction and are thus 50° askew to the trend of foliation. The cover above crown varies from a few meter in the inlet and outlet reaches to a maximum of 250 m.

The excavation of these tunnels have been carried out from both inlet end, in general the blasting was followed by defuming the tunnel, removal of muck, construction of wall beams erection of primary supports in the form of steel ribs, concreting/pumpcreting beyond ribs, drilling of holes and its charging for next blasting. The complete cycle for a pull of 2.4 m has taken about 36 to 40 hrs in general (Plate 4). In case of excavation from outlet end, since better rock conditions were met, after removal of muck, the face was again drilled, charged and blasted whereas the erection of support and related processes lagged 10-15 m behind face in order to reduce the cycle times.

The overbreak above the crown in general is of the order of 0.10 m to 1.0 m. The higher order of overbreaks above crown at certain reaches were mainly due to certain foliation/cross shear zones and specially when such reaches were water charged and in such case a maximum of 12 m of overbreak had occurred.

2.2. RESULTS OF INSTRUMENTATION AND STANDUP TIME

2.2.1. Instrumentation

In order to get a picture of behaviour of rockmass after tunnelling and specially to visualise the pressure exerted on the steel ribs, instrumentation were carried out in phyllite grade-I encountered in T4 tunnel by installing one load cell each at right and left springing at 699.3 m, one load cell at crown at 698.5 m and one stress meter below wall beam at 699.3 m from inlet side, immediately after excavation.

The load recorded at different positions are plotted against the time (Plate-5) and the plot indicates that the development of load is maximum at the crown and has recorded a maximum of 89.75 Tonnes whereas that on the left and right springing are comparatively less and have attained a maximum of 35.80 and 28.15 Tonnes respectively. It is also clear that after 40 days of excavation, the load is almost stabilised indicating thereby that the process of redistribution of stress is complete by 40 days time.
Similar observation can be drawn from the stress meter data which shows that after initial fluctuation from a stress of zero to a maximum of 5.4 kg/cm², the stress field becomes almost constant at 2 kg/cm² after 40 days time.

2.2.2. Standup time

As discussed in chapter 2.1 above, in the tunnelling operation of right bank diversion tunnels primary supports were immediately provided after excavation, but at certain reaches where grade-I phyllites were encountered and the span was considered relatively self supporting, a span of 4.8 m length was left unsupported between two supported spans of 4.8 m on either sides. From such an unsupported reach at about 341 m from outlet portal of T4 tunnel, a chunk of 0.5 m³ fell after seven months of excavation. A span of 70 m from 190 m to 260 m from outlet portal remained unsupported in phyllite grade-I for a period of 3 months, when the tunnelling operations were suspended. Although these values may not represent actual standup time but for practical purpose it may be taken as limiting value of standup time. In tunnel T3 at 590 m from outlet side, from grade-II phyllite, rock fall started from crown after 440 hours (i.e. about 18 days) of excavation.

These values of standup time have been plotted against the excavated tunnel diameter of 13 m (Plate 6). It is clear from the plate that the C.S.I.R. geomechanic score (Beinawaski, 1974) comes to be 85, 77.5 and 72 respectively for the above three cases. Thus the grade-I is definitely in the good rock category whereas the grade-II is in the marginal fair/good rock category and it is expected that the grade-III shall be in the poor/fair rock category.

2.2.3. Analysis of data

It is clear from the above discussion that in case of grade-I phyllite, where instrumentation have been carried out, about 90 Tonnes of load has developed above the crown and about 25 Tonnes and 15 Tonnes of loads have developed on the left and right springing within 40 days of excavations. These loads have developed on the steel ribs placed at 80 cm c/c. The compressive strength of grade-II phyllite is nearly half of that of grade-I and the standup time is also less as compared to grade-I is expected to have more load and grade-III with still less value is expected to have the maximum load.

3. EXTRAPOLATION FOR UNDERGROUND POWER HOUSE

3.1. From Diversion Tunnel Data

From the instrumentation results (Plate-5) it is clear that 50 Tonnes of load has been mobilised within 15 days of excavation in grade-I phyllite in case of diversion tunnel. The machine hall cavity of the power house which will have about 9 times more unit cross-sectional area than that of diversion tunnel, the period may be reduced to even 7 days or less for phyllite grade-I and still less for grade-II and grade-III phyllites, indicating thereby that the primary supports must be provided within such period, otherwise there are chances of heavy falls starting from the crown. This suggests that time between excavation and providing the supports should be minimised for machine hall cavity.

3.2. Rock Load Estimates

Shome and Kumar (1979) examined the pattern frequency and spacing of structural discontinuities and plotted them on the geological section along the short axis of the power house. On the basis of the wedge formed by the intersection of the prominent sets of discontinuities, a triangular rock load of 12 m to 16 m and equivalent rectangular rock load of 6 m to 8 m have been anticipated on the crown of the cavity.

3.3. Experience from Chibbro Power House Cavity

Chibbro power house for Yamuna Hydel Scheme Stage-II, Part-I is the only underground power house constructed in the Himalayan terrain. The engineering properties of limestone slate sequence of Mandhal series encountered in Chibbro cavity are almost similar to that of grade-I phyllite of Tehri and the expected rock load above crown is also similar (6 to 8 m rectangular wedge) in both the cases. On the basis of this
similarity, Shome and Kumar (1979) suggested that for stabilising the toe wall at Tehri underground power house a treatment similar to that of Chibbro (where 400 pre-stressed cable anchors each of 60 Tonnes capacity were installed) may be required. In addition, they suggested 10 to 15 cm thick shotcrete laid on chain link fabric, as has been done in Chibbro, immediately after excavation to prevent destressing of rock.

4. CONSIDERATION FOR LAYOUT AND SUPPORT SYSTEM

There is a wide variation in the physical and elastic properties of the different grades of phyllites and grade-I is the best type in engineering consideration. Such a large cavity should always be located on the best medium available, but the geological distribution of the lithounits in the project area (Plate-1) shows inability to locate a single grade-I band required to locate entire structure, the exposure of the thickest grade-I phyllite practically dictated the location of the power house cavity. In order to have the poorer grade-II and grade-III phyllite bands to the minimum possible length in the cavity to avoid differential load development along the crown, the cavity has been aligned at right angle to the trend of the foliation of phyllite. Incidentally, the best tunnelling conditions are expected in the direction normal to the foliation. Moreover, by doing so the foliation shears and foliation joints, which are the most prominent discontinuities will be cut in the shortest possible length along the crown of the cavity.

As discussed in chapter 3.1, that first 7 days or even less will be crucial and all primary supports should be erected within this time, in case of machine hall cavity, it may not be possible to complete excavations and provide support for the entire span within this period, multiple drift method of excavation is required for such a larger dimension cavity. Since for 13 m excavated diameter diversion tunnel, the excavation has been carried out in heading and benching reaches, for 50 m high and 22.5 m wide machine hall cavity, the heading portion should also be split up in at least three phases. Similar tech-

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GRAPH SHOWING STANDUP TIME PLOTTED AGAINST EXCAVATED DIAMETER
GEological Set-up and Prediction of TunnELLing Problems Along Tunnel Alignment, LOKtak HyDEL Project, MANIPUR, INdIA

Situation de l’Etude et la Prediction des Problemes de Percement de Tunnel Le Long du Trace de l’Installation Hyroelectrique de Loktak a Manipur (Inde)

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ABSTRACT

Six and a half kilometer long main head race tunnel of Loktak Hydel Project in Manipur passes through lake sediments, terrace deposits and rock units of Disang Group. The Disang Group of rocks show three generations of folding. The early fold is tight in nature having angular hinge, sub-horizontal axial plane and axis. The folds are of recumbent to reclined to inclined type.

The second generation of folds controlling the topography are upright having rounded hinge. The axial plane of this fold has general N - S trend and dip steeply towards East or West, marked by fracture cleavage. The general trend of the axis is NNW to NNE.

The third generation of folds are also upright to inclined type with rounded hinge. The axial plane of the fold is marked by another set of fracture cleavage having E - W to NW - SE trend with steep angle towards N or S. The trend of the axis is NW - SE.

Two vertical major faults have traversed the tunnel alignment having western block as upthrown side.

Flowing ground condition in unconsolidated saturated lake sediments and terrace deposit, roof collapse, heaving of floor, squeezing ground in soft and splintery shale-siltstone lithounits were predicted from the study of this geological set-up and compared favourably with actual tunnelling condition.
ABSTRAIT

Un tunnel étudié du canal de dérivation du central hydroélectrique à Manipur, long de 6 km, est percé à travers les dépots du lac, les exploitations de trop plein et les unités rocheuses du bloc Miseng. Les rocs du bloc structural Miseng présentent des pliements de trois âges. Le plissement ancien semble serre formant une articulation angulaire, entre le plan sub-horizontal axial et l'axe principal.

Les plus du second âge dominant la topographie et tiennent droits à articulation arrondie. Le plan axial de celui-ci a une orientation normale N - S, et s'incline abruptement vers l'ouest, marque plan de cassure. L'orientation normale de de l'axe est en direction vers nord-nord ouest à NE Est.

Coeurs du troisième âge sont également de genre droit ou incline a articulation arrondie. Le plan axial du pli est marqué par une nouvelle série de clivage en direction est-ouest ou Nord ouest-sud est, mais avec une inclinaison abrupte vers le nord ou le sud. Cet axe est en direction Nord-Ouest-Sud-Est.

Deux failles majeures et verticales passaient à travers le trace du tunnel dont la levre soulevée formait le bloc occidental.

D'après cette étude géologique on a prédit les conditions de la surface de l'écoulement en pente dans des sédiments lacustres saturees et non affaisées, l'effondrement de la terrasse, soulèvement du sol, du terrain schisteux et deux des unités filoniennes et on a rapproché propice à la situation actuelle de parois de tunnel.

Introduction

The Loktak Hydel Project in Manipur proposes transfer of water from Loktak Lake to Leimatak valley for generation of 105 MW power at 60% load factor. Power draft of 42 m and drop of 310 m will be utilized for this purpose. The essential components of this scheme consist of a barrage across Manipur River at Ithai to regulate water of Loktak Lake and 10.26 km long water conductor system to transfer water from one basin to another (Fig.1).

Out of this 10.26 km long water conductor system, a major portion (6.77 km) consists of a 3.81 m finished diameter tunnel, which pierces across a N-S hill range forming the water divide between the two basins. The construction of this project commenced in 1971 and tunnel boring was completed in October, 1981 through one adit and three shafts.
In order to prepare a dependable geological section along the tunnel alignment, systematic surface geological study was conducted at this project during its construction phase. Initially a photogeological base map was prepared on 1:60,000 scale (Fig.II), followed by traverse mapping of Loktak Hydel Project Manipur Interpreted Photogeological Map of Loktak Area.

25 sq.km. area in and around the tunnel alignment on 1:50,000 scale. Geological section has been prepared from the surface geological map to interpret the probable tunnelling condition and the anticipated problems. It may be mentioned that very meagre sub-surface drill hole data are available along the tunnel alignment which could be utilised for interpretations. Feasibility of rock exposures was another constraint which limited the observations along road and mule cuttings only.

Geology

The lithological units encountered in the tunnel area are lake sediments, terrace material and rock units of Disang (Eocene) Group from East to West. Silt, sand and pebbles of variable proportions constitute the lake sediments. The terrace material contains broken rock fragments and large size boulders in addition to silty and sandy fractions.

The lithological units represented by Disang Group are mainly sandstone, shale and siltstone in variable proportions. The sandstone is generally fine grained with silicious matrix and often contains thin calcite veins. Siltstone is the predominant rock
type in the area and is more abundantly exposed on the eastern side of the main hill range. Shales, varying in colour from grey to black are well exposed towards Laimatk valley. Based on the observations made in this area, the rocks have been classified into the following groups (Fig. III).

Shale
Sandstone
Shale with subordinate siltstone/sandstone
Siltstone with subordinate shale
Sandstone with shale partings

LOKTAK HYDEL PROJECT MANIPUR
MAP SHOWING ROCK TYPES ALONG TRAVERSE LINES IN LOKTAK AREA

Structure

Folds: The area shows three generations of folding. The early folds in bedding is marked by minor S, E and N shaped folds, which are very tight in nature, having angular hinge, sub-horizontal axial plane and axis. The axis plunges at low angle towards north or south (Fig. IV).

The second generation of folds control the topography with synformal axis more or less forming hills and antiformal crests cut by naias. These folds are upright having broad rounded hinge. No thickening of beds has been observed in the axial region of the folds indicating them to be of parallel type. The axial plane of these folds has a general N-S trend and dip steeply towards East or West marked by fracture cleavage. The general trend of the axis is NNW to NNE and plunge at low angles (Fig.V & VI).

The third generation of folds are also upright to inclined type with rounded hinge. The axial plane of these folds are marked by another set of fracture cleavage having E-W to NW-SE trend with steep angle towards N or S. The trend of the axis is NW-SE and dips at moderate to steep angle.
Faults: Photogeological study indicated three faults cutting across the tunnel alignment. They are located (a) near adit, (b) midway of the tunnel route and (c) close to construction shaft. Ground checks confirmed the fault close to adit as a major dislocation bringing shales of Lower Disanga in juxtaposition with arenaceous members of Middle Disanga. This is a vertical fault having western block as upthrown side. The throw of this fault could not however be estimated due to absence of any marker horizon. Along old road east of Lendan village fault trends NE – SW having 50° dip towards SE. This appears to be the same fault as identified in the photogeological scanning cutting across the tunnel alignment in the middle of its route.

Ground water

The ground water in the hilly area has been observed to circulate within the weathered mantle and open fractures in rocks and emerges out as springs at two elevations of (i) 675 to 835 m, and (ii) 1100 to 1145 m.
mainly along the contact plane of sandstone and shale. The majority of these springs emerge much above the tunnel grade and are the principal source of water for surface streams draining the hill slopes before debouching into the Loktak plains. The water table is high in the unconsolidated sediments in Loktak plains and occasionally occurs under artesian condition close to the lake periphery. Some of the drill holes drilled in the cut and cover section recorded low artesian head at a depth of 14 m below the ground level.

Prediction of tunnelling problems

Based on the data collected from surface observations an interpreted geological section has been prepared between Adit and construction shaft to bring out the broad geological features along the alignment and study their significance to tunnelling condition (Fig. VII).

A reference to this section shows that the alignment passes through shales from adit to 660.00 m, where a major vertical fault is intercepted. The tunnel alignment cuts the first synformal structure between 725 and 1060 m from adit and the axial plane at 935 m approximately. The second synform is intercepted between 2000 and 2250 m from adit along the tunnel grade. Both these synforms are likely to be occupied by highly fractured sandstone, while the rest of the tunnel will pass through siltstone/shale alternations in 80:20 proportion. Another major fault is likely to intercept the tunnel grade at 2450 m on the western limb of the second synform. Minor folds and faults may be encountered in between which is beyond the scope of interpretation.

The contact between the unconsolidated terrace deposits and rock strata will be met with at a distance of 3650 m from adit, close to the construction shaft. East of...
this shaft the unconsolidated sediments gradually merges into the lake sediments with progressive increase in the finer clastics.

Since lithology, structure and ground water regime control the tunnelling condition in a media, complex tunnelling problems were predicted for the Lekhak tunnel in view of the complicated geological set-up of the region.

Flowing ground condition was predicted in course of tunnelling through lake sediments and terrace deposits in view of the unconsolidated nature of the sediments and high water table condition. The presence of impervious clay layer towards lake periphery and more pervious horizon towards the hill top presented an ideal situation suited for artesian condition and indicated profuse seepage of water into the tunnel heading.

Roof collapse, heaving of floor and squeezing of strata were predicted in shale-siltstone lithounits which have been subjected to intense deformations due to folding and faulting. It was indicated that such phenomenon will be severe in the closure region of the major folds and highly crushed zones associated with faults. In sandstone reach roof falls would be in the form of large size blocks guided by the prominent divisional planes in rock unit. While the overall seepage condition would be favourable for tunnelling in rocky reach, moderate to profuse seepage may necessitate provision of adequate pumping arrangements in the sandstone zone at the axial region of the synform.

Release of methane gas trapped in joints, and fissures would pose constant safety hazard throughout the tunnel length, but the concentration would be more in crushed zone in shales; where special precautions would be necessary to dilute this inflammable gas.

Actual tunnelling conditions

After the completion of tunnel boring at Lekhak Project an attempt has been made to compare the actual tunnelling condition with the one predicted by surface geological observation. It has been observed that although the overall tunnelling condition compared favourably, there was some changes in the lithology at the tunnel grade due to variation in the geometry of the fold at depth. A reconstructed geological section based on the tunnel logs has been prepared and presented in Fig. VIII to highlight the variation.

Severe squeezing of strata and heaving of floor took place in shale-siltstone lithounits between 650 m and 1000 m resulting in heavy twisting of steel supports. Methane posed a continuous tunnel hazard throughout its length but the concentration was heavy in crushed zone in shales close to fault dislocation around 650 m. Seepage of water was moderately heavy in the synformal closure region at 2100 m from adit.

Flowing ground condition was a common phenomenon in saturated lake sediments and terrace deposits and posed tunnelling problems east of construction shaft. Special tunnelling techniques like forcopiling,
GEOLOGICAL SECTION
ALONG LOKTAK TUNNEL ALIGNMENT, MANIPUR.

INDEX
- : Terrace
- : Siltstone - shale with minor sandstone bands
- : Shale
- : Sandstone with shale partings
- : Sandstone with siltstone partings
- : Siltstone with shale partings
- : F Fault

FIG-VIII

advance grouting, heading and bench were adopted to advance tunnels in this reach.

Conclusions
Although interpretation of sub-surface geology from surface geological observations has some limitations in a complex geological set-up, it helps to bring out the broad geological features and its implications to tunnelling condition. While more reliance has to be kept on day to day geological observations at tunnel heading and advance probing for accurate interpretation of tunnelling condition, this study has a major impact on the overall planning in a tunnelling project. With advancement of tunnelling techniques it is expected that more accurate planning of a tunnelling project will be possible in future with the above guide lines and the surprises will be brought down to minimum.

IV.100
GEOLOGICAL ASPECTS OF TUNNELLING IN THE SEDIMENTARY FORMATIONS OF THE BHIMA SERIES, KARNATAKA, INDIA

ASPECTS GEOLOGIQUES DE PERCEMENT DE TUNNELS DANS LES FORMATIONS SEDIMENTAIRES DE CATEGORIE BHIMA, KARNATAKA, INDE

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ABSTRACT

Rock mass belonging to the Bhima series of pre cambrian age extend over an area of 4665 km² in Karnataka. They are equivalents of Kurnools which are more famous in Indian Geology. The Bhima series essentially consist of alternate beds of limestone and shale. On a regional basis the beds are horizontal, but locally a dip of less than 5° due south is noticed in both the beds. The limestone is high calcium limestone with more than 45% CaO with horizontal and vertical joints. The thickness of beds of both the formations varies from a few centimeters to a meter. The limestone is hard and competent whereas shale is soft and an incompetent rock mass. Shear zones of varying width occur at places in both the rocks. Two tunnels - one in shale and the other in limestone, in the Upper Krishna Project on the Narayanapur Left Bank Canal (NLBC) are being driven. The tunneling method of heading and benching is adopted to suit the physical and chemical properties of the rock mass. Based on the experimental blasts conducted by the author, the optimum overfall in the shale and limestone tunnels is estimated to be 10 to 12% and 11% to 14% of the cross sectional area of the respective tunnels. In view of the geological features, physical and chemical properties permanent support for the entire length of the each tunnel is recommended as against the 20% of the length of tunnels proposed earlier by the World Bank team. In order to avoid the accidents in the sheared zones especially in limestone, the permanent support even at closer intervals with the reduced pull per blast are recommended.

ABSTRACT

Une masse rocheuse, appartenant aux catégories Bhima de l'ère pré-cambrienne, s'étend sur une surface de 4665 km² dans le Karnataka. Elle présente des points communs avec les célèbres "Kurnools" de la géologie indienne. Les catégories Bhima sont formées de couches alternées de calcaire et d'argile schisteuse. A l'échelle régionale, les couches sont horizontales mais on observe localement une pente des deux couches inférieure à 5° vers le Sud. Le calcaire est à haut pourcentage de calcium, avec un taux supérieur à 45% d'oxyde de calcium, avec joints horizontaux et verticaux. L'épaisseur des couches des deux formations varie de quelques centimètres à un mètre. Le calcaire est dur et résistant tandis que l'argile est tendre et non-résistante. Des zones fissurées de taille variable se trouvent dans les deux formations. Deux tunnels, l'un dans la masse argileuse et l'autre dans la masse calcaire, sont percés à travers la rivière gauche du canal Narayanapur du projet "Upper Krishna". Les méthodes de forage sont celles de "heading and benching", c'est-à-dire forage de la moitié supérieure puis forage de la moitié inférieure des tunnels. Cette méthode est adaptée aux qualités chimiques et physiques de la masse rocheuse, selon les résultats des forages expérimentaux réalisés par l'auteur. La tolérance optimum pour les tunnels forés dans l'argile et le calcaire...
est estimée 10 à 12% et 11 à 14% de la surface de la section transversale de chacun des tunnels respectifs. En raison des caractéristiques géologiques, physiques et chimiques, un étayage permanent sur toute la longueur du tunnel est recommandé, de préférence à un étayage de 20% de la longueur du tunnel, qui avait été proposé auparavant par l'équipe de la Banque Mondiale. En vue d'éviter les accidents dans les zones fissurées particulièremment dans le calcaire, on recommande un étayage à intervalle rapproché, avec un avant-emplacement réduit pour chaque charge d'explosif.

1. INTRODUCTION

1.1. Upper Krishna Project is a multi-million Irrigation Project, which envisages construction of two dams across the river Krishna in the Northern Maidan region of Karnataka. The upstream dam is at Almatti and the other dam is at Narayanapura. The project is expected to be completed by 1993.

1.2 The Narayanapura Left Bank Canal (N.L.B.C.) which takes off from the Narayanapura dam passes through two Tunnels which are being driven in the sedimentary formation of Bhima series of Pre-Cambrian age (Fig. 1). There are a few tunnels elsewhere in Karnataka, driven in granitoids, but driving a tunnel in a sedimentary formation is a new experience both for geologists and engineers of the State. Hence, the geological aspects which have influenced the tunneling methods, overfall, seepage etc., are discussed in this paper.

2. DETAILS OF THE TUNNELS

Among the two tunnels the Rajankollur (R.K) tunnel and the Gundalgera (G'gera) tunnel are 3.003 km. and 856 m. in length with a clear height of 10.1 m., with a base width of 10.1 m. The R.K. tunnel is between 36,531 kms. and 39,534 kms. in length and the G'gera tunnel is between 55.5598 and 54,742 km. in length over the N.L.B.C. (Fig. 1). Both the tunnels are horse-shoe shaped.

3. GEOLOGY

3.1 Regional Geology - The formations at the sites of the tunnels are listed below in the chronological order:-

<table>
<thead>
<tr>
<th>Formation</th>
<th>Age</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>Recent</td>
</tr>
<tr>
<td>Deccan Trap</td>
<td>Cretaceous</td>
</tr>
<tr>
<td>Limestone</td>
<td></td>
</tr>
<tr>
<td>Shale</td>
<td>BHIMA</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Precam-</td>
</tr>
<tr>
<td></td>
<td>brian.</td>
</tr>
<tr>
<td>Conglomerate</td>
<td></td>
</tr>
<tr>
<td>Granitoids</td>
<td>Archaean</td>
</tr>
</tbody>
</table>

3.2 Local Geology - The R.K. Tunnel passes through entirely in shale overlain by limestone of Bhima series (Fig. 2). The Gundalgera tunnel passes through entirely limestone of Bhima series overlain by Deccan Trap (Fig. 3).

4. PHYSICAL PROPERTIES OF ROCKS

Physical properties of a rock through which a tunnel passes through influence the tunneling operations and the overfall. Therefore, the properties of shale and limestone are described below:-

4.1 Shale - It is a hard rock when fresh, but after a few weeks' exposure to air it starts swelling or scaling off due to absorption of moisture. Ultimately, as could be seen in the open cuttings and the experimental tunnel, the shale looses its strength & converts into a purple coloured powder. There is a tendency to break along the bedding plane, the major plane of weakness. The vertical joints further weaken this rock mass and act as planes for circulation of groundwater to a certain extent.

4.2 Lime Stone - It is light grey in colour. Between any two joint planes, it yields rock material which is hard and compact. It is high calcium limestone with CaO content of more than 40%. It is soluble at places along certain planes acting as conduits for circulating groundwaters.

5. STRUCTURAL FEATURES

5.1 Shale - Shale is thinly laminated and almost horizontal with a very low angle dip of 2° to 3° due South. It is intersected by vertical joints and crushed zones (Fig. 4). The thin horizontal beds are a few centimeters in thickness. However, thicker bedding planes ultimately separate into thin laminae due to absorption of moisture. When fresh and dry, the shale breaks with convolutional fracture. These structural features coupled with physical properties make this rock very incompetent rock, requiring careful tunneling. Therefore, heading and benching operations are carried out in two different stages to prevent roof collapses and to minimize the overfall in R.K. Tunnel.

5.2 Limestone - Limestone of the Bhima series on a regional basis, is considered as horizontal bedded, but a close examination at the G'gera tunnel site indicates a low angle dip of 3° to 5° due west. Limestone is highly jointed. The main joint is bedding plane itself and the thickness of the joint is hardly a few centimeters.
There are two sets of vertical joints spaced at a distance of a few centimeters to a meter. These three joints cut the rock mass of limestone into several small sized rock material. Limestone being a compact calcareous rock has incipient joint planes which open up due to a blast or a blow (Duncan 1969).

The Bhima limestone is generally devoid of shear zones. But limestone of this area in general and in the G'gera tunnel zone (including the deep cuts at both the ends) in particular is sheared and crushed locally. The shear zones have acted as conduits for circulating waters which have decomposed the limestone by chemical weathering leaving behind insoluble clayey material which is totally incompetent. Such zones have been encountered especially from approach end (Fig. 5 & 6), and are dangerous to men and material, if neglected. All these structural features and the physical properties have influenced to adopt heading and benching technique for tunneling in two different stages.

6. OVERFALL

6.1 Definition - In the present study, overfall is taken as a breakage in rock above the payline, in the semicircular zone, above the spring level. The percentage of overfall is worked out with reference to the entire cross section of the tunnel.

6.2 Causes of overfall - With the available technology, it is almost impossible to blast a hole in a natural rock to coincide exactly with the designed shape and size. Therefore, certain amount of overfall is bound to occur above the payline, which may be due to following reasons:

i) Physical and chemical properties of the rock concerned.

ii) Structural features of the rock formations.

iii) Seepage of groundwater.

iv) Pattern of blast holes.

v) Amount of charge per blast.

vi) Amount of pull per blast.

The first three causes are natural, and hence, cannot be controlled, whereas the rest are man-made and controllable.

6.3 Overfall in R.K. Tunnel and G'gera Tunnel - The overfall in both the tunnels recorded before the present study, was plotted along with the 2D geological log of the roof of the corresponding tunnel (Fig. 7). This indicated that the overfall has a tendency to increase in the areas which are traversed by closely spaced intersecting joints and shear zones. Overfall in shale of R.K. Tunnel varied from 5.95% to 16.88% whereas in limestone of G'gera tunnel it varied from 8 to 16%. In both the cases about 110 to 115 kg. of gelatine was used per blast (with total 84 different blast holes) to achieve the targeted pull of 2 m per blast.

7. EXPERIMENTAL BLASTS

7.1 Details of experimental blasts - As already stated the blasting is the only factor which can be controlled to reduce the overfall without sacrificing the progress. As the trimmers are responsible for shaping the crown, the quantity of explosive in these holes was reduced in the two experimental blasts each conducted in R.K. Tunnel and G'gera tunnel. The details of the experimental blasts are given in Table I & II.

7.2 Analysis of the results of the experimental blasts of R.K. Tunnel - During the experimental blast (Ex. Bl) No. 1 in the R.K. Tunnel the explosive was reduced to 93 kgs. The average overfall between 38+834.45 m. and 38+833.65 (Rib.No.899 & 900) was 11.43%. A crushed zone both at the crown and at the sides was noticed during this pull. The Ex. Bl. No.2 (90 kg. explosive) resulted in the tightness at about the spring level between 38+833.65 and 38+832.65 m. (901 & 902 ribs) which required secondary blasting.

7.3 Analysis of the results of the experimental blasts of Gundalgera Tunnel - The Ex.Bl.No.1 & 3 were carried out with the usual explosives of 108 Kg. whereas Ex.Bl.No.2 & 4 were carried out with 93 and 94.7 Kg. explosive. The experimental blast No.2, resulted in the tightness at the spring level and the secondary blast with 5 Kg. of explosive was carried out. The overfall was 13.57%. For the Ex.Bl. No.4, the quantity of the explosive was slightly increased to 94.7 Kg. which resulted in an average overfall of 14.08% with no tightness. This suggested that the reduction of the explosives beyond a certain limit requires secondary blasting which delays the cycle of operations. Therefore, the Ex.Bl’s. suggested that an overfall of 12 to 14% on the approach side and 11 to 13% on the exit side be reasonable, keeping the target of progress in view.

8. OVERFALL IN OTHER TUNNELS

The overfall in weathered granite of the Linganamakki tunnel and in massive greywacke of the Head Race tunnel is reported to be 20% and 25 to 50% (Gunasagar et. al, 1981). When compared to these
rocks the sedimentary formations of the Bhima series are incompetent. Therefore, the overfall of 10 to 12% in the shale of R.K. Tunnel and 11 to 14% in the limestone of G'gera tunnel seem to be reasonable without sacrificing the progress.

9. PERMANENT SUPPORTS AND LINING

In view of the physical and chemical properties, and the structural features of shale and limestone of the Bhima series, it is recommended that the entire length of both the tunnels be provided with permanent support and lining, instead of the 20% of the length of the tunnels as proposed earlier by a World Bank team. The normal interval of 1 m. between two ribs could be reduced in critical zones.

10. CONCLUSIONS

Based on the above discussions, the following conclusions are drawn:-

(i) The shale and the limestone of the Bhima series are rendered incompetent due to physical and chemical properties and due to the inherent structural features.

(ii) an overfall of 10 to 12% in the shale of R.K. tunnel is reasonable without sacrificing the progress.

(iii) an overfall of 12 to 14% on approach side and 11 to 13% on the exit side in the limestone of G'gera tunnel is reasonable without sacrificing the progress.

(iv) the total quantity of explosive should be around 95 Kg. and not more than 6 gelatine sticks be loaded in each trimmer hole to reduce the overfall.

(v) The pull per blast is at present is 2 m. Further increase in the pull is going to increase the overfall and is also dangerous to men and material at the working face. Hence, 2 m pull is to be continued.

(vi) Continuous geological mapping and study is recommended to make suitable modifications, if necessary, in the tunnelling operations.

11. ACKNOWLEDGEMENTS

The author is highly grateful to the Chief Engineer, Upper Krishna Project (Canal) Zone and the Chief Engineer, W.R.D.O. having given the opportunity to study the problem. He is thankful to the Executive Engineer, Tunnel Division, Assistant Executive Engineers of both the Tunnels for the facilities provided during the study. He is also thankful to Shri Narangundialah and Shri S.C. Bhairamadgi, Assistant Geologists for their able assis-

REFERENCES


Table I. Quantity of explosive used in different holes in each of the holes in the experimental blasts in shale of R.K. Tunnel.

<table>
<thead>
<tr>
<th>No. of Experimental blasts &amp; Date</th>
<th>Trimmers</th>
<th>Easers</th>
<th>Cut Holes</th>
<th>Baby Cut</th>
<th>Others</th>
<th>Total</th>
<th>Weight of explosives</th>
<th>Overall % of cross-sections</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. 13.6.81</td>
<td>18</td>
<td>6</td>
<td>17</td>
<td>7</td>
<td>8</td>
<td>12</td>
<td>15</td>
<td>8</td>
<td>94</td>
</tr>
<tr>
<td>2. 15.6.81</td>
<td>18</td>
<td>5</td>
<td>17</td>
<td>7</td>
<td>8</td>
<td>12</td>
<td>15</td>
<td>8</td>
<td>94</td>
</tr>
</tbody>
</table>

LENGTH PARTICULARS OF BLAST HOLES

<table>
<thead>
<tr>
<th>Type Hole</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Trimmers</td>
<td>10'</td>
</tr>
<tr>
<td>2. Easers</td>
<td>13'</td>
</tr>
<tr>
<td>3. Cut Holes</td>
<td>14'</td>
</tr>
<tr>
<td>4. Baby Cut Holes</td>
<td>10'</td>
</tr>
<tr>
<td>5. Others</td>
<td>10'</td>
</tr>
<tr>
<td>No. of Experimental blasts and date</td>
<td>Trimmers</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>No. 1 20.8.81</td>
<td>18</td>
</tr>
<tr>
<td>No. 2 21.8.81</td>
<td>18</td>
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<tr>
<td>APPROACH END</td>
<td></td>
</tr>
<tr>
<td>No. 3 21.8.81</td>
<td>18</td>
</tr>
<tr>
<td>No. 4 22.8.81</td>
<td>18</td>
</tr>
</tbody>
</table>

*5 kg explosive was used for secondary blasting.

**As the experimental blast No. 2 resulted in the tightness at the bottom, it was decided to increase charge per hole by one cartridge for 3 trimmer holes and 3 easier holes below the jumbo level on either side.
Fig. 4. Photo of the approach cut of the R.K. tunnel showing the thin laminated shale with diagonal intersecting shear zones filled with clay.

Fig. 5. The shear zone at Chainage 54,562 ft in the approach cut of the Gundalgera tunnel showing folding and shearing of the closely jointed lime stone.

Fig. 6. The working face as exposed after a blast at chainage 54,775 (46th rib) in the Gundalgera tunnel. At this face the lime stone was completely decomposed with relics of closely spaced joints in the clayey matrix which had filled up the shear zone. Slight folding and shearing was noticed.
ABSTRACT

The stability of the surrounding rock is one of an important problems on the mining underground excavation. According to the point of view on the engineering geological mechanics of rock mass, 4 fundamental researches should be worked. 1. The mining tectonic stress field, particularly, the neotectonic stress field should be determined. This is one of the principal factors of the rock mass deformation and instability; 2. according to the characteristics of rock groups and rock mass structure the classification of surrounding rock can be worked. It is a material base of the surrounding rock stability; 3. the stability status of the shaft-level in the construction should be considered, particularly, the stand-up time of an unsupported tunnel. It can most reflect the stability behavior of surrounding rock; finally, 4. on the basis of above-mentioned researches the engineering geologic evaluation and engineering geologic prediction can be made.

ABSTRAIT

La stabilité des roches encaissantes est un des problèmes importants dans l'excavation souterraine des mines. Au point de vue du mécanique de la géologie de l'ingénieur des masses, quatre types d'études ont été discutées comme la solution du problème: 1. Le champs de contraintes, en particulier celle actuel, doit d'abord être déterminé, il est considéré comme une valeur bien importante causant la déformation et la perte de la stabilité des roches encaissantes. 2. Des roches encaissantes sont nécessairement classifiées selon leurs groupement et structure qui constituent la base matérielle de la stabilité. 3. L'état des roches encaissantes doit être attentivement observé au cours de l'excavation pour savoir la durée propre d'un tunnel buit, directement liée à la stabilité. 4. L'évaluation et la prédiction de la géologie de l'ingénieur pourront enfin s'effectuer à la base des analyses cidessus.
The stability of rock mass surrounding the mining cavities is one of the important problems in mining excavation. According to the view of engineering geomechanics of rock mass, the character of the force and the medium should be considered in all respects. As far the action of forces, the attention should be paid sufficiently to the tectonic stress. Regarding the medium, although now, the fracturing of rock mass has been recognized, but the formation process and characteristics of the fractures in rock mass, especially the combinative action of the fractures have not been recognized properly. It is the basic focus of the present difference in understanding the natural characteristics of rock mass. Therefore, a fundamental engineering geological study of the surrounding rock stability should be developed. This problem is well instance in the 2nd area, Jinchung mine.

1. Determination of the tectonic stress field

A ground stress mainly includes the gravitational force and the tectonic stress. The tectonic stress plays a leading role in a high ground stress region or in a neotectonic active region. The tectonic stress does not follow the static water pressure law, \( \sigma \), is located in or near a horizontal direction. Thus, the surrounding rock deformation is of its specific character. The shaftlevel, stopes and other tunnel deformation is controlled evidently by the tectonic stress, thereby, it has a great influence upon the direction of a particular engineering location. Therefore, the determination of tectonic stress (mainly neotectonic) is very important.

In order to determine the neotectonic stress field, may be generally dealt with in 4 aspects: 1) According to the tectonic trace, on the basis of the developing history of tectonic movement, to analyse the direction of a max. principal stress; 2) In the view of regional historical earthquake, to interpret the mechanism of earthquake focus to make the first principal stress direction; 3) According to the data of ground deformation, to analyse, the max. principal stress direction; 4) According to the ground stress measurement, to give directly the three principal stress directions and its values. After that, by summarizing the above-mentioned all data, the superior direction of max. principal stress in the researched area can be given. Fig. 1 is a diagram of the max. principal stress of the neotectonic stress field in the 2nd area Jinchung mine, i.e. the direction of stress is about N35°E, the value of max. principal stress is 200-300 kg/cm². Once the tectonic stress of mine had been determined, it is to have a practical guiding meaning for the design and construction of the mining engineering, and to provide important basis for the engineering geological prediction of mining hazards.

Fig. 1. The neotectonic stress field map

1. mechanism of earthquake focus
2. ground deformation
3. ground stress measurement
4. earthquake before 1900 y.
5. direction of max. principal stress

2. Classification of the surrounding rock

To this problem many researchers paid close attention. The rock masses in nature are very various, it is not realistic to take the all-embracing classification of the surrounding rock. But, so long as the fundamental classification of a rock mass is given, for the particular engineering, according to the practice, on the basis of fundamental classification, a surrounding rock classification in engineering area can be made in consideration of the behavior and existent condition of the rock mass. As everyone knows, the formation and transformation are the two fundamental modes in the process of any rock mass formation. Therefore, a rock mass classification must be sufficiently dependent on this. The rock mass in nature may consist of one or many rocks it is mainly determined by the original environment of rock mass formation. In
order to reflect fully this basic status, the concept of engineering geological rock group has been presented, i.e., the natural rocks, having similar engineering geologic characters are divided into a group and it is called as a rock group. The principal bases of dividing the group may be considered as the lithology, stratum thickness and combination of rocks. In 2nd area Jinchung mine the rock groups have been divided as follows: fault fractured rock group, migmatite rock group, schist rock group, gneissose rock group, thick marble rock group, medium-thin marble rock group, granite rock group and ultrabasic rock group.

Under the action of engineering, the rock mass deformation is mainly influenced by internal surfaces and rock blocks, isolated by these surfaces. This researches gave us a deep enlightenment: i.e., the various rock masses: including faults, joints and stratum surfaces are an organic whole. The various surfaces (called the structure surfaces) and the rock block (called the structure body) may be considered as two fundamental units, in which, the structure surface is possessed of leading position. The forms and magnitudes of structure bodies are various, depending upon the behavior, development degree and combination model of the structure surfaces. The behavior of structure surfaces of various size is different, thus, the conception of structure surfaces class in dimension has been presented. According to the mechanical effect, the structure surfaces may be divided into two types: rigid structure surface and weak structure surface. According to the development and combination of the structure surfaces, can divide the rock mass various structure types. In 2nd area Jinchung mine the types of rock structure have been divided as follows: the entire blocky rock structure (includes the intact structure and blocky structure subtypes), layered structure (layered and thin layered), fractured structure (mosaic, layered fractured and fractured) and loosened structure.

Finally, according to the combination status of the rock group with the rock structure, the surrounding rocks may be classified. A certain influence has present between the rock group and rock structure. In the first process of transformation, the development of structures is often under the control of the rock group. But the various structure types may be formed in the same rock group with the variation of the tectonic development degree and tectonic location.

3. Rock fall - the shaft-level deformation

The mine of 2nd area is under capital construction, therefore, the stability status of surrounding rock may be directly researched. The rock fall is a first representation of the surrounding rock instability after the excavation. It will predict a tendency to the possible deformation in future. When the rock fall is researched, the close relation between the rock fall and the rock mass structure was found, for instance, for the loosened structure the rock fall is more easy and its scale and form are determined by the occurrence and width of the fractured zone. The amount of rock fall generally is about tens, hundreds and thousands cubic. Its form is unsymmetrical. For the fractured structure, the rock fall is easy and it is determined by the amount, magnitude and combination of the weak structure surfaces. The scale is large and forms are irregular. For the layered fractured structure, the rock fall is more easy and it is determined by the occurrence and scale of the weak structure surfaces. The form of rock fall is generally unsymmetrical, while the occurrence of a weak structure surface has present in accordance with the tunnel axis or in lesser angle with the axis. The scale of rock fall sometimes is more large. For the mosaic structure, due to development of the rigid structure surface and the structure bodies are bitten and pinned by the each other, therefore, the surrounding rock is considerably stable, only a local rock fall block falls. For the layered or blocky structure, the rock mass is generally stable and more stable. An individual rock block falls at most.

It has shown to synthesize the data of many rock falls that almost all rock falls are controlled by the structure surfaces. The general forms of rock falls for the various structure are shown in Fig. 2.

According to an incomplete statistics, the rock falls have been occurred at near 200 locations on this mining area. In which the rock falls in the loosened rock mass structure are 40%.
in fractured structure - 33%, in layered fractured structure - 22%, the others about 5%. And the scale of rock falls occurred in the layered and blocky structure is very minor, form the statistics of rock fall volume, it is less than a %. The statistics fully reflects the close relation between the rock fall and the rock mass structure.

After the tunnelling, the stand-up time of an unsupported can most comprehensively reflect the stability behavior of surrounding rock. This is listed in tab. 1 (see tab. 1).

The shaft-level deformation is a continuation of the surrounding rock instability. A creep deformation on the 2 # area is obvious. Its typical deformation curve is shown in Fig. 3.

The shaft-level deformation occurs in many locations of the mining area. In the statistics of 87 locations of the shaft-level deformation, which occurred in the loosened, fractured and layered fractured structures is about over 90%. The location of engineering is very important for the rock deformation. The amount of deformation occurred in the locations joint (connective) engineering is near 50% in the total amount. Therefore, the unsupported span should be a basic parameter in the surrounding rock stability. After researching all the shaft-level deformation, which are obviously characterized by the following: the force characteristic of deformation-main the lateral pressure; the space characteristic of deformation—internal heave and longi tudinal tension cracks, occurred mainly in or below the medium on the wall. The shear cracks occurred mainly above the arched line. The time of deformation is unequal, from the 4 hours to the many years, its continued time is longer. The deformation in east-west sub-shaft 1750 contact road may be a most typical example. The tunnel axis just is vertical to the max. principal stress direction. The shaft has located in the loosened fracture F16 fault zone. The deformation of shaft occurred in the initial stage after hours of lining in 1975. And the deformation goes on up to now. The deformation amounts max. to over 200 cm, and min. to near 100cm. According to the deformation characteristics, 4 types may be divided (see Fig. 3). According to the statistics the deformation induced mainly by the lateral pressure is about 80%. Therefore, general regularity of the level deformation in the region with high ground stress is well represented.
4. Engineering geological prediction and assessment. In general the engineering geologic condition in the 2# mining area is bad. Due to the mining area located in an arid zone, the factor of underground water is not of the important influence, otherwise, the matter will be a great trouble. In order to serve the design and construction, a great attention is paid to the engineering geological prediction and the satisfied results have been gained.

The engineering geological prediction of mining hazards should be one or the important subjects in the study of the rock mass stability. According to the surrounding rock classification and the rock mass quality estimation, considering the stability behaviour after the construction, the practical status of the surrounding rock in the particular engineering has been researched. These are a basis of a study of the engineering geological prediction. It is an obvious viewpoint to make the engineering geological prediction that the rock mass deformation is mainly determined by the development and space distribution of the weak structure surfaces and by the relationship between the particular engineering and the weak surfaces. Of course, the action of rigid structure surfaces can be not eliminated. In general, if the all surfaces are rigid in rock mass, the hazard of rock fall is not often easy to be induced, except that the rock blocks, combined by rigid surfaces, have in most unfavourable location.

Through the statistics and analysis, the relationship of an approximate exponential function curve is shown between the stand-up time and the rock mass quality. But the relationship between the surrounding rock has instability, i.e. the rock fall and the rock mass quality is not very close. In general, it is a straight line. But in the better rock mass, due to the some random of fault development and the influence of the construction factor, a some uncertain tendency is often presented.

On the basis of above-mentioned the engineering geological prediction of hazards has been made. For instance, in the transport level at the 1250 bottom, located in the medium-thin marble rock group of a layered frac-
tured structure, the rock fall will occur, because the level axis is vertical to the max. principal stress direction and the occurrence of weak surfaces inclines towards the tunnel interior from one side. After the construction, the practice of a large rock fall fully verified the conclusion of the prediction. For instance again, the 1250 power magazine originally was designed in the same engineering geological condition with the above-mentioned transport level. But, according to the prediction the location of the power magazine has been moved. It is located in the blocky marble rock mass. In consequence, the power magazine having 15m span is considerably stable when it is constructed.

From the above-mentioned examples it may be seen that the 4 factors are mainly considered in the engineering geological prediction, namely: the relationship between the level strike and the regional stress field; the relationship between the level strike and the weak structure surfaces; the characteristics and the occurrence of the weak structure surfaces; the relationship between the rock mass structure and the scale of mining cavities. These factors must be comprehensively analysed, thus, the quantitative prediction for hazard can be made.
ENGINEERING GEOLOGICAL PROBLEMS RELATED TO TUNNELING IN THE OLONOS-PINDOS ZONE OF WESTERN GREECE

PROBLEMES DE GEOLOGIE DE L'INGENIEUR CONCERNETS LE TUNELAGE A LA ZONE OLONOS-PINDE, GRECE OCCIDENTALE

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ABSTRACT
The geotectonic zone of Olonos-Pindos of Western Greece, covered mainly by limestone, shale, chert and flysch formations, is characterized by a complex geological structure and great instability.

In this zone the mechanical consequences of the stresses applied and the plastic deformation of rocks, because of the lateral loading, are very obvious. These phenomena lead finally to an accumulation of stresses in the rock and are better justified in the absence of the prealpine basement, facilitating the rock mobilization.

The development of tensional stresses and the intense faulting and fracturing of the rocks followed.

The geomechanical characteristics are examined in detail, a classification of the above limestone rocks is attempted and the problems expected during excavation of high-way tunnels are examined in two sites studied. These problems are mainly referred to the existence of fracturing zones in the rock mass, the unfavourable orientation of the bedding-fracturing planes, in relation to the tunnel axis, as well as the peculiar hydro-geological conditions. On the base of the field observations and the evaluation of the data gathered a series of proposals are given, concerning the confrontation of problems related to the supporting and the underground water flow.

ABSTRAIT
La zone géotectonique de l'Olonos-Pinde de la Grèce occidentale, qui se compose de calcaires, cherts et flysch, est caractérisée par une structure géologique complexe et une grande instabilité. Dans cette zone, les conséquences mécaniques des contraintes appliquées et de la déformation plastique des roches due au chargement latéral, sont très claires.

Ces phénomènes conduisent finalement à une accumulation des contraintes dans la roche et sont justifiés par l'absence du bassin préalpin facilitant la mobilité des roches. Les contraintes de tension y développées ont créé des zones de fractures et de failles dans la masse des roches.

On a examiné les caractéristiques géomécaniques des calcaires, on les a classifié et on a fait l'étude des problèmes attendus au cours de l'exécution des tunnels d’autoroute en deux sites particulières.

Ces problèmes concernent l'existence des zone de failles, l'orientation défavorable de la stratification et de la fracturation par rapport à l’axe de tunnel et les conditions hydrogéologiques particulières. D'après les observations du terrain et l'évaluation des
INTRODUCTION

The geological structure and the tectonic evolution of the Olonos-Pindos zone define primarily the engineering geological conditions which prevail and impose a general instability. Also, factors as neotectonic fracturing, seismic activity, strong morphological relief, state of stresses and peculiar hydrogeological conditions contribute to this instability.

The above conditions create serious problems in the various technical works and consist the determinant factors for their planning and execution. Especially, the national road network is referred, which has, as a rule, followed the main morphological axes with a N-S direction.

The two tunnels studied at Stalamata and Skorliga sites are enrolled in the new national road of Arta-Trikala, with a general direction across the above mentioned axes (Fig. 1).

GEOLOGICAL STRUCTURE OF OOLONOS-PINDOS ZONE

GEOLGY

The zone is characterized by intense morphology and peculiar geological structure and composition. These have been imposed by its geotectonic evolution, started from the upper Triassic, when the zone was formed as a part of the Greek geosyncline, through Mesozoic, when it consisted the Greek "eugeosyncline" between the adjacent submarine ridges.

The formations which consist the zone in the Triassic horizons, are limestones and dolomites with cherts and shales. The sedimentation continued up to the lower Cretaceous with deep sea chert-schist formations and thin-platy limestone beds, while during the upper Cretaceous thick-platy limestones predominate and finally the deposition of flysch follows up to the upper Eocene. The post-alpine formations consist of the neogene deposits and the quaternary, mainly loose, formations.

TECTONICS

The tectonism of the zone since the end of Miocene, is distinguished by the presence of strong tangential tectonic movements, being developed inside the frame of the migration of the orogenesis from E to W, having as a result the intense folding of the formations and the overthrust of the zone on the adjacent geotectonic zone to the West. So, the strong tectonic pressures exercised have caused the microfolding and fracturing of the formations, which reacted of course in different ways, according to their mechanical properties and the stresses applied (Fig. 2).

The increased mobility of the zone to the W is explained if we take into account the sediment composition, displaying considerable plasticity and flexibility and the submergence of the prealpine basement.

The faulting tectonics has followed the folding phase, with the development of vertical movements (neo-tectonic mainly ones of Plio-Pleistocene) and more intense fracturing of the rocks. This tectonism is continued up today with the main tectonic grabens formed being under geodynamic evolution in an extensional environment, having as a result the activation
Fig. 1 Map of Greece, indicating the extension of the Olonos–Pindos zone and the investigated sites.
Carte de la Grèce, montrant l'extension de la zone du Pinde et les sites examinés.

Fig. 2 Upper Cretaceous limestones of the Olonos–Pindos zone, multifolded and highly fractured.
Calcaires très plissés et fracturés du Crétacé supérieur de la zone du Pinde.
of normal faults or fault zones and the manifestation of strong earthquakes.

The above described geotectonic evolution dictates an increased mobility and instability in this zone. The stresses stored have not completely released, during the deformation stage and consist one of the main factors for the various catastrophic geological events such as earthquakes and landslides.

ENGINEERING GEOLOGICAL CONDITIONS IN THE TUNNEL AREAS

GEOLOGY and TECTONICS

Case I ("STALAMATA" tunnel)

In this area the rocks mainly consist of light grey, medium to thick-bedded Cretaceous limestones with plates, intercalations and modules of cherts, up to 10 cm thick. Occasionally, in these formations, intercalations and lenses of microbrecciated limestones with fragments of cherts, quartz and green schists are developed (Antoniadis et al. 1980).

The beds are almost horizontal with a mean strike and dip N30°E/20°SE, intensively jointed and faulted. On the surface, along the N30°W axis of the tunnel, two fault-zones were located with mean strike and dip N20°W/80°SW and width of five meters each, in which the rock mass was crushed and very loose. Also in the north portal of the tunnel a normal fault-plane with a strike and dip N16°W/65°NE was observed on which slickensides showing a pitch of 20°NE were measured (Fig. 3).

After the microtectonic measurements and the statistical treatment of the data were made, two main sets of joints were obtained, with a strike and dip N 50°E/80°NW and N20°E/90°NW respectively. The stereographic projection of the bedding-planes, the fault-zones, the joint sets and the axis of the tunnel is illustrated in Fig. 4.

Case II ("SKORLIGA" tunnel)

The geological formations consist of thin-platy to thick-bedded, light grey Cretaceous limestones with some chert intercalations up to 10 cm thick. Limestone layers have a N30°E/50°SE attitude and are characterized by intense and multifarious faulting and multifolded structure (Tsiambaos and Sabatakakis 1981).

Crushing and loosening of the limestone rock mass is very obvious in the fault-zones and the fold hinge lines. Two sets of joints were determined with N20°E/50°NW and N60°E/50°NW attitudes respectively. In Fig. 5 the stereographic projection of bedding planes, joints and tunnel axis is illustrated.

GEOTECHNICAL PARAMETERS and HYDROGEOLOGICAL CONDITIONS

Case I

In this area the limestones are highly fractured and fresh to slightly weathered. The uniaxial compressive strength, measured on intact specimens, ranges between 30 to 40 MPa (medium strong rock).

The joint frequency corresponds to eight joints per meter (close spacing) and the persistence of joints is medium to low, since these terminate against the bedding planes. These joints are described as tight to open, with rough undulating to planar walls (Fig. 6).

The orientation of the limestone bedding is unfavourable for tunnel stability, as the modal dip of beds is 20°. Conversely, the orientation of the first set of discontinuities (N50°E/88°NW) is characterized as very favourable for a tunnel
Fig. 3 Normal, almost vertical, fault-plane in the limestones of the "Stalamata" tunnel area.
Plan de faille normale, presque verticale, dans les calcaires de la région du tunnel "Stalamata".

Fig. 4
Stereographic projection of discontinuities data along the axis of "Stalamata" tunnel; S: bedding, J₁: 1st set of joints, J₂: 2nd set of joints, F₁: fault-zones, F₂: main fault-plane, TA: tunnel axis.
Fig. 5
Stereographic projection of discontinuities data along the axis of "Skorliga" tunnel; S: bedding, J₁: 1st set of joints, J₂: 2nd set of joints, TA: tunnel axis.

Projection stéréographique des discontinuités mesurées le long de l'axe du tunnel "Skorliga"; S: stratification, J₁: 1er système de diaclasses, J₂: 2ème système de diaclasses, TA: axe du tunnel.

Fig. 6 Limestone rock mass separated by two main subvertical systems of jointing and the nearly horizontal bedding planes.
Masse de calcaires séparée par deux systèmes principales de diaclasses et les plans de stratification presque horizontales.
driving from SE to NW and fair for an opposite driving, since the tunnel axis runs perpendicular to the strike of joints dipping steep to NE (Wickham et al. 1972). The orientation characteristics of the second set of joints (N20°E/86° NW) are considered as fair to unfavourable, because the angle between the axis of the tunnel and the strike of joints is 57° and the joints almost vertical. Finally the steep fault-zones intersect the tunnel axis at an angle of 15°, being thus characterized as very unfavourable.

Primary permeability of the limestones is very low but the secondary one very high, due to the intense fracturing and faulting and the solution phenomena as well. Isolated seasonal heavy flows of water are expected in association with fault-zones, solution pipes and cavities, but the correct estimation of the water inflow through these structures is very difficult. Here it is reported that during excavation of the access road to the north tunnel portal a large karstic cavern was located with a permanent water flow of 150 lit/sec (Fig. 7).

Case II

The limestone rock mass in this area is fresh to slightly weathered, with medium strength (30 to 40 MPa measured values of the uniaxial compressive strength) and highly fractured.

The joints are very tight, slightly rough, planar to undulating with a frequency of nine joints per meter. Strike and dip orientation of the limestone beds and joints is characterized as fair to unfavourable with relation to the axis of the tunnel.

As in the case I, seasonal inflows of water is likely to occur, through fault-zones and cavities of the limestones, but are expected to be at a lower rate.

ROCK MASS CLASSIFICATION

Case I

The limestone rock mass, which is the main formation in the zone of the tunnel, was classified according to Bieniawski (1979) for tunneling, taking into account the following parameters: compressive strength of the intact limestone, rock quality designation (RQD), spacing, orientation and condition of the discontinuities, and ground water inflow.

The rock mass along the tunnel is divided into two groups, according to its physical and mechanical properties. The first group comprises the main part of the tunnel and the second one the parts referred to the fault-zones, which constitute almost 20% of the total tunnel length. Table I illustrates the classification of these two groups, according to which the first is classified as class III (fair rock) and the second as class V (very poor rock).

Case II

Also in this case the limestone rock mass is divided in groups I and II, the first referring to the main part of the tunnel and the second to the zones of faults and microfolds, which represent 20-30% of the total tunnel length. The rating of group I is 50 and is classified as class III (fair rock) and group II as class V (very poor rock).

EXPECTED PROBLEMS OF TUNNELING

From the analyses of the engineering geological conditions which predominate in the tunnel areas, it is presumed that
Fig. 7 North portal of the "Stalamata" tunnel. A karstic cavity with high discharge of water has been revealed after excavation.

L'entrée septentrionale du tunnel "Stalamata". Une cavité karstique avec déchargement intense de l'eau révélée après l'ex cavation.

<table>
<thead>
<tr>
<th>GROUP I</th>
<th>GROUP II</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Description</strong></td>
<td><strong>Rate</strong></td>
</tr>
<tr>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>1. Strength of intact rock material in uniaxial compression (MPa)</td>
<td>30-40</td>
</tr>
<tr>
<td>2. Drill core quality (RQD)</td>
<td>50-75</td>
</tr>
<tr>
<td>3. Spacing of discontinuities (m)</td>
<td>0.12</td>
</tr>
<tr>
<td>4. Condition of discontinuities</td>
<td>Slightly rough tight to open</td>
</tr>
<tr>
<td>5. Groundwater, Inflow per 10 m tunnel length</td>
<td>10-25 1/min</td>
</tr>
<tr>
<td>6. Adjustment for orientation of discontinuities</td>
<td>Unfavourable</td>
</tr>
<tr>
<td>Rating</td>
<td>47</td>
</tr>
<tr>
<td>Class No.</td>
<td>III</td>
</tr>
<tr>
<td>Description</td>
<td>Fair rock</td>
</tr>
<tr>
<td>Average stand up time</td>
<td>1 week for 3 m span</td>
</tr>
</tbody>
</table>

**Table 1** Geomechanics classification of limestone rock mass in the tunnel areas (a: Stalamata tunnel, b: Skorliga tunnel).

Classification géomécanique de la masse de calcaire dans les régions des tunnels (a: tunnel Stalamata, b: tunnel Skorliga).

IV.124
some problems are expected due mainly to morphology, rock structure, state of "in situ" stresses and the peculiar hydrogeological conditions.

It is marked out, that in both cases of tunneling there were restrictive factors such a morphology and engineering problems which, in connection with the best route tracing, prevented any other site choice for tunnel excavation.

The expected problems of tunneling in these areas are:

a) Planar slides which are likely to occur during excavation along the bedding and fault planes of limestone close to the tunnel portals, because of the unfavourable orientation of these discontinuities (strike nearly parallel to the steep slope face).

To increase stability of the slopes, a series of rock bolts must be installed, anchored in polyester resin. The extent, orientation and length of the bolting required depend upon the local structure of limestone.

b) In the parts of tunnels where fault-zones and microfolds are crossed, the rock mass is very loose so immediately after blasting shotcrete 80 mm thick must be applied (Bieniawski 1974). Also sets of steel ribs spaced about 0.7 m with lagging are required for supporting of these unstable parts of tunnels.

On the contrary, the main sections of the tunnels require for their supporting rockbolts spaced 1.5 m plus wire mesh and 30 mm shotcrete in crown.

c) Residual stresses in the rock mass of the tunnels are expected due to the intense past tectonic activity. These have not yet relaxed and this fact in conjunction with the steep slopes and the inclined limestone beds create locally high lateral pressures. The determination of the stresses "in situ" is very important and this must be done by hydraulic fracturing or overcoring techniques. Depending on direction and value of the principal stresses, measures for the local support of the tunnels must be considered.

d) Finally the peculiar hydrogeological conditions will create locally serious problems and particularly in case I, during and after excavation. Correct estimation of the inflow of groundwater is difficult to be done, but this is expected to be seasonally very high. Drilling of small diameter boreholes on the face of excavation is suggested, in order to locate probable abnormal water inflows and estimate high water pressures.

Also, the general hydrogeological conditions must be investigated before the excavation, of the tunnels (by drilling, geophysical prospecting etc.), in order to consider the grouting and supporting measures of the zones of high water pressure and inflow.

CONCLUSIONS

In the frame of regional planning and development in the Greek territory, the extension of the national road network across the main morphological axes (E-W) is connected with serious technical works and especially tunnel excavations.

The problems related to these works become more acute in the Olnos—Pindos geotectonic zone, due to intense fracturing of the formations, rapid alternation of beds with different mechanical behaviour and peculiar hydrogeological conditions.

In the case of the two tunnels studied the basic determinant factors for
excavation are: the fracturing zones of considerable width, and other weakness planes with unfavourable orientation in relation to the pre-determined project axis, the state of stresses and the hydro-geological regime. The successful facing of these requires special measures related to stabilization of the rock in the tunnel portals, support of the loosening zones along the tunnels and grouting of the places of selective underground inflow.

REFERENCES


MAKKAH INNER RING ROAD PROJECT—ENGINEERING GEOLOGICAL SITE INVESTIGATIONS
FOR TUNNEL MIT-4

PROJET DE LA ROUTE PERIPHERIQUE INTERNE DE LA MAKKAH—ETUDE INGENIEUR-
GEOLOGIQUE DE SITE DU TUNNEL MIT-4

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ABSTRACT

The Makkah Inner Ring Road Project involves construction of four vehicular tunnels and connection roads around Holy Kabah in Saudi Arabia. The tunnels are horse-shoe shaped in cross-section, double-tube, and of varying lengths.

In this paper engineering geological site investigations for the 560 m long MIT-4 tunnel are described. During site investigation an emphasis is given to the assessment of rock material and rock mass characteristics of Precambrian migmatites, quartz diorites, dolerites, and felsites.

Engineering geological mapping is supplemented by coring drilling of boreholes, in-situ and laboratory testing, and discontinuity surveying. The field data are then used to evaluate rock mass quality for tunnelling stability.

INTRODUCTION

The town of Makkah, being the religious center of the Islamic World, has increased in population very rapidly. Especially during the pilgrimages period its population is almost more than doubled. Thus, the Kingdom of Saudi Arabia, Ministry of Communications, paying due consideration to the population growth, has decided to construct an inner ring road around Holy Kabah so as to ease the transportation problems within the town.

The so-called "Makkah Inner Ring Road (MIRK) Project" has been designed by consultants Dar Al-Handaasah (Shair and Partners) and it consists of four double-tube...
tunnels (Makkah Inner Ring Road Tunnels-MIT) and the connection roads.

In this paper engineering geological site investigations along the alignment of tunnel MIT-4 will be described. The site investigation involves detailed geological mapping, core drilling of boreholes, discontinuity surveying, and in-situ and laboratory testing of rock samples to determine their mechanical properties. The data thus obtained are used to evaluate rock mass quality for tunneling stability.

Location, Topography, Climate

Tunnel MIT-4 is located approximately 1.0 km north of Kabe (Figure 1). It is about 560 m long and its axis extends roughly in the northwest-southeast direction. It is a double-tube tunnel each with a horsehoe cross section (9.5 m high, 15.5 m wide). Its axes are slightly curved whose convex side is looking Kabe.

The tunnels cross a slightly conical hill having an elevation of 421.6 m at its peak. The southeast and northwest facing slopes of the hill are about 40% at the foothills and 20% at the uphills. The drainage pattern is radial with occasional dendritic systems. The tributary development does not indicate any obvious structural control.

The area is located within an arid climate. The summer season is hot and dry. During winter season occasional thundershower may be expected.

Natural vegetation is almost absent. Here and there a few short bushes may be encountered.

Regional Geology

The Arabian Peninsula, which is geologically a prolongation of the African continent, is set off from Africa by the Red Sea, from Iran by the Arabian Gulf and the Gulf of Oman, and is bounded on the south by the Arabian Sea and the Gulf of Aden.

The structural pattern of the Arabian Peninsula was set in Precambrian time with stabilization of the Arabian shield. The ancient rocks in themselves reveal a complex and mobile history.

The peninsula is subdivided into four stable divisions, namely the Arabian Shield, the Arabian shelf (Interior homocline), Interior Platform, and the Basins (Fig.2). The Arabian Shield, a vast complex of largely Precambrian igneous and metamorphic rocks, occupies about one-third of the peninsula in the west. Until Tertiary, the Arabian Shield was a projection of the shield of Africa but it is now separated from it by the Red Sea rifting.

The Arabian Shield consists of Western Arabian Shield, Yemen-Aden Plateau, and the Southern Arabian Shield. The investigation area is located within the Western Arabian Shield.

According to Brown (1972) basement gneiss is believed to represent the oldest rocks exposed in the Arabian Shield. Most of the rocks are orthogneisses derived from diorite, quartz diorite, granodiorite or granite, and are in the amphibolite metamorphic facies, although some are gneisses. The oldest radiometrically dated rocks are granodiorite and related quartz dioresites exposed in the vicinity of Makkah. The Rb-Sr ages of these rocks average about 1000 m.y.

Local Geology

The investigation area is located within the Precambrian belt of the Western Arabian Shield. The main rocks exposed in the area include migmatites and quartz dioresites. These rocks are crossed by a few felsite and numerous dolerite dykes (Figures 3 and 4).

In the following sections the rock material and the rock mass characteristics of these different lithological units are described.

Migmatite

Although the term migmatite implies intimate mixing of plutonic rocks with schists and gneisses, it is used here to denote metamorphic rocks (mostly gneisses) as distinct from quartz diorite.

Migmatite constitutes about one-third of the bedrock along tunnel MIT-4 (Fig.4). Its outcrops are mostly observed at the portal areas.

Migmatite is light gray where fresh and yellowish brown where weathered. The biotite-rich zones are slightly dark gray.

The grain size of migmatite is medium to coarse. It has a granoblastic texture and it consists of feldspar (+plagioclase), quartz, biotite, hornblende, and some epidote and chlorite. The feldspars are slightly altered to sericite.

The engineering properties of migmatite largely depend upon the discontinuities it contains and the effect of weathering.

IV. 128
Joints give rise to blocky structure to the rock. The rock shows well developed joints in three directions. Steeply dipping joints are separated at distances of 0.05 m to 0.50 m and gently dipping ones at an average separation of 0.75 m, thus giving rise to a small and medium sized blocks.

Weathering is mostly pronounced at the surface of natural rock exposures. Here the rock is slightly discoloured and became somewhat weaker than its fresh condition. The rock is partly disintegrated while still keeping its fresh parts as a continuous framework. Thus, the degree of weathering may be classed as slight to moderate. Penetrative weathering and/or alteration is restricted to the fractures or joints which may extend down to depths of 5 to 10 m below the surface. However, only discolouration rather than decomposition may be observed along such zones.

The strength of the rock, as assessed by means of simple field tests (i.e. geological hammer, point load test, schmidt hammer test) may be classified as extremely strong where fresh and strong to very strong where moderately or slightly weathered.

The initial porosity and permeability of the rock is very low. Even the joints do not significantly increase the permeability of the rock. No groundwater is encountered within the boreholes drilled in migmatite.

The rock shows poorly developed foliation. Due to the effect of weathering foliation becomes rather distinct at the surface. However, at the quarries where fresh rock is exposed foliation becomes very indistinct.

Migmatite is frequently crossed by dolerite and few felsite dykes. The boundary between migmatite and quartz diorite is rather sharp but highly irregular.

Quartz Diorite

It constitutes almost two-thirds of the bedrock along tunnel MIT-4. The rock is medium-to-coarse grained, grey to dark gray where fresh and yellowish brown where weathered. Locally, due to abundance of mafic minerals, mainly hornblende, its colour turns to dark greenish gray.

At the midway of the tunnel alignment quartz diorite possesses a porphyritic texture due to the presence of large crystals of hornblende some of which are 5 cm long and 2 cm wide. In addition to hornblende the rock contains plagioclase (an-

desine), quartz, and small amounts of biotite, chlorite and epidote.

At the upper levels of natural rock exposures the rock shows widely separated joints in three directions. The separation of the two steeply dipping sets generally range between 0.20 m to 0.60 m, whereas the third set, which is gently dipping, is separated at an average distance of 1.00 m thus producing medium sized blocks. At the quarries, where fresh rock is exposed, large block sizes are also observed.

Similar to migmatites, weathering in quartz diorite may be classed as slight to moderate. The intensity of weathering sharply decreases after 2.00 m to 3.00 m below the surface. Apart from the shear zones, the rock may be considered as practically fresh at the tunnel elevation.

The assessed strength of the fresh rock is strong to very strong. Where the rock possesses a porphyritic texture its strength is slightly reduced.

Both porosity and permeability of the rock is very low. At the tunnel elevation it is practically impervious. No groundwater was encountered within the boreholes drilled in quartz diorite.

The rock is frequently crossed by dolerite and felsite dykes. The dolerite dykes are by far the most abundant. Felsite locally shows pytmonic structure within quartz diorite.

Felsite

The term felsite is used here to denote light coloured, quartz and feldspar rich rocks. In addition to these, locally observed thin quartz veins are also included within felsites.

Felsite is generally observed in the form of dykes within quartz diorite and migmatite. The thickness of the dykes ranges between a few centimetres upto one metre. They are generally discontinuous. Both the thicknesses and the dip angles of the dykes change within short distances. During field mapping only the dykes having thicknesses greater than 0.20 m are mapped.

Felsite is whitish to light gray where fresh and yellowish gray where weathered. The sizes of individual crystals are small to medium. The dominant minerals include quartz, K-feldspar, plagioclase (albite), some hornblende, biotite, and chlorite. The mineral composition of the rock suggests granitic origin.

The joints give rise to a blocky and/
or columnar structure to the rock. Joints developed across the dyke are more obvious than those parallel to it. The sizes of individual blocks may be classed as medium to large.

The rock is slightly to moderately weathered. At the tunnel elevation it is practically fresh. The assessed strength of the fresh rock may be classed as strong to very strong.

**Dolerite**

Dolerite occurs in the form of dykes of varying thicknesses. Only the dykes having thicknesses greater than 0.20 m are shown on the geological map. The average thickness of the dykes is about 0.40 m. However, a dyke zone of more than 50 m wide is mapped close to the entry portal of the tunnel.

Dolerite is dark green where fresh and yellowish brown where weathered. It is fine grained, holocrystalline and slightly metamorphosed in the greenschist facies.

Due to jointing the rock possesses blocky structure. The sizes of individual blocks are generally small. Locally medium to large sized blocks are also observed.

The rock is slightly to moderately weathered. However, although rare, along several dykes which have undergone shearing, moderately to highly weathered zones may also be observed. The rock strength is assessed as strong to very strong.

The dykes show a well defined orientation. The general trend of the dykes are NEE-SWW/70°-80° SE and R-S/60°-80° W. The boundary between the dyke rock and the country rock is very sharp and smooth. There is always a thin film of alteration zone made of chloritic and clayey material along the contact surface which produces an easy splitting of the dyke rock from the country rock.

Dolerite is generally utilized as the only source for concrete aggregate. For this reason in the Makkah area numerous rock quarries were opened following the strike of dolerite dykes.

**Overburden**

The term overburden is used here in a sense similar to regolith, which includes any loose, unconsolidated sediments blanketing the bedrock. For this reason slope wash, colluvium, and residual soil are all included within the term overburden.

Overburden consists of gravel and boulder-sized blocks of the bedrock embedded within a silty and sandy matrix. The sizes of the blocks range between 0.10 m to 0.20 m. The material is generally unsorted, mostly angular and loose. The thickness of overburden is less than 2 m.

**Joints**

Investigation of joints is based upon such observations as orientation, frequency of spacing between joints, separation, persistence, infilling, and nature of surface. The description of various parameters of joints is carried out according to British Standards Institution (1981).

Joint line surveys are carried out at the portal areas of the tunnel. The dominant joint sets are determined from the stereographic plots of the joint measurements.

At the entry portal of the tunnel the Schmidt plot has yielded the following sets: N10°E/20°NW (Smax); N10°W/65°NE (Smax); and N40°E/20°NW (Smax). At the exit portal however, following sets were identified: N10°W/20°NE (Smax); R80°E/80°S (Smax); and N70°W/80°SW (Smax).

The spacing of joints is wide (200–600 mm) to very wide (600 mm–2 m) and their apertures are very narrow (2–6 mm) to tight. The joints are generally clean and along their surfaces occasional discoloration may be observed. Their persistence is generally less than 2 m, however, locally joints with persistence up to 4 to 5 m are also observed. Both rough (85%) and smooth (15%) surfaces are noted. Due to the absence of groundwater at and above the tunnel elevation all joints were dry.

The rough surfaces, very narrow apertures, and the absence of infilling materials and groundwater seepage through the joints provide favorable conditions from the stability point of view. In fact the author’s observations within the tunnels which were already opened in similar bedrocks elsewhere in Makkah also support this argument. Some minor overbreaks may however, be expected along the roof of the tunnel due to gently dipping joint sets.

**Classification Tests**

The classification tests include determination of hydraulic conductivity, uniaxial compressive strength, modulus of elasticity, poisson’s ratio, natural density, moisture content, and effective porosity of the rocks cropping out along the tunnel alignment. Both in-situ and laboratory tests are conducted.

The hydraulic conductivity of quartz
diorite and migmatite was determined by means of packer tests. The tests were carried out at three pressure increments and two decrements, equivalent to $1/3P, 2/3P, \text{and} P$, where $P$ is the total overburden pressure at the level of the test. Although no packer tests were conducted within the boreholes drilled along tunnel MIT-4, a number of test results are already available for migmatites and quartz diorites from other site investigation studies for MIRR project. The test results indicate that the hydraulic conductivities of quartz diorite and migmatite fall within the range of $10^{-11}$ m/sec to $10^{-14}$ m/sec (Anon, 1981 a,b), showing that the rocks are practically impervious.

In-situ determination of compressive strength of the rocks is carried out by means of "Schmidt Hammer" (I-type hammer with an impact energy equal to 0.073 mkg) and "Rock Point Load Tester" (Hydraulic pump with a maximum capacity of 5 000 kgf and a 2.23 sq.in ram.).

The average compressive strengths of the rocks determined by means of schmidt hammer are given in Table 1 and the point load test results in Table 2.

**Table 1. Schmidt hammer test results**

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Average Compressive Strength (MN/sq.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Migmatite</td>
<td>268.8</td>
</tr>
<tr>
<td>Q-Diorite porphyry</td>
<td>192.4</td>
</tr>
<tr>
<td>Q-Diorite (med. grained)</td>
<td>193.2</td>
</tr>
<tr>
<td>Felsite</td>
<td>213.0</td>
</tr>
<tr>
<td>Dolerite</td>
<td>118.0</td>
</tr>
</tbody>
</table>

**Table 2. Point Load test results**

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Average Compressive Strength (MN/sq.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Migmatite</td>
<td>220.2</td>
</tr>
<tr>
<td>Q-Diorite porphyry</td>
<td>125.3</td>
</tr>
<tr>
<td>Q-Diorite (med. grained)</td>
<td>200.8</td>
</tr>
<tr>
<td>Felsite</td>
<td>172.5</td>
</tr>
<tr>
<td>Dolerite</td>
<td>199.7</td>
</tr>
</tbody>
</table>

The uniaxial compressive strength, tangent modulus of elasticity, and poisson's ratio tests were carried out on the core samples having length/diameter ratio of 2. For the laboratory tests, the procedures described in International Society for Rock Mechanics (1972, 1979 a, and 1979 b) were adopted. The test results are given in Table 3.

For the laboratory tests the core samples were obtained from the depths corresponding to the roof elevation of the tunnel. Thus, the core samples for migmatite were not available for testing. In-situ and laboratory test results indicate that the rocks may be classified as very strong.

**Engineering Classification of Rock Masses**

Engineering classification of rock masses is acknowledged today as an essential adjunct for assessing rock mass conditions for engineering purposes. In this study the Q-system of Barton et. al. (1974) is adopted. Evaluation of six parameters for the determination of rock mass quality (Q) is based upon field observations, core drilling, and in-situ and laboratory testing.

Rock quality designation (RQD) is determined from the core samples according to the procedure described by Deere (1962). R.Q.D. values of the core samples corresponding to the roof elevation of the tunnel are 95% and 100% for the portals and the mid-portion of the tunnel, respectively. Joint set number, $J_n$, is determined from the line survey data. Since three joint sets dominate, a value of 9 was assigned for $J_n$. The values of joint roughness number, $J_r$, are based upon the observations of the core samples taken from the tunnel level. Here, the joint surfaces are either rough or irregular planar or slickensided. Thus, a value of 1.5 is assigned for $J_r$. Joint alteration number, $J_a$, is again determined from the core samples. The joint walls are generally fresh with occasional surface staining, thus a value of 1.0 is assigned for $J_a$. Joint water reduction factor, $J_w$, is taken as 1.0 since no groundwater seepage is observed through the joints. The values of stress reduction factor, SRF, range between 2.5 and 1.0. A value of 2.5 is assigned to the portals, whereas 1.0 is taken for the tunnel alignment away from the portals.

Based upon the values assigned for the six parameters the Q values are determined as 5 (fair quality) and 4 (good quality) for the tunnel portals and the alignment, respectively. The portals, however, constitute the most critical parts of the structure. Due to low topographic relief and relatively thin overburden the effect of weathering will be more pronounced at the tunnel portals. Thus, for the tunnel portals a more conservative approach may be desired. Therefore, it is recommended that class 6 (poor quality) be adopted at the portals.

**Seismicity**

Seismological investigation related to the Western Arabian Peninsula are mostly
Table 3. Summary of Laboratory test results.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Uniaxial Compressive Strength (MN/sq.m)</th>
<th>Tangent Modulus of Elasticity (MN/sq.mm(^4))</th>
<th>Poisson's Ratio</th>
<th>Natural Density (Tons/cu.m)</th>
<th>Moisture Content (%)</th>
<th>Effective Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse-grained</td>
<td>157</td>
<td>6.49</td>
<td>0.214</td>
<td>2.84</td>
<td>0.20</td>
<td>0.54</td>
</tr>
<tr>
<td>Q-Diorite</td>
<td>150</td>
<td>11.10</td>
<td>0.142</td>
<td>2.97</td>
<td>0.46</td>
<td>1.35</td>
</tr>
<tr>
<td>Medium-grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q-Diorite</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


According to Drake and Girdler (1964), the Red Sea and the Gulf of Aden have been developed in response to a relative displacement of Arabia with respect to the African continent. During this separation a deep axial trough bounded by normal faults has been formed. Many of the earthquake epicenters in the Red Sea are located along marginal faults (mainly associated with the eastern wall) rather than along the medial rift. The presence of shallow focus earthquakes supports the idea of sea-floor spreading where a new oceanic crust is formed as a consequence of separation of Africa and Arabia.

In the central Red Sea (17°N–25°N; 32°E–44°E) approximately 35 earthquakes have been monitored during the period January 1953 through December 1969 (Fairhead and Girdler, 1970). The magnitudes of these earthquakes range between 4.1–5.8. The earthquake epicenters for the Red Sea and the Gulf of Aden are shown in figure 5.

For the Makkah area no published data on the seismic phenomena were available. The Arabian shield, where Makkah is situated, is generally believed to be tectonically stable. Thus, no significant seismic activity is expected within this stable Precambrian terrain.

Conclusions

Along tunnel MIT-4 the bedrock consists of migmatisite and quartz diorite which are cut across by numerous dolerite and a few felsite dykes. Both the bedrock and the dykes possess very high compressive strengths.

The rocks are slightly to moderately weathered and the effects of weathering is mostly restricted to the upper few metres of natural rock exposures. However, at the portals of the tunnel the effect of weathering may locally become significant. During portal excavation the portal rock faces require protection against sliding and/or toppling failure of the rock masses. Use of pattern bolting, wire netting, and shotcrete is recommended for such sections.

Joints give rise to a blocky structure to the rocks. The block sizes increase with depth. However, the dyke rocks are likely to produce small to medium sized blocks. The flat lying and/or gently dipping joints may give rise to a local overbreak problems at the roof of the tunnel. Similar problems may also be anticipated along dolerite dykes due to their weak bonding at contact surfaces with the country rock. Otherwise both migmatisite and quartz diorite are competent enough to withstand heavy roof pressures and to produce good arching. During tunneling no groundwater inflow is anticipated.

At the portal areas the rock mass quality of class 6 is recommended whereas along the alignment class 4 can be adopted. The portals require pattern bolting, wire netting, and immediate shotcrete applications at the roof and spot bolting with shotcrete reinforcement along the walls. Away from the portal sections occasional spot bolting and continuous shotcrete application may be adequate. Geological investigations during the construction stage is essential for a successful and safe tunnelling practice. Only in this way any new situation can immediately be recognized and necessary precautions may be taken.

Acknowledgement

The author wishes to express his gratitude to TEKAR Ltd. Co. and ARTEC-BIMHOL J.V. for providing an opportunity for the realization of this work.
References


Figure 2. Structural divisions of the Arabian Peninsula (A: Western Arabian Shield; B: Yemen-Aden Plateau; C: Southern Arabian Shield) (Modified after Kent, 1978)
Figure 3. Geological map along tunnel MIT-4
Figure 5. Epicenter for the Red Sea, Gulf of Aden, and Afar depression for the period January 1953 through December 1968 (After Fairhead and Girdler, 1970)
EXCAVATION AND TREATMENT OF THE PRINCIPAL FAULTS IN THE TAILRACE TUNNEL OF THE RIO GRANDE I HYDROELECTRIC COMPLEX, ARGENTINA

EXCAVATION ET TRAITEMENT DES PRINCIPALES FAILLES DANS LE TUNNEL DE RESTITUTION DU COMPLEXE HYDRO-ÉLECTRIQUE RIO GRANDE I, ARGENTINA

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ABSTRACT

The RIO GRANDE HYDROELECTRIC COMPLEX is located in the heart of the country, in the Province of Córdoba. It consists of a pumped-storage scheme with the machine hall in a large cavern situated 200 m below the surface, excavated in rock the same as whole underground conveyance.

The scheme is sited mainly within rocks of the Precambrian basement. There are two kinds of rocks: foliate gneiss and massive gneiss, classified from fair to very good quality.

Three of the principal faults found in the Tailrace Tunnel are described mainly with respect to excavation methods, rock-falls, their causes and consequences, support system installed and anticrisis treatment adopted. Besides, the results carried out during studies for the project on these geological features are appraised as well as the excavation advances rate. Finally, a classification of the faults based on their characteristics and problems which arose during tunnelling, is proposed.

The authors feel that they are contributing in this way with a real experience to develop the rock tunnelling with conventional method of drilling and blasting in one of the world's largest tunnels.

ABSTRAIT

Le COMPLEXE HYDRO-ÉLECTRIQUE RIO GRANDE, est situé au milieu du pays dans la province de Córdoba. Celui-ci, consiste en un schéma d’accumulation par pompage avec une centrale placée dans une caverné à 200 mètres de profondeur, elle est excavée dans les roches, même que toute la conduite subterrestre.

Géologiquement l’aire correspond au soubassement cristallin précambrien, elle est formée par gneiss foliés et masives d’une qualité de régulière a très bonne.

On décrit ici, trois de los principales failles rencontré dans le Tunnel de Restitution, par rapport a la méthodologie d’excavation, éboulis de roches, ses causes et conséquences, le système de support exécuté et les traitements anticrisis adoptés. De la même manière on a évalué les résultats obtenus avec les études de ces structures pendant le projet, les vitesses d’avance d’excavation et on essaye une categorization des failles en fonction de ses caractéristiques et des problèmes qui ont occasionnées.

De cette façon on expose une expérience concrète dans la construction de tunnels en roches avec méthodes convencionnels des vrilles et sautage dans un tunnel que, par ses dimensions, est un des plus grands du monde.
INTRODUCTION

The object of this paper is to diffuse the valuable experience gained in the construction of the Río Grande I Hydroelectric Complex, fundamentally with regard to large excavations in rock, the technology of which is actually being fully developed.

The whole gamut of technical problems to be overcome during construction was very great; only a group of these will be dealt with here: The traversing and treatment of the principal faults which affect the excavation of the Tailrace Tunnel.

DESCRIPTION AND LOCATION OF THE WORKS

The Río Grande I pumped-storage scheme is being built by Agua y Energía Eléctrica, Sociedad del Estado. Panedile Argentina S.A. & Societa Italiana Per Condotte d’Acqua S.p.A. is the contractor in charge of civil works. It is located in the province of Córdoba, 700 km NW of Buenos Aires (fig. 1). The Río Grande, principal tributary of the Río Tercero, consists fundamentally of two great work groups; Dams and Spillways on the one hand, and Underground Hydraulic Conveyance on the other, which join the two reservoirs made on different levels (fig. 2).

GEOLOGICAL ENVIRONMENT

The Complex is located on the eastern slope of the Comechingones Range which belongs to the Sierras Pampas system. These last are distinguished by a structure of blocks limited by reverse faults of southern bearing, which plunge to the east, originating western escarpments and gentle eastern slopes.

The greater part of the Complex is located within the ambit of the Migmatitic Rock Mass of Cerro Pelado constituted by homogeneous tonalitic gneiss, the mineral paragenesis of which corresponds to high grade metamorphism: quartz-plagioclase-biotite-cordierite-garnet (Gordillo, Lencinas, 1979).

The last km of the Tailrace Tunnel and the Lower Reservoir is situated in the garnetiferous gneiss with marked foliation grading to schistous levels towards the east.

The relief reveals the presence of massive gneisses through the classic spheroid hills, Cerro Pelado being the best exponent and ball-shaped outcrops; while gentle slopes are produced in the schistous gneisses covered in debris and weathered material in the form of a mantle.

Two geological environments are defined in this manner: in the first place Ge-

Fig. 1 RIO GRANDE I location - Fig 2 RIO GRANDE I general profile
Fig. 3 Principal sets of geological discontinuities. Lambert's projection.

The Tailrace Tunnel traverses a third type of rock which it is important to mention, namely a bank of diopside amphibolite inserted in the schistous gneiss, of up to 50 m thickness, which, due to its gentle slope to the southeast, influences the excavation for a distance of 150 m.

FAULT SYSTEMS – NOMENCLATURE

The spherical diagram shows (fig. 3) the principal groups of alignments revealed in the photogeological survey of the zone, and the main groups of fractures surveyed in the excavations.

The nomenclature used to designate the structural features contains a category of size symbolized by: F (faults), fracture with gouge, breccia, mylonite, weathered and badly jointed rock. (This enumeration does not imply simultaneity of these attributes). L, fractures of very

continuous development, with signs of movement and hard walls.

The orientation of these fractures in space is symbolized by the letters N-O-P-Q R-V-E, according to the parameters of strike and plunge. Letter "E" designates undulating low plunge fractures which in the Contraembalse environment are parallel to the foliation planes (fig. 3).

In general, if the fracture contains an intrusive body it is designated by an "i". The placing of the fractures in the tunnel is determined by the chainage where this intersects the crown.

Example: FN 5170 designates a fault in the Tailrace Tunnel, group "N", with an intrusive body that intersects the crown, in the 5170 chainage.

DESCRIPTION OF THE TUNNEL

The Tailrace Tunnel has a vaulted section 12 m wide and 18 m high, with an area of 206 m². It develops along 5.5 km with an incline of 0.131% towards the higher reservoir. The rock overburden oscillates between 28 and 180 m. It will be under pressure during the whole of the useful lifetime of the Complex.

On account of the size of its section it was necessary to determine the excavation in 2 stages; one in vault 8 m high and another in benching 10 m high, both with a total width of 12 m. Owing to the importance of its length it was attacked on 4 working fronts, one from the end of the Oscillation Chamber and another from the Lower Reservoir outlet; and two fronts created by the construction of an Auxiliary "Window" Tunnel, in the middle of its course.

At the date of writing this paper the first stage of the excavation had been finished.

The method of excavation used is that of drilling and blasting. Smooth blasting technique was used to lessen the rock damage. The blasting pattern was of parallel drills, with headings of 1,60 to 3,80 m and the specific charge oscillated between 0,85 and 1,40 kg/m³ according to the type of rock.

TREATMENT OF MAIN FAULTS

A brief description is given of the items of the Contract which anticipated elements and accessories of support used during the tasks of headings and later complementary treatment.

a) Untensioned, grouted dowels.
b) Tensioned grouted and ungrouted rock bolts, diameter 19, 22 and 25 mm.
c) Welded steel mesh for rock support 100x100x4,2x4,2 mm.
d) Steel arches, double profile I 20 cm high, with spacers of different lengths.
e) Gunite of minimal thicknesses of 12, 25, 75 and 100 mm.
f) Welded steel mesh for gunite of 100x100x4,2x4,2 mm.
g) Concrete lining 0,50 m thick.
h) Grout for filling the remaining space between concrete lining and rock in the vault, and grout for rock consolidation.
i) Pilot tunnel excavation.

Following is a description of three of what are considered the most important and representative fractures in the order in which they were traversed. The rest are synthesized in the table of fig. 12, ordered according to the degree of importance that, in the opinion of the authors, these discontinuities had during the heading of the excavation.

1. Transversal Sur Fault

The basic documentation of the project says: "Regional fault, apparently sub-vertical with slight dip South. Strike approximately N 290°. The Geophysical studies detected a thickness of 40 m." (Agua y Energía Eléct. 1972).

This fault gave rise to a modification of the layout of the tunnel at this stage of the project with the view to traversing it as perpendicularly as possible. For this purpose two S-shaped curves were designed, as will be seen in the key plan of the figures, which increased the rectilinear length of the tunnel by 85 m approximately.

The systematic survey of discontinuities realized later in the tunnel and the study of the outcrops in the layout of this fault indicated that the most probable plunge was of a high angle towards the north (fig. 3, group "p"), which was confirmed by the appearance of features FP 3096, FP 3010, FP 2998, FP 2990 and FP 2966 in the first curve excavated in a pumping direction (east-west).

This group with a strike of N 290° and a plunge 60°N did not in general surpass 2 m in thickness, had neither water nor oxides, mylonites predominated and it was set in a rock mass of good quality.

The FP 3010 together with a joints system caused the vault to cave in by forming a gravitational wedge of 150 m³ with the apex in chainage 3000 (fig. 4), when the face of the heading was at chainage 2972, that is at 28 m; this distance is equivalent to 2,3 diameter of excavation (12 m).

It must be explained that between chainages 3041 and 2990 the excavation was effected with enlarged section (13 m wide by 8,5 m high) it being understood that the heading was approaching a fault zone when the aforesaid feature FP 3010 showed up in the south sidewall. This section was foreseen in the project, in the fault zones which were to take the covering of concrete 0,50 m thick. Later it became obsolete on account of being inconvenient to the stability of the section in the fault stretches, since it implied an increase of 1 m (8,3% more) in the width of the excavation. In this particular case it has been one of the concurrent factors which provoked the fall of rock. Immediately after this, a systematic bolting was effected (Ø 22 mm, 4 m long and 2 m apart) in the vault of the previous zone, in a space of 30 m, to avoid an increase in the relaxation of the rock mass (fig. 4).

Following this, beneath the cave-in, after applying a thin layer of gunite along the whole length of the fault, the placing of a group of 17 steel arches was resorted to, with a separation of 1 m between them and connected to the rock mass by means of steel spacers. In addition to this, a welded steel mesh was placed to retain small blocks of rock that continued to fall for a few days.

FP 2990 parallel to and with the same characteristics as the former fault made necessary the continuation of the placing of another 7 steel arches as far as chainage 2980. The size of the section had already been reduced to standard for the reasons stated above.

Eight months after the first group of arches was placed, during the construction of the concrete lining, the state of the rock mass was inspected in detail in the zone of the cave-in. The conclusions arrived at were that in spite of not having applied any active support but only a passive one, constituted by arches, the rock mass had stabilized itself and was not leaning on the contact profiles except for some small-sized isolated blocks.

After chainage 2980 the rock mass recovered its quality, although continuing to be fractured, and the excavations of the entire section continued, placing systematic bolting similar to that already mentioned, as the only support, until a new fault of the same group appeared, FP 2966, which began to present stability problems, which suggested advancing with a central pilot tunnel 6 m wide by 8 m high. This was prolonged from chainage
Fig. 4. Transversal Sur Fault - Plan View.
2963 up to chainage 2937, where as a consequence of FN 2937 a cave-in of totally desintegrated rock was produced in "chimney" form, which obliged a replanning of the whole system of heading and support that had been realized up to the moment (fig. 4).

FN 2937 was the largest fault found in the Rio Grande I excavations. It has a strike of N 338° and a plunge of 34° SW, with a weathered breccia and intensely jointed rock of up to 15 m of thickness.

There was little water with relation to the size of the structure (approximately 8/litre/minute in a 10 m stretch); the length of the section influenced by the fault was approximately 40 m.

Once the fall of rock in the pilot tunnel had been stabilized, three exploration borings were performed, one with extraction of core samples, and the other two by rotopercussion, with which a profile was made to support the one made with the samples, based on the velocity of the boring, the characteristics of the cutting and the colouring of the water of the drill hole.

Given the size of FN 2937 the stability of the wedge formed by it, and the FPI 2966 and "I" systems taken as sliding planes, was treated as being potentially unstable; it was in pyramidal form with a height of 14 m above the vault of the tunnel and a total weight of 2000 tn. One hundred bolts were installed of Ø 25 mm designed according to a square pattern with sides 2 m long with a bolt in the centre. Their length varied between 3 and 15 m, according to the structural feature they had to traverse; all were tensioned to 14 tn and grouted (Golder Ass., 1979).

Having dealt with this wedge the widening of the pilot tunnel was begun with 5 m headings, in both sidewalls alternately. In chainage 2945 the widening was reduced to the vault only, to permit the placing of steel arches without excavating the complete section (fig. 4).

To achieve good contact of the support, the empty space between the arches and the rock were filled with concrete.

Untensioned grouted dowells were placed at the top in the heading, trying to achieve an umbrella-shaped prereinforced vault for the next heading. This system was not successful owing to the rock being very breakable. Instead a thin layer of gunite was placed in all the headings, which permitted the cohesion of the clayey breccia to be maintained and the contact concrete was thrust in a projection towards the face of the heading (fig. 5).

To summarize, the excavation sequence was: 1 - Blasting of mushroom-shaped section with short heading (1,5 m). 2 - Scaling; 3 - The spreading of a thin layer of gunite in the whole of the section and in the face of the heading; 4 - The placing of a steel...
5 - Contact concrete in the space between the rock and arch. This methodology was continued up to chainage 2925 where complete section heading was begun, with short headings constructing the definitive concrete as the work progressed, up to chainage 2915, thus dispensing with the steel arches.

As a precaution, owing to the importance of the zone, starting from chainage 2915, the excavation of the central pilot tunnel continued and the corresponding widening was effected alternately, up to chainage 2881, that is for a space of 35 m, where that the quality of the rock gradually changed, This section was bolted normally, that is to say, in function of the geological features.

In numbers: the Transversal Sur Fault Belt extends from chainage 3040 up to chainage 2850, that is to say 190 m, taking in the whole group of fault referred to. It took 10 1/2 months to traverse it, using 1852 m of rock bolts, consisting of 65 of Ø 19 mm, 761 of Ø 22 mm and 1026 of Ø 25 mm; 24 steel arches; 60 m³ of gunite; 10 m of concrete for definitive covering and 20 m of contact concrete, totalling a volume of 603 m³, of which 320 correspond to the last quoted.

2. FT 20 Fault
The basic information says: “approximate strike 330°, incline not prove. Thickness estimated by electric sounding at 50 m depth: 200 m”. It was anticipated that this zone would be one of the most risky to excavate owing to its scarce covering of rock: 36 m (Agua y Energía Eléct., 1972).

The exploratory drillings realized from the surface indicated low quality rock mass. The geophysical studies detected an en echelon faulting zone with good quality rock intercalated.

This section was studied in detail with the object of defining the geotechnical characteristics of the rock mass involved, and evaluating the excavation methodology and support system to be employed (Casajus, Sarras Pistone, 1981). Synthesis of the survey in the tunnel: length of fault zone 280 m, the first 70 m (upstream) intensely jointed rock mass, structural transition surrounding the fault belt, which extended for a space of 210 m. The main discontinuities that integrate the fault belt FT 20 were: FNI 5170,
Fig. 7 FNI 5170 Fault

FNI 5212, FNI 5222, FNI 5257, FNI 5270, FNI 5304, (FNI 5325, 5329 and 5335) and FNI 5375. The rock mass involved was of good quality (fig. 6).

The most important faults were:
FNI 5170. In fig. 7, a cut perpendicular to the strike of this fracture is diagrammed, forming a standard design for the main FN of FT 20.

FNI 5222, responds to fig. 7 diagram. This fault put the two lithological environments of Contraembaise and Cerro Pelado in contact with one another (fig. 6).

FNI 5375, extrapolated to surface emerged on the left bank of a stream that ran along the zone of least overburden of the whole tunnel: 28 m.

From chainage 5250 to chainage 5400.

Fig. 8 FT 20 Fault - Support Design
the tunnel passed through a diopsidic amphibolite which inclined gently to the East with a thickness of approximately 50 m. The fresh rock was of optimum quality but, on being affected by the shearing zone, was strongly jointed (fig. 6).

During the excavation of the section '5110 - 5170, the vault was bolted in great detail with the object of supporting all the unstable blocks (fig. 8).

When FNI 5170 appeared it was decided to construct the concrete lining while advancing, also placing the necessary bolts to stabilize the rock mass. The length of the headings was reduced from 3,8 m to 2,1 m. The methodology of "concrete near the face while advancing" was executed from chainage 5136 up to 5204. As on other occasions, when the rock appeared to be very breakable, gunite was applied after the necessary bolts were installed, the thickness of which oscillated between 20 to 30 mm.

Between chainage 5170 and 5196 an unstable gravity wedge was formed in vault. In fig. 8 this wedge defined by the features FNI, LQ 5190 and LR, is indicated. When the face of the heading surmounted the first fault and uncovered the second feature, the presence of this block was inferred and by means of semi-spherical projections its volume was calculated, and according to the nature of the rock mass a weight to be supported was taken for granted while the headings were realising it. The support was effected with Ø 25 mm tensioned and grouted bolts. The face of the heading being in chainage 5186, long vertical bolts (10 m) were placed in order to "hang" the blocks. The subsequent ones were anchored onto the haunches passing through the most important discontinuities.

With the object of controlling the behaviour of the support, a mechanical extensometer of 2 positions was placed. In fig. 9 the graph obtained is shown. It is convenient to note that while the face of the heading was kept at a distance of less than 12 m (one diameter) from the instrument, the movements were produced in discrete form, the points where they increased coinciding with the days when the blastings were effected. From this distance onwards the movements were decelerated and the relation with the blastings was minimized. When the face of the heading surpassed the distance of two diameters with respect to the instrument, the readings were stabilized (Sarra Pistone, 1980).

In order to pass through FNI 5222 as from chainage 5204, the concrete at the face of the heading was dispensed with, and the long rock bolts (4 to 7 m) and the covering of the vault with gunite of minimum thicknesses of 20 to 30 mm were resorted to (fig. 8).

Having overcome this fault, localized rock bolts were used, that is to say for the most important structures.

To summarize: the FT 20 fault zone extends from chainage 5100 up to 5380 in 280 m. The excavation took 14 1/2 months, 2533 m rock bolts, 46 m of concrete were constructed, totalling a volume of 648 m³ and 150 m³ of gunite were placed.
3. FNI 1140 Fault

At the stage of exploration surveys, a bore with extraction of samples was practiced with the object of studying a photogeological alignment. The result of same indicated a shearing zone lying at a depth of between 60 to 120 m, where the lower boundary was constituted by the FNI 1140 fault found in the gallery (Agua y Energia Elec. 1972).

This fault appeared in chainage 1112 above one of the sidewalls (fig. 10), it had a medium thickness of 1.5 m in the shearing zone and a maximum of 5 m with the breccia belt and intensely jointed rock that borders it. There was an insignificant volume of water, and intensely weathered rock. It contained planes—very continuous and packed with gouge a few centimeters thick. The group of tectonites was composed by mylonites, breccia, weathered cataclastic gneiss and tectonized intrusions.

The angle between its strike and the axis of the tunnel was of 30° which made it very unfavourable to the stability of the excavation. As from chainage 1050 the vault was affected by a faulted pegmatite with a strike parallel to the axis of the tunnel and a 10° to 15° plunge south: FE1.

This discontinuity together with the FNI 1140 and the predominating set of joints produced a rock wedge of approximately 400 m³ in the sidewall, which slid one hour after the complete section blasting was effected (first stage), when the face of the heading was 10 m forward (fig. 11). It was then proved that the bolting applied had been insufficient in quantity and length, given the importance of the sliding plane discovered. The size of this cave-in produced a 75% enlargement of the section of the tunnel. Before the cave-in a two position extensometer 15 m long was installed in vault in chainage 1121, with the object of studying the behaviour of the shearing zone detected by the drilling already mentioned. The values of the measurements indicated that the vault had not undergone abnormal deformations, which indicated that the slide, which occurred at a distance of 13 m, cons-
stituted a localized phenomenon.

When the excavation had become stabilized, the extraction of material commenced, while the zone affected by the slide was bolted progressively. The fault and its surroundings was covered with a thin layer of gunite 30 mm to avoid the loss of humidity and the progressive fall of breccia fragments. Later an attempt was made to advance by 1 m to complete section to see the reaction of the rock mass. Only two headings were realized up to chainage 1144 due to the fact that the overbreak was becoming uncontrollable. It was then decided to concrete all the section as from chainage 1119 up to very close to the face of the heading, for a length of 24 m and also the fallen sidewall was reconstituted up to the face of the heading with concrete to avoid precisely that the rock should continue to relax and lose still more confinement in a longitudinal direction.

Having constructed the concrete arches, the excavation was continued with a sequence of short headings of 1 m to complete section, up to chainage 1154 where, owing to the excessive overbreak, it was decided to continued with a pilot tunnel 7 m wide by 8 m high, excavated in the right middle section (downstream). As entrance to the face of the heading was treated as a portal by constructing a concrete arch 4 m long which was thrust in a projection in the vault section in order to provide a prop for the top of the heading. The vault and the arches were previously treated with gunite to avoid the breaking up of the breccia, and long bolts (Ø 19 mm, 10 m long) were placed so as to anchor them 1 to 2 m beyond the tectonized rock.

As the pilot tunnel advanced, the whole section was treated with gunite immediately after the blasting, and the long bolts continued to be placed even in the sidewalls.

The pilot tunnel was excavated for the space of 41 m, up to chainage 1195. From there on the section began to be widened until it was normalized, due to the fact that the fault no longer influenced the stability of the tunnel, that is to say the overbreak was definitely controlled and the rock mass had gradually changed in quality until it reached the sound rock.

To synthesize: since FNI 1140 appeared in chainage 1112 up to the end of the pilot tunnel, chainage 1195, or rather 89 m, 5 1/2 months had elapsed; 28 lineal m of concrete were constructed with a volume of 1240 m³; approximately 95 m³ of gunite were placed and 267 m of bolts of Ø 19 mm and, 6,7 m average length.

4. Conclusions

From the treatments executed the following items may be emphasized:

-The good performance of gunite in slight thicknesses, 40 to 60 mm, applied soon after the blasting, as a preliminary
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<tr>
<th>NAME</th>
<th>STRIKE</th>
<th>PLUNGE</th>
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<th>TYPE</th>
<th>DESCRIPTION</th>
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<th>PREDOMINANT CATACLASES</th>
<th>TUNNEL ROOF PLAN</th>
<th>PROFILE</th>
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<th>ADDITIONAL</th>
<th>ANTIerosion LINING</th>
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Fig. 12 Tailrace Tunnel - Principal Faults
support, avoiding dismemberment of the rock, in fault zones with very fragmented rock.

- The effectiveness of structural bolting, even after the gunite, as active support also installed immediately after the blasting.

- The good behaviour of the concrete providing rigidity to the whole, fundamentally in a longitudinal direction.

- The utility of the pilot tunnel in making the heading possible with less deformations of the rock, and the observation of same. After traversing the fault the treatment of same may be undertaken without interrupting the rate of production.

- The inconvenience of the steel arches, which in addition to being expensive and difficult to install, are inefficient as passive support due to the absence of effective contact with the rock mass due to overbreak produced by the fault. The application of reinforced gunite is preferable with minimum thicknesses of 100 mm combined with the placing of tensioned and grouted rock bolts to construct the definitive concrete at a prudent distance from the face of the heading.

- The need to include the tasks of inspection and support of the excavation in the production cycle.

DEFINITIVE TREATMENT OF THE FAULTS

The Tailrace Tunnel was designed unlined except in zones of low quality rock.

Its function is to restore the water to the lower dam and take it from there to the higher one. In order to do this it must take a maximum flow of 500 m³/sec under pressure at a velocity of around 2.5 m/sec.

In order to guarantee its effectiveness it was necessary to design an additional support and linings against erosion in the fault zones enumerated.

In this manner additional rock bolts were placed in the vault and, above all, in the sidewalls of the first stage, which had not been taken much into account in the project of the support necessary for the advance.

Furthermore it was considered necessary to cover the faults with either gunite or reinforced gunite with mesh or concrete, according to the size of the discontinuities.

The proper functioning of gunite as lining of large hydraulic tunnels cannot be assured even today.

A fault with erosionable fill was lined with gunite in the Diversion Tunnel and it was possible to prove the good behaviour of the gunite in these cases after the tunnel had worked at pressure in various floods that occurred in the summer of 1981. That is why its application was accepted for cases of geological features with walls of good rock and scanty thicknesses (≤ 1 m) of tectonized rock.

For the rest, concrete lining was adopted. In the table of figure 12 a list is given of the support employed during the excavation of each fault, called primary; the one used to reinforce these zones, in the already excavated tunnel, with the object of minimizing the risks of excavation of the second stage, named additional support and antierosion treatment defined according to the characteristics of each fault in particular.

FINAL COMMENTARY

Although in general the stability of the tunnel was guaranteed in some way in accordance with the results of the analysis by the Finite Element Method (Agua y Energía Eléct. 1976), the extend of the works due to its section and length - presupposed localized problems - in the zones of detected faults of certain magnitude.

Owing to the good quality of the rock mass the problem posed were reduced in relation to the length of the tunnel; to give an example it suffices only to say that in 40% of its length no type of support was installed, that is to say the rock supported itself. This really implied, at the time, an overvaluation of the quality of the rock mass which lead to an excessive reliance on heading (in certain zones) and caused a relatively important fall of rock produced precisely by FP 3010.

This fact made the revision of the observation criterion, as also the paying of closer attention to the result of the excavations. At all times it was attempted to relate the geological accidents detected on the surface with those that emerged from the excavation. A detailed geological survey was made of all the tunnel. This permitted zoning the excavated tunnel and categorizing the most prominent discontinuities of the rock mass, which achieved a greater visualization of the problems and offered a coherent criterion of support for the rest of the tunnel.

Evidently the success of the heading in a gallery is due in large measure to the adequate use of the explosive and means of an adjusted blasting pattern and the efficacious utilization of the elements of support and reinforcement of rock, but above all in importance is the understanding of the rock mass behaviour where the tunnel...
is excavated. For this reason, in the opinion of the authors, geotechnical support is decisive during the heading of the excavation, as much as during the project itself. In this way the work done in underground projects is undertaken rationally and a real reduction of the costs may be achieved.

ACKNOWLEDGEMENT

The authors wish to thank AGUA Y ENERGÍA ELECTRICA for the authorization and support given them in the accomplishment of this paper.

REFERENCES

ABSTRACT

Data from interpretation of air-photos and landsat pictures and results of synthetical engineering geological exploration around Nanling Tunnel (6.07 km long) built in Karstic rock are thoroughly studied. In this paper is presented the prediction of following predominant engineering geological problems probably encountered during tunnel excavation. Such as: Lie concealed horizontal karst zone, its distributive rule, location and elevation; ground water discharge, its total capacity, outburst intensity and hydrographic curve; dynamit geological phenomenon, its type, possible location and scope of influence. In order to ensure safety to engineering work and transportation, and to reduce the ill-influence to environmental protection, necessary processes are suggested. The above predictions and suggestions are being proved much the same in the excavational practice of the tunnel which is under building.

ABSTRACT

D'après l'interprétation des photos aériennes et satellites, et les résultats des prospections synthétiques de géotechnique et hydrogéologie le long du Nanling tunnel de longueur de 6.07 km, en roche Karstique, l'auteur a strictement étudié tous les documentaires et a prévu avec accès les problèmes prédominants concernant l'excavation du tunnel dans les massifs rocheux karstiques au point de vue de la géologie d'ingénieur. Ainsi que les lois de distribution de la zone horizontale souterraine en karst, leur location et élévation; la capacité de décharge d'eaux souterraines, son intensité de l'injection brusque d'eaux, et sa courbe hydrographique; les phénomènes karstiques de dynamique géologique, leur type, location possible et influence ambianta. Pour affirmer la sécurité des excations du tunnel et des exploitations ferroviaires et pour réduire mal influence des protections environnantes, il est nécessaire de suggérer des processus technologiques visant à améliorer les conditions géotechniques pour les constructions, les prévisions et les suggestions sont bien prouvées par les observations après l'exécution du tunnel.
INTRODUCTION

Nanling Railway Tunnel, electrified and double tracked, to cross the Zheling Defile of Wugaishan arrangement in echelon and Qitianling is designed with a length of 6.07 km long and a sinking depth of only 39-170m into the ground. It is situated mainly on carbonatite formation of Lower Carboniferous Series where the geological structure is very complex. Thus, a thorough study of engineering geological problem about karst phenomena here will be the key to the tunnel excavation later on.

1. BASIC CONDITIONS OF THE KARSTIC DEVELOPMENT

The district concerned here, locating at a disintegrated medium-low

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FIG. 1: RIVER SYSTEM AND FLOW WITHIN THE REGION OF NANLING TUNNEL

FIG. 2 REGIONAL GEOLOGICAL MAP OF NANLING TUNNEL


mountain land, lying from the east to the west, wherein a watershed separates the System of Zhujiang River and Changjiang River, belongs to the sub-tropics, and with an altitude of 320-602m, a temperature average 17.6°C, a rainfall average 1139-1529 mm a year. Due to the Neocathaysian foldbelt most of the mountain ranges and water courses are directed to NNE (Fig.1). Except D_3X^2, C_1Y^2 and C_1D^2 which are clasolite formation, integrated with a thickness of 200-239m, giving a relative water resisting layer, strata emerged in this district surveyed are carbona-
**FIG. 3 KARSTIC ENGINEERING GEOLOGICAL MAP OF NANLING TUNNEL**

<table>
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<tr>
<th>Engineering geological division</th>
<th>Altitude in meter</th>
<th>Discharge in m³/s</th>
<th>Types of Country rock</th>
<th>Geological brief</th>
<th>Suggestion for Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td></td>
<td></td>
<td></td>
<td>Slight Karstification</td>
<td>Strengthen supports and close working steps.</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td></td>
<td>Medium Karstification</td>
<td></td>
<td>Sealed lining and close working steps.</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td></td>
<td>Intense Karstification. Mud-sand spout, surface collapse and attenuation or dry up of discharge of well-spring are probable.</td>
<td></td>
<td>Lead drilling, radio-detection karst cavities under roadbed, dredge the surface water, block up ground water and close working steps.</td>
</tr>
<tr>
<td>IV</td>
<td></td>
<td></td>
<td>Intense Karstification. Surface collapse and attenuation or dry up of discharge of well-spring will occur with all probability.</td>
<td></td>
<td>Sealed lining and close working steps.</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td></td>
<td>Intense Karstification. Heavy Mud-sand spout, Surface collapse and attenuation or dry up of discharge of well-spring is probable.</td>
<td></td>
<td>Lead drilling, radio-detection karst cavities under roadbed, dredge the surface water and close working steps.</td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td>Medium Karstification</td>
<td></td>
<td>Sealed lining and close working steps.</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td></td>
<td>Slight Karstification and water confined.</td>
<td></td>
<td>Strengthen supports and close working steps.</td>
</tr>
</tbody>
</table>
## Fig. 3 (Contd.)
### Synthetic Geological Column

<table>
<thead>
<tr>
<th>Time</th>
<th>Columnar Section</th>
<th>Lithological Characteristic</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Phaneroma</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sabulous clay with sand gravel.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Heavy phanerocrystalline limestone and dolomitic limestone.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Agglutinate carbonate shale and limina silicarenite intercalated some beds of limina smut.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Apparent lamina heavy phanerocrystalline limestone, carboniferous limestone, dolomitic limestone and dolomite, rich in fossils as.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Laminated calcareous shale and siltstone interbedded, rich in fossils as.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Neocomitris elegans sternberg. Medium-heavy phanerocrystalline carbonate limestone and dolomitic limestone, rich in fossils as.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Conglomerate</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coal bed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Forecasted water outburst point</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Limestone</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dolomitic limestone</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dolomite</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Braccia limestone</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Chart limestone</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Road</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compression fault</td>
</tr>
</tbody>
</table>

### Legend
- **Q**: Quaternary system
- **Cgt**: Hutian Formation, Upper Middle Carboniferous Series
- **Cdb**: Datang Formation, Lower Carboniferous Series
- **Cdp**: Zimenqiao Stage, Datang Formation
- **Cdp**: Ceshui Stage, Datang Formation
- **Cd**: Shidai Stage, Datang Formation
- **Cy**: Yanguan Formation, Lower Carboniferous Series
- **Cy2**: Upper Shale Stage, Yanguan Formation
- **Cy1**: Lower Limestone Stage, Yanguan Formation
- **Dsx**: Xihuanshan Formation, Upper Devonian Series
- **Bxt**: Upper Shale Stage, Xihuanshan Formation
- **Dx**: Lower Limestone Stage, Xihuanshan Formation
- **Mr**: Granite of Indo-China Period
- **Sabulous clay**: Karst gravity spring
- **Sand**: Inlet of ground river
- **Gravel**: Outlet of ground river
- **Shale**: Surface water system
- **Sandstone**: Lie-concealed karst zone
- **Conglomerate**: Forecasted water outburst point
- **Limestone**: Division of rift zone karstification
- **Dolomitic limestone**: Division of rift zone karstification
- **Dolomite**: Division of mixed zone karstification
- **Braccia limestone**: Drilling and its number
- **Chart limestone**: Railway
- **Agglutinate limestone**: Highway
- **Compression fault**: Place designed by engineers
Hengxialong

<table>
<thead>
<tr>
<th>Section</th>
<th>Number of drilling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tw—47</td>
</tr>
<tr>
<td></td>
<td>Tw—49</td>
</tr>
<tr>
<td></td>
<td>Tw—50</td>
</tr>
</tbody>
</table>

| Messhanli | Tw—56               |
|           | 57.00               |

| Longtang | Tw—60               |
|          | 60.00               |

| Shengchaolong | Tw—64       |
|               | 60.00         |

| Xialianxi | Tw—65               |
|          | 60.00               |

| Dengliang | Tw—4                |
|           | 41.40               |

| Detongban | Tw—5                |
|           | 50.15               |

| Luyuanpu | Tw—19               |
|          | 50.15               |

<table>
<thead>
<tr>
<th>Table1: Attitude of water-filling cavities of Nanling Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hengxialong</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Messhanli</td>
</tr>
<tr>
<td>Longtang</td>
</tr>
<tr>
<td>Shengchaolong</td>
</tr>
<tr>
<td>Xialianxi</td>
</tr>
<tr>
<td>Dengliang</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Detongban</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Luyuanpu</td>
</tr>
</tbody>
</table>

Heritance and superposition of the tectogenesis the rotation fault of Qingmingtang Convolute Structure initiated at Indo-China Period, partly those directing to NNE are under tension at first and compression at last, partly those directing to NWW or EW are continuously under tension.

2 BASIC CHARACTERISTICS OF THE KARSTIC DEVELOPMENT

(1) Depth, size and filling of the karstic cavities
Karstic cavities in this district develops intensely at a depth over 40m and to its most 70m near fault belt. Most of the cavities are fissures with a width of 0.3-26m, height 0.3-16.5m, length to its lo-

Photo 1 The distributive feature of karst depressions from Messhanli to Shengchaolong
Table 3: Discharge Calculated results of Nanling Tunnel

<table>
<thead>
<tr>
<th>Order</th>
<th>Section</th>
<th>Calculated Length (m)</th>
<th>Method of hydrologic statistics</th>
<th>Method of karstic rate</th>
<th>Method of experience</th>
<th>Method of feeding from well-springs</th>
<th>Method of hydrodynamic balance</th>
<th>Method of Value adopted</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Huijian</td>
<td>213</td>
<td>m²/d</td>
<td>4.82 484 4882</td>
<td>2.20 2084</td>
<td>20846</td>
<td>211 242</td>
<td>492</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Jiaqian</td>
<td>700</td>
<td>m²/d</td>
<td>2180 1311 135280</td>
<td>4.55 69509</td>
<td>0.034 2355</td>
<td>4.9 41.8</td>
<td>3587</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Hengxiao</td>
<td>122</td>
<td>m²/d</td>
<td>1162 4710 119162</td>
<td>4.35 69509</td>
<td>0.034 2355</td>
<td>4.9 41.8</td>
<td>3587</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Mangshan</td>
<td>150</td>
<td>m²/d</td>
<td>1540 2800 288050</td>
<td>4.35 69509</td>
<td>0.034 2355</td>
<td>4.9 41.8</td>
<td>3587</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Lianghai</td>
<td>50</td>
<td>m²/d</td>
<td>2020 190 21100</td>
<td>4.35 69509</td>
<td>0.034 2355</td>
<td>4.9 41.8</td>
<td>3587</td>
<td></td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>2310</td>
<td>m²/d</td>
<td>566554 46604</td>
<td>346 26235</td>
<td>286444 16620</td>
<td>4.9 246</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Lianghai</td>
<td>180</td>
<td>m²/d</td>
<td>2980 1175 115680</td>
<td>325 43839</td>
<td>0.014 2916</td>
<td>4.9 246</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Shengshan</td>
<td>600</td>
<td>m²/d</td>
<td>1050 2350 236500</td>
<td>325 43839</td>
<td>0.014 2916</td>
<td>4.9 246</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Xiaolai</td>
<td>188</td>
<td>m²/d</td>
<td>3660 5710 53650</td>
<td>325 10339</td>
<td>0.014 2916</td>
<td>4.9 246</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Liangjiang</td>
<td>412</td>
<td>m²/d</td>
<td>2115 641 65515</td>
<td>205 5738</td>
<td>0.026 4794</td>
<td>1.82 735</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Shuangliang</td>
<td>400</td>
<td>m²/d</td>
<td>402 634 60282</td>
<td>205 5738</td>
<td>0.026 4794</td>
<td>1.82 735</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td></td>
<td>South</td>
<td>3200</td>
<td>m²/d</td>
<td>565537 13030 242</td>
<td>205 5738</td>
<td>0.026 4794</td>
<td>1.82 735</td>
<td>57282</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>6070</td>
<td>m²/d</td>
<td>608643 107341 225</td>
<td>205 5738</td>
<td>0.026 4794</td>
<td>1.82 735</td>
<td>57282</td>
<td></td>
</tr>
</tbody>
</table>

Qcp = (19724.1+197500+171632+256077)/4 = 227104 m³/d

(The value of Q or Qc is discharge in dry period but it must be increased by 4-5 times in raining period.)

(e.g. Hengxiaoliuyi concealed horizontal karst zone, Huangjiawan ground river, and Y-4012-Y-4064 ground river, Picture 3) and places where formed by superposition of the above conditions (e.g. Huangjiawan ground river) etc. (Fig. 1.3).

3 DISCUSSION ON KARST ENGINEERING GEOLOGY PROBLEMS

(1) Conditions of storage, supply and draining of karst water and predictions to its capacity and its outburst intensity

1. Conditions of storage, supplying and draining of karst water.

The linear anticlinorium in NNE direction which the tunnel passes through obliquely is cut by a series of longitudinal compression fault and thus the surface drainage is relatively diffused and takes longitudinal movement, but connected across with tension fault. And furthermore, the district locates at a watershed where rocks are naked and no effective water-resisting layer, the karst water is mainly supplied by surface water that makes

Table 4: Water outburst points and their discharge of Nanling Tunnel

<table>
<thead>
<tr>
<th>Order</th>
<th>Section</th>
<th>Range of water outburst (m)</th>
<th>Length of water outburst Point (m)</th>
<th>Type of water outburst Point</th>
<th>Method of calculation</th>
<th>Discharge Q (m³/d)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Huijian</td>
<td>0.055x0.040x0.020</td>
<td>200</td>
<td>Horizontal Karst zone</td>
<td>Method of feeding from well-springs</td>
<td>7153</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>2</td>
<td>Jiaqian</td>
<td>0.036x0.030x0.010</td>
<td>100</td>
<td>Cutting through confining bed</td>
<td>Method of feeding from well-springs</td>
<td>5952</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>3</td>
<td>Mangshan</td>
<td>0.035x0.035x0.015</td>
<td>190</td>
<td>Horizontal Karst zone</td>
<td>Method of feeding from well-springs</td>
<td>3713</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>4</td>
<td>Lianghai</td>
<td>0.036x0.030x0.010</td>
<td>110</td>
<td>Horizontal Karst zone</td>
<td>Method of feeding from well-springs</td>
<td>3176</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>5</td>
<td>Shuangliang</td>
<td>0.035x0.035x0.015</td>
<td>200</td>
<td>Horizontal Karst zone</td>
<td>Method of feeding from well-springs</td>
<td>2549</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>6</td>
<td>Shuangliang</td>
<td>0.035x0.035x0.015</td>
<td>180</td>
<td>Horizontal Karst zone</td>
<td>Method of feeding from well-springs</td>
<td>1567</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
<tr>
<td>7</td>
<td>Jiaqian</td>
<td>0.055x0.040x0.020</td>
<td>100</td>
<td>Cutting through confining bed</td>
<td>Method of feeding from well-springs</td>
<td>4715</td>
<td>Accompanied with mud-sand spout probably.</td>
</tr>
</tbody>
</table>

IV.162
Photo 4  S-1033, an outlet of ground river

supplying and draining very quick (e.g. dynamic time-lag of springs are only 2-3 days), the discharge is small (e.g. the discharge of a single spring during dry period is generally below 600 m³/d, the most is but 1954.1-2163.8 m³/d).

iii Prediction to the discharge of the ground water

Different methods, such as method of hydro-statistics, method of hydro-dynamic balance (including method of karstic rate and empirical formula), method of hydro-dynamics and method of feeding from well-springs, etc., has been taken for the prediction to the total ground water discharge in the tunnel. Results are shown in Table 3. On the table, it indicates that the method of hydro-statistics give a result too small to be adopted, while the result yield from the method of karstic rate and method of feeding from well-springs are reasonable.

(2) Analysis of the lie concealed horizontal karst zones

1 Huayuanli to Hengxiaolong horizontal karst zone

Most of the surface water here goes into the ground and partly overflows to the surface at S-1033 (Photo 4). While boring, heavy leakage discovered the existence of cavities filled with water, heavy dissolution and on account of the passing through of grand rotation fault which is under compression but was under tension before, indicates the lie concealed horizontal zone might stand at an altitude of 265-275m and its vicinity. Karst water in this zone flows toward the NNE direction. Because of the resistance of the compression fault and a neighbourhood of watershed at the south the discharge might be small.

Photo 5  The Section of Lianxi River which developed along a cross tension joint

Photo 6  Water swallow in the interposition of tension joints

Photo 7  The near feature of subsequent tension joints in water swallow
Maoshanli to Shengchaolong horizontal karst zone

Based upon the test of electrifying charging method and boring from Maoshanli Y-3062 (where ground water table 341m) via Linghaitang S-3056 to Shengchaolong S-3030 (altitude 324,97m) the karst cavities in this zone are discovered mostly to be filled with clay, yet there are unfilled cavities but inundated which are under serious dissolution. It indicates that except there is a shallow ground river at an elevation of 300-310m, there might be a water filled horizontal karst zone at an elevation of 260-280m and somewhere nearby.

Futangshangtong to Lianxi horizontal karst zone

A regular variation of the hydrochemical composition of the linear distribution karst springs from Futangshangtong via Shengchaolong to Lianxi indicates C$_1$$^4$ and C$_1$$^y$ might contact directly due to the fault in its depth. Based on the dynamic investigation of the water discharge in Lianxi River, a large amount of water leakage of 988.3 (at dry period)-4703 (at raining period) m$^3$/d at the intersection of the cross tension joint along the river section in NWW direction and the subsequent longitudinal tension joints at Shengchaolong (Photo 5-7) is recorded. Meanwhile, boring discovered that karst cavities about Shengchaolong are well developed and there are inundated cavities under serious dissolution. In addition, Lianxi S-3050 react sensitively only to the rainfall about Futangshangtong. All these indicate that along NW direction from Futangshangtong to Lianxi tenso-shear fault somewhere about to Shengchaolong which collects karst water of the horizontal karst zone originated from Maoshanli and then between the Lianxi River cross tension joint and Lianxi there might be a horizontal karst zone. The discharge of the karst zone might be comparatively very large, and the altitude of the karst zone near the tunnel might be approximate 260-280m.

Futang Farm to Lianxi horizontal karst zone

A part of the ground drainage in Futang Farm karst depression overflows to a very large extent at Y-3087 (Photo 8) during raining period, yet from Y-3087 to Lianxi S-3049 the surface water varies in and out of the ground at place to place and the hydrochemical composition of karst well-springs give a regular variation between apparently. These indicate in C$_1$$^y$ there might be a horizontal karst zone along C$_1$$^y$-contact zone which collects water from Futangshangtong to Lianxi horizontal karst zone at the south and flows downward to Xialianxi.

Xialianxi to Liaojiawan horizontal karst zone

Boring in this district discovered a serious water leakage near Xialianxi, most of the karst cavities are filled with clay yet still there are unfilled cavities but filled with water which is under serious dissolution. Besides, water flow in dry period at S-4080 amount to 2163 m$^3$/d and at the confluence of Daixiajiang River and Lianxi River the water leakage at dry period amount to 3024 m$^3$/d. All these indicate from Xialianxi to Liaojiawan there might be a horizontal karst zone which catches the water from the north two karst zones of Lianxi and rainfall as well. The flow of the horizontal karst zone would be in a great extent and flows southward to the Liaojiawan direction. And its altitude near the tunnel might be about 260-280m.

IV.164
(3) Analysis of karst dynamic geological phenomena

New drain gallery shall be made while excavating the tunnel and a large degradation of the ground water level, an increase of the flowing pressure, subsurface erosion and consolidation of earths, negative pressure in the karst cavities will happen as well. When tunnel crosses the karst valley, the depressions, such as Hengxialing (0.78km long), Maoshanli to Shengchaolong (1.08km long) and Xialianxi (0.55km long) many dynamic geological phenomena as earth's surface settle, collapse, will happen in the vicinity 0.3-0.5 km on both side of the tunnel, and when karst cavities being exposed and water outburst being concentrated, mud-sand spouts, earth's collapse and well-springs about the tunnel to be attenuated or even dry up will be the key to the engineering geological problems happen to three sections of the tunnel in its excavation. These three sections are Hengxialong, Maoshanli to Shengchaolong and Xialianxi which total 2.4 km long. In order to ensure the safety under construction and traffic operation and to reduce ill-influence to environment protection the following suggestions are proposed:

IV.165
tunnel and close working steps, scheduled in dry season of the year, well be adopted.
(2) In construction, beside reasonable measurement must be taken in time to the exposed karst cavities, radio detection through the whole carbonate section of the tunnel must also be taken to discover the concealed karst cavities if there is any under the roadbed in time or in advance.

5 PRIMARY CONCLUSIONS

(1) The dissolution intensity of the carbonate formation is largely lies on its lithological characters, structure and supplying and draining of the ground water. Therefore a thorough analysis to every aspect of these basic conditions which may give reasonable predictions must be the only key to the karst zone that may happen in tunnel excavation.
(2) In general dissolution intensity of carbonate formation is non-homogeneous but in a same engineering geological division it provides a relative regularity. That makes a possibility of estimating the karst engineering geological problems happened to the tunnel section as a whole.
(3) The method of hydro-dynamic balance and the method of feeding from well-springs for calculating the total water discharge of the tunnel in karst rocks which deals with a whole dissolution intensity is adaptable. And the method of hydro-dynamic and method of feeding from well-springs for calculating water outburst which deals with the karst zone is adaptable.
(4) As tunnel locates at the transitional belt of the vertical and horizontal karst zone and sinks a little depth to the ground, whereupon concentrated water outburst, mud-sand spout, earth's surface collapse, well-spring discharge attenuate or dry up will be unavoidable and becomes essential!
(5) Leading drilling, roadbed detections, draining to the open water and blocking up to the ground water, close working steps are necessary processes to ensure safety of tunnel construction and traffic operation as well as to prevent ill-influence to the surroundings.
COST AND BENEFIT OF SITE INVESTIGATIONS FOR TUNNELING IN PORTUGAL

ANALYSE COUTS-BENEFICES DES TRAVAUX DE PROSPECTION GÉOTECHNIQUE DANS DES CAS DE TUNNELS AU PORTUGAL

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Professor UNL
Portugal

ABSTRACT

The costs and the benefits of site investigations for tunnels on rock masses were assessed, by using six case histories of tunnels designed in Portugal during the last decade. Based on, one side, upon the cost of the different types of site investigation works and, on another side, upon the amount of information supplied by each of them for the assessment of the geotechnical behaviour of the rock mass at design stage, a cost-benefit analysis is presented. A coefficient (Return Coefficient) which reflects the return of the capital invested in doing a certain type of work in terms of actual knowledge of site conditions it provided, enables a comparison between the different types of works to be made.

Finally the paper includes a comparison between the costs and the benefits of site investigations for tunnelling purposes with those for dam foundations obtained through a similar research.

ABSTRAIT

Les coûts et les bénéfices des travaux de prospection géotechnique pour des tunnels dans des massifs rocheux ont été définis à l'aide de six "Case histories" dont le projet a été exécuté au Portugal pendant les années 70.

Ayant comme base, d'une part, le coût des différents types de travaux de prospection et d'essais et, d'autre part, le pourcentage d'information fourni par chacun de ces travaux envers la connaissance du comportement géotechnique des massifs rocheux à la phase de projet, les auteurs présentent une analyse coûts-bénéfices et tirent des conclusions sur l'intérêt relatif des méthodes qui sont utilisées plus fréquemment.

À la partie finale de la communication les auteurs présentent une comparaison entre cette analyse coûts-bénéfices et une autre du même type faite pour des études de sites de barrages.

1 - General

The goal of site investigation (S.I.) is to predict the ground conditions to be encountered and to provide relevant information which the design will be based on.

The benefits of S.I. can be regarded in two ways: geotechnical and financial. Advances in the understanding of both geological and geotechnical conditions (geotechnical benefits) make the predictions to be closer to the actual conditions and so decreasing the risk of inadequate designs, this affecting both safety and cost (financial benefits). From the
above, geotechnical benefits and financial benefits are related in real life.

Site investigations for tunnels on rock masses require, in general, great costs to achieve high standards of knowledge in that respects the ground and so to obtain adequate parameters for design. This is due to the particular character of those structures – long and frequently with thick overburden. This has been one of the paramount reasons why the levels of knowledge achieved by S.I. for tunnels are normally lower than the ones for dams, as far as the design needs are concerned (CARVALHO, 1981).

A thick overburden constitutes in many cases what is some times called "dead ground" which has to be penetrated to gather information at depth concerned by the structure. However this is not true in most situations due to the fact that the geologic formations which form the overburden in some sections of the tunnel occur at tunnel depths some distance away from those sections. This means that the situations encountered along a specific borehole drilled from the surface of the ground (with the exception of the weathered top ground) are representative of what is to be found at tunnel depths. If, as it should be, the extent of site investigation works is such that a statistical appraisal of the rock mass is possible, no "dead ground" has to be considered and all the information obtained from the surface down to the tunnel depths is relevant for the design of the structure.

This concept of statistical appraisal of the rock mass quality based on a significant number of results of tests and on a reliable sampling of the formations leads to their engineering geological zoning and classification (OLIVEIRA, 1974, 1979).

This paper deals with the site investigations performed for six tunnels on rock masses designed in Portugal during recent years – table I.

The costs of the investigations and their geotechnical benefits will be discussed.

2 - The costs

The costs of each type of exploration and testing works for the six tunnels are presented in table II. Each cost is expressed as a percentage of the total money spent with the site investigations performed for each project.

The figures quoted for "Engineering Geological Staff" correspond to the amount spent in payment of the staff involved in the studies, either from the owners or/and from the consultants. In some cases both owner's and consultant's staff can be involved.

The task of the engineering geological staff is to gather relevant information already available and also to undertake the reconnaissance of the ground; to plan the programmes for subsequent exploration and testing; to follow the progress and closely supervise the execution of these works which, in general, are contracted out; to suggest alterations or modifications to the established programmes when advisable and to assess the information and produce appropriate reports to assist in design.

The costs of the engineering geological staff belonging to the site investigation contractors are not accounted for in the previously mentioned ones since they are con...

<table>
<thead>
<tr>
<th>TUNNELS</th>
<th>ALTO LINDOSO</th>
<th>CASTELO DO BODE</th>
<th>FUNCHO BENACIATE</th>
<th>SABUGAL MEIMOA</th>
<th>SADO MORGAVEL</th>
<th>Sta. CLARA- MTE. DA ROCHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>LENGTH (m)</td>
<td>5729</td>
<td>5090</td>
<td>1410</td>
<td>4073</td>
<td>1315</td>
<td>9860</td>
</tr>
<tr>
<td>DIAMETER (m)</td>
<td>9.0</td>
<td>3.0</td>
<td>2.9</td>
<td>3.0</td>
<td>2.9</td>
<td>2.7</td>
</tr>
<tr>
<td>OVERBURDEN MAX.</td>
<td>400</td>
<td>90</td>
<td>50-90</td>
<td>100</td>
<td>130</td>
<td>100</td>
</tr>
<tr>
<td>BETWEEN PORTALS MIN (m)</td>
<td>95</td>
<td>10</td>
<td>30</td>
<td>10-30</td>
<td>20-30</td>
<td></td>
</tr>
<tr>
<td>ROCK MASS TYPE</td>
<td>GRAVITE</td>
<td>GNEISS</td>
<td>MAINLY SCHIST</td>
<td>SHALE</td>
<td>SHALE</td>
<td>SHALE</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>LIMESTONE AND DOLERITE</td>
<td>AND GREYWACKE</td>
<td>AND GREYWACKE</td>
<td>GREYWACKE</td>
</tr>
</tbody>
</table>

* Three contiguous tunnels with lengths of 400 m (initial tunnel), 150 m (intermediate) and 800 m (final).

Table I – Some characters of the tunnels

IV.168
<table>
<thead>
<tr>
<th>TYPE OF WORK</th>
<th>ALTO LINDOSO</th>
<th>CASTELO DO BODE</th>
<th>FUNCHO- BENCACATE</th>
<th>SABUGAL- MEIMOA</th>
<th>SADO- MORGAVEL</th>
<th>Sta CLARA- M.t DA ROCHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEOPHYSICS</td>
<td>-</td>
<td>.57</td>
<td>1.5</td>
<td>-</td>
<td>10.7</td>
<td>8.5</td>
</tr>
<tr>
<td>TRENCHES</td>
<td>7.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOREHOLES</td>
<td>39.8</td>
<td>51.1</td>
<td>50.6</td>
<td>75.5</td>
<td>61.7</td>
<td>63.3</td>
</tr>
<tr>
<td>(including Lugeon tests)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IN-SITU TESTS</td>
<td>24.1</td>
<td>16.2</td>
<td>26.9</td>
<td>-</td>
<td>12.8</td>
<td>19.4</td>
</tr>
<tr>
<td>LABORATORY TESTS</td>
<td>4.6</td>
<td>1.4</td>
<td>1.9</td>
<td>-</td>
<td>2.1</td>
<td>1.0</td>
</tr>
<tr>
<td>ENG. GEOL. STAFF</td>
<td>24.4</td>
<td>256</td>
<td>19.0</td>
<td>24.5</td>
<td>12.8</td>
<td>7.9</td>
</tr>
</tbody>
</table>

Table II - Site investigation costs - details. (Figures are given as percentages of the total cost of S.I. for each tunnel).

<table>
<thead>
<tr>
<th>ENG. GEOL. STAFF COSTS (100%)</th>
<th>ALTO LINDOSO</th>
<th>CASTELO DO BODE</th>
<th>FUNCHO- BENCACATE</th>
<th>SABUGAL- MEIMOA</th>
<th>SADO- MORGAVEL</th>
<th>Sta CLARA- M.t DA ROCHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>FOR DESK STUDIES AND REPORT PREPARATION</td>
<td>3.3%</td>
<td>7.0%</td>
<td>5.0%</td>
<td>3.5%</td>
<td>4.0%</td>
<td>4.3%</td>
</tr>
<tr>
<td>FOR FIELD STUDIES</td>
<td>6.7%</td>
<td>2.4%</td>
<td>5.0%</td>
<td>6.5%</td>
<td>6.0%</td>
<td>5.7%</td>
</tr>
<tr>
<td>FOR GATHERING EXISTING DATA AND RECONNAISSANCE ONLY</td>
<td>17%</td>
<td>3%</td>
<td>7%</td>
<td>12%</td>
<td>16%</td>
<td>11%</td>
</tr>
<tr>
<td>ENG. GEOL. STAFF COSTS / COST OF S.I.</td>
<td>24.4%</td>
<td>256%</td>
<td>19.0%</td>
<td>24.5%</td>
<td>12.8%</td>
<td>7.9%</td>
</tr>
<tr>
<td>ENG. GEOL. STAFF COSTS / COST OF CONSTRUCTION</td>
<td>0.02%</td>
<td>0.32%</td>
<td>0.23%</td>
<td>0.17%</td>
<td>0.08%</td>
<td>0.04%</td>
</tr>
<tr>
<td>COST OF S.I. / COST OF CONSTRUCTION</td>
<td>0.1%</td>
<td>1.3%</td>
<td>1.2%</td>
<td>0.7%</td>
<td>0.6%</td>
<td>0.5%</td>
</tr>
</tbody>
</table>

Table III - "Engineering Geological Staff" costs: Details and comparisons

sidered in the costs of the works contracted.

Details and comparisons of the "Engineering Geological Staff" costs for the six cases under study are presented in Table III.

The reduction to a common basis, of the costs actually paid by the owners for the S.I., is necessary to enable a comparison both within the different items in each project and between different projects. This is also necessary if a comparison between the costs of the site investigations and the costs of construction is to be assessed, and also to try a comparison between the costs and the benefits of site investigations. The costs cannot be used without such a reduction due to variations caused by inflation over a number of years in Portugal.

It was decided to convert all the costs which were obtained from the owners, consultants, contractors etc., to 1980 prices. The Bank of Portugal price indices proved to be a satisfactory way for that purpose (CARVALHO, op. cit.).

It was after that conversion that tables II and III were worked out.
3 - The benefits
   3.1 - Definitions
   The benefit of each type of investigation work is assumed to be the amount of effective knowledge it provides with respect to the geological and geotechnical conditions of the ground that are relevant for each design, expressed as a percentage of the knowledge which would be desirable (complete knowledge) at completion of site investigations.

   Complete knowledge is taken to mean that designs require no further relevant information about the ground.

   Total knowledge is the total information available at the end of a site investigation, regardless of whether it answers all the design geotechnical questions. Total knowledge is, therefore, usually less than complete knowledge which, as a rule, is never reached.

   The benefit of a site investigation as a whole is defined in this paper as the percentage: total knowledge ÷ complete knowledge.

   3.2 - Approach used for assessing the benefits
   After all the information related to the site investigations for each tunnel has been gathered, several meetings were arranged with the individuals responsible for each investigation who are called assessors here. They were asked to provide, for each project:

   i) marks for the benefit obtained from each type of work in terms of a percentage of the total knowledge (assumed as 100% for this purpose in order to make the answer easier),

   ii) a mark for the site investigation as a whole, as a percentage of the complete knowledge, according to their judgement, after the design has been completed.

   Answers were given individually by each assessor, in a total of seven for the six tunnels under study. Some of the assessors were involved in more than one investigation and for each tunnel two to four assessors were inquired. Age group and profession status of those assessors are presented in table IV.

   The marks obtained in (i) had to be corrected taking into account the marks in (ii) so that the benefit of each type of work in terms of complete knowledge could be assessed. Let us consider, for example, a programme of boreholes. If the answer to question (i) was 25% and the answer to question (ii) was 80%, then the benefit of that programme in terms of complete knowledge (effective geological and geotechnical knowledge gained from the site) will be 25% of 80%, i.e., 20%.

   The marks given by each assessor to both the benefit of each type of work and to the total benefit of S.I., the first ones after the above explained correction, are presented in table V. Table VI contains the averages of the different marks.

4 - Cost - Benefit analysis
   A cost-benefit ratio for the different types of works performed during an investigation, as it is understood in economic terms cannot be used here. For it to be meaningful, the benefits should be expressed in the same way as the costs, i.e., in terms of money. As previously stated, the benefits of S.I. dealt with in this work (the geotechnical benefits) cannot be quantified in that manner. The method selected for their quantitative assessment is rather subjective, the results reflecting the personal feeling of the individuals who were responsible for the S.I., i.e., the assessors. Nevertheless, the benefit/cost relationships, using the data on tables VI and II, can be regarded as a coefficient—the Return Coefficient (RC) which shows the return for the money spent in a given type of work in terms of actual knowledge of the site conditions it provided. For a given percentage of the total cost of a site investigation spent in doing the work, RC will increase as the effective knowledge of the site conditions it provides, increases.

   Any conclusions which can be derived from relating the benefits and the costs can only be of a general character and respecting to the Portuguese case. They are conditioned: a) by the feeling of the assessors as previously stated,
### Table V - Individual marks assigned by the assessors to the benefit of S.I.

<table>
<thead>
<tr>
<th>ASSESSOR OF WORK</th>
<th>ALTO LINDOSO</th>
<th>CASTELO DO BODE</th>
<th>FUNCHO - BENACIATE</th>
<th>SABUGAL - MEIMOA</th>
<th>SADO - MORGAVEL</th>
<th>Sta CLARA - Mte DA ROCHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>RECONNAISSANCE &amp; GATHERING EXISTING INFORMATION</td>
<td>17.0</td>
<td>31.5</td>
<td>16.0</td>
<td>12.0</td>
<td>5.0</td>
<td>8.0</td>
</tr>
<tr>
<td>GEOPHYSICS</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>8.5</td>
<td>12.5</td>
<td>13.5</td>
</tr>
<tr>
<td>TRENCHES</td>
<td>8.5</td>
<td>22.5</td>
<td>4.0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOREHOLES (incl Lugoan tests)</td>
<td>25.5</td>
<td>13.5</td>
<td>4.0</td>
<td>24.0</td>
<td>42.8</td>
<td>42.3</td>
</tr>
<tr>
<td>&quot;IN-SITU&quot; TESTS</td>
<td>12.0</td>
<td>13.5</td>
<td>12.0</td>
<td>6.0</td>
<td>19.0</td>
<td>9.0</td>
</tr>
<tr>
<td>LABORATORY TESTS</td>
<td>12.0</td>
<td>9.0</td>
<td>8.0</td>
<td>6.0</td>
<td>19.0</td>
<td>16.2</td>
</tr>
<tr>
<td>TOTAL BENEFIT OF S.I.</td>
<td>8.5</td>
<td>9.0</td>
<td>6.0</td>
<td>6.0</td>
<td>9.5</td>
<td>9.0</td>
</tr>
</tbody>
</table>

### Table VI - Averages of the marks given to the benefits of S.I.

<table>
<thead>
<tr>
<th>RECONNAISSANCE &amp; GATHERING EXISTING INFORMATION</th>
<th>ALTO LINDOSO</th>
<th>CASTELO DO BODE</th>
<th>FUNCHO - BENACIATE</th>
<th>SABUGAL - MEIMOA</th>
<th>SADO - MORGAVEL</th>
<th>Sta CLARA - Mte DA ROCHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.1</td>
<td>6.1</td>
<td>8.7</td>
<td>11.6</td>
<td>15.5</td>
<td>15.5</td>
<td></td>
</tr>
<tr>
<td>GEOPHYSICS</td>
<td>-</td>
<td>12.1</td>
<td>8.7</td>
<td>-</td>
<td>16.0</td>
<td>26.8</td>
</tr>
<tr>
<td>TRENCHES</td>
<td>11.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BOREHOLES (including Lugoan tests)</td>
<td>25.7</td>
<td>41.3</td>
<td>41.5</td>
<td>38.3</td>
<td>32.5</td>
<td>28.6</td>
</tr>
<tr>
<td>&quot;IN-SITU&quot; TESTS</td>
<td>12.1</td>
<td>12.3</td>
<td>19.7</td>
<td>-</td>
<td>12.0</td>
<td>12.2</td>
</tr>
<tr>
<td>LABORATORY TESTS</td>
<td>10.0</td>
<td>19.2</td>
<td>8.7</td>
<td>-</td>
<td>4.0</td>
<td>4.4</td>
</tr>
<tr>
<td>TOTAL BENEFIT OF S.I.</td>
<td>78.7</td>
<td>916</td>
<td>675</td>
<td>500</td>
<td>800</td>
<td>875</td>
</tr>
</tbody>
</table>

(Recovery and gathering existing information) (Laboratory tests) (Geophysical exploration) (Boreholes and "in-situ" tests)

b) by the prices demanded in Portugal, c) by the approach in conducting the site investigations and b) by the statistical meaning of the number of results which it was possible to consider.

The RC connected with each type of work performed in the site investigations for each of the tunnels studied was assessed and plotted against the cost (Table I) of the same work, in fig. 1.

The above graph clearly shows a great difference in the RC values within the different types of work performed which can be considered in four groups:

A - Works with high to very high RC (Reconnaissance and gathering existing information)

B - Works with high to moderate RC (Laboratory tests)

C - Works with high to low RC (Geophysical exploration)

D - Works with low RC (Boreholes and "in-situ" tests).
Fig. 1 also shows that the most expensive works - boreholes and "in-situ" tests are those which present lower RC's. This contrasts with the works which by rule take less money in site investigations for tunneling purposes - reconnaissance and gathering existing information, and laboratory tests.

People experienced in site investigations will certainly agree that the trend of the graph in Fig. 1 would be expected but its quantification has never been tried before. It appears to the authors that the trend of that graph would not be very different if a greater number of results could be considered.

The RC graph represents a genuine reflection of a well established conceptual appreciation of the development of the knowledge during a site investigation. It explains, in some way, why certain types of works (e.g., those included in groups A, B and C above) are used in the early stages of the studies when a compromise exists between attempts to obtain a general picture of the rock mass and the amount of money usually allowed to be spent.

The lower RC's obtained for boreholes and "in-situ" tests are mainly due to the costs involved in their performance. It is believed that, some times, better RC's may be obtained by improving the information which these exploration and testing techniques can provide, in terms of both type and standard.

The results also suggest that special care should be paid to these works in that respects their planning and execution conditions, in accordance to the expenditures required by their performance.

5 - Conclusion

Table V shows that in some cases the judgement of the assessors in that respects either the benefit of each type of work or the benefit of a site investigation is quite different. This reveals personal feelings related not only with the value, amount and reliability of the information obtained but also with the assessor's background and idea about the design needs in terms of that information, and reflects in fact, what happens in real life. As a result, a site investigation (type of works, their extent and location) can be quite different if conducted by different people. Obviously, this can also introduce great differences in the costs of the investigations.

The above emphasizes the importance of good training and expertise in that concerns the people charged with the responsibility of
It seems also to be interesting to compare some of the results presented in this paper with results obtained in the case of site investigations for dam foundations on rock masses, assessed during a similar research performed by one of the authors. This involved the costs and the benefits of S.I. performed for 8 concrete arch dams, 6 concrete gravity dams and 6 earth dams.

For the total benefit of S.I. the following results were achieved:

ARCH DAMS - 80% to 95%; average 89%
GRAVITY DAMS - 70% to 85%; average 80%
EARTH DAMS - 75% to 95%; average 85%
TUNNELS - 50% to 92%; average 79%

The above figures show that there is a trend towards more detailed studies for concrete arch dams.

It would be expected that the figures for tunnel site investigations were lower, in comparison with those for dams. This situation can be regarded as normal according to what has been the Portuguese methodology for tunnel site investigations. The detail of final design of tunnels is made only during construction taking into account the additional knowledge provided by excavation, mainly in the first stages. The engineering geological information for the design should, however, give a clear picture of the rock mass (in terms of its geotechnical zoning) and provide the designer with information about the various options deemed appropriate for the excavation and support of the tunnel.

Also, the cost of S.I./cost of construction is an index which deserves to be compared. It may, in some way, give an idea about the standard of the investigations. The values obtained for that relationship as regards the cases studied were as follows:

ARCH DAMS - 0.4% to 8.9%; average 2.6%
GRAVITY DAMS - 0.4% to 0.8%; average 0.6%
EARTH DAMS - 0.4% to 2.5%; average 1.2%
TUNNELS - 0.1% to 1.3%; average 0.7%

REFERENCES:


RISK ASSESSMENT AND CONTROL ON TUNNELLING CONTRACTS IN THE UNITED KINGDOM

ESTIMATION DES RISQUES ET CONTROLE DANS DES CONTRATS POUR LA CONSTRUCTION DE TUNNELS DANS LE ROYAUME UNI

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ABSTRACT

The paper defines the term 'risk' in a civil engineering context and indicates how it affects the cost of tunnelling contracts in the United Kingdom. Several methods of reducing the effects of uncertainty are discussed from a contractual viewpoint, with emphasis being given to the use of alternative forms of contract. Some details are given of research at Durham University in which attempts are made to assess the adequacy of site investigation in past tunnelling contracts. Site investigation and contract cost data are presented.

ABSTRAIT

L'objet de cet exposé est de définir le facteur risque dans des ouvrages de travaux publics et ses répercussions financières sur les contrats pour la construction de tunnels dans le Royaume Uni. Nous présentons plusieurs méthodes pour réduire les effets des facteurs imprévisibles sur le contrat et, particulièrement, la possibilité d'opter pour diverses formules de contrat. Nous donnons également des détails de certains projets de recherches réalisées par l'université de Durham sur la tentative d'évaluation de l'efficacité des études d'implantation qui ont eu lieu pour des contrats de tunnels donnés. Ces détails portent sur les études d'implantation ainsi que sur l'aspect financier des contrats.

INTRODUCTION

Many people would agree that tunnelling is one of the most risky of all civil engineering operations. However, would they all agree on what is meant by the word 'risk'? Reference to the dictionaries provides a selection of definitions: danger; peril; hazard; chance of loss; and consequence etc; exposure to mishap; amount covered by insurance. To the non-engineer the word is synonymous with danger, peril, hazard, especially to life and limb. In a wider sense risk implies a potential monetary loss or gain, and without some ability to quantify insurance would be impossible.

Many different factors can contribute to the overall risk of a tunnelling project. Their relative importance depends on the type, location and duration of the work. Government influence is imposed, both directly through laws controlling safety of labour and indirectly through control of the economy, inflation rates and the stability of society. The project client introduces numerous risks into a contract, since he ultimately controls the cash flow and can cause serious disruption by delaying information access.
payment or the settlement of disputes and claims. The Engineer to a contract is also responsible for the smooth running of the works and as such is involved in a mediatory capacity with respect to financial problems. On the opposite side the contractor can also be responsible for delays as a result of his negligence or inefficiency.

However, all sides of a contract would agree that the most important factor in tunnelling is that the cost of the enterprise is much more closely controlled by the nature of the ground – in other words, the geology and hydrology – than are most other civil engineering operations. The close control arises from the presence of an unexpected hazard which can escalate the original costing and generate a chain of decisions each of which concatenates a quantifiable risk. On the other hand, if a potential hazard is foreseen, and is allowed for in some way in the costings, then a substantial saving over the expected contract cost can be made if the hazard does not arise or is less severe than forecast.

It is useful to note in Table 1 some comparisons between estimated contract cost, as defined by the tender (bid) price and the final costs of some tunnelling contracts in England. The percentage that the winning tender is below the average of the second and third tenders is an indication of the ‘keenness’ of the winning bid, and one might perhaps expect to observe a tentative trend of increasing construction and contract costs with increasing margin between tender prices. The wide variation in percentage construction and contract costs above winning tender values can be said to be caused mainly by the results of risks and actions taken under conditions of uncertainty, but since this variation is expressed on an individual contract basis it represents a smoothing of what could well be more severe variations over (usually) and under a forecast price with respect to individual lengths of a particular tunnelling contract. It is also noted that the comparative realism of the construction or contract cost to tender value ratio can be affected by the level of cost inserted in the bill of quantities by the Engineer under the Provisional Sums and Prime Cost Sums headings. Whereas several years ago there may have been a proneness to use provisional sums to 'pad out' a tender value, current experience shows that any such sums are much more specifically designated within the contract.

An obvious conclusion that emerges is that on any multi-million pound contract, any reductions that can be made in the variations from forecast conditions and prices are usually likely to lead to a financial saving for both sides of the contract.

**RISK CONTROL**

The problem of risk reduction can be approached in a contractual setting in several different ways, while noting that full disclosure of ground investigation information – of both a factual and interpretive nature – provides the initial basis for a more equitable sharing of geological risk between the contracting parties (CIRIA, 1978). A first approach accepts that there is an inherent indefinable level of risk in any construction project and tries, by means of manipulation of contract procedure, to accommodate potential risk. In order to understand these alternative forms of contract there needs to be an appreciation of current practice, and a precis of the most commonly-used form of contract in the U.K. is included in the Appendix.

Two alternative forms of contract which appear especially favourable for use on tunnelling works are Target Cost and Cost-Reimbursement contracts. Both place more of the risk on to the Client. A Target Cost contract works on a method-related Bill of Quantities incorporating all the heads of expenditure - salaries, wages, insurance, materials, plant, consumables - but excludes the fee or profit element. This fee and profit element would be agreed as a percentage of the target estimate formed from the priced Bill of Quantities. In addition, there would be provision for adjustment of the target estimate for variations in quantities and design or fluctuation in costs of labour and materials. To act as an incentive, there is the facility to adjust the fee, either positively or negatively, on a pre-arranged share-out between Client and Contractor.

Cost-reimbursement contracts eliminate this profit motive, with payment being made at cost price for work carried out, and profits and overheads being paid on a pre-arranged fee basis. By utilising this form of contract there is an increased responsibility on the Engineer and his staff to ensure that the Client is getting value for money.

Research by Ashley (1977) has indicated that there are benefits to contractors in sharing the risk of a contract. Not
<table>
<thead>
<tr>
<th>Contract</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual Tender Value (£)</td>
<td>2,299</td>
<td>853</td>
<td>532</td>
<td>7,548</td>
<td>326</td>
<td>524</td>
</tr>
<tr>
<td>July '81 Tender Value (£)</td>
<td>2,170</td>
<td>720</td>
<td>579</td>
<td>9,229</td>
<td>12,229</td>
<td>524</td>
</tr>
<tr>
<td>Percentage Winning Tender</td>
<td>+5.15</td>
<td>+5.77</td>
<td>+9.65</td>
<td>+10.53</td>
<td>+10.94</td>
<td>+28.11</td>
</tr>
<tr>
<td>Below Average of 2nd and 3rd Tenders</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual Construction Cost (£)</td>
<td>1,120</td>
<td>948</td>
<td>1,031</td>
<td>3,613</td>
<td>281</td>
<td>788</td>
</tr>
<tr>
<td>July '81 Construction Cost (£)</td>
<td>1,080</td>
<td>675</td>
<td>1,179</td>
<td>3,679</td>
<td>104</td>
<td>764</td>
</tr>
<tr>
<td>Actual Contract Cost (£)</td>
<td>540</td>
<td>248</td>
<td>1,227</td>
<td>4,141</td>
<td>311</td>
<td>764</td>
</tr>
<tr>
<td>July '81 Contract Cost (£)</td>
<td>827</td>
<td>012</td>
<td>2,057</td>
<td>8,291</td>
<td>108</td>
<td>238</td>
</tr>
<tr>
<td>Percentage Construction Cost</td>
<td>+35.7</td>
<td>-13.6</td>
<td>+61.8</td>
<td>+2.2</td>
<td>+13.6</td>
<td>-13.6</td>
</tr>
<tr>
<td>Above Tender Value (Actual)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percentage Construction Cost</td>
<td>+4.1</td>
<td>-36.2</td>
<td>+1.8</td>
<td>-30.9</td>
<td>-17.4</td>
<td>-19.6</td>
</tr>
<tr>
<td>Above Tender Value (July 1981)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percentage Final Contract</td>
<td>+47.9</td>
<td>-0.5</td>
<td>+73.5</td>
<td>+7.2</td>
<td>+19.9</td>
<td>-4.4</td>
</tr>
<tr>
<td>Cost above Tender Value (Actual)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percentage Final Contract</td>
<td>+12.8</td>
<td>-24.8</td>
<td>+9.4</td>
<td>-26.9</td>
<td>-11.7</td>
<td>-11.1</td>
</tr>
<tr>
<td>Cost above Tender Value (July 1981)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
Actual construction cost is the total certificated cost.
Actual contract cost is the construction cost plus design and supervision costs, compensation costs plus any other ancillary costs (land and legal fees, etc).
July 1981 costs are the actual costs, as incurred, but index-linked by date to July 1981.
* Total costs for sub-aqueous tunnel.
+ Excavation and primary lining of tunnels and shafts but excluding all ancillary works.

only do joint ventures allow contractors to participate in broader engineering fields than their specialist skills might encompass, but such ventures also permit them to spread their overall risk over a wider number and range of contracts, and thus exercise better control over corporate cash flow. However, some drawbacks are apparent in the arrangement.

In accepting a partnership arrange- ment, perhaps for a large construction job, a contractor automatically accepts liability for non-performance by any, or all, of his partners. If a position of mutual trust exists, this liability can be borne without redress to legal loopholes. In the situation where trust does not exist partners will want 'backout' clauses to avoid responsibility for the actions or 'non-actions' of others. Thus an intricate web of contract documentation is liable to exist, so making overall contract management difficult.

An alternative scheme, which is gaining popularity within the civil engineering industry, is the use of insurance to cover possible risks. The client is already insured by the contractor, through the contract documents, against certain eventualities. However,
### Table 2: Cost Data for some Tunnelling Contracts in England

<table>
<thead>
<tr>
<th>CONTRACT</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basis of cost</td>
<td>Total Actual</td>
<td>Total Adjusted to July '81</td>
<td>Primary Actual</td>
<td>Primary Adjusted to July '81</td>
<td>Adjusted to July '81</td>
<td>Adjusted to July '81</td>
</tr>
<tr>
<td>Tunnel external diameter (m)</td>
<td>3.4</td>
<td>1.78</td>
<td>3.51</td>
<td>3.35</td>
<td>3.55</td>
<td>3.55</td>
</tr>
<tr>
<td>Tunnel internal diameter (m)</td>
<td>3.2</td>
<td>1.20</td>
<td>2.93</td>
<td>2.83</td>
<td>2.75</td>
<td>1.83</td>
</tr>
<tr>
<td>Materials excavated</td>
<td>Coal measures mudstone and sandstone, shale coal seams</td>
<td>Fill, glacial deposits, Coal measures rocks</td>
<td>Clay, silt, sand, gravel, alluvium</td>
<td>Clay, silt, sand, coal, sandstone, mudstone</td>
<td>Clay, silt, sand, coal, sandstone, mudstone</td>
<td>Loose household and industrial fill; boulder clay</td>
</tr>
<tr>
<td>Contract dates</td>
<td>May '73–June '77</td>
<td>May '73–May '75</td>
<td>May '73–May '75</td>
<td>April '74–March '77</td>
<td>May '75–May '77</td>
<td>Feb '76–April '79</td>
</tr>
<tr>
<td>Cost/lin. m. (£/m)</td>
<td>6972</td>
<td>16880</td>
<td>2516</td>
<td>6392</td>
<td>288</td>
<td>60</td>
</tr>
<tr>
<td>Number of boreholes</td>
<td>32</td>
<td>16</td>
<td>35</td>
<td>77</td>
<td>49</td>
<td>85</td>
</tr>
<tr>
<td>Average site investigation cost per borehole (£/BH)</td>
<td>649</td>
<td>2466</td>
<td>1116</td>
<td>4233</td>
<td>103</td>
<td>411</td>
</tr>
<tr>
<td>SI (Total) Cost/Construction Cost (%)</td>
<td>2.67</td>
<td>3.29</td>
<td>7.88</td>
<td>8.42</td>
<td>0.69</td>
<td>1.11</td>
</tr>
<tr>
<td>SI (Total) Cost/Actual Tender Cost (%)</td>
<td>3.63</td>
<td>7.64</td>
<td>5.00</td>
<td>5.07</td>
<td>0.60</td>
<td>1.58</td>
</tr>
<tr>
<td>SI (excluding probing) cost/Construction Cost (%)</td>
<td>0.92</td>
<td>1.44</td>
<td>2.59</td>
<td>5.55</td>
<td>0.55</td>
<td>0.93</td>
</tr>
<tr>
<td>Value of claims + VO's attributable to deficiencies in SI</td>
<td>1.50</td>
<td>1.22</td>
<td>1.54</td>
<td>1.26</td>
<td>3.21</td>
<td>2.60</td>
</tr>
<tr>
<td>Cost of SI</td>
<td>1.50</td>
<td>1.22</td>
<td>1.54</td>
<td>1.26</td>
<td>3.21</td>
<td>2.60</td>
</tr>
</tbody>
</table>
NOTES FOR TABLE 2.

'Primary' cost basis: relates to the tunnel alone and excludes ancillary works. Tunnel A was sub-aqueous; the others were driven below land.

Cost adjustment to July 1981 base date is via the UK General Retail Prices Index.

GI (Total) Cost includes any probing ahead of the tunnel face.

Construction cost means here the total certificated value of the work undertaken (completed); together with the design costs, supervision costs, compensation costs, land and legal fees etc, it constitutes the Contract Cost.


CPF means Contract Price Fluctuation.

'Claims' here means claims for extra payment; for example, Clause 12 of the 5th Edition on the grounds of 'unforeseen physical conditions'.

VO's means Variation Orders, or variations to the Specification for the Works, issued by the Engineer.

the use of insurance policies to cover the contractor is relatively new. An example of a policy for contractors is the Contractor's All Risk (CAR) cover. However, the cover does not include the major uncertainty in tunnelling - ground conditions. A real appreciation of what causes cost escalation in tunnelling is needed before the drafting of any policy with specific provision for geological risks can be contemplated.

The alternative to manipulating contractual arrangements in order to accommodate an undefined risk is to attempt to identify and where possible quantify the risk. Theoretical cost models (Hoovenzadeh and Markow, 1976; Wheby and Citének, 1973) have been constructed to represent all or part of the complex problem of forming reliable tunnelling cost estimates. All, by necessity, have to make generalisations about variables, such as ground conditions, inflation adjustment, working rate and so on. It is the fact that these assumptions are required which has led to the non-acceptance of such models by practising engineers. It is with the aim of investigating whether any empirical relations can be formulated between theory and actual conditions that research has been carried out at Durham University, England over several years.

RESEARCH

Although decisions concerning soil and rock sampling locations hinge on informed judgement, a contractor will base his costing of a tunnelling job on his own interpolations and extrapolations from the sampling evidence. The density of the sampling clearly conditions the contractor costing, and it is known that decision theory can be used to predict the optimum density (Raiffa and Schlaifer, 1967). Although the decision theory approach has been applied to rock tunnelling (Einstein et al., 1978) its application to general tunnel construction has not yet been assessed in detail.

The basic question still remains: is there any method of predicting whether an increase in site investigation outlay on a particular contract would be rewarded with a corresponding or greater decrease in overall contract cost of the job?

The research undertaken by the authors into site investigation and tunnelling contract costs has taken two general forms: theoretical analyses based on decision theory, and historical reviews of costs actually incurred. These latter cost reviews have involved the circulation of detailed questionnaires to numerous Consultants, Contractors, Water Authorities and other Public Authorities in the UK with a view to creating a substantial data base from which cost trends can be projected for future guidance. Approximately fifty contracts have been studied. For this present paper further cost data from the first six of the contracts to be studied in depth can be seen in Table 2. These contracts are the same as those given in Table 1.
For each contract two sets of figures have been shown, one giving the actual costs and the other providing adjusted series of costs to a base-date figure of July, 1981. It was necessary to derive a method of adjusting costs to allow for inflation, in order to enable cost comparisons to be made, not only between contracts but also within contracts. Some research effort was expended in investigating and comparing methods of index-linked adjustment, and, in the end, one of the simplest methods, that of relating costs to a retail price index, was employed as a standard. This index-linking, performed on every cost as it was incurred, is, of course, a quite separate operation from the contract price fluctuation adjustments performed as contract accountancy operations in accordance with the ICE (5th Edition) Conditions of Contract - the so-called Baxter Indices published by HMSO*.

In attempting to link theory with empirical data two questions are fundamental. The first question is: can (SI) information level/value be related directly to cost?

To answer this question it is first necessary to understand what is meant by information level. The number of results from a series of tests in a particular location does not, in itself, constitute an information level since a proportion of that information is liable to be redundant in the sense that it does not contribute in any real way to the decision-making process. However, unless investigation is pre-planned in great (perhaps excessive) detail there will be an inevitable degree of information redundancy. Thus, in relating information level to cost, total information must realistically be included. Comparison between information levels on different contracts is difficult to achieve, since the amount of information required is dependent upon type of construction and ground conditions, but because (with the exception of Contract A) the studied contracts were similar in type and general location it is possible to relate the average July 1981 cost (£) per borehole (y) to the average number of tests per borehole (x). The equation based on July 1981 values is:

\[ y = 71 + 90.45x \]

Consideration must be given to the difference between information level and information value. The information required by an Engineer for the design of a contract may not necessarily be the same as the information required by a contractor for accurate pricing and work programming, although it can be argued that in an ideal world both parties to a contract should be looking for and using the information in the same way. In practice, the Engineer will veer towards a pessimistic appraisal of the ground information from the site investigation. The contractor must tend towards optimism for his tenders if he is to remain in business. Thus, from the outset, and in the absence of a mutually-agreed set of detailed ground reference conditions, the fact that, in effect, two different sets of ground reference conditions exist can and does lead to a climate of misunderstanding, conflict and claims.

Contract A, which is atypical and which is not included in the above equation because it shows a large deviation from the general trend, also happened to have a large percentage value of claims to contract cost. This may be taken as an indicator of relatively low value of information. However, care must be taken when drawing such a conclusion, since other factors could have contributed to the additional costs. A contract offering several construction options with different degrees of financial tolerance implicitly nested in each could attract higher cost penalties. Another point to be borne in mind is that the reported costs will have been smoothed over the whole of a contract whereas much of the contract might have been trouble-free, only a small percentage of the construction having taken place against a background of low information value with respect to the conditions encountered.

The value of information is dependent, therefore, on the use to which it is put in the decision-making process. Unfortunately, little documentary evidence about decisions is ever retained. However, in situations where it is possible to predict that the value of the information will be low and the cost of obtaining it high, for example deep boreholes in a known fault zone, alternative methods of risk control, as outlined previously, should be considered.

The second fundamental question of the research is whether the risk of

contract cost variation can be related to information level. Because research is carried out from a position of hindsight, it also views the contracts from a position of 'realised' risk or, to express the point another way, the recorded level of cost variation is not synonymous with the potential risk of cost escalation of a contract, when viewed from its start. To be effective in the decision-making process there has to be a transfer from post-to pre-posterior analysis. Empirically, this transfer appears to be the most difficult step in linking theory with practice. However, there are indications that variation in contractors' estimates could be used to provide an anticipated maximum value to the financial risk.

Assuming that there is a method of assessing the totality of risk inherent in a contract, the question still remains as to whether there exists a direct relationship between contract cost variation and errors/deficiencies in the site investigation. Although firm conclusions are not yet available on this question, it is possible to show that the types of problem affecting tunnelling costs can be categorised into two groups.

The first group contains additional works which only marginally alter overall cost; for example, an extension to pre-programmed excavation not on the critical path schedule. The second category concerns problems which necessitate alterations to the working method during construction, with the consequential delay and disruption. It has been shown using cost models (Thompson and Barnes, 1977) that delay prior to commencement has little overall effect on contract cost. The present research would indicate that, in the case of this second group, it is important to identify and quantify, in terms of possible cost escalation, the uncertainties remaining after the completion of the site investigation. If a quantifiable result could be obtained, it would allow contract programming to include flexibility, both in contract documentation and construction method.

REFERENCES


Raiffa, H. and Schlaifer, E. Applied statistical decision theory, Studies in Managerial Economics, Graduate School of Business Administration, Harvard University, Boston, Mass., 1967.


APPENDIX

Current Contractual Procedure in the U.K.

The most commonly used form of contract for civil engineering work in the U.K. is the Admeasurement contract, which is based on Bills of Quantities incorporating the principle of payment by a client (Project promoter) to a contractor through re-measurement of completed work at initially-tendered or subsequently-negotiated rates. A client, represented by an Engineer, attempts to achieve a fair price for the work by allowing contractors to compete for the work through competitive tendering. These bidding contractors are usually pre-selected on the basis of financial standing, size, experience and ability to undertake the work efficiently.

Contracts are usually let under the U.K. 5th Edition of Conditions of Contract and Forms of Tender, Agreement and Bond for the use in connection with Works of Civil Engineering, a document accepted by the parties to the Contract. These Conditions of Contract are subject to periodic revision, and the major revisions are intimately involved with the more equitable apportionment of risk between the parties to a contract.

One of the major advances came during the early 1970's when it was appreciated that, because of inflation, contractors were being placed in the untenable position of having to estimate, for the purposes of tendering, the costs of materials and labour up to two years in advance of their use. Estimates were becoming increasingly conservative and clients were required to meet the additional costs of a 'front-end-loading' nature. During the beginning of 1974, with the introduction of the
Variation of Price (VOP) clause, a method of adjusting costs was introduced, which took into account the effects of inflation. This clause was refined in the Contract Price Fluctuation (CPF) clause of the 5th Edition, with both clauses generally operative on contracts longer than twelve months duration, and basing their variation on monthly indices of labour, plant and materials which reflect fluctuations in cost.

Other forms of risk are less easily quantified. Clause 12 of the Conditions of Contract deals with the compensation available to an experienced contractor who could not reasonably have foreseen the actual conditions, particularly ground conditions, or obstructions that affected construction progress. This is the clause that relates most frequently to inadequacies (deficiencies in information value) in site and ground investigation.

Attempts have been made, through the Conditions of Contract, to set up a procedure whereby reassessment can be made during and after a particular contract period. Written into the Conditions of Contract under Clauses 51 and 52 is the facility for the Engineer to order and value a variation on any part of the work. If the work is of a similar nature to work already priced in the Bill of Quantities then payment will be made at those billed rates. However, if the work is dissimilar in nature, then valuation will be based either on a dayworks rate or on starred rates (materials, labour, plant etc. used on that work, plus a profit element).

The situation may arise, however, where the contractor feels that the rates applied to the additional work do not fairly recom pense him for the cost of doing the work. If the additional work is Clause 12 related, then the contractor can claim for a higher rate or price under that clause of the Conditions of Contract. All such claims must be substantiated and quantified by detailed calculations and it is not always the case that the premises upon which the calculations of the contractor on the one hand and the Engineer to the contract on the other will be the same.

An adjudication procedure is included in the Conditions of Contract, where Clause 66 deals with the settlement of disputes. In the first instance the Engineer is required to arbitrate between client and contractor and it is considered desirable that agreement should be reached at this level. Even so, it is often felt by contractors that, as the Engineer is either paid by, or, in the case of public bodies, employed by the client, he has an inevitable tendency to be biased in the client's direction.

Should there still be disagreement about the justification or make-up of a claim then either side has the right to request arbitration, a system deemed to be both quicker and cheaper than litigation. It also has the advantage that arbitrator(s) are independent and should have a sound technical knowledge on the subject of the claim. However, in many instances it is likely that the outcome of a problem will hinge on legal arguments and procedure for which the engineering arbitrator has received little or no specific training. Another weakness of this system is that the arbitrator(s) do not have to give reasons for their judgements; in fact there is liable to be a reluctance to do so, as it may provide the grounds upon which an order for award can be quashed by a court. However, recent revisions to the act allow either party to apply to the courts to enforce the arbitrator(s) to disclose reasons for decisions and also allow the arbitrator(s) to penalise any delaying tactics by either or both parties to the claim.

Accordingly, for all the attempts to use a contractual system to define the apportionment of risk, the prevailing feeling remains that 'if you can't win then try to ensure you don't lose', which at the very least provides an insecure base for setting up a multi-million pound project!
ABSTRACT

There is an ever-increasing need to study tunnels constructed several decades ago especially since cracks in the lining have appeared. The loss of or lack of previous geological studies plus the fact that construction has taken place make it necessary for the geologist and the engineer to adopt a kind of study which is rather different from the usual: the tunnel is already constructed and the lining prevent the nature and state of the rock from being seen. In such cases, the resistivity prospecting can adopt itself to these circumstances and be of great use. Five cases in which the study of the state of the lining, the injections of cement made, and the presence of hollows, the state of the concrete and the presence of decomposed rock are discussed.

ABSTRAIT

Très souvent on a besoin d'étudier des tunnels bâtis il y a quelques dizaines d'années. La défaut des études géologiques et des rapports de la construction posent à l'ingénieur et au géologue des modalités un peu différentes des usuelles: le tunnel c'est déjà percé et le revêtement cache la nature et l'état de la roche. Dans ces cas la prospection électrique, la méthode des résistivités, on peut adapter à ces circonstances et être de grande utilité. On commentent cinq exemples dans lesquels on a étudié l'état du revêtement, l'efficacité des injections du ciment faites, l'existence de vides, l'état du béton et la présence de la roche décomposée.

1. THE ELECTRICAL RESISTIVITY PROSPECTING METHOD AND TUNNELS

The electrical resistivity - prospection method is not widely used among engineers who devote themselves to the planning, construction and management of tunnels. The limited nature of its application plus the errors in interpretation when the terrain is neither very regular nor tectonically smooth are the reasons why its application in the study of ground where the construction of a tunnel is to take
place is not commonplace.

These shortcomings do not always justify its exclusion; in over-looking this method, a great potential in obtaining qualitative and semi-qualitative economic interpretations of ground distribution is lost. By this we mean interpretations that provide an approximate distribution of ground-rocks and indicate the approximate situation of faults, weak points in the terrain etc.

The adaptation of this method to actual problems can give results which are often surprising and sometimes conclusive in deciding on and orientating the solution of problems of the designing engineer or constructor.

Geology does not stop helping the engineer when the construction of the tunnel is completed. Often cement is injected immediately after construction, or a few years later. On other occasions, flaws in the lining are revealed; this makes investigation into their cause necessary. In such cases as these the electrical resistivity method can be of great help, as can be seen in the examples that we present.

2. THE BASICS OF ITS APPLICATION INSIDE A TUNNEL

An electrical resistivity survey can be done on the coating of a tunnel. It is enough to solve the means of applying the four basic electrodes of this method: A and B for the intensity circuit and, M and N for the potential circuit. The electric sounding will make it possible to determine the resistance and thickness of the concrete and that of the extrados. The surface of the coating cannot resemble a plan, that limits the semispaces, in which the classic formulas of the method are based. For lengths of AB inferior to the radius of the circle equivalent to the section of the tunnel, the errors can be discounted, for greater distances they will have to be modified. Nevertheless, the phenomena that affect the coating of a tunnel are usually a result of factors in adjacent terrain, not in the ground where the coating itself is, and as a consequence the traditional formulas are applicable.

The existence of heterogeneities in the concrete and in the terrain complicate the interpretation, but at the same time, the anomalies in the curves that are obtained reveal the presence of these zones, and with the aid of other external observations, they allow them to be located and even identified.

With the electrical profiling it is possible to achieve a quicker study, the obtaining of the resistances for fixed lengths of AB at each point of measurement. In this way, and with only one measurement we can know the resistance of the concrete, together with its state and quality, and knowing these qualities of the concrete, with another measurement we can gauge the resistance of the zone adjacent to the extrados of the lining, also together with the nature and state of the rock, water-content, existence of hollows, etc.

From table 1, it can be deduced that the resistance of the concrete can be known with sufficient exactness, that is if the measuring errors caused by the microdevices used can be eliminated. It is not possible to know the resistance of the terrain to the same degree, given that if the length of AB is increased, other more uncontrollable factors come into evidence.

If we use microdevices different from the Schlumberger ones (four electrodes aligned A and B occupying the extremes and M and N symmetrical in relation to the
TABLE 1.
Resistivities ohms-m
Schlumberger device

| AB = e 2e 4e 6e |
|-------------|---------|---------|---------|
| 1.02       | 1.23    | 2.05    | 3.05    |
| 1.02       | 1.21    | 1.90    | 2.65    |
| 1.01       | 1.19    | 1.75    | 2.40    |
| 1.01       | 1.15    | 1.60    | 2.05    |
| 1.00       | 1.10    | 1.35    | 1.60    |
| 1.00       | 1.00    | 1.00    | 1.00    |
| 0.96       | 0.96    | 0.88    | 0.80    |
| 0.99       | 0.94    | 0.72    | 0.57    |
| 0.98       | 0.93    | 0.59    | 0.38    |
| 0.97       | 0.90    | 0.51    | 0.28    |
| 0.96       | 0.86    | 0.47    | 0.23    |
| 0.96       | 0.82    | 0.42    | 0.17    |

g = ground resistivity
c = concrete resistivity
e = thickness of concrete
AB = length of Schlumberger device

centre of them and a distance less than 0.2. AB) we use tables similar to those in table 1. If the coating is of reinforced concrete it is necessary to use a somewhat different methodology, but this is not the purpose of this paper.

I am presenting a series of Spanish cases that I have studied personally, whose presentation makes it feasible to know the possibilities of these methods; this presentation is all the more important if the following factors are taken into consideration:

a) the equipment used is relatively cheap and b) 2 to 3 people apply them, obtaining in the cases that are shown here, a performance ranging between 200 m and 900 m per working day, with an approximate average of 400 m of tunnel studied per day.

3. STUDY OF REVETMENT

3.1. The La Grandota Tunnel (1962)

Situated four kilometres east of Oviedo (Asturias). Length 3,742 m; section 51.6 m²; coating of concrete 0.8 m. It passes - through a complex tectonic structure (Fig. 1) determined by a thorough geological survey made after the construction of the tunnel. Considerable deterioration in the coating was observed; this being caused by the effect of gypsum ferous water. This was the reason for a detailed study not only of the lining, but also of the massif as a prerequisite to the repairs in which injections of cement next to the lining were considered necessary. Lodes and veins and fissures with secondary gypsum were indentified, especially in the Permian Sandstones. The distribution of gypsum in the terrain neither explained the damage to the concrete, nor the intensity of the attack which affected the entire lining some parts. In any case it was necessary to pinpoint the areas where the lining suffered damage, and also the degree of the attack.

To achieve this objective, the application of the method described was proposed and accepted, with two series of measurements in the left wall, separating two points of measurement every 10 metres. The first series with lengths of AB = 0.6 m, and the second with AB = 1.5 m. With the first series we obtained the resistance of the concrete near of the surface, with the second, that of the whole lining, not greatly -
Fig. 1.- Geological section of the "La Grandota Tunnel" (F. Macau)
H = carboniferous; D = Devonian;
P = Permian; C = Cretaceous

Fig. 2.- "La Grandota Tunnel". Resistivity of the lining.

affected by the terrain. With a view to controlling, isolated measure-ments were taken two months later, the values of which were practically the same as the original ones.

In figure 2 we show the results in one stretch. The study enabled us to determine the areas where the damage to the concrete affected the whole lining, the areas where the damage was in initial stages and other areas which were unaffected.

It is situated in the "Depres-sion of the River Ebro" consisting of gypsum marls of the Continental Miocene, about 48 kilometres from Zaragoza, to east, and is a part of the Monegros Canal. It was complet-ed in 1962, but did not come into use for various reasons. The total
length is 6,200 metres, of which 1,037 were constructed as a false tunnel, that is, a trench was dug and later filled in. The section dug is 48 m² and the average thickness of concrete is 0.60 m. The land consists of marls, marl sandstones and sandstone all of Miocene, arranged close to the surface, without appreciable folding or tectonic faulting. The gypsum has infiltrated some of the sandstones and centimetric veins of this mineral can be observed.

Sulphur-resistant cement was not used during the construction, but it was used in some repairs made three-four years after the completion of the tunnel.

The lining showed signs of attack by water containing gypsum in about 1,500 m, which made an intensive geological study of the area and the lining necessary; this involved boring, the taking of samples, chemical analysis etc.

In order to know which parts were attacked by the gypsum, these only being evident when the attack had reached the visible face, and in order to know which were the parts contaminated by the gypsum-water and those attacks merely incipient, to guide the sample-taking in the lining, and in essence to centre the problem, I decided to use a method similar to that used in 1962 in the La Grandota tunnel.

Measures of resistance with AB = 0.5 m, were done, 1.0 m apart in the right sidewall, and in areas of 10 metres in the left sidewall and crown. A second series of measurements with AB = 3.9 m, each 5.0 m were taken in the same right sidewall.

In the first series we looked for the resistance of the concrete in the half near to the intrados of the lining. With the second series the resistance of the terrain and the possible existence of hollows,

**Fig. 3.** Alcubierre Tunnel Resistivity on right wall.
as a series of injections of concrete was foreseen.

The interpretation enabled us to identify the areas contaminated by gypsum, to guide and reduce the sample-taking in the lining and later to analyse them in the laboratory (uniaxial compression, chemical analysis, diffraction of X rays, etc) and also to decide on repairs, evaluate their cost, in study together with the results of all the investigations that took place. In Figure 3 we show the results of one stretch.

4. CONTROL OF INJECTIONS IN TUNNELS

The resistant lining of concrete in tunnels are intended to absorb the pushes of the terrain, that has been fractured, fragmented and meteorized during the digging. The importance of the resistance purpose grows as the quality of the rock diminishes or when there are faults. In such cases the use of systematic injections of cement in the extrados is recommended, this has as its objective filling in holes in the nearby area, especially in the vault, homogenize the decompressed and altered area, and regulate the distribution of tensions and finally complete the impermeability of the coating, if this also forms part of the work.

The injections are done using a series of drillings that go through the lining and are more frequently at 4-5 metres intervals. There are few fixed criteria for deciding when the injections have been correctly done, filling in the holes and fissures in the areas that correspond to each drilling. Sometimes a systematic first injection is done, and once this is finished a second one is done for testing and reinforcing in the areas considered to be doubtful, these are usually those which have allowed the greatest volume of concrete to be injected.

Any procedure which improves the criteria of diagnosis about the local efficiency of the injections will be of great use, though it is not a method which can be interpreted quantitatively, and it only provides indices and probabilities in which it is necessary to persist in specific areas.

We show two cases in which the electrical resistivity prospection has been applied. These cases make it possible to draw conclusions about the methods possible contributions.


The transportation for the irrigation areas on the right bank of the River Guadalhorce occurs principally on marls, clays and sandstones in flysh facies of Eocene. In this terrain, 26 tunnels were bored, with a total length of 8,143 metres. In the most important of these, the Sabinal tunnel, 5,150 metres long, various accidents occurred during the construction, and after the completion, it was necessary to reconstruct several stretches, increase thicknesses, modify sections etc. Once the construction was completed, injections were made in all the tunnels, in order to fill in holes, reinforce the impermeability of the coating and homogenize the area next to the lining. This work was considered of vital importance in the vault of the tunnel where the rock falls had been numerous and of importance.

The final thicknesses of the coating in all the tunnels ranged from 0.6 m to 2.0 m. The sections were horse-shoe shaped, oval and circular. The free height and width ranged from between 6.0 m and 7.5 m, and 4.0 m and 5.5 m respectively.

In order to make sure that the injections were efficient we decided to apply electrical resistivity in the crown with the AB = 3.0 or
Fig. 4.– El Sabinal Tunnel.

Resistivity in the vault and ton of cement injected. The number 1, 2, 3 show areas with insufficient injection.

2.5 m device, depending on the areas, applying it in a parallel way to the axis of the tunnel and with measuring points every 3.0 m.

The interpretation was complex because on the one hand it was necessary to take into account the kind of section and its thicknesses and on the other hand, the volume of injected cement, wetness, cracks, etc.

The study made it possible to locate the areas where it was advisable to reinject; this having been done, it was possible to confirm in almost all cases that the holes that existed had not been completely refill, it was thus possible to show that the method had provided nearly exact information. In figure 4, we show a stretch together with the results obtained.

4.2. Guadalefeo Tunnel (1964)

It forms part of the transportation system for the irrigation areas of the River Guadalefeo (Granada). It is 3,200 m long, to which an access gallery that is 354 m long must be added. The free section is about 20 m², and the thicknesses of concrete in the coating are 0.7 to 1.2 m with an average of 0.8 m.

It goes through slates and limestones.

Dampness and fairly intense dripping ad been observed in the tunnel, plus fissures and cracks in the vaults and sidewalls; this made it advisable to inject cement in order to fill in the holes and make the lining impermeable.

The vault was considered to be an area which indicated the effectiveness of the injections, and AB = 3.2 m device was used; it was usually situated in crown perpendicular of the axis of the tunnel with measuring points at intervals of 2.5 m. The resistivities obtained are affected by the shape of the vault, which is semicircular. The length and positioning of the device make it possible to accept that the entire area of the extrados and the lining, influence the resistivity obtained, and that the distance between the measuring points, in the same way, makes it possible to observe the whole of the zone near the vault.

Systematic measurement were made in the whole tunnel and the access gallery after the injections, and in 2,300 m of the tunnel, before and after the injections. Control measures were also made several months after the systematic measurements were made, and finally, simultaneous measurements with alternating and direct currents, which provided identical results.

The study made it possible to test the effect of the injections to located areas where new injections should be made and to find a relationship between the volumes injected and differences in resistivity before and after the injections (Figures 5, and 6). These permit a theoretical prediction of volumes to be injected into the tunnel if the resistivities before and after injection in the representative stretches, are known.
Fig. 5. - Guadaleo Tunnel. Resistivity in the vault before and after of the injections and tons of cement injected.

Fig. 6. - Guadaleo Tunnel
Relationship between ton of injected cement and differences in resistivity before and after the injections.
5. REPAIRING OF OLD TUNNELS

There are many tunnels, constructed several decades ago that reveal damage. Sometimes this is damage logical and predictable in all human work, on other occasions it indicates processes of degeneration in the concrete lining or in the rock where they are situated. Very often only the section-type of these tunnels is known, this, plus more or less reliable references as to the terrain that they crossed and the important incidences of construction.

Before repairing these tunnels, it is thus necessary to do a complete study of the lining, and quite often a geological study of the area is also necessary in order to determine the kind of rock and the state of the rock, where it comes into contact with the lining at the points where repairs and reinforcing are to take place. The electrical resistivity methods can help in these ever increasing cases, as can be seen in the following example.

The Railway Tunnels El Ferrol (La Coruña) to Aviles (Asturias) (1979)

This railway line links the above mentioned towns, along the north and north-west coast of Spain. Its length is 293 kms and there are 114 tunnels with a total length of 30,238 m.

In 1978 a systematic study of the tunnels in this railway line was begun, in preparation for their repairing and reconditioning. Given the importance of the work anticipated, it was necessary to establish an order of priorities for the repairs and reconditioning.

The tunnels have a horseshoe section: 2.70 metres wide in the invert and 3.25 metres in the starting point of the vault, which is semi-circular: the total free height is 4.5 metres. The coating is concrete with a thickness ranging from 0.25 and 0.60 m depending on the project with which it was constructed and also on the nature of the rock. The tunnels go through almost all possible terrains dating from Precambrian to Jurassic, and in some cases Quaternary.

Mechanical drillings in crown were done; these went through the lining and penetrated the terrain between 0.5 and 1.0 m. With these drillings 30 m apart, are tried to find the thickness of the coating, obtain samples of it and also samples of the terrain, and at the same time evaluating the holes that were there. In this investigation we included resistivity measurements with dispositives parallel to the axis of the tunnel: one with $AB = 2.5$ m and another with $AB = 9.0$ m, the measuring points being separate 2.5 m and 9.0 m respectively. At each measuring point we made these observations to see if the following existed: wetness, water penetration, cracks of fissures, signs of decomposing in the concrete and any other incidence. In addition to this, in each tunnel a dispositive of $AB = 0.3$ m was applied in order to measure the resistivity of the concrete in dry and wet areas. At the end we made an inspection of the geology which made possible the construction of a geological section simplified by the axis of the tunnel. All this data was taken into consideration in the interpretation, and the results given can be seen in figure 7. After all this had been done, it was possible to make a list of priorities for repairs in all tunnels, these being ordered, taking the following factors into account. (Those factors obtained using electrical resistivity are underlined): index of quality of terrain (coefficients analogous to those used for the geomechanical classifications of rocky masses), percentage of length of tunnel with
decomposing or altered rock, thickness of concrete lining and its dispersion on average in every tunnel, percentage of length of tunnel with altered or low quality concrete, percentage of length of fissures or cracks, percentage of length with holes in crown, (with aid of borings in crown), and length of each tunnel. The most dangerous areas were added to this classification, plus singular areas where provisional reinforcing had taken place.

The electrical resistivity method did not, as is only normal, solve the problems posed in isolation but it did make it possible to extend and to generalize precise observations, and the method also furnished other data not detected by the techniques used.

6. SUMMARY

The geology of the engineer does not stop assisting engineering as soon as the construction is finished. The relationship work-terrain evolves with time. Tunnels are an artificial factors put into rocky massif, and it often is a strange one, this can help to precipitate the decomposition of
of the rock. The consequences are attacks on the concrete coating, and decomposition in the rock; these increase and sometimes exceed the pushes foreseen in the project and so crack and break the coatings.

When these phenomena are evident in a tunnel, it is necessary to study the terrain next to the lining even more so when the construction dates and geological studies have been lost. Geology plays its part in these cases, as one element more in the investigation providing the investigating team with all possible methods of prospecting. Among these methods the electrical resistivity methods offers very favourable facets, this is because of the characteristics this method determines and the speed and economy with the method can be applied.

REFERENCES


ANALYSE DES FACTEURS QUI CONDITIONNENT LES PREVISIONS DES AÉLAS EN TUNNEL

FACTORS AFFECTING THE PREDICTION OF HAZARDS OF TUNNELING

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ABSTRAIT

La question de la prévision des aléas est envisagée de différentes façons selon le point de vue particulier de celui qui aborde le problème, géologue, ingénieur, promoteur ou entrepreneur, et selon l'étape particulière dans la réalisation du projet de tunnel considéré: études préliminaires, appel d'offres, construction, période post-construction. La communication analyse les facteurs importants qui encadrent et conditionnent la façon de réaliser et d'utiliser la prévision. Ces facteurs ne sont pas uniquement d'ordre géologiques, mais les géologues doivent en être conscients lorsqu'ils s'attachent à la prévision des aléas.

ABSTRACT

The prediction of hazards and problems of tunnelling is viewed differently at the various phases of a project by the participants in its accomplishment. The author discusses the most important factors which create the different contexts of the prediction from the standpoint of the engineers, geologists, owners and contractors. Hazards can affect cost through the decision making processes which are dependent on many other non geologic elements. Being involved in the whole interrelationship, the geologist must recognize how they play at the planning, construction, and post-construction phases.

INTRODUCTION

La question de la prévision des aléas dans les projets de tunnels est beaucoup plus complexe en fait, que ce qu'une analyse d'un seul point de vue, celui du géologue par exemple, peut laisser supposer. Dans un contexte géographique, politique et économique donné, on peut même constater que chacun des intervenants dans cette question, possède sa façon propre de voir le problème. La communication vise particulièrement à analyser ce contexte et non pas à commentter les divers phénomènes géologiques qui peuvent produire des aléas, ce qui a déjà été présenté à plusieurs reprises (ex. Bergman, M. S., 1978, Jethwa et al., 1980, Yagge, A. G., 1978, Wahlstrom, E. E., 1973).

Nous situons notre analyse dans le contexte nord-américain en prenant des exemples dans ce qui s'est fait à Montréal; au cours des dix dernières années, 120 km de tunnels nouveaux ont été excavés. Les phénomènes géologiques qui ont eu une importance dans la réalisation de ces travaux, sont commentés par Durand (1978).

La prévision des aléas dans les travaux en tunnel dépend de la connaissance des éléments de détail dans la géotechni-
Figure 1. Schéma organisationnel représentant de façon simplifiée les intervenants et les étapes des projets de tunnels urbains discutés dans le texte.
que de chaque projet individuel; mais elle est grandement conditionnée par le contexte général dans lequel se situent cette collecte et analyse de données. Nous allons analyser ce contexte et le point de vue des principaux intervenants dans trois grandes étapes des projets :
1) période pré-construction, études préliminaires,
2) période de la construction,
3) période post-construction, opération et entretien.

La figure 1 schématisée les liens organisationnels entre les principaux intervenants et les étapes dont on parlera dans ce texte. Dans ce schéma et dans notre analyse, nous situons le géologue dans le bureau d'étude, qui est rattaché au maître d'ouvrage, ou qui agit pour lui comme consultant. Sa tâche est la plus souvent limitée à l'étape exploration, mais il peut aussi assurer un suivi du chantier, quand l'ingénieur résident lui signale des difficultés liées au terrain. Nous n'avons pas rattaché de géologue à l'entrepreneur, car dans les travaux urbains que nous avons étudiés ce cas était l'exception plutôt que la règle.

ÉTAPE PRÉ-CONSTRUCTION D'UN PROJET EN TUNNEL

- Le géologue défend habituellement le principe d'études préliminaires élaborées; il lui faut le maximum de données d'exploration sur chaque site avant de se prononcer de façon détaillée. Sa prudence tient compte de la grande complexité du milieu naturel. Son approche est scientifique et l'analyse qu'il fait, porte sur des paramètres bien localisés dans chaque tronçon de l'ouvrage projeté. Le géologue, ou le géotechnicien, cherchera à localiser, quantifier et évaluer les risques particuliers de chaque phénomène individuel sur le tracé, si on lui fournit, bien entendu, les moyens d'aller chercher toutes les données requises. Les aléas de la catégorie III sur la figure 2 présenteront encore pour le projet des inconvénients, mais ceux-ci seront minimisés car le design pourra en tenir compte avec exactitude.

Le type d'investigation (forages, mesures, essais... etc) dépend du contexte géologique, du type d'ouvrage et de la technique de construction qui sera retenue. Gill et Bally (1976) ont analysé pour les tunnels de la région de Montréal, les types de campagnes d'essai les plus pertinentes. Leur analyse porte surtout sur les projets où l'on envisage l'utilisation de taupes ou machines foreuses à pleine section, projets qui selon eux demandent des investigations géologiques plus poussées que pour les tunnels excavés en méthode conventionnelle.

Quelle que soit la précision des relevés, la prédiction des aléas ne peut jamais être exhaustive et absolue; le géologue ajoute à son rapport des mises en garde quant à ce qui reste indéterminé selon son analyse.

- L'approche du promoteur du projet est différente. À une étape préliminaire, il cherche avant tout à démontrer la faisabilité du projet à l'intérieur d'une enveloppe de fonds alloués. Conscient de l'existence d'aléas de toute nature, et pas seulement d'ordre géotechnique, il en tient compte habituellement de façon globale selon une probabilité statistique de majoration de coût et de délais. Dans l'estimation de ce facteur dû aux aléas de toute nature (conditions mauvaises du terrain, bris d'équipement, problèmes de relation de travail en chantier, retard de fourniture... etc), il doit se fier autant sur ses données administratives relatives à des projets comparables, que sur les relevés géotechniques du projet envisagé. En Amérique du Nord, les projets importants passent invariablement par le système d'appel d'offre. Dans ce contexte, le promoteur cherche à obtenir des soumissions qui cadrent avec l'enveloppe des coûts qu'il a préalablement estimés. Les grands projets sont fractionnés en plusieurs éléments. Chaque appel d'offre portera sur une partie du projet de façon à favoriser la concurrence et le plus grand nombre de constructeurs locaux possible.

Le promoteur n'a pas intérêt à être alarmiste, c'est-à-dire à annoncer aux soumissionnaires le détail de tous les aléas, qu'une analyse souhaitée par le géologue pourrait idéalement produire ; 1° les prix demandés seraient majorés et le projet risquerait d'être bloqué par les bailleurs de fonds; 2° les promoteurs des corps publics souhaitent en général que les grands travaux aient un maximum de retombées économiques locales, c'est-à-dire que les entrepreneurs locaux, disposant parfois de moines d'expertise, puissent aussi soumissionner.

Il est probable que les grandes entreprises de construction disposant de spécialistes appropriés arriveraient à un degré de prédiction plus poussé que celui de la pratique courante, pour un prix de soumission plus réaliste, mais aussi plus élevé.
Le promoteur laisse en fait au soumissionnaire une responsabilité dans l'évaluation des aléas, tout en sachant que si la soumission de ce dernier est basse, les imprévus risqueront un peu plus d'affecter l'exécution du projet.

- L'entreprise qui soumissionne, s'appuie sur les données fournies par le promoteur dans le cahier des charges. À cette étape-ci, les données géologiques se trouvent également présentées avec un ensemble d'autres conditions techniques et financières, relatives à l'ouvrage, aux délais pour la réalisation des éléments du projet, aux conditions particulières de sécurité ou d'environnement. L'entrepreneur qui soumissionne est lui-même lié par d'autres contrats en cours (figure 1) ce qui l'amène à estimer les aléas divers d'une façon globale, pour le projet, mais aussi en fonction de leur interrelation entre les divers travaux où il est impliqué. Si des conditions plus difficiles sur un chantier l'amènent à déplacer une partie de son équipement et de son personnel, c'est l'ensemble de son fonctionnement qui sera touché. Son estimé des aléas et leur conséquence est donc fait dans une perspective qui déborde le projet en cause et qui tient compte de son activité d'ensemble à cette période de temps. Au moment de la soumission, il se satisfait souvent d'une prévision de la catégorie II (figure 2).

L'élément concurrence avec les compétiteurs qui soumissionnent en même temps que lui, l'amène parfois à minimiser dans sa soumission les risques liés au terrain. De par la structure même du système d'appel d'offre, dans un contexte de compétiteurs dans des situations comparables, l'entrepreneur trop pessimiste (ou trop réaliste ?) se voit déclassé par son prix plus élevé. Il se peut que le soumissionnaire retenu soit celui qui, par manque de compétence ou par choix délibéré, a indiqué un prix qui ne tient aucun compte des aléas géologiques. C'est aussi celui qui se trouve le plus embêté par les problèmes qui ne manquent pas de survenir. Les corps publics acceptent malgré tout de payer des rallonges financières, et sont bien forcés d'accepter les délais. La pratique actuelle et les contrats permettent un partage des risques entre l'entrepreneur et le maître d'œuvre, pour les augmentations de coûts qui résultent des mauvaises conditions du terrain (Waggoner, 1981).

En résumé, tous les intervenants souhaitent une collecte de données et une analyse des conditions géologiques pour prédir les risques possibles dans la
construction des tunnels. Mais il y a cependant des objectifs différents dans l'esprit des divers intervenants, dont certains favorisent, au contraire de l'approche du géologue, un estimé global des aléas et une minimisation des risques géologiques, à l'étape où le projet est lancé.

**LES ALEAS RENCONTRES DANS L'ETAGE DE LA CONSTRUCTION DU TUNNEL**

**Les Aléas Majeurs**

Pendant l'étape de la réalisation de l'ouvrage, l'entrepreneur se retrouve devant mille et une difficultés, la plupart sans grande conséquence, mais parfois aussi, devant des difficultés majeures. Crice et Durand (1979) ont analysé les causes des retards survenus dans des tunnels excavés par sautage conventionnel et dans des tunnels percés à la machine fo-reuse pleine section.

Nous avons réexaminé les données des 120 km de tunnels urbains excavés à Montréal durant la période 1971-1981, pour le Métro (45 km) l'épuration des eaux (45 km) et les aqueducs (30 km). Les aléas peuvent être rattachés à des causes géologiques, à des causes techniques reliées à l'équipement, ou à des causes liées à des grèves du personnel.

Pour les tunnels réalisés à Montréal au cours des dix dernières années, les aléas géotechniques arrivent au 1er rang des causes, suivi des problèmes d'équipement et de personnel.

Parmi les imprévus que l'on peut décrire comme géotechniques, les plus fréquents sont les fontis ou éboulements à la voûte près du front de taille. Pour vingt accidents de ce type dans les dix dernières années, trois éboulements ont remonté au travers des dépôts meubles jusqu'en surface, où un cratère d'effondrement s'est formé. Un de ces cas était au milieu d'une artère très passante. Aucun des trois cas n'a occasionné de perte de vie, mais il est bien évident que ce type d'accident survenant dans les tunnels urbains est fort préoccupant, tant pour le chantier, que pour ce qui se trouve au-dessus.

Aucun des vingt effondrements ne peut être considéré comme prévu, en ce sens que l'entrepreneur a été pris de surprise et n'a pu réagir qu'après le fait. Ils se rangeaient dans les catégories I et II de la figure 2. Parfois même, le design du tunnel a dû être modifié par le maître d'œuvre. Cependant l'examen du chantier montre, dans la majorité des vingt cas, que le massif exca-vé dans la zone de l'éboulis présentait divers indices utilisables pour une pré-diction à court terme: augmentation de la densité des diaclasses, début d'une zone de faille, changement non prévu du penda-ge des strates, augmentation du nombre des venues d'eau. Souvent même un éboulement important fut précédé de petits éboulis, un peu avant dans le tunnel. Un suivi minutieux de l'avancement d'un front de taille permet d'augmenter de beaucoup la prévision d'accidents. De fait, l'importance, la localisation précise et la nature exacte des caractéristiques de discontinuités relativement petites, ne peut être évalué qu'en tunnel. Les relevés préliminaires et complets doivent soient-ils laisser toujours une possibilité de faire passer la prédiction de la catégorie II à III, comme l'indique la flèche sur la figure 2.

La cartographie géologique au fur et à mesure de l'avancement du tunnel n'est pas demandée cependant dans les devis. L'entrepreneur demeure responsable de l'évaluation du roc en cours d'excava-tion. Le degré d'analyse des signes avant-coureurs est très variable d'un projet à l'autre.

Le Bureau de Transport Métropolitain, qui est le maître d'œuvre des tunnels et des stations du Métro de Montréal, fait exécuter une cartographie des nouveaux tronçons en cours de réalisation (Chayer, 1977). Pour d'autres projets, seules les sections où un problème se manifeste, font l'objet d'un relevé.

Dans bien des sections, environ le triple du nombre de fontis, l'état du massif inspira à l'entrepreneur une saine prudence et ce dernier, en accord avec le maître d'œuvre, a pris des mesures spéciales: par réduction des volées, mise en place de soutènement lourd, forages d'exploration. Le fait que des accidents surviennent malgré tout, illustre la difficulté pour les constructeurs d'arriver, dans les conditions actuelles, à une analyse déterministe satisfaisante.

**Les Aléas Mineurs**

Bien qu'il soit possible d'interpréter le détail d'un relevé géologique en tunnel pour affiner le calcul des poussées sur le soutènement et le revêtement final (Székely, 1970), ces méthodes demeurent peu utilisées. Le design est fait pour convenir à des conditions assez larges. C'est pour un design fixe, que la
soumission a été retenue et qu'elle lie l'entrepreneur et le maître d'ouvrage; ce fait diminue à leurs yeux l'intérêt d'un relevé systématique. Les aléas dans ce contexte peuvent être définis comme toute condition géologique qui ne convient plus au design.

On peut surdimensionner le design pour diminuer le nombre d'aléas et c'est ce qui se rencontre le plus fréquemment. Cette pratique est la suite logique de celle qui, à l'étape préliminaire, optait pour une évaluation globale et probabiliste des aléas. Par exemple, on peut boulonner systématiquement la voûte des tunnels et non pas seulement là où les conditions du roc les rendent nécessaires. Les aléas mineurs s'en trouvent réduits et, en termes de rentabilité, cette pratique peut être souvent justifiée.

Il faut insister cependant pour dire que si les aléas mineurs sont diminués, les risques plus importants demeurent. Quand l'estimé global indique, par exemple, un besoin de support additionnel de 10% pour un ouvrage donné, cette valeur n'exprime qu'une moyenne. Ajouter 10% de soutènement partout uniformément, ne pourra supprimer qu'une toute petite partie des aléas, que cette valeur de 10% cherche à évaluer statistiquement.

Les Conséquences des Aléas

L'importance pratique des aléas ne se mesure pas par l'importance intrinsèque de chaque cause, mais plutôt par les conséquences de chaque incident. Il n'y a pas de corrélation simple entre l'importance d'une discontinuité, comme une faille ou une diaclasie, et les difficultés qu'elle provoque. Des diaclasies mineures ont causé de graves éboulis; des zones de failles importantes ont été traversées sans trop de difficultés. C'est bien souvent l'état de préparation de l'entrepreneur pour faire face à la difficulté, donc la qualité de la prévision, qui contrôle l'importance des conséquences.

Les conséquences peuvent être regroupées en trois catégories montées à la figure 2. Les augmentations de quantités surviennent fréquemment, mais n'affectent les coûts que de façon modérée; par exemple les sections de tunnel où l'on se voit obligé de mettre en place du soutènement lourd (centres d'acier) n'ont coûté que 10 à 20% de plus que les sections normales.

Si l'aléas entraîne des retards en plus, la perte économique risque d'être beaucoup plus importante. Les contrats exigent l'achèvement des diverses étapes du projet, avant des dates limites sous peine de pénalités financières pour l'entrepreneur. D'autre part les retards à pouvoir utiliser un ouvrage pour lequel des fonds importants ont été empruntés et investis, coûtent très cher à la collectivité en frais d'intérêts et en inconvénients divers. Dans le bilan des majorations de coûts, les retards occupent une place prépondérante.

Les incidents les plus importants, en plus d'occasionner des retards et des augmentations de quantités, peuvent encore être considérés comme non prévus dans le cahier de charges. Ils peuvent être dus à des erreurs de conception, à des mauvaises réactions de la roche ou à des erreurs de mesure.

LES ALEAS QUI SURVIVENT UNE FOIS L'OUVRAGE COMPLET

Pendant la durée de vie du tunnel d'autres aléas peuvent survenir:
- à la suite de nouvelles constructions dans le voisinage immédiat;
- à la suite de la détérioration lente du tunnel ou de son revêtement.

En milieu urbain, le premier cas demeure assez fréquent. La juxtaposition de galeries souterraines rend encore plus complexe l'évaluation très précise du comportement des massifs. Dans ce cas précis, la cartographie géologique des excavations constitue un outil précieux pour la modélisation exacte des interférences entre des excavations nouvelles à construire et les tunnels existants.

Une station de Métro s'est déformée et fissurée de façon importante lors du creusage en tranchée d'une section d'auto-route urbaine (Benussouan et al., 1982). Malgré les précautions prises, le massif peut se déformer de façon imprévue, le long d'un plan de discontinuité, qui se manifeste pas dans le premier projet, ou qui n'est pas observé et cartographié correctement.

A Montréal maintenant, quand une ex-
cavation se situe dans le voisinage immédiat d'un tunnel existant et risque de l'affecter, on installe, soit un soutènement additionnel, soit des instruments de mesure dans l'ouvrage existant, ou encore, les deux simultanément.

Les détériorations qui se manifestent à long terme, ont surtout affecté le béton des revêtements des tunnels de Métro à Montréal. Ces aléas mineurs ont eu pour origine des venues d'eau dans des diaclases ouvertes. Les réparations ont nécessité des travaux d'injection et d'auscultation des voûtes pour détecter la présence de vide entre le roc et le revêtement.

Ces problèmes qui surviennent après la construction, entrent toujours dans la catégorie des aléas imprévus, les difficultés détectées avant et pendant la construction ayant en principe été surmontées lors des travaux.

C'est au hasard du fonctionnement ou de l'inspection de l'ouvrage que l'on détecte ces nouveaux aléas. Ceux qui originent du roc et qui sont masqués derrière un revêtement de béton ou autre, se manifestent de façon sournoise et tardive, quand la partie visible du tunnel subit l'effet de la détérioration.

L'examen ou l'investigation du massif à cette étape est beaucoup plus difficile que pendant la construction car le terrain est masqué par plusieurs centimètres de revêtement. Il arrive souvent que l'on n'arrive pas à se prononcer sur la cause exacte du problème, même après des forages et des essais. Si le fonctionnement de l'ouvrage ne peut être stoppé trop longtemps, la priorité est donnée à la réparation des dommages visibles et à la reprise du trafic.

Parce que les aléas post-construction sont heureusement peu répandus, il n'existe pas comme tel de compagnie d'étude visant à leur prévision, en dehors des procédures normales d'inspection.

CONCLUSION

La prévision des aléas dépend de l'intérêt de celui qui la fait dans le projet, et dépend aussi de l'étape où se situe la prévision par rapport au projet.

Avant la construction, chacun cherche à évaluer l'importance des aléas pour prévoir les difficultés et les coûts qui en découlent. Le géologue favorise une approche déterministe dans l'analyse des données géotechniques, tandis que le maître d'œuvre et le soumissionnaire fondent leurs estimés sur une approche probabiliste plus globale, qui inclut tous les types d'aléas.

Pendant la construction, on peut diminuer le nombre d'incidents en surdimensionnant certains éléments de l'ouvrage, comme par exemple le soutènement temporaire. Par contre les risques géotechniques doivent être localisés dans le temps et l'espace, et pour ce faire, il n'y a que l'approche déterministe du géologue qui peut prévenir les coups. Il n'est pas possible de prédire tous les incidents importants; cependant presque chacun d'eux est précédé de signes, qu'un relevé géologique au front de taille peut détecter.

Après la construction, il n'y a plus d'analyse visant à la prévision des aléas géotechniques. Dans le cadre du programme régulier d'inspection des ouvrages, on pourra cependant détecter des détériorations du revêtement dont la cause présumée se situe dans le massif rocheux.

BIBLIOGRAPHIE


IMPORTANCE OF DETAILED MORPHOTECTONIC AND GEOLOGICAL STUDIES AS A MEANS OF PREDICTING POTENTIAL HAZARDS AND PROBLEMS IN TUNNELLING AND SITE INVESTIGATIONS

IMPORTANCE DES ETUDES DETAILLEES MORPHOTECTONIQUES ET GEOLOGIQUES DANS LA CONSTRUCTION DES TUNNELS ET LE CHOIX DES SITES

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ABSTRACT

The paper lays stress on the importance and need of detailed studies of the geomorphology and geology of the local site areas as well as the regional catchment and surrounding areas of major engineering projects. Special reference is made to the major engineering projects in the Pre-Cambrian terrain of Sri Lanka from 1960-1972 and the current projects under construction since 1980.

The lack of emphasis on detailed geological and morphotectonic investigations of site areas in Sri Lanka is partly the result of the misconceived idea that all regions underlain by Pre-Cambrian crystalline rocks are stable tectonically. Another factor is the lack of appreciation of special geological and structural conditions - differential tilting and neotectonic movements along deep lineaments and shear zones that may exist in the project areas.

In most engineering project areas in tropical countries, on account of the deep surficial cover of highly weathered residual and colluvial materials - soils, landslide debris, boulder trains and in some places, flat dips of unweathered rock layers, the airphoto interpretation of the geology and structure and even field reconnaissance by the geologists are not very effective. Hence, in most engineering projects in Sri Lanka, the preliminary information regarding the geology and tectonics of the site areas has been inadequate. Unexpected bad rock conditions have been encountered during tunnelling, dam and power-house site excavations resulting in long delays and heavy losses to government authorities and the contractors.
At various stages during construction of these engineering projects, the author has undertaken several detailed geological and structural mapping on 1:120 and 1:10,000 scales as "rescue operations" to locate the danger zones and predict the potential hazards during tunnelling and excavations. The paper will highlight the field techniques used in such surveys and the usefulness of these detailed geological studies for construction programmes.

ABSTRAIT

Ce travail insiste sur l'importance de faire les études détaillées morphotectoniques ainsi que géologiques concernant l'emplacement potentiel du tunnel, ses environnements et son bassin d'alimentation. Dans ce travail l'auteur se réfère largement aux projects entrepris entre 1960 et 1972 et aussi aux projects actuels.

La manque de ce genre d'études s'explique d'une part par la notion malconçue de la stabilité tectonique de tout terrain précambrien ou se trouvent les roches cristallines et d'autre part par la connaissance insuffisante des conditions locales.

Dans un pays typiquement tropical comme Sri Lanka, les processus d'altération sont assez repandus et ceux-ci donnent lieu souvent aux surfaces bien épaisses et riches en matériel très décomposé résiduel et colluvial. De telles surfaces prennent des formes diverses morphologiques. C'est ainsi que les études des photographies aériennes peuvent parfois susciter de fausses interprétations. Donc une interprétation erronée peut provoquer l'apparition inattendue des conditions géologiques pendant la construction d'un tunnel. A Sri Lanka, l'insuffisance de l'information sur la géologie et la tectonique dans les sites géotechniques a parfois donné lieu aux accidents entraînant bien des dégats et des retards à l'état ainsi qu'aux entrepreneurs.

L'auteur, en tant que géologue-conseil, a été appelé "au secours"a plusieurs reprises au cours des constructions des tunnels de Sri Lanka. Dans ce travail, l'auteur fait une synthèse sur les méthodes employées à résoudre les problèmes qui se produiraient pendant la construction des tunnels.

INTRODUCTION

In Sri Lanka, civil engineering structures (reservoirs and dams) mainly for irrigation were built by the ancient kings. However, as in the case of most recent hydro and irrigation projects, e.g. Castlereagh Tunnel (1963), Polpitiya Power House (M.O.P.I, 1968), Canyon Project Tunnel (1981) and Kotmale (1980), there is geological field
evidence to show that the lack of preliminary appraisal of the morpho-tectonics and geology of the critical areas of the projects had given rise to serious problems during and after construction throughout the long history of civil engineering works in the 'Island.'

The geological mapping of the ancient Sigiriya reservoir (500 AD) area shows the ruins of about 8 km long bund located along a thick marble band. Very likely, on account of the heavy seepage through the exposed karstic marble, the King Kassyappa's reservoir was never filled up with the exception of the small section of the reservoir underlain by the biotitegneisses. Another example is the breaching and failure of the long dam (10 km) of the "Sea of Parakrama Bahu", 1100 AD, which occurred probably along the permeable quartzite band running oblique to the tank bund (Vitanage 1959).

Without the knowledge of modern sophisticated techniques of field mapping and subsurface exploration, it is not surprising that ancient engineers encountered unexpected conditions. But it is rather surprising that in most of the present day large engineering projects in Sri Lanka, unexpected bad rock conditions have been encountered during tunnelling, dam and power house site excavations resulting in long delays and heavy losses. According to the writer's opinion the main reason is the lack of proper preliminary appraisal of the morphology, geology and structure of the project areas during both feasibility and design stages.

At various stages during construction of these engineering projects in Sri Lanka, the writer has been called upon to undertake several detailed surveys. The comments and the results of these field surveys will be discussed in the following sections along with some details regarding the mapping techniques successfully employed for large scale mapping of the geology and morphology of the highly weathered and eroded hard-rock terrains under tropical climatic conditions.

FIELD MAPPING TECHNIQUES EMPLOYED

With the exception of Maduru Oya Project in north-east of Sri Lanka, almost all of the major hydro-irrigation projects are in areas subject to deep differential weathering and erosion (Fig.1). In these areas, preliminary reconnaissance geological mapping is often of limited value for large civil engineering projects. Even detailed field geological mapping of these areas is extremely difficult on account of several factors - thick vegetation, over-burden of residual weathered rock and soil cover, colluvial deposits (boulder trains and landslides), inaccessibility and lack of accurate topographic maps for the whole project area. However, good air photo cover for the
whole Island and more recent (1979) large-scale air-photos of some project site areas in the Mahaweli basin provide good base maps for field mapping.

For detailed surface regional and local site mapping of the geology and structure of the project areas on scales of 1:10,000 and larger scales, the writer has successfully employed additional field methods - mapping of break-in-slopes and minor morphological features measuring of slope angles, colour and texture of soils, identification and plotting of distinctive heavy minerals in soils and weathered rock (by panning) and location of springs and flowing wells. These physical parameters help to trace the contacts of rock units, especially the marker beds - quartzites, marble, white granulites and charnockite bands in areas, where fresh exposures are scarce (Vitanage, 1981).

Hydro/Irrigation Projects in Sri Lanka and Geological Field Investigations

In the Uplands and Highlands of Sri Lanka, the morphology of the landforms - very accurately reflect the underlying geology and structure (Vitanage, 1970 a). Unfortunately, the importance of morphology and morphotectonics has not been appreciated in engineering geological site investigations. The costly unexpected conditions encountered during tunnelling and large excavations e.g. at the power house site at Polpitiya (Fig. 4) extensive caving in along weathered rock zones along Canyon Tunnel and subsequent new alignment of tunnel (Fig 6 d), and heavy seepage with mud and debris slides along Polgolla tunnel (Fig. 6b) could have been anticipated or probably avoided, if careful attention was given to the diagnostic morphological and morphotectonics features during the preliminary field mapping of the above site area.

As regards the other projects - Castlereagh (1964), Kotmale (1980) and Victoria (1980), the field data collected during the feasibility stage were incomplete and inadequate. According to the Victoria Scheme Feasibility Report (1978) "there are several significant deficiencies in the information obtained to date which prevent final decisions being reached on the most suitable location of the dam" and "on the selection of engineering design parameters for detailed design of the component parts of the engineering scheme". The comments on the feasibility reports (1966-1978) on the Kotmale Project site by the Panel of Experts (Deer 1980) appointed to report on the water-tightness of the reservoir and the site are as follows: "knowledge on geologic structure is deplorably incomplete. There are 3 geologic maps, covering only the reservoir elaborated on reconnaissance level. The maps contradict each other in important items and in some instances
they are not compatible with drill hole data. They are not considered an acceptable basis for final decisions.

CASTLEREAGH PROJECT

As shown in Fig. 1 and Fig. 2, Castlereagh scheme is in the Central Highlands of Sri Lanka. The geology and the geotechnical data of the tunnel section 0-730 m (0+00 to 23+00 ft.) are shown in Fig. 3.

During the excavation of the tunnel, highly decomposed and weathered kaolinized zone 160 m width was encountered in 1963 between station 270 m to 400 m very close to the centreline of the Castlereagh dam in the hillside (Fig. 3). "Huge amounts of water started flowing into the tunnel and as the excavation was carried on, new springs were frequently discovered." The contractor "feared that tunnel might collapse or even worse suddenly would drain Castlereagh Lake through a fault zone or a band of decomposed rock extending from the tunnel to the Lake."

At the request of the contractor in 1963, the writer carried out a detailed geological and geotechnical survey of the surface and the subsurface along Adit 0 and the already excavated sections of the tunnel. The results of this survey is shown in Fig. 3. On the basis of the field data from the geological mapping and the borehole data along the study area, weathered zones and places of heavy seepage from the reservoir were located to help further tunnelling. But as expected, the tunnelling operations "came to a stand still" at station 80 m (250 ft.) on account of bad rock and heavy seepage from the reservoir. The completion of the tunnel was delayed by one year.

- The results of this investigation at the Castlereagh and subsequent excavations along the tunnel showed that it is possible to locate seepage and weathered zones very accurately by detailed mapping of even narrow (2 metres thick) marker beds-quartzites, marble (Fig. 3) in spite of the variable strike and dips of the surface rock formations (strikes N40W and N55W and dips 30-70W) making use of average regional strike and dip along with the amplitude and wave-length of the minor folds, it was possible to predict within ± 2 metres, the weathered and seepage zones at stations 160 m 125 m, 100 m, 85 m, and 50 m (Vitanage, 1963). The persistence of the narrow quartzite beds from adit 0 to the tunnel bend near intake confirmed the absence of appreciable fault offsets as proved by surface mapping above this area. Probably, the weak zones occur along the shear zones parallel to the strike of rock units in the area.

Detailed field mapping of the tunnel route and a few more boreholes across the tunnel trace near the intake could have located these series of narrow kaolinized shear zones with seepage associated with the quartzites, calc-gneisses and
marble bands. Very likely the occurrence of sound charnockite rock along the right abutment of the dam near the intake portal and the absence of extremely adverse conditions during tunnelling up to adit O upstream had given the false impression that no tunnel hazards would be encountered along the last 400 m. The occurrence of a perched aquifer indicated in a bore-hole above the tunnel was also not taken into serious consideration. As stated earlier, the results of this lapse was one year's delay of the completion of the project with heavy losses to the contractor and the owner.

MASKELIYA OYA, M.O.P. I
Polpitiya Power-house site,M.O.P.I

The results of the detailed mapping of the powersite and the tail race area on a scale of 1:480 is shown in Fig. 4. The site is located on the right bank of the Maskeliya Oya at the confluence of a small tributary, Bata-ela. As the fresh rock exposed after excavation was very limited (10%), the boundaries of the weathered rock layer were traced by noting the colour and texture of the partly weathered rock layers, especially the variation in colour of minerals in the pegmatites, conformable veins and layers. In areas where excavation was done in bedrock, mapping was done after each blasting. As shown in cross-section Fig. 4 (II) by this round-wise geological mapping it was possible to trace the differentially weathered bedrock profile and the rock layers from surface to the floor of the power-house pit (Vitanage 1968a and 1968b). The detailed mapping of the power-house site area indicated that almost 90% of the site is covered by alluvial and colluvial deposits - peat, clay, river terrace gravel, boulders and slide deposits. Bed rock consisting of unweathered Precambrian gneisses, banded charnockites and quartzites between the eastern bank of the power house excavation and anchor block C showed a series of sheared and fractured zones (Fig. 4). Along these weak zones there was heavy seepage and downslope movement into the excavated area. These conformable shear zones parallel or oblique to the rock layers are the most treacherous during tunnelling and in large excavations in Sri Lanka. As discussed earlier the tunnelling problems along Castlereagh tunnel were all due to the presence of similar shear zones.

No laterites was found in the power-house pit site area, with the exception of a small narrow patch of laterite, although the preliminary investigations have reported "that in the vicinity of the basin near creek laterite overburden ranges from 75 to 100 feet in depth, between B11 and B4" (Campbell, 1963).

On the contrary, the site selected for the power-house and the tailrace was a marsh along an earlier river channel 20 metres above the present Maskeliya Oya.
The river terraces 10 - 20 m above the present channel levels and buried channels are common features in the river morphology in Sri Lanka. Unfortunately, this important morphological feature has not been observed before the selection of the site. Besides, possible occurrence of the extensive lateritic deposits referred to in the feasibility report was probably based on scanty bore-hole evidence in drill holes B11 and B4, "where laterite cover ranges between 10 & 30 ft. over 50 percent of which comprised of blocks of fresh bedrock referred to as the unlateritized charnockite river boulders. These boulders and rounded gravels of the earlier river channel of the Maskeliya Oya were well exposed near the batching plant (Fig. 4-1).

The statement given in the Geology Report - "the excavation for the power-house will be mostly in laterite to depths of 75-100 ft. The slopes of such an excavation will stand steeply and securely with berms at 30 ft. lifts" (Campbell, 1963)- was completely wrong. The contractor asked for Rs.8,000,000/= as damages as a result of changing conditions, the Ceylon Electricity Board agreed to pay 3,000,000/=. It should be pointed out, in addition to the compensation paid for changing conditions, the construction was delayed for months for which the Government had to produce thermal power for electricity with imported oil.

CANYON POWER PROJECT

The Canyon Power Project currently under construction is the third stage of the hydro-power development in the Maskeliya Valley (Fig.2)

During the excavation of the tunnel, extremely adverse tunneling conditions were encountered at stations 2200 m and 4320 m and nearly 130 m of tunnel had to be abandoned (Fig. 6d). Here again, it is surprising that no reconnaissance mapping of the geology and the morphology of the surface had been carried out along the tunnel route. Apparently, careful analysis of the reports and maps already available in the earlier investigations of the two projects downstream - M.O.P. I (Vitanage 1970) and M.O.P. II(Godfrey, 1964, Cabrera, 1970, Vitanage, 1970b) - had not been made. No drilling had been recommended along and across the tunnel route to ascertain the subsurface conditions of the tunnel trace area.

The inspection of the Canyon tunnel and reconnaissance study of the geology and morphology of the area indicates that the selected old tunnel route was along the foothills of a north-west-south-east trending scarp ridge and along a line of swallow-holes and boulder-filled depressions. All these morphological features generally reflect obvious bad tunneling conditions which unfortunately have not been taken into account in recommending the tunnel route.
KOTMALE PROJECT

Under the accelerated Mahaweli Development Programme, Kotmale Project consisting of a reservoir dam, tunnel and power house is under construction since 1978 (Fig. 1). Project site is a difficult and complicated one on account of a variety of adverse geological features - (i) unstable soil and rock masses in the reservoir area, (ii) cavernous limestone in the reservoir and below the proposed dam sites (iii) deep irregular weathering of rocks with soft shear zones at the dam site along with regional and local lineaments parallel and oblique to the dam and the reservoir rim (Fig. 6a).

During the long feasibility stage (1968-1978), a series of investigations by several foreign and local organisations had been carried out. Several reports and reviews (eight) on the geological and geotechnical conditions of the Kotmale dam site and reservoir were made available (Podhalitz, 1980). However, according to the reviewers and the Panel of Experts (1980) the available knowledge on the "geology of the project area is as yet incomplete and is ambiguous and does not permit to enunciate a final judgement..." As regards subsurface exploration, Olson (1978) writes "Kotmale dam site and reservoir basin have been explored extensively with core drilling and tunnelling but poor recording and data processing have meant that the information available is less than would be expected for a project of this nature for which feasibility studies have continued for many years".

The basic deficiency in the preliminary investigations during the feasibility stage was the lack of detailed geological and structural map of the whole Kotmale Project area to be used as a reliable basis for subsurface exploration and also to enunciate a final judgement during the design stage on potential practical problems especially the watertightness of the reservoir and the nature (active or not) of the regional lineaments. Referring to the limitations of the available information, the Panel of Experts (Deer, 1980) very rightly commented that "knowledge on geologic structure is deplorably incomplete. There are 3 geologic maps, covering only the reservoir, elaborated on reconnaissance level. The maps contradict each other in important items and in some instances they are not compatible with drill-hole data. They are not considered an acceptable basis for final decisions". They recommended that a detailed geological map of the reservoir and potential seepage outlets to be prepared as the first and most important step.

To rectify the deficiencies referred to above. The writer was requested by the joint consultants of the Kotmale Project to undertake a detailed regional study of the whole site area on a scale of 1 :
10,000. The detailed geological field mapping was very difficult on account of extremely poor exposures of the key beds in the critical areas, extensive surficial deposits-landslides, boulder trains and mud flows - and thick vegetation cover. In critical areas, especially along the reservoir bed and upstream of the dam axis, the marker beds - the quartzite and marble beds (crystalline Limestones) - were completely covered by mud-flows, slides deposits and alluvium. Additional problem was the lack of an accurate base topographic map on 1 : 10,000 scale to cover the area mapped.

Normal exposures (outcrop) mapping was not possible in most of the area mapped. Therefore, different field techniques were employed in locating and tracing of the field boundaries of the key beds especially the main limestone bed (Mb0). With the help of minor morphological features - swallow holes, collapsed depressions, hydrogeological features - line of springs and flowing wells, the presence of residual marble boulders, and place names, e.g. Hedunawewa = collapsed pond; Hunugaloya = stream along limestone bed, it was possible to trace the approximate contacts of the main limestone bed (Fig. 6a). The charnockite beds (Ch0) immediately below the main marble band (Mb0) was traced along the reservoir bed and rim and into the upstream and downstream area over 15 - 20 kms (Vitange, 1981). The contacts of marker quartzite beds were mapped carefully where the contacts are exposed. The offsets of the regional lineaments so well shown in the air -photos were determined with the help of the marker beds - quartzites and charnockites. Probable seepage zones and monitoring points for checking the direction and amount of possible seepage were located (Fig. 6a).

It must be pointed out if this detailed geological survey was carried out earlier, with a careful follow up of subsurface drilling, good deal of money and time could have been saved during the designing stages. In addition, shifting of the dam site in 1980 and the unnecessary expensive core drillings (over 100) around the 1979 dam site could have been avoided.

**VICTORIA PROJECT**

Unlike in the Kotmale Project for the Victoria Project which is under construction since 1979, considerable preliminary information though deficient in some respects were available during the feasibility and designing stages. In November 1981, during the excavation of 5 km tunnel (7.2 m diameter) extremely adverse tunnelling conditions - heavy seepage, caving of roof and bad rock (shear zones) - were encountered along the downstream section of tunnel (Station 5100 m). On account of continuous bad ground, the 100 m of old 7.2 m diameter tunnel
and partly constructed surge shaft were abandoned and the tunnel route relocated on a new trace (Fig. 6a).

Surface mapping of the area above section of the new tunnel route seems to indicate that the rock formation designated as "gneiss and quartzites" by the earlier survey is responsible for these adverse subsurface conditions. The preliminary indications are that the weak rock conditions may repeat again along the new tunnel route, if these quartzites and the interlayered gneiss are encountered during tunnelling. Besides, the section between the downstream portal and the surge (old and new) shaft is very close to the east-west Mahaweli Minipe 20 km long megalineament.

It is interesting to note that the longer dog-leg route of the tunnel was recommended mainly to reduce the length of the tunnel to be driven through the marble and also to decrease water head at tunnel level (Victoria Project Report, 1980). The presence of artesian springs and travertine was another consideration. However, the significance of the occurrence of a series of narrow quartzite (4 quartzite beds) interlayered with thinly bedded biotite-gneiss had not been considered. In addition as in other projects referred to earlier, no regional study of the geology and tectonics around the project site area had been made.

As a result, the indirect or direct influence of the major east-west Mahaweli-Minipe lineament, along which the Mahaweli river runs for over 20kms in a deep narrow valley and has not been considered. Apparent regional structures and tectonic features of the east-west culmination and cross folds and the associated differential stress pattern especially the potential neotectonic movements also had not been taken into account (Vitanage, 1972)

The preliminary field evidence suggests potential adverse tunnelling conditions along the new tunnel route. Intensely fractured rock layers with series of minor overfolds and buckling of the earlier lineated rock layers indicate probable shear zones parallel and oblique to the rock foliation, which reflect adverse subsurface tunnelling conditions. On account of the steep valley slopes, the marble beds which will be encountered by the tunnel, will not give rise to such bad rock as expected earlier (Victoria, 1978). The tunnel excavation during the next few months will confirm these observations.

POLGOLLA PROJECT AND M.O.P. II PROJECT.

The two most outstanding examples of hydro projects in Sri Lanka where systematic comprehensive geological surveys have been carried out during the preliminary stages are the Polgolla Hydro-Irrigation Project in middle Mahaweli basin and M.O.P. II Hydro Project in the
Maskeliya Oya basin (Fig. I, 2, 5 and 6b). During the construction of both these projects, there was only minimum construction problems as a result of unexpected conditions.

At Polgolla, surface geology and structure along the tunnel route was mapped by the writer and the probable geological and geotechnical problems that might arise during tunnelling were indicated on a longitudinal geological and structural profile to the contractor before the bidding for the contract was finalised. As shown in Fig. 6b mainly on morphological evidence, probable location of two deep shear or fault zones were shown in the longitudinal profile. Similar map showing the weak tectonic zones was provided to the consultants by the geologist of the Ceylon Government Irrigation Department.

Unfortunately, both the contractor as well as the consultants — did not pay adequate attention to the likelihood of encountering these bad rock zones. Probable reasons for not carrying out further investigations in spite of a prior warning of a potential hazard may have been that the bad rock cover along this stretch of the tunnel was over 330 m, which is the highest rock cover for any tunnel in Sri Lanka. As a result on November 5th, 1971, during "blasting of the tunnel, water appeared in the upper section of the right hand wall" at station 7800 m (259+08 ft.). This was the beginning of a series of so-called "eruptions", which brought nearly 1606 m³ (2100 cu yds) of sand, silt and rock into the tunnel within a period of 15 days. Water flooded the tunnel to a depth of \( \frac{1}{2} \) meter. Progress of tunnelling along the downstream face delayed by about three months. No other serious unexpected problems were encountered during further tunnelling.

A good example of geological advisory service of the feasibility and design stage is the comprehensive information given in the three reports of the M.O.P. I Maskeliya Project (Godfrey, 1964; Cabrera 1970 and Vitanage 1970). As shown in Fig. 5, a detail study of the area along the 5.6 kms long tunnel route was carried out by three geologists at various stages. Route No. 1 with three adits with full "umbrella" rockbed cover was recommended by Godfrey (1964). Because of the excessive length of the tunnel and the adits, this tunnel route was not accepted by the Ceylon Electricity Board. In 1970, two other geology consultants were asked to consider the feasibility of shifting the tunnel along a shorter route. New tunnel route No. 3 with only two adits (Vitanage, 1970) was accepted by the Electricity Board.

On account of the exhaustive study of the geology along and around the tunnel route, no unexpected tunnelling problems were encountered and the project was completed according to schedule.
SUMMARY AND CONCLUSIONS

The case histories of major engineering projects in Sri Lanka discussed in the previous sections show that the common fault in a majority of engineering undertakings is the neglect of geologic factors or insufficient study of them.

With the exception of M.O.P. II and Polgolla Projects almost in all the other projects heavy losses and delays involving millions of rupees resulted from a lack of proper detailed knowledge of the geology and morphology of the site areas.

The main defects in the geological investigations undertaken in Sri Lanka are:

(i) Lack of detailed large-scale regional mapping of the morphology and geology of the Project Area.

(ii) Lack of knowledge of most up-to-date information on the local and regional geology and structure.

(iii) No critical appraisal of earlier case histories.

(iv) Limited time devoted to field mapping, and reliance mainly on airphotograph interpretation to work out the geology and structure of the site area.

(v) Poor mapping techniques—prominent marker beds not made use of to work out the rock succession and structure.

As pointed out by Deere(1969) Geological materials are so variable in short distances that "it is nearly impossible to uncover all the important variations by present day exploration techniques". However, as pointed out decades ago by Berkey (1950) it is the writer's belief that a detailed regional geological survey preferably on a large scale e.g. 1: 10,000 of the whole site area with a follow up of well directed subsurface exploration and proper geophysical surveys could eliminate a large part of the inherent risks involved.

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FIGURE 2:
Main Projects in the Upper Kolani Valley Basin-Castlereagh Project (1964) along Kehelgamu Oya; Polpitiya Project (M.H.P.I), 1969; Mankelliya Oya Project (M.O.P. II, 1974); Canyon Project 1981.
FIGURE 5:
Geology of the area around M.O.P.II Project.
FIGURE 5: (a) Geology and structure of the Kothale Reservoir and Dam Site; (b) Geological Profile of Polgolla Tunnel from Station 7500m to downstream portal showing the fault seepage zone, (c) Sheared zone along Victoria Tunnel, (d) Tunnel route of Canyon Tunnel.
EVALUATION OF RADIUS OF BROKEN ZONE AROUND A TUNNEL IN SQUEEZING ROCK CONDITIONS

ESTIMATION DU RAYON DE LA ZONE DE RUPTURE AUTOUR DE L'OUVERTURE D'UN TUNNEL DANS DES CONDITIONS DE TASEMEMENT

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ABSTRACT
The determination of radius of the broken zone around a tunnel opening under squeezing rock conditions is a very important issue because the rock pressure is greatly influenced by the same. In the present paper a graphical procedure has been proposed for the estimation of radius of the broken zone. The procedure is simple and gives an average value of the radius of the broken zone. Its efficacy has been demonstrated by analysis of the field data obtained from instrumented sections from a tunnel constructed in the Himalayan region.

ABSTRAIT
La détermination du rayon de la zone de rupture autour de l'ouverture d'un tunnel dans des conditions de défaillance de roches, est une question très importante en de son influence sur la pression exercée par les roches. La présente étude propose une méthode graphique pour l'estimation du rayon de la zone de rupture. La méthode est simple et fournit une valeur moyenne du rayon de cette zone. On a démontré l'efficacité de la méthode en se basant sur l'analyse des données obtenues sur le terrain dans des zones équipées d'instruments, lors de la construction d'un tunnel dans la région de l'Himalaya.

Introduction
The virgin rock masses are usually in a state of static equilibrium attained after a long cycle of events in the geologic past. The process of excavation within the rock mass disturbs the state of the equilibrium and a new set of forces are brought into existence. The mechanism of manifestation of these forces and their influence on the behaviour of tunnelling media is a very important aspect. These forces affect the rock pressure building process and are instrumental in bringing about a physical change in the rock mass behaviour under certain conditions.

The forces or stresses induced due to tunnelling, sometime exceed the in-situ strength of the rock mass and consequently the surrounding rock mass fails. There is thus a physical change and the behaviour of such a failed rock is very different to that of the intact rock.

The stress distribution is changed drastically and for the analysis of stresses, elastoplastic material model had been advocated by many researchers. The physical behaviour of the broken rock mass is responsible for 'squeezing pressure' or genuine mountain pressure. It had been observed that the support pressure reduced with the increased tunnel-wall deformation.
With the help of the elasto-plastic stress analysis (Dube, 1979) it is possible to estimate the quantum of the rock pressure under a given situation. For the mathematical simplicity it has been assumed that the rock failure extends only up to a certain radial distance within the rock mass. This radial distance is called the radius of broken zone and is a very important parameter influencing the rock pressure.

**Brief Review of Literature**

An approximate stress analysis of the broken zone was first given by Terzaghi (1925) but he did not propose any method to predict the displacement within the broken zone. This discrepancy was overcome by Labasse (1949) who introduced the concept of volumetric expansion of the broken zone. A comprehensive stress and strain analysis was done by Daemen (1975) who also took into account the elastic displacement at the interface of the broken and elastic rock mass.

The failed rock mass within the broken zone had been assumed to expand volumetrically at a constant rate throughout the extent of the broken zone. The expanding rock is kinematically free to move only towards the opening effecting the displacement of the tunnel periphery towards its centre. Figure 1 illustrates this situation. The radial displacement at the tunnel periphery (tunnel wall) is \( u_r \) and at the boundary of broken zone \( u_b \). Following the labassian concept of constant volumetric expansion of broken zone and accounting for \( u_r \), the overall coefficient of volumetric expansion \( k \) may be calculated from the following equation:

\[
k = \frac{2(a_u - b_u)}{b^2 - a^2} \quad (1)
\]

To obtain the local variations of \( k \) within the broken zone, Eq. 1 may be generalised as:

\[
k_{1,2} = \frac{2(r_1 u_{r1} - r_2 u_{r2})}{(r_2^2 - r_1^2)} \quad (2)
\]

where

- \( r_1 \) = smaller radius of annular zone
- \( r_2 \) = larger radius of annular zone
- \( u_{r1} \) = radial displacement at \( r_1 \)
- \( u_{r2} \) = radial displacement at \( r_2 \)
- \( k_{1,2} \) = coefficient of volumetric expansion

If \( k \) is known then it is possible to predict the displacements at various radial distances within the broken zone.

Daemen (1975) proposed the following expression for displacements at the outer boundary of the broken zone in the case of hydrostatic primitive stress field:

\[
u_b = \left(1+\nu\right) \left(\frac{P \sin \varphi_p + c_p \cos \varphi_p}{E}\right) \quad (3)
\]

where

- \( \nu \) = Poisson's ratio of rock within the elastic zone
- \( E \) = Young's modulus of rock within elastic zone.
- \( P \) = hydrostatic stress field
- \( \varphi_p \) = peak angle of internal friction of rock mass applicable within elastic zone.
$c_p =$ peak cohesion of rock mass applicable within the elastic zone.

Daemen's assumption of hydrostatic stress field is too idealistic to be relevant to field conditions. Dube (1979) therefore modified the equation (3) for the non-hydrostatic primitive stress field. The following expressions have been derived for vertical ($u_{bv}$) and horizontal ($u_{bh}$) displacements at the boundary of the broken zone:

$$u_{bv} = b \left\{ (0.5P(1-\lambda)(5+\nu) + (1+\nu) \\
0.5P(3-\lambda) \sin \varphi_p c_p \cos \varphi_p \right\} . \quad (4)$$

$$u_{bh} = b \left\{ (-0.5P(1-\lambda)(5+\nu) + (1+\nu)) \\
0.5P(3-\lambda) \sin \varphi_p c_p \cos \varphi_p \right\} . \quad (5)$$

Development of Graphical Method Theory

The radius of the broken zone enlarges with the advance of tunnel face. For each stage of development of the broken zone, radial displacements may be calculated from Eq (2) in which factor $k$ is a function of time. All the displacement-distance curves must converge to a point, say $x$, where the fully developed broken zone meets the elastic zone as shown in Fig 2. The ordinate of this point $x$ represents the value of elastic displacements $u_{bv}$ or $u_{bh}$ whereas the abscissa gives the radius ($b_{v}$ or $b_{h}$) of the broken zone. However, these displacement-radial distance curves will not converge before the full development of the broken zone, instead they terminate on the line $ox$ at the points say, $A$, $B$, and $C$. The line $ox$ represents the displacements at the interface of elastic and broken zone and it should be a straight line passing through origin according to Eq (4) and (5). The projections of these points on the abscissa give the radius of the broken zone at the different stages of its development with the face advance or time. The corresponding elastic displacements at these stages are given by the ordinate of $A$, $B$, and $C$ for different radii of broken zone.

The theoretical radial displacement vs radial distance curves are ideal and in practice it may not be possible to measure the displacement instantaneously after excavation of an opening under such a situation, as some initial displacements go unrecorded due to inevitable delay in the installation of instruments for measuring displacements. If the installation of instruments is delayed by say, $T$ days, then it is necessary to deduct the displacement which may have occurred during this period from the displacement at other points. In this way, Fig 2 may be modified to obtain Fig 3 which accounts for delay in installation of instruments. It may be noted that these curves also converged at the same point $x$ as in Fig 2, thus giving the radius of the broken zone correctly. Hence, the extensometers installed later may also be useful in evaluating the extent of the broken zone.

**Fig. 2. Typical Variation of Rock Displacements Within Broken Zone with Time**

**Fig. 3. Theoretical Variation of Rock Displacements Within Broken Zone If Extensometers Are Installed Later**

However, it could not be possible to obtain either the total displacements or value of the coefficient of volumetric expansion ($k$) correctly.

Proposed Graphical Procedure

The radius of broken zone may be obtained if the displacements at various radial distances have been observed. From the bore hole extensometer data...
and the closure observations it is easy to obtain radial displacements at various radial distances within the surrounding rock mass. To determine the radius of the broken zone, radial displacements vs radial distance curves are plotted for the various periods of observations. These plots should be analogous to the theoretical curves of Fig 2 and hence it is possible to assess the development of the broken zone in stages. Further, the displacements at the boundary of the broken zone may also be estimated. Further more, the variation of the broken zone with time may also be plotted against the corresponding face advance or time. Such a plot should resemble the theoretical slope in Fig 4, depending upon the magnitude of primitive stresses.

![Diagram of Stress Levels](image)

**FIG. 4. DEVELOPMENT OF BROKEN ZONE WITH ADVANCE OF TUNNEL FACE (AFTER DAEMEN AND FAIRHURST, 1972)**

To define the shape of broken zone as truly as possible, it is necessary to have the displacement observations within the rock mass. Such observations are possible with multipoint extensometers having 8 to 10 points.

In the cases of non-homogenous rocks the above procedure will be valid only for obtaining the average radius of broken zone on both sides of the tunnel-walls, because normally no information is available on independent tunnel-wall displacements on either side. One, therefore, assumes that the tunnel-wall displacement is the same on the sides of the tunnel.

The advantages of this graphical method are that the errors in borehole extensometer data are easily identified when:

(i) the borehole extension is more than the tunnel-wall displacement and

(ii) the point of convergence lies some what below the abscissa (in case of non-hydrostatic primitive stresses the point of convergence may lie somewhat below the abscissa indicating negative displacements at the boundary of broken zone.

Thus, it may be noted that the proposed graphical procedure is quite general in application.

The validity of the graphical procedure developed in the foregoing paragraphs had been verified in the field. Extensive instrumentation as detailed in the following paragraphs had been done in a tunnel during its construction in Himachal Pradesh.

**Field Instrumentation**

Instruments were installed to measure the displacements. Since the field studies were conducted in a prototype tunnel during construction it was felt necessary to install simple but sturdy instruments. Such instruments were mostly mechanical type and were fabricated at the project workshop. The following instruments were employed in the field study:

1. Closure bolts - These were used as plugs for measurement of both the tunnel-wall and the support displacements.

2. Closure meter - To measure the (diametrical) distance between two opposite closure bolts.

3. Single point bore hole extensometers - To measure the displacements at a desired point within the rock mass with respect to the tunnel-wall.

All the above instruments were protected so that these were not damaged by the flying pieces of rocks after blasting at the tunnel face. Fig 5 shows the scheme of installation of these instruments.

**Analysis of Field Data**

The following assumptions were made while analysing the field data:

1. Horse-shoe shaped opening was considered to be circular.

2. The rock mass around the tunnel opening was in a state of failure.

3. The radial displacement within the
The broken zone is given by:

\[ u_r = u_a - e_{av} \]

where,
- \( u_r \) = radial displacement at radius \( r \),
- \( u_a \) = tunnel-wall displacement
- \( e_{av} \) = bore-hole extension, considered positive when the extensometer tube moves towards the opening.

The displacements attain their final value within a face advance as equal to tunnel diameter. The tunnel-wall displacement data was examined and it was found out that the displacements were definitely higher than 1 percent of the tunnel diameter and they stabilised after the lapse of considerable time. This confirmed that the rock mass was in a state of failure around the tunnel. The displacements under failing rock conditions exhibit the following tendencies:

(i) The tunnel-wall displacements are significant and stabilise after a long time of 100-300 days.

(ii) The rock pressures may reduce with increasing tunnel-wall displacements.

(iii) The bottom may heave and the rock at the tunnel face may be under distress.

It was thus concluded that the rock mass was in a state of failure where the experiments were conducted.

The observed data was reduced to radial displacements at the tunnel-wall periphery and within the rock mass around the tunnel cavity. According to the graphical procedure developed earlier these displacements for a given period of time are plotted against corresponding radial distances and the points are joined by smooth curve. A number of such curves are shown in Fig 6 for the right tunnel-wall and in Fig 7 for the left tunnel-wall. The numbers against each curve indicate the corresponding period in days for which the observations were taken.

It may be seen in Fig 6 that all the curves for the right tunnel-wall appears to converge after a period of 96 days. However, such a trend is not evident for left-wall (Fig 7) presumably because only one extensometer was functioning at this location. The extensometer at radius of 9.62m could not be installed and the one at radius of 4.62m did not function well. This reduced the number of points for plotting the displacement curves and so the straight lines were plotted. There was no clear trend of convergence as seen in Fig 6 and therefore, the line ox of the right wall has been superposed on Fig 7 of the left-wall for the
The purpose of obtaining radius of the broken zone. The elastic displacements at the boundary of the broken zone and the variation in radius of the broken zone with tunnel face advance are also determined by the graphical procedure.

Broken zone stabilised after a face advance of 8 and 15 times the radius of the tunnel for left and right tunnel-wall respectively (See Table 1). Further, the radius of the broken zone on the opposite tunnel-wall also differs significantly. From Fig 8a radius of the broken zone on the right wall is 16.5m whereas on the left wall it works out to be about 17m. Thus, the broken zone does not appear to be concentric with the tunnel opening.

**DISCUSSION OF RESULTS**

The development of broken zone with advancing tunnel face is similar to that proposed by Daemen and Fairhurst (1972). However, the stabilisation process is delayed. Daemen and Fairhurst (1972) suggested that the broken zone should stabilise within a face advance equivalent to 3.5-4.5 times the radius of the tunnel opening. The results of the field study indicate this limit between 8 to 15 times the tunnel radius.

The shape of the broken zone as inferred from field data is elliptical having its major axis along the horizontal
plane as 27.5m (Fig 8a,b). The radius of the broken zone along vertical axis may be about 13.75m because the vertical closures were nearly half of the horizontal closures. The elliptical shape partially corroborates Kastner’s (1962) observations for \( \lambda \) (ratio between horizontal and vertical primitive stress) being less than unity. Since horse-shoe supports installed in the tunnel were less stiff in the horizontal plane, it might have allowed the broken zone to expand more in the horizontal direction. This suggests that \( \lambda \) might not have been the sole cause of larger broken zone in the horizontal direction. Further, the broken zone seems to be eccentric with respect to the opening. Further more, the axis of this elliptical broken zone might be tilted in reality because the tunnel had been excavated through highly dipping anisotropic phyllites. The precise shape of the broken zone could not be ascertained because of inadequate number of borehole extensometers. It is suggested that multi-point borehole extensometers should be preferred over several single point borehole extensometers and the maximum depth of such an extensometer should be at least 4 times the diameter of the opening.

It is surprising to note that the ratio between the radius of broken zone and that of the tunnel opening is as large as 5-8 in the horizontal direction. Whereas, Daemen and Fairhurst (1972) predicted this ratio between 2.0 and 2.5 for moderate to high primitive stress conditions.

It is clear from the above discussions that there were large differences between the observed and the theoretical predictions. There may be several reasons for such a discrepancy e.g. time dependent behaviour of broken, rock mass, failure of support systems and heterogeneous nature of the rock mass. All these factors had not been accounted for in the theoretical formations. Hence, there is a need to re-examine the basic theory with a view to incorporate these factors.

The rock mass beyond the boundary of the broken zone remains in elastic state and the displacement may be calculated from Eq. 4 and 5. The actual elastic displacement at the boundary had been obtained graphically and are compared in the Table II.

The observed and the calculated values are seen to differ widely. However the case of \( \lambda = 1 \) represents nearest fit. Keeping in mind the limitations of extrapolations, it is encouraging to note that common assumption of hydrostatic primitive stress field (Jaeger, 1972) popularly known as Heim’s hypothesis appears to be valid for the case studied. It may be remarked here that similar comparison for other tunnels has been more encouraging (Dube, 1979).

CONCLUSIONS

The significant achievement of this study is the development of a graphical method and the demonstration of its validity in determining the average radius of broken zone in actual field conditions. The method had been useful in arriving at the following conclusions:

1. The shape of the broken zone may not always be circular and concentric with the tunnel opening.

2. The radius of the broken zone increases with the tunnel-face advance and the same was stabilized at distance equal to 5 to 15 times the radius of the tunnel opening.

3. The radius of the broken zone is 5-8 times the radius of the tunnel.

ACKNOWLEDGEMENT

The authors are grateful to the authorities of Giri Hydel Project (HP) who sponsored the studies and provided all necessary assistance for the execution of the instrumentation programme. They are thankful to Mr. S. Roy Chowdhury and Late Mr. K. Roy for helping the authors by taking regular observations at the site. They are also thankful to Mr. P.K. Ghosh of the Geological Survey of India for providing the relevant geological informations of the test sites. Thanks are also due to Mr. S.C. Sharma for illustrations and Mr. Prit Pal Singh, CBRI for excellent typing.
### Table I: Development of Broken Zone with Tunnel Face Advance

<table>
<thead>
<tr>
<th>Rock Mass</th>
<th>Radius of Broken zone/ Tunnel Radius</th>
<th>Face advance at the Time of Stabilization Radius of Tunnel</th>
<th>Support Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phyllites</td>
<td>(a) Right wall 8.0</td>
<td>15.0</td>
<td>Failed</td>
</tr>
<tr>
<td></td>
<td>(b) Left wall 5.2</td>
<td>8.0</td>
<td>Failed</td>
</tr>
</tbody>
</table>

### Table II: Elastic Displacements at the Boundary of Broken Zone

<table>
<thead>
<tr>
<th>Rock Mass</th>
<th>Displacements Range of Theoretical</th>
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<tbody>
<tr>
<td></td>
<td>inferred from Field observations,</td>
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<td></td>
<td>in mm</td>
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<tr>
<td></td>
<td>Crown Springing</td>
<td>Crown Springing</td>
<td></td>
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<tr>
<td>Phyllites of Giri Tunnel</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>$P = 52\text{kg/cm}^2, \phi = 35-45^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$c_p = 1.0-1.5 \text{kg/cm}^2$</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$v = 0.25, E=1.2\times10^5 \text{kg/cm}^2$</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>$b_v = \text{N.A.}, b_h = 11.0-16.5m$</td>
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<td></td>
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<td></td>
<td>18.0</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>20.0</td>
<td>-</td>
</tr>
<tr>
<td>i) $\lambda = 1.00$</td>
<td></td>
<td></td>
<td>4.33, 5.23</td>
<td></td>
</tr>
<tr>
<td>ii) $\lambda = 0.75$</td>
<td></td>
<td></td>
<td>1.27, 1.87</td>
<td></td>
</tr>
<tr>
<td>iii) $\lambda = 0.50$</td>
<td></td>
<td></td>
<td>-0.87, -2.8</td>
<td></td>
</tr>
</tbody>
</table>

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PROBLEMS DURING CONSTRUCTION OF THE VARDØ TUNNEL—A 2.6 km LONG SUBMARINE ROAD TUNNEL

LES PROBLEMES DE LA CONSTRUCTION DU TUNNEL DE VARDØ—TUNNEL ROUTIER SOUS-MARIN DE LONGEUR 2.6 km

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ABSTRACT
The excavation of the first submarine road tunnel in Norway started in 1979. This tunnel, of 53 m² cross section, width 9.5 m and length 2600 m had its breakthrough in July 1981. The object of this enterprise is to link the island of Vardø to the mainland. Vardø is situated 1.5 km from the mainland. The latitude is approx. 70°N. The lowest point of the tunnel is 80 m below sea level and the minimum rock cover is 32 m.

The field investigations consisted of detailed geological mapping, seismic refraction measurements and core drillings. Evaluating the data obtained, three major faults and several minor ones could be predicted below the sea bottom. Only few severe water leakages were to be expected.

The tunnel is excavated in a flaggy sandstone of Late Precambrian Age. In some fault zones the rock mass stability was so poor during construction that special safety measures had to be carried out and a special system for concrete lining on the working face was invented.

As a result 561 m length of tunnel was concrete lined of which 350 m at the tunnel face in addition to 2500 m³ of shotcrete and 18000 rock bolts. In order to detect possible water bearing zones and poor rock conditions, extensive exploratory drillings were carried out ahead of the tunnel, totalling 9500 m of percussive holes drilled from the working face and 1500 m of core holes drilled from special recesses in the tunnel.

ABSTRACT
L'excavation du premier tunnel routier sous-marin de Norvège a commencé en 1979. Le percement du tunnel, dont la section nette est de 53 m², la largeur de 10 m et la longueur de 2600 m, a eu lieu en juillet 1981. Le but de la construction de ce tunnel est de joindre l'île de Vardø à la terre ferme. Vardø est à 70° degrés de latitude Nord, à 1 km de la terre ferme. Le plus bas niveau du tunnel est de 80 m au-dessous du niveau de la mer, et l'épaisseur minimum des roches solides au-dessus atteint 32 m.

Pour les recherches préalables, on a fait les cartes géologiques détaillées et on a utilisé la prospection sismique et le carottage. En évaluant les données, on a prédit une grande faille et plusieurs plus petites au-dessous du fond de la mer. On a prévu de grands écoulements d'eaux par occasions seulement.

Le tunnel a été excavé dans les grès schisteux formés vers la fin de l'ére précambrienne. Dans quelques zones de faille, la stabilité du massif rocheux était si faible pendant la construction qu'on a dû effectuer des dispositions spéciales pour des raisons de sécurité,
et on a inventé un système particulier pour le bétonnage au front de taille.

Le résultat en a été le revêtement en béton de 561 m du tunnel, dont 350 m exécutés au front. De plus, on a utilisé 2500 m³ de béton projeté et 18000 boulons. Pour découvrir des couches aquifères possibles et des roches instables, on a effectué de nombreux sondages exécutés de la front de taille, comprenant 9500 m de perforation par rotation-percussion à partir du front, ainsi que 1500 m de carottage à partir de niches parti-
culières.

1. Introduction

Vardø is a small island off the far north-eastern point of Norway, latitude 70° 20' North. (Fig. 1). It is inhabited by approx. 4000 people who work mainly in the fish industry. The island, which is situated 1.5 km from the mainland, borders on the Barents Sea. The climate is therefore strongly influenced by the arctic conditions here resulting in average temperatures during winter and summer of -9°C and +6°C respectively. The area is rather windy with gales or stronger winds than 200 days a year.

Ever since the town of Vardø was being rebuilt after the Second World War, the planning of a permanent connection with the mainland has been going on. The depths in the sound Bussesundet (Fig. 3), are quite moderate, and a bridge was looked upon as the best solution until 1977 when money was allocated for the project. Then it was found that a tunnel beneath the sound was feasible and even cheaper than a bridge (Ref. 2).

Before the planning of this first submarine road tunnel in Norway started, considerable experience had been gained during the preceding 5 years when 2 other submarine tunnels, one for sweet water supply and the other for a gas pipeline, had been constructed. Experience had also been gained from hydro power tunnels passing beneath lakes or rivers.

The excavation of Vardø Tunnel (Photo 1) started in June 1979 and the breakthrough occurred two years later. The tunnel will be opened for traffic in December 1982.

This tunnel descends at 80 per cent from both sides, producing minimum rock cover of 32 m under the eastern part of the sound. The mean rock cover is approx. 40-50 m. Some 1700 m of the 2600 m long rock tunnel is beneath the sea (Fig. 9). The cross sectional area of the tunnel is 53 m², the width 9.9 m. The tunnel was attacked from both ends, using normal drill and blast method for the excavation. The average tunnelling progress was 17 m/week with a maximum progress of 55 m/week working 75 hours a week (10 shifts).

Because of the loose rock masses in the tunnel walls caused by the jointing, extreme care was exacted in blasting, using weak explosives in the contour holes.

2. Field investigation

The first field investigations for a connection between Vardø and the mainland was started as early as 1958, for one of the bridge alternatives. For the tunnel alternative (Fig. 2, the following investigations were made:

Detailed geological mapping both on the island and on the mainland. The results were presented in maps scaled 1:1000. Loose deposits over large areas on the mainland reduced the accuracy of the geological interpretation here.

Boomer-sparker profiling, which made it possible to work out a map in scale 1:1000 of the seabed, indicating the thickness of the loose deposits.

25 seismic refraction profiles with a total length of 12.7 km. The profiles cover a 500 m wide zone across the sound. The measurements have recorded data about the thickness of the loose materials as well as seismic rock mass velocities, indicating weak zones or faults as low velocity zones. The rock mass seismic velocity distribution is shown on Fig. 3.

36 soundings in the ocean in order to detect the exact position of the rock surface in the depressions. As shown on Fig. 4 short core drillings were carried out in the bottom of some of the holes to obtain samples of the rocks. All the holes were submitted to water pressure tests. These soundings showed that the rock surface found by the preceding seismic method had an accuracy of ± 0.5 m, not counting clefts along weakness zones.
7 core holes totalling 660 m drilled from the coast. The holes were submitted to water pressure tests and the water leakages recorded as Lugeon-values.

From these field investigations the rock surface had been determined relatively accurate. This was significant for the evaluation of the tunnel alignment with respect to necessary rock cover and length of the tunnel.

3. Geology

The bedrocks are of Late Precambrian Age and consist of slightly metamorphic quartzitic sandstones, siltstones and clay-schists (Fig. 5). The rocks are folded along a N-S fold axis. In the middle of the sound Bussesundet a faulted and unsymmetric anticline occur. On the mainland the bedding dips steeply towards West and on the island Vardsø the dip is 45-50° East.

Also, several 0.5-5 m thick dolerite veins occur, striking NE-SW. The rocks are mainly composed of unaltered metadolerite without any pronounced weakness along the boundary.

In addition to frequent joints along the bedding planes, three other steep-dipping joint sets occur. The joint spacings are as follows:

Bedding joints, spacing 0.1-1 m
Vertical joints, strike N-S " 1-2 m
Vertical joints, strike E-W " 0.2-1 m
Flat-dipping joints " 0.5-2 m

The degree of jointing can be classified as moderate to high with a volumetric joint count (Jv) of 5-19 joints per m³ of rock mass, with a mean value of 8. Most of the joints are planar and have thin clay coatings.

The overall Q-factor, Ref. 1, outside weakness zones was found as:

\[ Q = \frac{ROD \times Jr \times Jv \times SRP}{Jn} = \frac{90 \times 12 \times 4}{4} = 1.9 \]

assuming a normal stress situation and only minor water leakages in the tunnel. The tunnel, with a span of 9.4 m is given a support class of 22/23 according to Ref. 1. The corresponding support measures will be systematic bolting 1x1 m and mesh reinforced shotcrete. As will be shown later, this was in fact the rock support most commonly used outside the weakness zones, except that the shotcrete was only seldom reinforced.

The core drillings all showed small leakages except for a few zones. This indicated that the permanent leakage in the tunnel would be low when the main water bearing zones were sealed by grouting. A permanent water leakage of 1,0 m³/min. was predicted based on the water pressure tests in the boreholes.

4. Exploratory drillings

Although the core drillings from the shore and in the sound had shown small leakage values, the relatively thin rock overburden beneath the sound (32-50 m) and the possibility that major water leakages should cause trouble for the excavation of the descending tunnel was the reason for performing extensive exploratory drillings. For the entire length of tunnel under the sea, 4 percussive holes were drilled from the tunnel face. The holes were 25-30 m long with an overlap of 5-8 m, see Fig. 6. During the drilling operation, variations in the drilling rate were roughly recorded, yielding information about possible fractures ahead of the tunnel face. The leakage out of the holes was recorded and water pressure tests were performed. According to Ref. (3) the time consumed was 2 hours including drilling of the 4 holes.

These exploratory percussive drillings showed that it is often impossible to drill through clay seams or highly fractured zones. The drillings gave, however, valuable information about the leakage conditions.

For a submarine tunnel under construction it is essential that no rock falls or cave-ins can develop up to the surface (sea bottom). It is therefore important that the best possible information about any poor rock quality ahead of the tunnel face is recorded. Such information is especially valuable where the percussive drillings cannot penetrate through weak or crushed zones. This was the main reason for using core drillings as a part of the exploratory drilling programme. The core holes were drilled from specially provided recesses, and were parallel to the excavation (Fig. 7). The length of the holes was approx. 200 m and the distance between recesses 170-180 m. For each 6-12 m length, water pressure tests were performed and the Lugeon-values calculated.

The experience gained from core drillings gave valuable information about the rock mass condition ahead of the tunnel, specially interesting where the
The tunnel was excavated through unstable rock masses. The core drill hole gave also early information about the water leakage conditions. In a few instances, however, core drilling results were interpreted wrongly because the zones had other orientation than indicated. In two cases the intersection of weakness zones with the tunnel appeared in locations other than expected.

5. Sealing works

Previous experience from tunnelling in Norway seem to indicate that the sealing of water bearing zones is most successfully done by pre-grouting. An estimate based on the pumping cost of permanent leakage water gave as result that leakages in excess of 2.5-3 Lugeon were economically sealed by grouting. 2.5 Lugeon was therefore used as the limit for performing pre-grouting.

Water leakages did not cause any special problems during the tunnel excavation. Only in one place, under the middle of the sound, major water losses occurred in the core hole and in the exploratory percussive holes, and about 36 tons of cement was grouted. In three other places, moderate water leakages were recorded in the boreholes and between 3.5 and 13.5 tons of cement was grouted.

The permanent water leakage in the entire tunnel is 1.0 m³/min., which happens to coincide with the estimate.

6. Construction experience

As mentioned above, the evaluation of the field investigation results predicted slightly unstable rock mass conditions. The degree of jointing proved to be somewhat higher than expected. There was also a greater number than expected of joints with clay coating or clayey fillings. The immediate support in these rock conditions was carried out by scaling works, rock bolts and shotcrete. Later this support was often strengthened by one more layer of shotcrete, and some more rock bolts. In the clay schist the bedding joints were often closely spaced. Where other joints or fractures occurred in addition, the stability was severely reduced, and extensive supporting works resulted. This was the situation in large parts of the tunnel excavated from the mainland. The unfavourable direction of the tunnel during the first 400 m here had also a great impact on the amount of the supporting works.

The predicted large size and moderate size weakness zones were mostly found in the tunnel (Fig. 8). Some of the zones had, however, a poorer stability than expected because of the overall prominent clay-coated bedding joints. An example of this is shown on Fig. 9 where the necessary linings of the predicted weakness zones were longer than expected. All the large and moderate weakness zones, therefore, had to be stepwise lined with in situ concrete. Concrete lining was also necessary in many other parts of the tunnel where unfavourable fractures or smaller zones cut through the flaggy clayschists.

On two occasions the stability of weakness zones was so poor that special care had to be taken during excavation in order to prevent cave-in on the working face. A special excavation and supporting procedure was invented for these situations. The principles of this method, which was successfully applied, are shown in Fig. 10. Experience showed that the shotcreting should be carried out soonest possible after blasting. The spiling bolts should be installed from stabilized rock masses (or from the concrete lining). The final and most important rule is that the length of the round must be adjusted to the actual situation. In a few instances the length was reduced to 0.8 m with stepwise in-situ concreting before the next round could start.

7. Concluding remarks

A total of 561 m of the tunnel was concrete lined, 350 m at the working place. 2500 m³ of shotcrete was placed, most of it without reinforcement, 18,000 rock bolts (which equals a little less than 7 bolts per m tunnel), and some 9500 m of steel bands and 7500 m² of nylon nets were used. The water leakages are distributed along most of the tunnel as drips. As a result approx. 2000 m length has to be water protected by aluminium shields as described by Ref. (4) and (5).

The price per m tunnel will be USD 8,000. The excavation costs amount to USD 3,000 per m, which is close to the estimate. Salt leakage water caused some difficulties for the drilling and loading machinery and special care should be taken to protect the electrical equipment.

Even though the rock mass tunnelling conditions were of poorer quality than normally encountered in Norway, the Vardø Tunnel project has been a success, and
has given valuable information about submarine tunnelling. Plans have been worked out for several other submarine tunnels, 3 of which are now under construction. These 3 are gas pipeline tunnels of lengths from 3 to 4.5 km. The same procedure of exploratory drillings ahead of the tunnel face is being used here and a similar procedure for excavation through possible particularly poor and unstable rock masses will be applied.

8. References


Fig. 1
KEY MAP

IV. 235
Fig. 3 DISTRIBUTION OF SEISMIC VELOCITIES OF THE ROCK MASSES

Photo 1

PHOTO OF VARDØ AND THE TUNNEL
Fig. 4

EXAMPLE OF PREINVESTIGATIONS IN THE
SOUND TO DETERMINE THE ROCK SURFACE

IV.238
LEGEND:

- MAINLY RED QUARTZITIC SANDSTONE
- MAINLY GREY QUARTZITIC SANDSTONE
- MAINLY CLAYSCHIST/CLAYSTONE
- ALTERNATING CLAYSCHIST/SANDSTONE
- DOLERITE VEIN

Fig. 5

GEOLOGICAL MAP
Fig. 8
CORRELATION BETWEEN THE ASSUMED AND THE
RECORDED POSITION OF WEAKNESS ZONES

LEGEND:

- Alternating Schists/Sandstones
- Mainly Clayschists
- Mainly Sandstones
- Dolerite Vein
- Strike and Dip of Bedding
- Major Crushed Zone
- Moderate Crushed Zone
- Strike and Dip of Zones
- Concrete Lined Tunnel
- Assumed Weakness Zone, Not Encountered
- Assumed Weakness Zone, Encountered
Fig. 9

CORRELATION BETWEEN ASSUMED WEAKNESS ZONES,
ROCK MASS CONDITIONS AND SUPPORTING WORKS
IN THE TUNNEL

ASSUMED
WEAKNESS ZONES

SEISMIC LOW
VELOCITY ZONE

SEISMIC PROFILE
**STEP I**
STEP CONCRETING WERE PERFORMED BEFORE EXCAVATION INTO THE EXTREMELY UNSTABLE ZONE AND SPILING BOLTS WERE CARRIED OUT THROUGH THIS CONCRETE AND INTO THE ZONE.

**STEP II**
EXCAVATION INTO ZONE WERE DONE BY SHORT BLASTING ROUND AND THE BLASTED MATERIAL WERE REMOVED. CEILING, FACE AND WALLS WERE SHOTCITED. PREBUILT FORM WAS POSITIONED AS CLOSE TO FACE AS POSSIBLE.

**STEP III**
TUNNEL SPOIL WERE PLACED AGAINST THE FACE. FORM WORK WERE PLACED BETWEEN TOP OF TUNNEL SPOIL AND THE CEILING OF THE PREBUILT FORM.

**STEP IV**
TUNNEL SPOIL, PREBUILT FORM AND THE FORM WORK WAS REMOVED AND SPILING BOLTS WERE PLACED THROUGH THE NEW CONCRETE AND INTO THE ZONE. A NEW EXCAVATION ROUND COULD START.

![Diagram of excavation process](image)

*Fig. 10* SPECIAL SUPPORTING PROCEDURE FOR EXCAVATING THROUGH EXTREMELY UNSTABLE ROCK MASSES
AN ASSESSMENT OF THEORETICAL MODELS FOR THE DESIGN OF MINE TUNNELS

UNE EVALUATION DES MODELES THEORIQUES POUR LE DESSEIN DES TUNNELS DE MINE

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ABSTRACT

The prediction of rock movement in mining has become the subject of modelling by known mathematical solutions. In coal mining the closure is generally an order of magnitude greater than in hard rock and it is important for the correct design of a support system to know the amount of closure expected. Formulae for the prediction of coal mine tunnel and roadway closure are therefore desirable.

Four formulae for such prediction are herein compared with data from ten test sites typical of British Coal Measures rocks. The order of magnitude of the closure experienced presents special difficulties, in particular because the plasticity solution on which many model solutions are based is no longer statically determinate for large deformation problems.

The agreement between formulae predictions and test site observations are of similar order of magnitude for at least the incremental formula. However, there are clearly other factors which need to be taken into account, since there are variations in closure observations unexplained by the formulae. These variations and their causes are discussed.

ABSTRACT

La prédiction du mouvement du rocher dans l'exploitation minière est devenu le sujet de modèles théoriques à base de solutions mathématiques connues. En ce qui concerne le charbonnage le degré de fermeture dépasse généralement le degré de fermeture dans le domaine de l'exploitation de rochers durs. Il est donc important pour une formulation exacte d'un système de soutien de savoir le degré de fermeture attendue et des formules pour la prédiction de la fermeture des tunnels houillers deviennent souhaitables à ce propos. On trouve ci-jointes quatre formules pour une telle prédiction qui sont à comparer avec les données tirées de dix stations d'études typiques pour les couches carbonifères britanniques. Le degré de fermeture observé présente des difficultés spéciales très particulièrement parce que la solution obtenue de la théorie de plasticité à base de laquelle un grand nombre de modèles sont formulées ne déterminent plus rien de façon statique en ce qui concerne les problèmes d'une grande deformation.

Les predictions obtenues à base de formules et les observations des stations d'études se rapprochent du moins en ce qui concerne les formules d'accroissement. Il y a cependant d'autres éléments à considérer du fait que les formules n'expliquent pas certaines variations de fermente observées. On discute ici de ces variations et de leurs causes.

IV.245
1. Introduction

The prediction of closure by mathematical models follows a pattern which does not necessarily relate to the problem encountered underground. Known mathematical solutions are combined with established material behaviours to give solutions to problems which purport to represent drivages and tunnels in mines. Whilst the models are not yet of sufficient sophistication to be worthwhile as predictive tools, despite their use as such in many papers and texts, it is perhaps valuable to compare predictions with observations in an attempt to check whether the adopted path is the correct one.

2. Theoretical Models

The usual starting point for models is the elastic solution for a hole in a 2-dimensional, isotropic, homogeneous, infinite medium, in a (plane strain) hydrostatic stress field. Application of a failure criterion establishes whether failure has occurred at the hole boundary, and the assumption of infinitesimal deformations makes the problem statically determinate. This enables solutions to be obtained for the stresses and extent of failed rock without requiring a solution for displacements.

The assumptions made can be roughly summarised and classified as to their worth. A 2-dimensional stress system is not present close to the heading, and this is where high stress concentration and the majority of closure occurs. Rocks are inhomogeneous, but many may be thought of as statistically homogeneous on the scale of mining. Isotropy is less justifiable, particularly in Coal Measures rocks, and many of the effects of inhomogeneity may be classified more accurately as anisotropy. Plane strain is justifiable inasmuch as the problem is 2-dimensional, but hydrostatic loading is definitely not. Hoek and Brown (1980) show the deviations from hydrostatic conditions commonly encountered, and the degree of deviation is clearly a major factor. The differences in failure criteria, from the simplest straight line (σ₁, σ₃) envelope to parabolic or power-law envelopes, are likely to be swamped by other effects.

Assumption of rock behaviour is usually elastic-brittle/plastic. If small deformations are assumed the elastic displacement is comparable with the total displacement; if this condition is relaxed it is permissible to neglect the elastic component, in which case arguments against the elasticity of rock are less valid. The brittle/ductile behaviour of rocks has been extensively discussed (see e.g. Murrell 1965) and is well established. However, for large displacements the solution is not statically determinate, and a complete solution for strains and displacements is necessary. This, therefore, considerably complicates the solution.

3. The Test Sites

Comparison with case histories must take account of the assumptions made, and attempt to match conditions reasonably closely. However, assumptions may inter-compensate; for instance, an arched opening subject to a ratio of vertical to horizontal far field stress greater than unity, may behave in a similar manner to a circular opening in an hydrostatic field with judicious choice of shape and ratio. Thus are the tunnels and drivages in mines usually declared comparable with theoretical models. In this study all possible precautions were taken to match test site conditions with the pre-requisites of theoretical models, but with the aim being to check the overall trend of model goodnes-of-fit, and not to prove the validity of a particular model, provided the test sites are representative the onus is on the models to fit the experimental data, not vice versa.

Figure 1 shows a typical geological section from a test site. It is clear that the ground is inhomogeneous, with the macroscopic appearance of transverse isotropy, both of elastic properties and failure and post-failure parameters. Methods for estimating equivalent elastic moduli of composite materials exhibiting macroscopic transverse isotropy due to layering are well known (e.g. Salamon, 1970) The failure and post-failure properties will, however, have different macroscopic average values depending on the type of failure expected. Ductile failure caused by plastic slip will occur in-plane at the lowest value of combined stresses at which anyone plane would slip, but across the plane at the highest value. Brittle failure will not exhibit such pronounced anisotropy. A first solution will be, therefore, to obtain bounds based on the weakest and strongest materials in the
layering. Geometric and arithmetic averages, by analogy with the overall elastic moduli, may have some significance, but only experimental results are of real value in these circumstances.

The ten sites chosen are typical of British Coal Measures; ranges of values of mechanical and physical parameters are available (e.g. Hassani, 1980), typical strong rocks being sandstones \( (E = 5 \times 10^4 \text{ MPa, } v = 0.3) \); unconfined compressive strength \( (\sigma_c) = 74 \text{ MPa} \); triaxial strength factor \( (k) = 4.4 \); and weak rocks sandshale \( (E = 3 \times 10^2 \text{ MPa, } \nu = 0.13, \sigma_c = 15.8 \text{ MPa, } k = 2.4; \text{ where } \sigma_c = k\sigma_0 + \sigma_0 \text{ is the failure criterion). Depths are also representative; the deepest coal mine in the world is only 1400 m deep, in the Ruhr in West Germany, and shallow deposits are taken by open cut unless extreme political or ecological reasons dictate otherwise. 200 -900 m is therefore a typical range in which predictive formulae should be expected to produce good results. Full site data is given in Table 1.

4. Models for Comparison with Test Site Data

Four models were chosen, not necessarily covering the whole range of those available, nor being the most sophisticated. However, two are recommended either by their authors or by others as realistic predictive models (Ladanyi, 1974; and Wilson, 1980); and two have particular features which may be worthy of study. Muir Wood's (1975) model is based on elastic material behaviour, and may therefore help to demonstrate whether there is any possibility of elastic models giving accurate predictions in certain restricted cases. Ladanyi's and Wilson's models are both of the failed zone type; Ladanyi's is essentially geometric with a plasticity analysis to determine the expansion factor, Wilson's examines an idealized microstructure of axi-symmetric broken rock. The fourth model (Wells, 1982) is not significantly different analytically from Ladanyi's or Wilson's but allows for the more accurate prediction of dilatational properties. It is obvious that the dilatational characteristics of the rock, together with the extent of failed rock, determine the closure; it is therefore surprising that Wilson's formula is sufficiently insensitive to these characteristics to enable the parameter to be treated as a universal constant, and that Ladanyi's formula has a value based on infinitesimal plasticity theory, effectively restricting all deformations to be of similar order of magnitude to the elastic deformations. Since it is found experimentally (Murrell, 1965, and many others) that post-failure dilatation depends on the confining pressure, and since this pressure typically varies by an order of magnitude across the region subject to post-failure dilatation, it is only reasonable to allow for an expansion factor varying with stress, across this region. The form of this assumed dependence is based on results of Price and Farmer, 1979 Displacements are calculated on an incremental basis from simple analytic formulae, with the theoretical advantage of continuity of strains across the elastic-failed boundary (Wells, 1982). Of course, the need for accurate experimentally determined data on the post-failure expansion of rock, as the pressure is reduced from very high hydrostatic, is in this case even more important.

5. Comparisons of Model Predictions with Test Site Results

Calculations were made on the basis of the four models mentioned, with parameters as given in Table 1. Where appropriate the rock parameters have been chosen most, advantageously for the model i.e. in the cases where closure is predicted to be less than that found in practice the maximum predicted closure has been chosen from all possible interpretations of the data. For failure envelopes this means allowing failure of the whole rock when failure conditions are met in any one rock comprising the laminae.

If it were desired to rigorously compare various formulae such manipulation of the data would be undesirable; the theoretically most justifiable combinations of data should be used, not those experimentally most expedient. However, it is not the purpose of this comparison to say whether any particular formula is useful as a predictive tool. Indeed it was stated at the outset that none should be regarded as workable formulae, but as steps on the road to a solution, and that this comparison is to monitor that direction. If the prediction does not approach the observation even under the most favourable assumptions then some fault must exist in the formulation of the prediction. This is clearly the case. The elastic model of Muir Wood is not of the order of magnitude required. This is expected since Muir Wood
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Table 1 Test Site Data

Table 2 Comparison between actual and predicted closures at ten test sites

* No yield zone is predicted

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<td>0.06</td>
<td>0.20</td>
<td>0.02</td>
</tr>
</tbody>
</table>

IV.249
formulated his model to describe the behaviour of buried pipes in shallow ground. The pipe will clearly be elastic under such conditions, but the ground around a roadway or drivage supported by 1 metre spaced arched steel girders is not, by observation. The values of closure predicted by Ladanyi are smaller than the observations. It has already been noted that the expansion factor in Ladanyi's formula is based on a linear plasticity analysis. With Labasse's (1949) analysis closure values closer to observation are predicted. (Labasse expansion factor = 0.1, Ladanyi for typical coal mine drivage = 0.006). The Wilson formula has been devised specifically for coal mines, and should therefore give good predictions. The discrepancy may be due substantially to neglect of stress-dependency in parameters such as expansion factor and post-failure rock properties. This is borne out by the improved accuracy of the incremental model. The observation variations, however, are once again not able to be predicted.

6. Possible Improvements to Models

The models are more uniform in their predictions than are the sites in their closure data. There is considerable scatter in the measured closure (Figure 1) but the trend is clearly discernible and averages are consistent. At present it is possible to predict closure in coal mine tunnels and drivages more accurately using a formula based on statistical analyses of past performances than on a theoretical analysis (Kammer 1980). The prospects for a model of a gate roadway, with all the added complications of a longwall face alongside, are therefore poor. However, theoretical models are potentially as useful for the understanding they give as for the results and, if for this alone, it is worth considering ways to improve models. These may come under two headings: more accurate use of existing parameters and incorporation of other influential factors.

The post failure behaviour of the rock is certainly of paramount importance, and yet in the formula of Wilson all post-failure properties are replaced with estimated representative constants. This may seem to be unjustifiable, particularly since variational analyses show these to be the most influential parameters in the formula, but examination of the results from the test sites shows that the poor correlation with prediction is not accounted for by this approximation. Also required to be estimated due to lack of knowledge of exact value is the support pressure. However, the support offered by colliery arches varies only within a small range (Miller 1981). Of perhaps more importance is the in situ pressure. This is typically several orders of magnitude larger than the support pressure. It is currently estimated from the depth, using the rule that vertical load is due to overburden pressure, average density of overlying is rock is 2.4, and horizontal pressure maintains hydrostatic conditions. That this is not a good estimate of in situ pressure has been demonstrated (e.g. Hoek and Brown 1980) and variations are frequently found of 100% or more from this estimated vertical load. These marked variations are, however, usually in tectonically altered rocks, not the strata of the workable Coal Measures. The poor correlation found between depth and closure would suggest, that a more reliable method of estimating in situ pressure should be used unless the relationship with depth can be established.

More important would seem, therefore, to be factors which are not yet accounted for. Deviations from hydrostatic conditions, anisotropy of rocks and their failure characteristics and non-circular shape of opening are all obviously important judged by the differences between vertical and horizontal measured closure, typically 35% (but occasionally very much lower horizontal than vertical).

Closure usually attains a steady state within 3 or 4 roadway diameters of the face. This represents 3-5 days advance in a typical heading and 15 arch sets. The question naturally arises as to whether either the time of support installation relative to closure or the rate of closure has any effect on overall closure, and whether the delay in attainment of steady state closure is due to time-dependent effects or a changing stress profile due to the proximity of the heading. Of these considerations the difference between time-dependency and dependence on distance from the heading would not seem to be able to account for the variations in the data, since the effect is the same in all cases. The rate of closure is not a strain-rate dependence, since this is only found in Coal Measures rocks under exceedingly rapid loadings (of the order of seconds) or a creep effect, since this is evident much further from the heading. It is presumably therefore, due to the proximity of the face.
and related to face advance. This parameter will therefore affect the time delay before support installation, which in turn affects the overall closure. Daeman (1975) has shown how critical is the time delay theoretically, and whilst the practical effects would be blurred it is worth consideration.

7. Conclusions

The model of Muir Wood does not have the capacity to predict closure to within acceptable orders of magnitude. The elastic effect can therefore be considered negligible. The model of Ladanyi therefore reduces to that of Labasse. This model can be made to fit reasonably well with many observations, but is missing the parameters which characterize the variations apparent in the test site results. Inclusion of more detailed information in the model is desirable, including the true state of in situ stress, the anisotropic failure condition and the amount of stress relation before support installation. The model of Wilson suffers these same disadvantages but is otherwise more amenable to manipulation by the skilled on-site engineer, since many parameters are approximations to rock properties or "general impression factors". The improved accuracy of the incremental model is encouraging for this approach, and therefore is helpful in the aim of improving understanding of physical mechanisms through theoretical modelling, but since it introduces no new parameters it cannot help to explain the unexpected variations in test data.

8. Acknowledgements

The work reported herein forms part of a wider Rock Mechanics research programme and as such acknowledgement must be made of the contributions of Professor T Atkinson and Dr B N Whittaker, Rock Mechanics research co-ordinators.

9. References

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IV.251
ABSTRACT

Les facteurs principaux qui déterminent les aspects géochimiques et géologo-techniques d'altération des roches dans les pays à climat tropical sont envisagés. La description de l'installation expérimentale de simulation artificielle d'altération dans le laboratoire est présentée aussi que la justification des conditions des limites, les paramètres thermodynamiques et hydrochimiques des modèles ce qui détermine le choix de quatre types des modèles. La discussion des résultats des expériences est donnée. Les études de la composition, de la structure et des propriétés physico-chimiques sont effectuées à l'aide des méthodes chimiques, microscopiques, laboratoires ainsi que des méthodes structurales à rayons-X.

L'influence importante sur le déroulement du processus d'altération de l'ambiance hydrochimique, des conditions thermodynamiques, de la composition des roches est établi. Les facteurs envisagés ci-dessus sont évalués quantitativement ce qui permet de donner le pronostic de l'altération dans les conditions naturelles.

Les résultats des expériences ont permis le pronostic de la formation de la composition, de la vitesse et de la puissance de la croûte d'altération dans le cas concret de la construction très importante.

ABSTRACT

The main factors, determining geochemical and engineering geological aspects of weathering of rocks in conditions of tropical climate are considered. The description of experimental plant for the modeling of process of weathering in laboratory conditions is given. The boundary
conditions, the thermodynamical and hydrogeochemical parameters of modelling are grounded and accordingly four types of models are chosen. The results of experiment are discussed. The composition, the structure and physical-mechanical properties are researched by means of chemical, microscopic, laboratory and X-rays methods. The essential influence of hydrogeochemical surroundings, thermodynamical conditions and compositions of rocks on weathering is ascertained. For the enumerated factors there are obtained the quantitative estimates, which permit to predict to prognose the natural process of weathering of rocks. In virtue of results of forming of composition, velocity and power of weathering crust is elaborated for the most important object of construction.

L'altération des roches dans les pays tropicaux est totalement différente de celle qui se passe dans les pays boréaux. L'abondance des précipitations dont la quantité atteint 2000-10,000 mm par an, des hautes températures (25-41°), l'abondance des acides organiques et CO₂ qui se produisent grace à la décomposition de végétation tropicale (la quantité des feuilles tombées constituent 35-tonnes par an) grace à quoi les roches subissent les changements chimiques très profonds produisant ainsi la croûte d'altération de type latéritique. Bien que les problèmes géochimiques de la latérisation soient un objet d'études très actif (car ils sont la clé des prospections des minéraux utiles d'origine hypargénée, le nombre des travaux, consacrés aux aspects géothecniques du processus d'altération est très limité. Or cette question est très importante vue le développement de larges programmes de construction dans les pays d'Afrique, d'Asie, d'Amérique Latin, réalisé à l'aide très active et la participation directe des spécialistes soviétiques. Aux particulier ce travail a été fait pour la solution des nombreuses questions, qui se sont posé lors la projection de l'hydrocentral Hoabigne sur la rivière Da ou Vietnam du Nord. Le choix du matériel de construction pour le barrage, la configuration et les angles de versants du chenal de navigation ainsi que des excavations de construction, la protection des versants contre l'influence d'altération, la possibilité des glissements grace aux changements de l'humidité des roches altérées lors du remplissage du barrage - constituent l'ensemble de problèmes dont la solution dépend de l'évaluation géothecnique des roches de la croûte d'altération sur tout le territoire de la ville de Hoabigne.

Les prospections des profils de la croûte d'altération sur la terrain, l'absence des données des observations stationnaires ne permettent pas la solution des problèmes qui se posent. Vue cette situation, nous avons fait la tentative de simuler artificiellement ce processus dans le laboratoire.
Le but des travaux était suivant :
- de démontrer l'influence de l'ambiance hydrochimique ;
- de l'influence de la composition et des propriétés structurato-texturales de la rochemère sur la formation de la composition et des propriétés des matériaux de la croûte d'altération ;
- de l'influence des zones de la destruction tectonique sur la formation des croûtes d'altération linéaires, ainsi que l'évaluation des apports des facteur (agents) dans le processus d'altération des roches ;
- la détermination de la vitesse d'altération des roches dans les pays tropicaux ;
- la définition des tendances dans le changement physico-mécanique des propriétés des roches en fonction de classe d'altération .

Le processus d'altération des roches dans les pays tropicaux est complexe et multivalent. Il va de soi qu'il est pratiquement impossible de reconstituer ce processus dans les conditions artificielles .

De ce fait en choisissant le modèle nous avons essayé de reproduire les facteurs les plus importants, qui conditionnent le changement de la composition ainsi que des propriétés des roches de la zone de hypergénèse, d'intensifier ces facteurs et d'observer le caractère et la vitesse des changements chimico-minéralogiques en composition et propriétés des roches.

L'expérience a été basée sur l'utilisation d'appareil Soxcel, qui est le plus convenable du point de vue de la simulation artificielle du processus d'interaction des roches avec des agents atmosphériques et biologiques y compris le processus d'altération. Cet appareil a été utilisé pour la première fois en 1964 pour des études expérimentales de géochimie d'altération des roches cristallines par le géochimicien François Pedro. L'appareil se compose du cylindre principal qui sert du réservoir pour les roches à étudier, le frigorifique et le cylindre - récepteur qui s'installe sur l'appareil de chauffage. Le principe de fonctionnement de l'installation : le fluide chauffé dans le cylindre s'évapore et par le syphon latéral parvient dans la partie supérieure du cylindre principal. Une fois condensé dans la partie inférieure, qui est en contact avec le frigorifique sous forme des soutes, elle tombe sur la roche. Ce processus va jusqu'à ce que le cylindre est rempli. Une fois le tuyau supérieur du syphon est atteint, la liquide se décharge instantanément dans le cylindre-recepteur .

Du point de vue "d'hydrologie" cet appareillage reproduit parfaitement le cycle naturel de l'eau : les précipitations atmosphériques - infiltration - écoulement - accumulation - évaporation. Les syphons de bypass permettent d'inter deux zones à caractère est d'hydrologique différentes : la supérieure est

IV.255
la zone d'aération dans laquelle les échantillons sont humectées par "la pluie", et la zone inférieure dans laquelle les échantillons des roches immérent périodiquement et qui ainsi constitue la zone d'humidification alternante.

Socxlet permet d'assurer la durée d'expérimentation assez longue, ainsi que de délimiter deux phases de décompositions qui sont propres à toutes les types d'altération: la phase compacte résiduelle, qui se produit dans le cylindre principal et qui se compose des minéraux originaux et secondaires dont l'origine est liée avec la répartition des compositions lessivées et la phase liquide - le filtrat, qui contient les éléments et leurs compositions, emmenés par la solution lors sa percolation à travers les roches. Ainsi il est possible de faire le bilan géochimique du processus d'altération, des roches à chaque étape de l'expérience en faisant la comparaison des phases compacte et liquide avec la composition d'une roche originale.

L'appareil permet de réaliser tout l'ensemble des conditions et de facteurs déterminés de processus de l'altération ainsi que les varier dans les limites très larges. L'appareil peut être changé par des échantillons de toutes les dimensions et de toutes les formes. Les propriétés physico-mécaniques peuvent être étudié avant, après et aussi au cours des essais ce qui permet d'établir la vitesse des changements des tels ou tels paramètres en fonction du temps.

De principaux paramètres qui constituent l'ambiance spécifique thermodynamique et hydrochimique du processus d'altération et qui ont été étudié lors de la simulation artificielle sont: l'énergie thermique les précipitations atmosphériques, CO₂ et les acides organiques. Les conditions limitatives ont été imposées de telle sorte que les paramètres intensifier ne depassent pas les valeurs extrêmes existant dans la nature.


La norme diurne des précipitations dans l'appareil socxlet était de l'ordre 440-620 mm, dans les conditions naturelles pendant l'hivernage elle est égale à 13 mm seulement. Cependant en Inde et dans les Philippines on Connais les Cas, quand les précipitations diurnes était de 1000 mm.

Répartition des précipitations a unité la suite de l'eau saisoni; durant le de la pluie de 10-12 heures a été percolée presque la moitié de norme annuelle des précipitations a travers les roches (ce qui représentaient l'hivernage); la partie restante du jour a imité la période sèche dont la température était égale à -18 -20°C.
Vitesse d'infiltration dans l'appareil était égale à 60 ml/h, la valeur des précipitations dans les conditions du Viet-Nam du Nord représentent près de 800 mm/an car de 2000 mm des précipitations près de 40% constitue l'évaporation et 20% - l'évaporation superficielle.

On sait que conformément aux conditions acido-alcalines les eaux des sols se divisent en 4 groupes principaux (Perelman, 1977) fortement acides (pH 3), acides, faiblement acides (pH = 3 - 6,5), neutres - faiblement alcalines (pH = 6,5 - 8,5), alcalines (pH 8,5) on a décidé de réaliser 4 modèles de processus.

Modèle I. Le milieu fortement acide, pH = 2,65 - 3,45; les agents; la température, les précipitations atmosphériques, la vitesse d'infiltration - 60 ml/h, l'acide organique (CH₃COOH).

Modèle II. Le milieu acide, pH = 3,65 - 4,65; les agents; la température, les précipitations atmosphériques, la vitesse d'infiltration 60 ml/h, l'acide organique, le dioxyde carbonique.

Modèle III. Le milieu neutre, pH = 6,3 - 7,2; les agents; la température, les précipitations atmosphériques, la vitesse d'infiltration 60 ml/h, le dioxyde carbonique.

Modèle IV. Le milieu alcaline, pH 8,5, les agents; la température, les précipitations atmosphériques, la vitesse d'infiltration, H₂O.

Les échantillons des roches prélevés dans les mines et des tunnels de prospection sur l'emplacement du barrage Hoabinh sont des porphirites basaltiques, les brèches de porphirites - basaltiques, les roches des zones schisteuses à composition chlorite - carbonique et des zones des brèches à composition quartz-calcite-dolomitique; c'est-à-dire des roches qui serviront de base du barrage. Les porphirites-basaltique ont été soumis à simulation selon 4 modèles; les roches d'autres types n'ont été soumis qu'à simulation selon le premier et le troisième modèles. La durée d'expérience pour chaque modèle était de 4-5 mois, et quelque fois 15-24 mois. La durée générale de l'expérience pour tout ensemble de modèles était de 72 mois. Le tableau 1 représente de principaux paramètres de expérience.

L'étude détaillée de la composition des roches par des méthodes chimiques pétrographiques et à rayons X a été effectuée avant et après des essais. L'étude de la composition ainsi que pH des solutions ont permis d'établir la capacité à déplacement des éléments lors de l'altération artificielle et ainsi d'évaluer quantitativement les éléments lessivés et les changements du milieu. Les études des propriétés physico-mécaniques des roches (la masse volumique, la densité, la porosité, les indices de plasticité, les limites de la résistance à compression le coefficient de rigidité ont permis de
<table>
<thead>
<tr>
<th>Nom d'une roche</th>
<th>NN des modèles</th>
<th>Facteurs</th>
<th>Dure d'expérience</th>
<th>Quantité Payé d'une des pré-roche dans la cylindrotorsion, mille mm</th>
<th>Lessivage des roches au cours d'expérience, gr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porphyrites</td>
<td></td>
<td>CH₃COOH+T°</td>
<td>5</td>
<td>25,1</td>
<td>203,5</td>
</tr>
<tr>
<td>-basaltiques</td>
<td>II</td>
<td>H₂O+T°+CO₂</td>
<td>4</td>
<td>41,9</td>
<td>791,7</td>
</tr>
<tr>
<td>III</td>
<td>H₂COOH+T°+CO₂</td>
<td>15</td>
<td>100,0</td>
<td>243,8</td>
<td>1,3</td>
</tr>
<tr>
<td>IV</td>
<td>H₂O + T°</td>
<td>24</td>
<td>199,8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chlorite-</td>
<td></td>
<td>CH₃COOH +T°</td>
<td>4</td>
<td>17,3</td>
<td>259,6</td>
</tr>
<tr>
<td>carbonates</td>
<td>II</td>
<td>H₂O+T°+CO₂</td>
<td>3</td>
<td>10,8</td>
<td>783,2</td>
</tr>
<tr>
<td>Quartz-</td>
<td></td>
<td>CH₂COOH+T°</td>
<td>4</td>
<td>36,4</td>
<td>711,4</td>
</tr>
<tr>
<td>carbonates</td>
<td></td>
<td>H₂O+T°+CO₂</td>
<td>3</td>
<td>22,9</td>
<td>237,3</td>
</tr>
</tbody>
</table>

Tab. 1. Principaux paramètres d'expérience

mettre en évidence les fois de formation des propriétés sous l'influence d'altération en fonction de la capacité à migration des changements de la composition chimique et mineralogique. Les résultats obtenus de quatre modèles réalisés sur les porphyrites-basaltiques ont démontré.

L'intensité des processus géochimiques et leur orientation dépendent du régime adidoalcalin du milieu aquatique à mesure que l'acidité baisse et l'alcalinité monte, la quantité de résidu lessivé sommaire diminue. Les calculs montrent que dans la nature, compte tenu de la différence en densité des roches, insitu et dans le modèle le résidu lessive dans le milieu acide a constitué 660 gr. d'un m³ de roche, dans le milieu neutre-alcaline - 28 et 10 gr. pas an (l'infiltration étant de 800 mm). Le lessivage le plus intencif des composants est observé au début, ensuite il baisse ce qui s'explique par la forte densité des roches (2-7%) et par l'action des pellicules protectrices recouvrant les produits de destruction.

Le milieu acido-alcalin détermine la capacité à migration différente des éléments. Les milieux acides ainsi que Ca ont conditionné la capacité élevée de déplacement de Fe; en présence de CO₂ on observe le lessivage intensif de Si; la capacité de déplacement de Fe et Al étant faible.
On peut constater, que dans les roches de la croûte d'altération se passe la répartition des éléments qui est suivie par le lessivage des éléments les plus mouvants et l'accumulation, des éléments faiblement mouvants dans les conditions données. L'interaction dans le système l'eaux-roche aboutit à son évolution y compris les fases: dure, liquide et gazeuse; la variation de pH lors de l'expérience en est la preuve.

<table>
<thead>
<tr>
<th>Mois Avant l'expérience</th>
<th>Etapes I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>I 2,8 3,5 3,1 3,4 .-  -</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II 6,8 6,3 7,2</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>III 2,7 4,6 4,0 4,6 4,4 3,6</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IV 7,6 9,9 9,8 7,8 7,9 8,5</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Tab.2)

La saturation des laux du sol par les composantes lessivées des roches au cours de la filtration explique leur transition des acides en faiblement acides - neutres - alcalines.

Lors de l'altération de différentes zones hydrochimiques se produisent en se repartissant suivant des intervalles de la profondeur ce qui conditionne la variation du caractère de lessivage des éléments à ces intervalles.

Les résultats d'expérience montrent que la capacité de migration dans des milieux acido-alcalins variés conditionne la diversité de la composition minéralogique des roches. La tendance du processus d'altération consiste en diminution progressive du nombre des minéraux la formation des agrégats monominéraux dans le développement des profils hygènes. Les essais pétrographiques à rayons X montrent qu'au cours. Des expériences, réalisés dans les milieux acides les minéraux primaires des grains et également ceux de la masse microcristalline se transforment en chlorites tandis que les vitres se transforment en argiles.

Les changements chimico-minéralogiques conditionnent la diminution du poids spécifique de 3,0 à 3,6 - 2,8 - 10^-3 kg/m^3 ce qui correspond au poids spécifique des chlorites. Dans les milieux neutres et faiblement neutres les processus d'oxydation très actifs ont lieu dont la conséquence est la formation de la croûte ferrugineuse superficile, mais puisque la quantité des oxydes de fer n'est pas grande, le poids spécifique ne se varie que d'une manière insignifiante.

La chasse des composantes chimiques est la cause de la distorsion des minéraux primaires et secondaires, la formation des poses et de cavères la fissuration. La porosité de I-er modèle a augmenté de 2-7 à 36%, celle de II-ième modèle s'est accrue de 5-14%. La tendance unique des processus est la diminution de l'indice volumique de la masse.

Plus le lessivage des composantes, plus la diminution de la masse volumique est grande.
<table>
<thead>
<tr>
<th>Modèle Zone</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>d'aération</td>
<td>2.88</td>
<td>2.91</td>
<td>2.93</td>
<td>2.91</td>
</tr>
<tr>
<td>de humidification</td>
<td>2.90</td>
<td>2.96</td>
<td>3.01</td>
<td>2.96</td>
</tr>
<tr>
<td>avant l'expérience</td>
<td>2.96</td>
<td>3.00</td>
<td>3.03</td>
<td>2.97</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Modèle Zone Statistiques II III IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>d'aération X 191 194 181</td>
</tr>
<tr>
<td>de humidification R 134-259 83-205 -</td>
</tr>
<tr>
<td>déficience R 114-270 182-244 191-243</td>
</tr>
<tr>
<td>variable n 13 8 4</td>
</tr>
<tr>
<td>avant X 238</td>
</tr>
<tr>
<td>l'expérience n 38 10</td>
</tr>
</tbody>
</table>

Tab.3. Changements des propriétés physico-mécanique des porphyrites après des expériences de l'altération

A. De la masse volumique,
10^{-3} \text{ kg/m}^3

B. De la résistance des roches à la compression, MPa.

En se basant sur le caractère des changements de la résistance des porphyrites basaltiques on peut constater que la baisse de l'indice de la résistance des roches à compression est liée à la transformation chimique, à la destruction des liens, à la formation de la porosité et des fissures d'alévation et à l'ouverture de fissures primaires. La baisse la plus importante est dans la zone d'aération surtout dans sa partie supérieure et sur le contacte avec la zone de l'humidification alternante.

Les expériences ont permis d'établir les nuances de l'altération dans les limites des zones tectoniques. Les changements des roches dans les zones de schistosité et celles de breches sont totalement différents comme dans la nature, et aussi bien dans des expériences et démontrent l'influence de la composition, des particularités structurales des roches ainsi que du milieu hydrochimique.

La capacité de migration des éléments dans les modèles I et II était égale à celle des porphyrites basaltiques Ca, Mg-Fe-Si, Al - dans le milieu acide, Ca,Mg-Si-Fe, Al - dans le milieu neutre.

La quantité de lessivage dans le modèle I a atteint 28-58% dans le milieu neutre - 4 - 18%, car la composition des roches des zones de schistosité et des breches est formée de Ca et Mg qui sont très mouvants ce qui détruit les minéraux dont le réseau cristallin en est constitué. Ce sont les carbonates dont la lessivage en état de solution était égale à 79,5 de 237 g de roche dans le modèle I. Or le même processus de solution donne les résultats différents: la formation de la porosité et des cavernes
dans des roches quartzo-carbonates et la dispersion des roches chlorito-carbonates donnant naissance au matériau chlorito-argileux. Cela est dû aux particularités structurales des roches des zones de brèches et de schistosité: dans le premier cas les roches se caractérisent par la structure à gros et à moyens grains; dans le deuxième cas les carbonates se placent en forme de noeuds parmis des cristaux allongés des chlorites. La migration intense des éléments des roches provoque les changements de pH du milieu de 2,67 à 5-7,20. Dans les milieux neutres la présence de CO₂ et des carbonates conditionne la formation de la solution de contact dont pH est constant jusqu'à moment de l'équilibre qui a pourfum lieu dans le milieu alcalin (dans notre cas à pH = 9,1-9,3). L'activité de migration de Ca, Mg dans le milieu alcalin tombe, l'équilibre de système: roche - eau s'établit très vite, la solution des carbonates est suivi par leur déposition et leur recristallisation.

Le résultat de la transformation des roches est les changements de leurs propriétés, les plus accentués dans le modèle I. Elle s'exprime par le changements de la composition granulométrique (l'augmentation en teneur en fractions agrileuse et poressièruse), l'augmentation de plasticité, la diminution de la densité des roches chlorito-carbonates. Le devancement de la vitesse de solution sur celle de hydration ne change les chlorites que très faiblement ce qui s'exprime selon les données d'analyse par les rayons X par la formation des minéraux à structure schisteuse-mélangée. Dans le cas des roches quartzo-carbonates il se manifeste par l'augmentation de la porosité jusqu'à 20-64%, la diminution de la masse volumique à 23-49%, la baisse de la résistance en 5 et même plus de fois.

Dans les milieux neutres (II modèle) la destruction des roches quartzo-calcite-dolomitiques est faible car la capacité de déplacement et le lessivage totale sont faibles. Cela a influencé sur les indices des propriétés physico-mécaniques des roches qui sont restés les mêmes. La dite solution de carbonates même très faible provoque la destruction des liens structuraux des roches chlorito-carbonates et sollicite la formation de la phase dispersif de la composition chlorito-argileuse.

Ainsi, la situation générale permet de constater, que:
- l'intensité d'altération des roches formant les zones de schissténité et de breches est plus grande en comparaison avec les porphyrites basaltiques; que ce sont ces zones qui constituent les points faibles du massif et qui servent des voies par lesquelles l'altération pénètre aux grandes profondeurs en donnant naissance aux croûtes linéaires; les zones chystenses présentent les conditions plus favorables pour la formation des croûtes
lineaires d'altération dans les milieux acides aussi bien que dans les milieux neutres; dans les zones des breches le milieu acide est favorable pour le développement du processus carstique; la constatation purement pratique: la diminution de la pente pour en assurer la stabilité peut être inutile, car les pente douces augmentent l'infiltration qui à son tour peut intensifier l'altération (toutes les autres conditions restent les mêmes), et par conséquence la perte de sa stabilité.

Bibliographie:
SEISMIC EVALUATION OF ROCK ELASTICITY IN TUNNEL MEDIA

L'EVALUATION SEISMIQUE POUR ELASTICITE DE ROCHE PAR LE MOYEN DE TUNNEL

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ABSTRACT

Analytical formalisation and correlation of seismic wave velocities with elastic parameters of rock media is presented in this paper. The propagation of compressional and shearing stresses in the elastic wave motion is resolved in terms of the displacement vector potentials. Seismic recordings with three component geophones determine the primary and shear wave velocities of the rock formations which are analysed in correlation with the Young's Modulus and Poisson's Ratio of the media. Rock status is of prime consideration in the feasibility and design studies of any engineering project. Seismic velocity two dimensional mapping provide indirect measure of subsurface stratification and vital structures. The technique of seismic evaluation of rock competency, compressibility, load bearing capacity, anisotropic trends have vital field applications in simulating the litho-variations, shear zones, and void spaces along the tunnel sections and dam sites particularly in the rugged Himalayan terrains. Theoretical curves are simulated for evaluating the rock elasticity parameter and status from the seismic velocities distribution and trends.

ROCK ELASTICITY

Strain components are homogenous linear functions of the stress for perfectly elastic rock media (Hooke's Law); but beyond the 'proportional limits' the functions are generated by the nonlinear strain
theory. The propagation of compressional and shearing stresses in wave-motion of rocks are analytically identifiable in terms of displacement vector potentials. In isotropic rock materials the stress and strain tensors have the same principal axes showing characteristic linear tensor relation of the type:

\[ P_{ik} = \lambda \delta_{ik} + 2 \mu e_{ik} \]

where \( P_{ik} \) is stress tensor, \( e_{ik} \) is strain tensor, \( \delta_{ik} \) is Kronecker's constant involving the elastic moduli; Young's Modulus \( E \) and Poisson's Ratio \( \nu \) as \( \lambda = \sigma E/(1+\nu) (1-2\nu) \) and \( \mu = E/(1+\nu) \).

Young's Modulus \( E \) is measured ratio of acting longitudinal stress (compressive or tensile) to corresponding relative strain; and Poisson's Ratio Indicates ratio of relative longitudinal strain \( (dL/L) \) to lateral strain. The rigidity (shear) modulus \( G \) expresses ratio of shear stress to corresponding strain. In a state of volume stress a relation between compression and proportional change of volume is indicated by the Bulk Modulus \( K \). A number of interrelations are given between the different elastic moduli of rock formations.

Analysing the stress components, nature of unbalanced surface force on volume element and point displacement vector potentials it has been possible (Grant-West) to derive the standard wave vector equation for propagation of longitudinal and transverse waves in rock media in the form:

\[ P \frac{d^2 \Theta}{dt^2} = (\lambda + 2\mu) \cdot (\text{DEL})^2 \theta \quad \ldots (2.3) \]

\[ P \frac{d^2 (\text{curl} \ u)}{dt^2} = U (\text{DEL})^2 (\text{curl} \ u) \quad \ldots (2.4) \]

These are the vector relations for propagation of elastic dilatation in isotropic rock media with longitudinal velocity

\[ V_L = \sqrt{\frac{(\lambda + 2\mu)}{\rho}} \]

and transverse waves with velocity \( V_T = \sqrt{\frac{(\mu)}{\rho}} \). Primary waves propagation involve the principal stresses, and shear waves of the shearing stresses at different speeds. It is also the basis of evaluating the dynamic elastic moduli by recording the P and S wave speeds in the rock formations.

The displacement vector is separated into dilational and rotational components satisfying P and S wave equations. These are body waves, whereas at the free surface boundary there are surface waves (Rayleigh and Love wave) characterised by logarithmic decrease in amplitude with depth and velocity of the order of 0.32 \((U/p)^2\).

A broad variety of geological materials like clays, sands, silts and shales do not behave as perfectly elastic under any kind of stresses, and certain modifications of viscosity terms is required in the rock mechanics relations (Srivastava, V.N. 1969) for identifying the wave motions. The mechanical behaviour of such viscoelastic materials does not modify the theory significantly.

Geological formations consolidated through successive stages have an anisotropy (two dimensional symmetry) in elasticity. In such media a pure P wave can be propagated along and perpendicular to axis of symmetry both with different velocities; in other directions there is coupling of shear waves. In geological materials P wave velocities may vary by few percent with change of direction; and P, SH, SV waves are separated identifiable.

Due to compressive loads earth materials harden giving rise to both horizontal and vertical velocity gradients (sandstone, shales calcification) so that P and S waves are coupled in complicated way. In vertically inhomogeneous media the SH motion separates from the coupled P-SV motion. Random inhomogeneity is common in the geological settings; and assuming small random Variations in the mean value of \( V_p \) it has been shown that attenuation due to scattering increases rapidly with decreasing wavelength. Due to this factor the earth materials almost act as high cut filter to seismic signals, applying strong attenuation to all frequencies above 100 cps. The wave motion equations indicate that the elastic wave velocities in rocks are directly proportional to \( (E)^2 \) and inversely to \( (\rho)^2 \), but the range of density variations (1.5 to 3) is limited so that the elastic moduli of rock formations are comparably controlled by the velocity changes in comparison to density.

Seismic velocities of geological formations change less with depth and greater geological age. Cementation of elastic sediments by mineral solutions have more influence on elastic constants. Metamorphic rocks have higher elasticity than those from which they are derived; and are different in direction of texture. In shales...
almost 50% change in seismic velocity along and perpendicular to stratification is reported by Snell.

Any decrease in silica content of igneous rocks is reported to increase their elastic constants, but variations with degree and depth to its crystallisation are less pronounced. Similarly petrological composition texture and history has marked influence on elasticity of sedimentary rock formations. Rock porosity and decomposition factors tend to reduce the elasticity. Analytical correlation of the elastic parameters with in situ rock formation velocities is discussed and formalised for rapidly evaluating the subsurface rock strength and load bearing capacity in the feasibility studies of engineering projects.

**DYNAMIC ELASTIC MODULI DERIVATIONS**

Rock in situ elastic parameters may be studied by recording of 'Primary' and 'Secondary' wave velocities and using the three component sensitive geophones. The ratio \( V_p/V_s \) of \( P \) and \( S \) waves velocities of rock formations determines the Poisson's Ratio and \( (E) \) Young's Modulus as

\[
\frac{1}{2}(\frac{V_p}{V_s})^2 - 1
\]

Poisson's Ratio \( \sigma = \frac{1}{2}(\frac{V_p}{V_s})^2 - 1 \) (1)

Young's Modulus \( E = V_p^2 \times p \times (1- \frac{2\sigma^2}{1-\sigma}) \) (11)

In this paper curves are presented for the Poisson's Ratio \( \sigma \) in terms of the velocities ratio \( (V_p/V_s) \) by the set

\[
\begin{align*}
Poisson's Ratio & \quad \sigma = 0.1 \quad 0.2 \\
& \quad 0.3 \quad 0.4 \quad 0.5 \\
V_p/V_s & = 1.45 \quad 1.5 \quad 1.63 \quad 1.87 \quad 2.45
\end{align*}
\]

Another set of curves is given for evaluating the Young's Modulus \( E \) from the formation velocity \( V \), variations from 1 to 8 km/sec; and the following set of variables \( (p, \sigma) \) generally observed for the rock formations in the engineering geological studies:

\[
\begin{align*}
Density \ p & = 1.5 \quad 2.0 \quad 2.5 \quad 3.0 \quad \text{(gm/cc)} \\
Poisson's Ratio \ \sigma & = 0.2 \quad 0.3 \quad 0.4 \quad 0.45
\end{align*}
\]

It is possible to estimate the band width of the Young's Modulus \( E \) of various geological formations from these curves; by determining the \( P \) wave velocities and limits of density and Poisson's Ratio variations. The seismic stratification of the engineering geological domains in terms of the surface material, overburden and bedrock characteristic velocities (Srivastava, V.N. 1981) provide parametric guidelines for evaluating the dynamic elastic moduli of in situ rock formations. Under the static loads the Young's Modulus is usually less than that with dynamic stresses; and the differences may even be upto 35% or
Fig. 2 A & B. Curves drawn on the basis of the correlation parameters (Appendix II) of P-wave velocity, \( V_p \) with the Young's Modulus of rock formations. In the set A curves are for lower density ranges 1.5, 2.0, and 2.5 with Poisson's Ratio 0.4, 0.45 (soft formations); and set B for densities 1.5 to 3.0 and Poisson's Ratio 0.2, 0.3.

more. It is mainly due to processes of elastic after effect and relaxation phenomena that occur under static load; and to microstructures jointing porosity, etc. increasing rock strains under heavy loads.

A correlation between the dynamic and static values of the Young's Modulus has been given as

\[
E_{\text{dynamic}} = 0.3 E_{\text{static}} + 0.97
\]  

(III)

E values for the rocks in state of triaxial compression is considerably higher due to pores closing and increases of contact areas of grains. The static modulus of elasticity rises sharply with increase of triaxial stress by factor of about 3 at pressures upto 1000 atm; but the corresponding dynamic modulus increases by 50-60% only. The moduli of elasticity characterise the rigidity of rocks and their capacity to resist external influences. In majority of solid bodies the Young's Modulus is of the order \( 10^4 \) to \( 5 \times 10^5 \) kg/cm²;

being greater in metals than in rocks. It is very high in ferro-magnesian and ore minerals (olivine, garnet, pyrite upto \( 2.7 \times 10^6 \) kg/cm²). Mineral composition exerts a great influence on the elastic properties of porous rocks. The Poisson's Ratio for majority of rocks lies between 0.2 and 0.4. Quartz has anomalous value of 0.07 on account of it's three dimensional lattice structure. High values of Poisson's Ratio \( \sigma \) (0.35 to 0.45) are characteristic of the basic rocks; and 85% of the rocks with specific gravity less than 2.7 g/cm³ have Poisson's Ratio less than 0.25.

Griffith expressed the tensile and compressive strength of rocks by

\[
T_{\text{ten}} = \left(2 E \cdot W_0 / 11 a\right)^{1/2}
\]  

IV

\[
T_{\text{com}} = 8 T_{\text{ten}}
\]

E = Modulus of elasticity;

\( W_0 \) = Specific surface tension of rocks

a = Half the length of greatest fracture.

IV.266
Mineral composition has significant factor in rock strength. Quartz has maximum compressive strength of about 5000 kg/cm² as compared to 2-5 x 10³ kg/cm² for the ferromagnesian minerals, feldspar, olivine pyroxenes, etc. Calcite shows compressive strength of 100-200 kg/cm² only. Rocks having quartz matrix (sp. gr. 2.65-2.75 gm/cm³) show much higher compressive strength in comparison to those having calcite-mica contents. Rocks can withstand compressive stresses much better than tensile stresses which is not even 10% of the compressive strength. Elastic moduli and strength constants of rock formations common in engineering geological studies are appended in this paper; and the curves for evaluating dynamic elastic parameters are also presented. Seismic techniques for evaluation of elasticity and strength properties of rock materials has been effectively used in various tunnel sections and dam sites of major projects in the rugged Himalayan terrain for studying the nature of rock strata, weak zones, structural trends/follations pattern and foundation grade rock material etc. It has been able to provide seismic stratification measures and correlation significant for the project design and feasibility considerations.

SUMMARY AND CONCLUSIONS

Evaluation of dynamic elastic properties of in situ rock formations is based on determining the P and S wave velocities, using the three component sensitive geophones. Curves are presented for estimating the Young's Modulus and Poisson's Ratio of in situ geological materials from the longitudinal and shear wave velocities; with the density variations (ρ = 1.5, 2.0, 2.5, 3.0) and the Poisson's Ratio values (σ = 0.2, 0.3, 0.4, 0.45) generally encountered for the rocks in the engineering projects.

Seismic stratification of the engineering geological domains has been discussed in a different paper (Srivastava, V.N., 1981) so that the relevant elastic parameters—Young's Modulus, E; Poisson's Ratio σ and Bulk Modulus of Rigidity etc. of the surface materials (Vp = 200-1000 m/s); overburden section (Vp = 1100-2000 m/s) and the bedrock materials (Vp = 2500-6000 m/s) can be estimated for the design and feasibility consideration. Seismic velocities and anisotropic patterns are useful in correlating the rock fracture and joints system. Some basic data on the static and dynamic elasti-

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REFERENCES

### ROCK MECHANICS DATA AND ANISOTROPIC FACTORS

<table>
<thead>
<tr>
<th>Rock</th>
<th>Young's Modulus $E$ x $10^{-5}$ kg/cm²</th>
<th>Poisson's Ratio</th>
<th>Bulk Modulus $K$ x $10^{-5}$ kg/cm²</th>
<th>Shear Modulus $G$ x $10^{-5}$ kg/cm²</th>
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### SEISMIC CATEGORISATION OF GEOLOGICAL MATERIALS

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<th>Geological formations</th>
<th>Seismic wave velocity (m/s)</th>
<th>Young's Modulus $E$ x $10^{-5}$ kg/cm²</th>
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IV.268
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SEISMIC VELOCITY AND YOUNG'S MODULUS CORRELATION PARAMETERS

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SEISMOLOGICAL STUDY OF THE ROCK BURSTS AT THE KOLAR GOLD FIELD, INDIA

ETUDE SEISMOLOGIQUE DES EXPLOSIONS DE ROCHE A KOLAR GOLD FIELD (INDE)

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North Eastern Council
Shillong
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ABSTRACT

Seismographic records of rock bursts occurring at the Kolar Gold Field, India, are systematically available for over half a century. Analysis of these records of the Wiechert seismograph offers an unique opportunity of assessing the pattern of variation of occurrence of rock bursts, influence of geotectonics in the area on the same and stress level and b-values (logN = a-bM Gutenberg - Richter relationship) associated with larger rock bursts upto Richter magnitude (M) 5.0 prevalent in the area. Though there are innumerable number of rock bursts recorded since installation of the Wiechert seismograph at the inception of this century, there also occurred, at long intervals, some very large rock bursts (M ~ 5.0) which were recorded in seismographs at several thousand kilometers away from Kolar.

There have been a few destructive larger rock bursts in the early seventies when ground intensities exceeded VI IM scale, thus causing damages to the structures in the area. Detailed studies of the occurrence of the rock bursts, amount of mining and b-values suggest that the larger rock bursts are preceded by low b-values as in case of earthquakes. Also significant natural earthquakes in the peninsular shield seem to follow low b-values obtained from the rock bursts. Thus, this association of rock bursts and the natural earthquakes is of great significance indicating that the geotectonic pattern of the peninsular shield could influence both the natural earthquakes and occurrence of the rock bursts as well apart from mining activity.

ABSTRAIT

Les enregistrements séismographiques des explosions de roche arrivant à Kolar Gold Field, Inde, sont systématiquement disponible pour plus d'une moitié de siècle.
L'analyse de ces sismographies de Wiechart offre une occasion unique d'évaluer les types des variations de l'occurrence des explosions de roche, l'influence des géotectoniques dans la région sur le même, le niveau de contrainte et les b-valeurs ($\log N = a - bM$ règle de Gutenberg - Richter) associées à plus nombreuses explosions de roche jusqu'à la grandeur de Richter (M) 5-0 prédominante dans la région. Quoiqu'il y ait un nombre innumerable des explosions de roche enregistrées depuis l'installation du sismographe de Wiechart au commencement de ce Siècle, la sont arrivées aussi, aux longs intervalles, quelques fortes explosions de roche ($M>5.0$) qui ont été enregistrées dans les sismographes à divers milles Kilomètres loins de Kolar.

Il y avait quelques plus grandes explosions de roche destructives au commencement des soixante-dix quand les intensités de terre ont dépassé l'échelle de VI MM, causant ainsi les dégâts aux structures dans la région. Les études détaillées de l'occurrence des explosions de roche, la quantité minière et les b-valeurs suggèrent que les plus fortes explosions de roche sont précédées des b-valeurs plus basses comme dans le cas des tremblements de terre. Aussi, les tremblements de terre significatifs, naturels dans le bouclier péninsulaire semblent suivre les b-valeurs basses obtenues des explosions de roche et des tremblements de terre. Ainsi, cette association des explosions de roche et des tremblements de terre naturels est d'une grande importance indiquant que le type géotectonique du bouclier péninsulaire pourrait influer à la fois les tremblements de terre naturels et les occurrences des explosions de roche en dehors de l'activité minière.

'Rock bursts', 'coal bumps', 'out bursts' etc. are well known phenomena following mining operations, specially involving deeper strata. Minor disturbances in mines such as rock falls etc. involving readjustments subsequent to mining are common even in shallow mines; but in mining involving deeper horizons, large rock bursts, rock falls and rock slips of unprecedented dimensions could take place (Milne and Berry, 1976).

These larger rock blasts could disrupt the mining operations seriously and the same could be associated with large scale rock falls, rock slips in mining horizons (McGarr, 1971; Cook, 1976).

The intense ground movements could, on the other hand, cause wide spread damages to residential and other structures in the mining area. Though smallest of the rock bursts (Blake et al, 1974) could be recorded only locally with very highly sensitive equipments having frequency response up to $10^4$ Hz, the larger ones could be recorded by seismographic instruments stationed at few thousand kilometers.

Some of the largest rock bursts in South African deep Gold mines could be equivalent to 4.0 to 5.0 in Richter earthquake magnitude scale and could be felt very strongly near the mining area and have been recorded at few thousand kilometers from the source. Some of the recent very large rock bursts in 1973 at Kolar.
Gold field (KGF) caused severe damages to mining tunnels underground and to residential complex and other structures situated nearby. These rock bursts have been felt over about hundred kilometers or so and have been recorded practically at all the seismological stations in India and neighbourhood, even upto Quetta in Pakistan. The magnitude of one of the largest rock bursts in 1973 was 4.6 in Richter Scale (Ghah et al, 1981).

An Wiechart seismograph of magnification about 80 was in operation at the Gold Field for the past about 70 years or so almost continuously giving most vital data of quantitative record and observation of rock burst occurrence in the Gold Field area. The low magnification of the seismograph reduces its capability of recording the rock bursts smaller than Richter magnitude 2.0 or so but in view of availability of rock burst observations over such a long period, the data are of great use for studying the significance of variations of rock burst frequency and energy, specially the occurrence of the largest rock bursts whose energy compares with medium size earthquakes commonly prevalent in the southern shield of India. Except the marginal areas of the shield, the interior part of the Southern shield is highly aseismic, and only scattered low magnitude earthquakes occur very infrequently along some of the prominent faults in the area. The marginal areas of the shield, on the other hand, is moderately seismic including occurrence of swarm like activity in some parts such as in Godavari valley, Nilgiri Hills, some areas in Tapi-Narmada graben system etc. The historically largest earthquakes in the shield area have been the Koyna earthquake (M = 7.0) of Dec. 10, 1967 near the Koyna dam site. An aftershock at Koyna dam was of magnitude 5.2 (1973). The other prominent recent earthquakes in the marginal area of the shield have been the Bellary earthquake, 1843 (M = 5.8); Coimbatore earthquake, 1900 (M = 6.0); Satpura earthquake, 1938 (M = 6.2); Kothagudum earthquake, 1969 (M = 5.7); Brouch earthquake, 1970 (M = 5.4) and Shimoga earthquake, 1975 (M = 5.0). The seismicity of the shield broadly investigated by Varma et al (1970), could be described as moderate with very infrequent occurrence of earthquakes of magnitude upto 6.0. Similar moderate activities along margins of the other shield areas are also present.

In addition to the above natural seismic activities, the marginal areas of the shield have exhibited 'triggering phenomena' in respect of seismicity following impoundment of certain reservoirs specially Koyna and Iduki. This favourable geohydrological and geotectonic environment of the marginal areas responsible for 'triggering phenomena' requires detailed study. The moderately low tectonic environment of the shield could be main contributory factor responsible for 'triggering phenomena' though other factors responsible for this peculiar phenomena have not so far been isolated.

The Kolar Gold Field (13°0'N;
78°06′E) is situated in a highly seismic part of the interior of the peninsular shield. No sizeable earthquake excluding the rock bursts is known to have occurred in the vicinity of the Gold Field in recent times. Similar observations have been observed in respect of occurrence of rock bursts in South Africa and Canadian Shield. Medium earthquakes as prevalent in the peninsular India, specially along its marginal areas, and the largest rock bursts at Kolar Gold Field compare favourably both in respect of magnitudes and frequency of occurrence. Thus the occurrence of large rock bursts commonly observed in such seismic areas, requires indextudy regarding mechanics of the 'triggering phenomena' as commonly believed responsible for their origin. The magnitude and frequency of occurrence of the medium size earthquakes in peninsular margin and the largest rock burst at Kolar being comparable, the occurrence of any one of them is expected to influence the other—a fact which could be of great tectonophysical significance in understanding the inter-dependence of large rock bursts and medium earthquakes. Thus the study of rock burst history both in respect of magnitude and frequency of occurrence at Kolar could indicate seismogenic pattern of the peninsular India, i.e. time variation of geotectonics. The long period data of rock burst phenomena at Kolar (~70 years) could thus be an ideal data base for such study.

The data base as mentioned earlier has been the record of rock bursts in Wiechart mechanical seismograph installed in the Gold Field area for about 70 years or so. This being a direct inking mechanical seismograph, suffers from frictioanal effects. The data have been subjected to usual seismological analysis, namely, frequency of events (rock bursts), occurrence of very large events, seism-statistical analysis specially application of Gutenberg-Richter or Ishimoto-Ida statistics and evaluation of probability of occurrence of the largest events etc.

\[ \log N = a - bM \]  \hspace{1cm} \text{(1)}

where \( N \) = number of earthquakes of magnitudes \( M \leq M \)
\( a, b \) = arbitrary constants

\[ \log N = a - b \log \frac{A}{A_0} \]

where \( \log A_0 \) is the constant depending on distance as prescribed by Richter.

\[ \log N = a + b \log A_0 - b \log A \]

\[ \log N = K - b \log A \]  \hspace{1cm} \text{(2)}

where \( K = a + b \log A_0 \).

Hence the constant \( b \) can be evaluated periodically from eqn.(2) from rock burst records of the Wiechart seismograph. In actual practice, one thousand and five thousand events (rock bursts) have been used for evaluation \( b \)-values at different periods so that time variation of this statistical coefficient \( b \) in equation (2) could be obtained to assess the probability of occurrence of largest events. It is known that the coefficients \( a \) and \( b \) have parallel variations, and hence only \( b \) values have been used in this paper.
Fig. 1 shows record of a rock burst of magnitude about 1.0 in a high gain micro-seismograph installed at the Kolar Gold Field. The closeness of the epicentre of the rock burst could be ascertained from very small time difference (∼0.1 second) between body wave phases, P and S. A short period, about a month, survey of rock bursts was made with the help of the above micro-seismograph to assess the characteristics of the rock bursts such as, nature of recorded phases, magnitudes of the rock bursts from duration of the records (as usually the practice in micro-seismology), mechanism of the events etc. Magnitude of the largest events recorded in the micro-seismograph was 2.5 while some smallest ones could be easily − 1.0. In view of local noise level due to mining and other activities, the micro-seismograph could not be set to very high gain.

Figs. 2 and 3 show the frequency of occurrence of rock bursts, b values, amount of mining in tonnage and largest rock bursts and earthquakes in peninsular India during the period under investigated. It is interesting to note that both the largest rock bursts and the earthquakes occur generally following low b-values as in tectonically active regions (Ghia, 1979; Padale et al., 1979). The low b-values indicate high tectonic stress and hence the occurrence of largest rock bursts could be expected. The rock bursts could also be expected to release the stress thereby increasing the b-values. Thus variation of b-values with time could thus be used to predict the largest rock bursts − the low b-values could be expected to precede largest rock bursts.

As mentioned earlier, the earthquakes in peninsular India, vide Fig.3, occur simultaneously with the largest rock bursts and follow the low b-values obtained from rock burst population recorded at Kolar Gold Field. These observations on occurrence of earthquakes, rock bursts and b-values obtained from rock bursts, directly confirm that largest rock bursts and earthquakes in peninsular India are both controlled by broad geotectonics prevalent over the peninsula as the b-values computed from rock burst population could be used to correlate occurrences of both rock bursts and earthquakes as evident from Fig.3.

The rock bursts like the earthquakes follow the seismo-statistical law of Gutenberg and Richter and hence the phenomena has close relationship with the earthquake processes. Not only the broad statistical similarity, two processes are analogous in detail also such as body wave phases P and S, polarisation of energy, range of frequency of ground vibrations and duration of vibratory motion which is an indication, like earthquake, of energy content of the event. The fault plane solutions obtained for some of the largest rock bursts recorded at wide distances broadly indicate the direction of faulting in the area. Thus, the similarity of mechanism of the two processes, namely rock bursts and earthquakes, could be made use of in
analysis of the rock bursts specially in prediction of the same. The rock burst phenomenon at KGF has been occurring within the mining complex following mining operations, similar to other artificial seismicity. The two trigger:
ing agents could be as follows:

1. Effects of blasting used in mining operations.
2. Superimposed stress field due to formation of cavities following removal of mined material.

Brune (1970) has suggested the following type of relationship for induced seismicity due to stress modification:

\[ P = P_0 - P_1 \]

(effective (tectonic (induced stress)) stress)

where \( P_1 \), the induced stress, could be due to any artificial process, namely, reservoir load, mining operations, fluid extraction and injection, thermal process etc.

The influences of the above two triggering agents, combined or separately, in affecting changes in tectonic stress \( P_0 \) may result in lowering the effective stress \( P \) thereby inducing these artificial rock bursts or rock-slip. The induced stress \( P_1 \) could be as a result of number of factors as indicated above. In case of high tectonic stress where \( P_0 > P_1 \), \( P_0 - P_1 \)
i.e. \( P \) is always large and hence, no rock-slip could take place while for \( P_0 \sim P_1 \) the effective stress \( P_0 - P_1 = P \) could be reduced sufficiently thereby causing rock-slip. Hence, moderately low tectonic stress field like that at KGF and Southern Indian shield, is favourable for induced seismicity. The maximum magnitude of the induced earthquake may be equivalent to stress drop of the order \( P_0 - P_1 = P \). In case \( P_0 \) is very small, the magnitude of induced seismicity will also be very small i.e. micro-earthquake activity. These theoretical deductions have been broadly corroborated from observations of induced seismicity due to different triggering agents in varied geotectonic environment.

Thus measurements of ambient tectonic stress through hydrofracturing experiments could lead to basic understanding of the cause and extent of induced seismicity. Also continuous observations of stress in mining strain could be a very effective method of predicting the size of impending induced earthquake as maximum magnitude of induced earthquake could be estimated from stress drop equivalent to effective stress \( P_0 - P_1 \) (i.e. \( P \)).

As mentioned earlier, the largest rock bursts in Fig.3 follow the low b values computed from rock burst population. It is interesting to note that the larger earthquakes in peninsular India, Fig.3, also follow low b values obtained from rock burst population. Similar observations have been made from earthquake data when large earthquakes follow low b values. Thus rock bursts at KGF and earthquakes in peninsular India are dominantly controlled by common geotectonic pattern prevalent in the shield area.

The larger rock bursts at KGF and the larger earthquake in peninsular shield are of similar magnitude range (4.0 to 6.0) and hence the occurrence of one will
naturally influence the other. It is thus evident that common stress field influences both the earthquakes and rock bursts. The peninsular shield acting as a mini-plate could transmit the stress field very effectively over long distances. Thus the mini-plate (Indian shield) acts as a highly elastic and rigid body as has been the case with other plates.

Thus the seism-statistical analysis (b-values) of rock bursts could lead to basic understanding of the rock bursts at KGF as well as earthquakes in peninsular shield (mini-plate). 'Rock burst' phenomenon at KGF could be treated as 'tectonic thermometer' of the Indian peninsular shield which acts as a highly elastic and rigid mini-plate.

Acknowledgement

I am greatly indebted to my former co-workers in Central Water and Power Research Station, Pune, India who have been responsible for much of the ideas developed in this paper.

References


IV.277
Fig. 1 Microseismogram of rock burst ($M \approx 1.0$) recorded at Kolar Gold Field.

Fig. 2 Tonnage mined, b-values and rock burst activity at Kolar Gold.
Fig. 3 Tonnage mined, b-values and rock burst activity at Kolar Gold Field and significant earthquakes in peninsular shield of India.
A PRELIMINARY STUDY ON THE PHENOMENON OF CORE DISCING

ETUDE PRIMAIRE SUR LE PHENOMENE DE GATEAU FRACTURE DES CAROTTES DE SONDAGE

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Wuhan Institute of Hydraulic and Electric Engineering
Wuhan, China

ABSTRACT

In this paper the investigation is based on core discing of syenite at Ertan Hydro Power Station. The main conclusions are

1. The phenomenon is a result of a high natural stress (residual stress) field in rock mass. The fractured discs of syenite core can be subdivided into three zones: a central zone (about 0.46R), an intermediate zone (0.46R-0.81R) and an outer zone. Where R is borehole radius three diameters of borehole: φ56, φ108 and φ127 mm are used.

2. The increase of the ductility of rock is due to the high natural stress in the rock mass. Therefore, according to calculation of fracture mechanics for fractured discs of syenite, $K_{IC} = 100, 156$ and 176 kg/cm² correspond to above-mentioned different diameters of borehole cores are obtained respectively. They are near and higher than that obtained by three-point bending test in laboratory ($K_{IC} = 118.1$ kg/cm²).

3. A natural stress in rock mass of $\sigma_1 = 240-250$ kg/cm² is obtained according to the calculation of fractured core disc, which well coincides with the in situ measured result $\sigma_1 = 260$ kg/cm².

ABSTRACT

L’étude et l’analyse dans cet article sont basées sur les carottages du gateau fracture de syenite dans la station hydro-électrique. Les conclusion principales sont:

1. Le phénomène est relation avec le résultat de haute contrainte naturelle (la contrainte résiduelle) dans les massifs rocheux. Le gateau fracture de la carotte de syenite peut être subdivisé en trois zones: la zone centrale (0.46R), la zone intermédiaire (0.46R-0.81R) et la zone extérieure. On R est rayon de forage. Il y a trois diamètres de forage: φ56, φ108 et 127 mm.

2. L’augmentation de ductilité des massifs rocheux est due à la haute contrainte naturelle dans les massifs rocheux, par conséquent, selon le calcul de mécanique de fracture pour le gateau fracture, $K_{IC} = 100, 156$ et 176 kg/cm² correspondant à la diamètre différente mentionnée de forage sont respectivement obtenus. Leurs résultat est plus grand que celui ($K_{IC} = 118.1$ kg/cm²) d’essai a la flexion de trois points dans le laboratoire.

3. Selon le calcul de gateau fracture des carottages de sondage, le résultat de contrainte naturelle dans les massifs rocheux, $\sigma_1 = 240-250$ kg/cm² et sa valeur de mesure in situ, $\sigma_1 = 260$ kg/cm², sont obtenus. Tous les deux sont approches.

IV.281
1. INTRODUCTION

In recent years, a phenomenon of core discing ("discing" in short) is observed in some hydro-electric investigations in China. It has been little studied from the engineering geological and rock mechanics point of view. In this paper the author would try to make a preliminary study to cause further research on this theme.

2. THE PHENOMENON OF "DISCING"

The geological background of occurrence of the phenomenon "discing" may be various, but it has a common condition of having a high ground stress field and hard intact rock quality. This characteristic confines the phenomenon mostly occurring in the igneous rocks, but sometimes may be in some sedimentary or metamorphic rocks, too. This phenomenon was recorded during the process of hydro-electric engineering geological explorations in South-West China, for example, in the syenite at the Ertan Hydro-electric Power Station, the basalt of Baihetan, the granite of Daigangshan, the dolomite of Lubuye, etc. This phenomenon occurs not only under the natural internal stress conditions, but also in the residual rock stress field created by underground nuclear explosion both abroad and in the home (Lu, 1980). This phenomenon clearly indicates that rock mass possesses a property of storing high residual stress.

The primary characteristic of this phenomenon includes that the cores taken from the drill holes rupture into discs covering one by one, the thickness of the discs is approximately the same for every disc and has some relation with the diameter of the drill core, usually about 1/4 of the core diameter; the surface of the cores is approximately horizontal, always a fresh fracture, without any marks of weathering and water corrosion. On the fractured surface of the Ertan syenite core the cleavage surface of feldspar and pyroxene keep their looking glass and bright. The slight scratch and tension ridge clearly appeared on the disc surface. Any two of the nearby upper and lower discs cannot be reset again. This is quite similar to the case of unrecoverable rock bursting fragments. It indicates that core has dilation during discing.

This "discing" phenomenon encountered in some hydro-electric power stations in South-West China mostly occurred in the river-bed area at the bottom of a valley or the end of a slope, which is an area of stress concentration as a rule (Tao, 1976).

According to the statistic data, the dimensions of the disc from the syenite of the Ertan Power Station are shown in Table 1. From the sketch of the core (Fig. 1) it can be seen that on the core surface only the scratches of one direction exist, but the tension ridges have two directions and they are in a form of a circle. This is a very important feature. It indicates that core was under a complex fracture form of tension-shear or compression-shear during discing and under a process of stress release. Because core discing occurred during drilling, when it was taken out it already became discs.

3. RESIDUAL STRESS IN ROCK

The phenomenon of residual stress in rock causes wide attention of rock mechanics workers in the recent ten or more years. There are plenty of evidences showing the existing of residual stress in different rocks. Although there are some research reports on residual rock stress, generally speaking, there is a lack of comprehensive knowledge of that. This brings about some important effect to the development of rock mechanics: firstly, we must take prudent attitude towards the results of recent method of stress measurement. This is because in recent measurements we can obtain only the complicated state of stress from the residual and attached stresses in rock mass. Under the present technical conditions, it is difficult to divide the residual and the attached stresses strictly. Secondly, the existing of ununiform residual stress in rock shows that even in the case of geological time dimension these rocks may have a preliminary strength - in other words, the properties of rock processing residual stress differ from which of pure viscous material. Maybe there is some relation between residual rock stress and properties changing from time, though the concrete characteristic of such relation is unclear up to now.

The phenomenon of core discing discovered during the engineering geological investigations in many Chinese hydro-electric power stations may be helpful for the research of residual stress.

Take Fig. 1 as an example, we noticed that there is neither scratch, nor tension ridge in the central part of disc. The radius of this zone is $a = 0.46R$. Where $R$ is the radius of disc. And there is an annular zone along the periphery of the core also without scratch and tension.

IV. 282
Table 1. Statistic data of disc dimensions of syenite from Ertan Hydro-electric Power Station

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<th>Diameter of drill holes (mm)</th>
<th>Amount of cores taken into statistic</th>
<th>Dimension of cores (mm)</th>
<th>h/D</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Diameter D on average</td>
<td>Height h on average</td>
<td></td>
</tr>
<tr>
<td>$\phi 56$</td>
<td>38</td>
<td>38.766</td>
<td>9.953</td>
<td>0.257</td>
</tr>
<tr>
<td>$\phi 108$</td>
<td>19</td>
<td>80.955</td>
<td>21.432</td>
<td>0.265</td>
</tr>
<tr>
<td>$\phi 127$</td>
<td>26</td>
<td>100.765</td>
<td>27.146</td>
<td>0.269</td>
</tr>
</tbody>
</table>

Table 2. Division of zones of residual stress

<table>
<thead>
<tr>
<th>Rock</th>
<th>a</th>
<th>b</th>
<th>Notice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Syenite of Ertan</td>
<td>0.46R</td>
<td>0.81R</td>
<td>R-radius of drill core</td>
</tr>
<tr>
<td>Basalt</td>
<td>0.50R'</td>
<td>0.83R'</td>
<td></td>
</tr>
</tbody>
</table>

ridge. The radius of this zone r is between R and b, i.e. $R_{>r}>b$, and $b = 0.81R$. This fact is not occationary, it is quite similar to the division of zones of residual stress disspucted by Bock (1979). The three zones discribed by Bock (1979) are:

- The central zone with a diameter about 4.5 cm, predominantly with compressive stress,

- The intermediate annular zone with an internal radius of 4.5 cm and external radius of 7.5 cm, predominantly with tensile stress.

---

The outer zone predominantly with compressive stress. If take the conversion radius of the basalt cylinder $R' = 9$ cm, we can gain Table 2. From Table 2 it is seen that there is a close similarity between the two. Probably it is possible to make an estimate on fracture mechanics using this statistic material.

4. FRACTURE ESTIMATION

According to above discussed conditions of scratch and tension rise it may be considered that in the zone of $r < a$, there is no influence of stress release during drilling; but zone $R_{>r}>a$ is a zone of its influence, and there are two occasions: in the zone of $a<r<b$ when there is ultimate equilibrium the surface of disc undue normal tensile stress which approaches uniaxial tensile strength; in the zone of $R_{>r}>b$ there is some state of stress release because of increasing outside influence during drilling. Just under such stress condition the fracture expands and the disc breaks. Therefore we can carry out some estimation of its fracture toughness, when

$$K_{\text{mc}} = K_1$$

fracturing occurs. Where $K_1$ is calculated by formula (Tada et al, 1973) (Fig. 2):

$$K_1 = \frac{b (\pi b^2)}{(\pi a)^{\frac{3}{2}}} \left\{ \cos^{-1} \left( \frac{a}{b} \right) + \frac{a}{b} \sqrt{1 - \left( \frac{a}{b} \right)^2} \right\}$$

$$K_{\pi} = K_{\pi} = 0$$

Fig. 1. Sketch of core disc of Ertan Syenite.
tively. We have obtained the $K_{IC}$ value for the Ertaan syenite equal to 118.1 kg/cm$^{3/2}$ using three-point-bending test. Here the fracture toughness is near and higher than that obtained by laboratory fracture test. If we consider that Ertaan is situated in high ground stress area, the fracture toughness is enhanced, the result seems as if reasonable. In fact, this is an influence of three dimensions stress state on the toughness of rock mass. It is a possible fact. Once this fact is verified by scientific experiment, the important influence factor which may affect the stability of hard ambient rock of underground cavity in such an area may be transformed into a rheological characteristic performed by toughness of rock mass. Therefore, establishment of this fact has an important theoretical and practical significance as well.

In fact, from above calculated results it may be seen that $K_{IC}/p$ increases as the borehole diameter increases (Fig. 3). This coincides with the feature of ground stress accumulation.

$$K_{IC}/p \text{(cm}^{3/2})$$

![Graph showing the relationship between borehole diameter and $K_{IC}/p$.](image)

5. ESTIMATION OF INITIAL STRESS IN ROCK MASS

After the phenomenon of core discing has occurred in some high ground stress area some Chinese researchers are exploring how to use the phenomenon of scratch on the core surface and estimate the dimension of ground stresses according to the shear loading condition. They consider that the direction of the maximum major stress can be determined by the direction of scratches. This is a significant research subject. But after such estimation, they have obtained the initial stress of rock mass in the depth no more than 100 m equal to about

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Fig. 2. Stress - Intensity Factor.

where $p$ is the uniform tensile stress loaded on the annular zone of $a \leq r \leq b$, and $p$ is equal to the tensile strength of rock $R_t$, $a$ and $b$ are the inner and outer radii of the ring respectively. In our case of considered disc, $a=0.46R$, $b=0.81R$, where $R$ is the radius of disc.

For different drill cores of the Ertaan syenite, the values of $a$ and $b$ are shown in Table 3.

Thus we can obtain from Formula (2):

For the discs of $\phi 56$ drill cores,

$K_{IC}/p = 1.0 \text{ cm}^{3/2}$

For the discs of $\phi 108$ drill cores,

$K_{IC}/p = 1.56 \text{ cm}^{3/2}$

For the discs of $\phi 127$ drill cores,

$K_{IC}/p = 1.76 \text{ cm}^{3/2}$

Because the tensile strength $R_t$ of the Ertaan syenite is about 100 kg/cm$^2$, i.e. $p = R_t = 100 \text{ kg/cm}^2$, the $K_{IC}$ value for $\phi 56$, $\phi 108$ and $\phi 127$ borehole cores are about 100, 156 and 176 kg/cm$^{3/2}$ respectively.
Table 3. The inner and outer radii of the tension ridges of Ertan syenite discs

<table>
<thead>
<tr>
<th>Borehole diameters (mm)</th>
<th>Disc radius, R (cm)</th>
<th>Inner radius of tension ridge, a (cm)</th>
<th>Outer radius of tension ridge, b (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>φ 56</td>
<td>1.94</td>
<td>0.89</td>
<td>1.57</td>
</tr>
<tr>
<td>φ 108</td>
<td>4.05</td>
<td>1.86</td>
<td>3.28</td>
</tr>
<tr>
<td>φ 127</td>
<td>5.04</td>
<td>2.32</td>
<td>4.08</td>
</tr>
</tbody>
</table>

600~800 kg/cm². It is obviously too much. The measured results indicate that the estimated value is about 2.5~3.2 times higher than the measured one.

It has been mentioned above that there is only scratches of one direction existing on the core surface. It shows that it is reasonable to estimate the initial stresses of rock mass using shear loading condition. The problem is how to estimate. On the basis of the fracture estimation above we can assume: firstly, in the zone of r<a there is a cohesion force c existing to resist shear, but in the zone of R>r>a there is no such a force. Secondary, on the full surface of core disc, this is an internal friction force existing to resist shear. Its friction coefficient in f=tgφ, where φ is angle of internal friction, but in the annular zone of a<r<b because there is tensile stress existing, the normal stress σ₂ should be at a tensile strength R2 smaller than in any other zone, i.e. the normal stress in this zone is (σ₁−R₂). Thus, the shear strength on the half a core disc S is

\[ S = \frac{\pi b^2}{2} \cdot c + \sigma_1 \cdot f \cdot \frac{\pi}{2} \left[ R^2 - (b^2 - a^2) \right] + (\sigma_1 - R) \cdot f \cdot \frac{\pi}{2} (b^2 - a^2) \]  

(3)

Because there is σ₁ acting along the thickness h on the diameter section of core disc and due to the certain stress release during drilling, we may assume σ₁ distributes along the thickness in a triangular form, with zeno on its top, and σ₁ at its bottom (Fig. 4). Then the acting shear stress τ is obtained equal to

\[ \tau = \frac{1}{2} \cdot \sigma_1 \cdot h \cdot 2R = \sigma_1 h R \]  

(4)

In the ultimate condition we have

\[ \tau = S \]  

(5)

that is

\[ \sigma_1 \cdot h R = \frac{\pi a^2 c}{2} + \frac{\pi f}{2} \left[ R^2 - (b^2 - a^2) \right] + \frac{\pi}{2} (\sigma_1 - R) (b^2 - a^2) \]  

(6)

In this equation we have two unknown figures of σ₁ and σ₁, therefore we should find out a definite condition.

Fig. 4. The loading condition of half a disc.

The dam region of Ertan is an area of valley, the width of the valley bottom is about 100 m, the angle of the slopes is 26°~40°, and they are about 300 m higher than the river level. The phenomenon of discing occurred mostly under the surface of rock foundation in the bottom of the valley, usually with a depth of 20~100 m, and with a maximum density in the area between 20~50 m. It is obvious that it has an intimate relationship with the influence of geography. We assume that σ₁ is equal to the weight of the rock column γH (where γ is the density of the rock, take γ = 2.7 t/m³, H = 300 m), then we have

\[ \sigma_1 = \gamma H = 2.7 \times 300 = 810 \text{t/m}^3 = 81 \text{kg/cm}^2 \]

Put it into Equation (6), and according to the data taken from in-situ shear test of Ertan syenite rock cylinder, we have

\[ c = 25 \text{ kg/cm}^2 \]
\[ f = 2.10 \]
\[ R_c = 100 \text{ kg/cm}^2 \]

then σ₁ can be calculated.

IV.285
Table 4. Measured results of three-dimensions rock stress at Ertan Power Station

<table>
<thead>
<tr>
<th>Rock</th>
<th>Situation of measured points</th>
<th>$\sigma_1$ kgf/cm$^2$</th>
<th>Azimuth $\alpha$</th>
<th>Dip $\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Syenite</td>
<td>Adit No.3 of Tunel No. 2 in the left bank</td>
<td>$\sigma_2 = 260$</td>
<td>$N_{34^0W}$</td>
<td>$23^0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\sigma_3 = 90$</td>
<td>$N_{40^0W}$</td>
<td>$-30^0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\sigma_4 = 25$</td>
<td>$N_{83^0W}$</td>
<td>$40^0$</td>
</tr>
</tbody>
</table>

Notes: 1. The angle of elevation is considered as positive and angle of depression negative.
2. The compressive stress is considered positive.

For the discs of $\phi 56$ drill cores

$\sigma_1 = 250$ kgf/cm$^2$

For discs of $\phi 108$ drill cores

$\sigma_1 = 242$ kgf/cm$^2$

For discs of $\phi 127$ drill cores

$\sigma_1 = 240$ kgf/cm$^2$

These results quite coincide with the measured results by the Research Division of Chengdu Designing Institute of Hydroelectric Power, Ministry of Electric Power, China (Table 4).

6. CONCLUSIONS

Following above mentioned we can make some short conclusions:

1. The phenomenon of core discing is an important characteristic of rock mass in high ground stress area. It can be investigated not only in natural state, but also under artificial conditions (e.g. under ground nuclear blasting).

2. Because there are scratch and tension ridge during the process of discing, it is possible to estimate the orientation and dimension of the initial stresses according to the stress trace.

3. The method of estimation fracture toughness and initial stresses of rock mass suggested by this paper is only a preliminary one. But it may be an essentially believable one because of its good coincidence with the measured results.

4. The estimation of fracture toughness of the Ertan syenite indicates that the fracture toughness of rock mass in high stress area is slight higher than that of sample tested in laboratory. This is a new physical phenomenon which may be due to the three dimensions ground stress state and worth further researching.

7. REFERENCES


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Theme 1: Engineering geological studies for environmental evaluation and development & Auxiliary papers

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Theme 2: Engineering geological problems of tunnelling and excavation of cavities

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