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FOREWORD

Due to various reasons, more tunnels are being constructed all over the world to cater to various demands like irrigation, hydro generation, drinking water, industrial needs, highway and railway tunnels. Migration of bulk population to major cities warrants creation of underground space to accommodate transportation communication and utility works and complexes for handling processing and storage of many kind of materials. Many of the largest and most complex urban tunnelling projects are being implemented in the large and rapidly growing countries of Asia.

Keeping in view the relevance and importance of the topic and to provide a forum for exchange of experiences for flow of advanced technology, the Central Board of Irrigation and Power in conjunction with the Adhering Committee of International Tunnelling Association (India) organised Tunnelling Asia '97, from 20–24 January 1997 at New Delhi, India.

Ample input of information and experience both from the national and international scene was collected in the form of technical papers to be discussed during the conference to enable some conclusions to be drawn up to recommend concrete action both on a short-term as well as long-term perspective for more effective and efficient management of larger public-works including underground construction projects.

The organisers would like to express sincere gratitude to all the experts who have contributed papers for discussion in the conference. It is our hope that discussions at this conference will give fillip for introducing modern methods for the quick and economic execution of tunnelling projects eliminating the delays in the completion of projects. We also welcome the participation of the experts from abroad.

It is hoped that this comprehensive volume dealing with tunnelling for various purposes like Water Resources, Highways, Railways and Metro will be found useful by the engineering fraternity.

(C.V.J. Varma)
Member Secretary
Central Board of Irrigation and Power
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ENGINEERING CLASSIFICATION AND CHARACTERIZATION OF ROCK MASS FOR TUNNELLING
THE ENGINEERING CLASSIFICATION OF WEATHERED ROCK MASS

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ABSTRACT
For a good understanding of the engineering performance of the weathered rocks, an appreciation of the degree of weathering is very important. Classified grades and zones provide a framework within which strength behaviour can be interpreted and linked to engineering performance. It has always been desired that the rock mass must be assessed quantitatively rather than qualitatively which can serve as basic input in design and analysis.

In the present study an attempt has been made to study the weathering profiles and develop a rating based classification system for the assessment of weathering stages.

INTRODUCTION
The unpredictable nature of weathering enhances the complexity in the assessment of rock masses. The difficulties in the assessment of rock masses are more pronounced in the humid-subtropical countries like India where soil development is marked by weathering profiles of varying thickness.

The rock mass assessment can be explained as the judgement and analysis of engineering geological conditions and mechanical characteristics of rock mass. In practice, assessment of the weathering extent of rock is accomplished through the standard guidelines of a classification system. A number of classifications have been recommended for the assessment of weathering in rock material and mass. Most of these classifications are descriptive and use the general terms which do not adequately characterize the variable nature of the zones of weathering.

Present study reviews the existing classification systems and argues their limitations. Based on extensive field and laboratory study an approach has been developed to classify the weathered rock mass at profiles.

LITERATURE REVIEW
Many authors have described the weathering of rocks by setting up classification schemes based on visual identification of the weathering features. Most of the classifications are developed for the typical location, condition and purpose.

Two of the earliest classifications which were relevant to the engineering uses were developed by Moye (1955) and Ruxton and Berry (1957) for granitic rock from Australia and Hong Kong respectively. Later these classifications have provided the basis for other classifications. Moye's (1957) classification was based on visual characteristics and material slakability. These were followed by classifications given by Lumb (1962), Little (1969) for granite and Chandler (1969) for clays. Though they described the weathering in different scale from rock core to zones of profile but they did not appreciate the importance of difference of weathering in material and mass scale. Comparisons between these classifications are

Several professional organizations have also attempted to standardize the description of weathering for common rocks and for engineering uses. Geological Society of London (Engineering Geological Group) (Anon, 1970) has developed their classification following Moye's scheme. The numbers of grades were enhanced from six to seven. After this, the society has revised this classification several times in 1977, 1981 and very recently in 1995 (Anon 1995). International Association of Engineering Geology (IAEG, 1981) has also advanced a classification based on description of weathering features and recommended a scale of % of weathering for which no guideline has been provided. This has been directly adopted by International Society for Rock Mechanics (ISRM 1981a). Martin and Hencher (1986) reviewed the above classifications and noticed lack of definition and guidance for the description of rock material grades. They also pointed out limitations of the existing descriptive classification in a way that they involve rigid term "grade" which was used as rock type or zone of heterogenous rock mass rather than a state in scale of rock weathering. Martin and Hencher (1986) have also given some useful suggestions and guidelines for the classification of weathered rock and proposed a general zonal scheme for the weathering classification.

In some late classifications purposely developed indices have been used for the quantification of weathering state in rock material. The indices used for the quantification for different kinds which expresses influences of weathering in different properties. Moye (1957) used a crude index i.e. slakability of material at field. Hamrol (1961) devised a quantitative classification of weathering and weatherability based on quick water absorption and degree of microfracturing respectively. Lumb (1962) defined an index based on weight ratio of quartz and felspar. Another index i.e. coefficient of weathering (K) for granitic rock was suggested by Iliev (1966). This index was based on ultra sonic wave velocity in rock material. Irfan and Dearman (1978) also developed a quantitative method of assessing the grade of granite in terms of microscopic petrographic characteristics.

Some the indices were proposed for the field application and few were laboratory indices but all these were suggested for the characterization of weathering at material scale. Unavailability of any field index test which are representative at the mass scale reflects the reason and little scope for the development of any reliable rock mass weathering index.

FIELD AND LABORATORY STUDY

The profiles developed on three rocks namely granite, basalt and quartzite were carefully studied at several exposures sites. The collected samples were tested in the laboratory for engineering properties. As many as 13 profiles have been selected for the profiles study at fresh cuttings of recent quarry slopes, road cuttings and foundation excavation sites.

Field Study: Observation on Weathered Profiles

(i) Granite of Precambrian age is widely occurring as a country rock at Malanjkhand copper open pit mines and provided huge profile at quarry section of more than 90m depth. Large amount of staining, particularly at the joints and good development of soil horizons were marked in the field. Different zones were marked having different degree of weathering and jointing.

(ii) Basalt of Deccan Trap (Eocene age), exposed at Nagpur was studied at road stone quarries. Weathering profile is marked with high degree of corestone development in shallow weathered zones.

(iii) Quartzite of Precambrian age, which outcrops in and around Delhi were observed as considerably weathered up to the limited depth. Chemical weathering of quartzite rock resulted intense staining at surface and joints.

Laboratory Study

For the detailed laboratory study, the identified and collected samples were further sorted by Schmidt hammer test, carried out over the saw-cut surface of each block of rocks. Number of cylindrical, and discoidal specimens were prepared depending upon the requirement of test. Special care was taken while preparing the specimens from rocks of highly and completely weathered grades. Physical index properties
and strength properties e.g. point load, Brazilian and uniaxial compressive strength tests were conducted for number of specimens. All the tests were carried out by following the methods suggested by ISRM (1981b). Thin section studies under the stereo polarizing microscope were carried out to infer the micropetrographic and mineralogical variation due to weathering.

COMPREHENSIVE ROCK WEATHERING CLASSIFICATION

Based on study of existing rock weathering classification and observations on weathering profiles, an approach is developed to devise a rating based classification for weathered rocks. The proposed system of classification involves detail assessment of weathering by visual identification and engineering characterization in both material and mass scale. All the important weathering parameters of rock mass have been considered and expressed as relative weightage in the classification. The procedure for the classification is given in following steps:

1. Identification and characterization of weathered rock at materials scale.
2. Delineation of weathering zones by observation of marked changes in spatial distribution of weathering material.
3. Assessment of representative weathered material predominant in the delineated weathering zones.
4. Identification and description of rock mass parameters influenced by weathering.
5. Assigning the rating for different weathering zones and classification as per given range total rating for classification for weathering zones.

Identification of the Material: Classification of Weathered Rock Materials

Considering the importance of visual identification and descriptive classification of different weathered rocks in the field, five recognition factors have been selected on the basis field identification of rock materials and available in literature.

The following five recognition factors are used to classify the rock materials in six categories.

a) Discolouration and staining
b) Texture and fabric
c) Disintegration
d) Decomposition
e) Relative strength

The important observations with description and identified grades for granite are mentioned in Table 1. To provide input data in classification system and the quantitative assessment of the rock material an index (Wi) has been developed. Applicability and usefulness over the other existing indices have been discussed by Gupta and Rao (1996). Below, ranges of the value of index of weathering (Wi), quick absorption index (QAI), coefficient of weathering (K) and the rating for the classification system are given in Table 2.

Delineation of the Weathering Zone

The zones were recognized through the visual changes in spatial distribution of weathering grades. The problem of delineating the weathering zones is resolved either by dividing the exposures in small units (zones and subzones) and classifying each one separately. Other way is, after studying the material grade of each subzone intermediate rating can be selected for the zone. Since the boundaries may be gradational (overlapping) or sharp (abrupt) care must be taken in delineating the margins.

Generally, the predominant material of any grade with in the delineated zone is used to classify the zone. This approach is developed after Martin and Hencher (1986) suggestion for weathering classification in heterogeneous weathered rock mass. The spatial distribution of weathered material is assessed as volumetric percentage (to the nearest of 5%) of different weathered material and average of proportion decides the
Table 1. Classification of Weathered Granite by Visual and Micropetrographic Identification.

<table>
<thead>
<tr>
<th>Grades of Weathering</th>
<th>Visual Identification Description</th>
<th>Micropetrographic Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh Rock</td>
<td>Grey or pink coloured. No discolouration. Grains are having vitreous luster. Virtually no major cracks present.</td>
<td>Quartz grains are fresh and unfractured. Felspar sericitized and gives cloudy appearance. Microcracks are very few. Grains are closely packed.</td>
</tr>
<tr>
<td>Slightly Weathered</td>
<td>No significant staining. Dull luster of minerals. Grains are tightly bonded. Few felspars are gritty. Hair line crack are visible in small quantity.</td>
<td>Altered boundaries of sericitized felspar are visible. Biotite are partially altered. Grain boundaries are slightly open. Numerous intergranular cracks are observed.</td>
</tr>
<tr>
<td>Moderately Weathered</td>
<td>Slightly stained. Few grains are gritty. Altered microcracks are visible, but they are tight. Few felspars (plagioclase) are decomposed. Feldspar can be scratched. Sample can be broken by one firm blow of geological hammer.</td>
<td>Except sercite felspar, other plagioclase crystals are altered. Numerous intergranular crack passing through grains are visible. Width of microcracks ranges from 0.01 to 0.2 mm. Broken quartz are visible.</td>
</tr>
<tr>
<td>Highly Weathered</td>
<td>Discoloured and highly stained in to pale brown colour. Most grains are gritty and clayey. Loosely bonded and fractured grains of quartz are numerous. Microcracks filled with clays. Few felspar are undecomposed.</td>
<td>Numerous, shattered grains of quartz and undecomposed felspar. Crack width increases more than 0.2 mm. Large amount of decomposed felspar.</td>
</tr>
<tr>
<td>Completely Weathered</td>
<td>Completely discoloured. Specks of white clays are present. Very loosely bonded grains. Microfractures are open filled with clay and air. Sample can be crumbled by fingers.</td>
<td>Shattered grains appear like floating in decomposed matrix. Very wide cracks are often observed. Zig-zag pattern of cracks observed. No unaltered plagioclase is found.</td>
</tr>
<tr>
<td>Residual Soil</td>
<td>Original texture is lost. Sample becomes granular with virtually no strength.</td>
<td>Only few remanent grains of quartz and mica can be seen in decomposed ground mass. Very poor visibility of the section.</td>
</tr>
</tbody>
</table>

Table 2. Range of Weathering Indices for the Classified Weathering Grades.

<table>
<thead>
<tr>
<th>Grades of Weathering</th>
<th>W0</th>
<th>W1</th>
<th>W2</th>
<th>W3</th>
<th>W4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terms</td>
<td>Fresh</td>
<td>Slightly Weathered</td>
<td>Moderately Weathered</td>
<td>Highly Weathered</td>
<td>Completely Weathered</td>
</tr>
<tr>
<td>W,</td>
<td>80-100</td>
<td>50-80</td>
<td>25-50</td>
<td>10-25</td>
<td>&lt;10</td>
</tr>
<tr>
<td>K</td>
<td>0.0</td>
<td>0-0.4</td>
<td>0.4-0.7</td>
<td>0.7-0.8</td>
<td>0.8-1.0</td>
</tr>
<tr>
<td>QAI</td>
<td>&lt;0.2</td>
<td>0.2-1.0</td>
<td>1.0-2.0</td>
<td>2.0-4.0</td>
<td>&gt;4.0</td>
</tr>
<tr>
<td>Rating</td>
<td>30</td>
<td>25</td>
<td>15</td>
<td>7</td>
<td>3</td>
</tr>
</tbody>
</table>

predominant representative grade is used for the rating. This approach is found to be less useful at the profile where corestone development is prevailing. Such difficulty has been observed in basalt profiles very prominent and can be overcome by following a simplified consideration proposed by Dearman (1974) for the assessment of distribution of weathering in a rectangular block (as corestone and rim). Idealized diagram of the stage and distribution of weathering in a block is given below as Fig.1 (after Dearman, 1974).
It is interesting to note that a thin layer within the cube of rock is sufficient to give rise to a 50% change in to different weathered grade. This much amount of weathering is sufficient to change the rippability and excavation methods in rock mass (Dearman, 1974).

**Rock Mass Parameters Influenced by Weathering**

In the assessment of state of weathering of rock mass, study of influence of weathering on joint planes is vital part. The joints and bedding planes results the heterogeneous distribution of weathering and also makes the geotechnical behaviour highly variable and unpredictable. Roughness of joints and joint wall strength are two important parameters influenced by weathering. Strength of joint wall which reduces significantly by weathering controls the overall strength and deformational behaviour of rock mass. Barton and Choubey (1977) have explained five progressive stages of weathering that bring changes in strength of wall rocks. Though roughness of wall rock depends on type of failure along the discontinuity, weathering at joint also changes the preexisting roughness at the surface. Hencher and Richards (1989) observed that in practice weathered rock joints can exhibit higher shear strength than their less weathered counterpart. But in case of highly weathered wall where highly weathered material will act as gauge material, the strength along the wall will be reduced significantly even in absence of any filled materials. The use of Schmidt hammer test is recommended as most suitable tool for the assessment of degree of weathering at joint walls surface. In the present study state of weathering is assessed by visual identification wherever it is exposed. Number of blows of hammer at the corner of joints are used as a crude index.

Although advancement of weathering is closely related with joint spacing in any rock mass, however irregular distribution of weathering shows virtually no relationship with jointing. Bieniawski (1974), and Ramamurthy and Arora (1994) showed how the spacing of joints modifies the strength of rock mass. Separation of joints also controls the frictional strength along the joints as well as the flow of water and the rate of weathering of wall rock. In the present study consideration of filling material is also taken under this section with an assumption that joint infill is generally composed of the weathered product of same wall rock material.

**Assignment of Rating**

All the important elements of rock mass, effected by weathering, has been considered by using a rating system. Not all the parameters are of equal importance in the assessment of rock mass strength thus it is necessary to assign a numerical weightage to each parameter according to influence of weathering on it. The final rating for the rock mass is made by summing up the weighted values determined for the individual parameters at each zone. Higher value of final rating (Rw) reflects less weathering. The recommended classification with rating for each parameter and each class is presented in Table 3.

Joint number is given less weightage because in this classification effect of physical weathering (i.e. jointing) is assessed less influencing rather than chemical weathering which is predominant in the subtropical regions. Rating for joint number and joint width is combinely given 1/3 rd weightage out of total rating and rest 2/3 rd weightage is given to chemical weathering and also microfracture which are combined result of chemical and physical weathering.
Table 3. Rock Weathering Classification Based on Rating System.

<table>
<thead>
<tr>
<th>Weathered Material Grade</th>
<th>Symbol</th>
<th>Fresh</th>
<th>Slightly Weathered</th>
<th>Moderately Weathered</th>
<th>Highly Weathered</th>
<th>Completely Weathered</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$W_i$</td>
<td>80-100</td>
<td>50-80</td>
<td>25-50</td>
<td>10-25</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td></td>
<td>25</td>
<td>15</td>
<td>7</td>
<td>3</td>
</tr>
<tr>
<td>State of Joint Weathering</td>
<td>Jwt</td>
<td>Fresh</td>
<td>Slightly Weathered</td>
<td>Moderately Weathered</td>
<td>Highly Weathered</td>
<td>Completely Weathered</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td></td>
<td>28</td>
<td>17</td>
<td>8</td>
<td>3</td>
</tr>
<tr>
<td>Number of Joints /m</td>
<td>Jn</td>
<td>&lt;2</td>
<td>2-4</td>
<td>4-8</td>
<td>8-16</td>
<td>&gt; 16</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>25</td>
<td></td>
<td>20</td>
<td>13</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Joint Width (mm)</td>
<td>Jw</td>
<td>&lt; 1.0</td>
<td>1-2</td>
<td>2-5</td>
<td>5-20</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10</td>
<td></td>
<td>8</td>
<td>5</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

A SITE EXAMPLE

A typical example of the gradational weathering profile developed over the granite at Malanjkhand copper ore mines is illustrated graphically in fig. 2. This shows the distribution of fractures and the weathering state in delineated zones following the proposed rock mass classification. Assigned rating and classes of the weathering zone are given in Table 4. The observation of the features at the zones are described briefly in Table 5.

Table 4. Ratings and Weathering Classes of The Delineated Zones of Weathering Profile.

<table>
<thead>
<tr>
<th>ZONE</th>
<th>Rw</th>
<th>RMR</th>
<th>CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>15</td>
<td>Residual Soil</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>19</td>
<td>Completely Weathered</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>35</td>
<td>Highly Weathered</td>
</tr>
<tr>
<td>4</td>
<td>35</td>
<td>52</td>
<td>Moderately Weathered</td>
</tr>
<tr>
<td>5</td>
<td>41</td>
<td>65</td>
<td>Moderately Weathered</td>
</tr>
<tr>
<td>6</td>
<td>59</td>
<td>65</td>
<td>Slightly Weathered</td>
</tr>
<tr>
<td>7</td>
<td>70</td>
<td>75</td>
<td>Slightly Weathered</td>
</tr>
<tr>
<td>8</td>
<td>77</td>
<td>79</td>
<td>Slightly Weathered</td>
</tr>
</tbody>
</table>

Fig. 2. Different Zones of a Weathering Profile Observed at Malanjkhand Copper Mine. Zones are marked in numbers (1-8).
Table 5. Observation on the Different Zones of the Weathering Profile Developed Over Granite Rock.

<table>
<thead>
<tr>
<th>ZONE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Structureless layer of soil of variable thickness. Various shades of dark to light reddish brown shows different horizons of vegetation, leaching and accumulation. Rock fabrics are absent.</td>
</tr>
<tr>
<td>2</td>
<td>The zone contains decomposed rocks predominantly. Most of the materials are W4 (around 70%). Floating rounded regoliths can be seen. Original rock fabric preserved. Relict joints are visible. The weathering is heterogeneous with very little development of corestones.</td>
</tr>
<tr>
<td>3</td>
<td>The zone is mostly constituted by Highly Weathered material (W3 grade more than 70%). The rectangular blocks are separated by thin seams of decomposed and friable materials.</td>
</tr>
<tr>
<td>4</td>
<td>This zone is little suffered from decomposition. Friable materials are restricted at narrow seams of joints. Highly and Moderately Weathered material dominantly constitutes the zone.</td>
</tr>
<tr>
<td>5</td>
<td>Moderately Weathered material zone represents the exposed part of zone. At few places joints are clearly visible and thinly apart (aperture width = 5-10mm). Joint surfaces are highly weathered in state. The joint no./m is observed ranging from 2-4.</td>
</tr>
<tr>
<td>6</td>
<td>The zone is mainly composed of Slightly and Moderately Weathered material grade. Joints shows high roughness and low width of 1-3mm. At few places joints are filled with highly weathered materials. Very thin layer of rim can seen at the margin of few block.</td>
</tr>
<tr>
<td>7</td>
<td>The features are same as observed in zone 6 with almost no sign of corestone development.</td>
</tr>
<tr>
<td>8</td>
<td>Slightly weathered rock material is more prevalent in this zone. Massive huge blocks are present in this zone with slight staining at surface. Very less number of joints and most of them are not continuous and randomly oriented.</td>
</tr>
</tbody>
</table>

PREDICTION OF ENGINEERING PROPERTIES

Based on the observations and determined values of Rw for different zones of 13 profiles and 3 case studies, relationships were drawn for prediction of material constants for Rao (1984) and Ramamurthy et al. (1985) criterion and in-situ deformibility. The relation between Rw and RMR (Bieniawski, 1974) is also established for all 16 weathering profiles. Important relationships are given as:

\[ R_w = 0.9 \times \text{RMR} + 10 \]  \hspace{1cm} (1)

\[ B_j/B_i = \exp\left(\frac{(R_w - 100)}{30}\right) \]  \hspace{1cm} (2)

\[ E_{(\text{in-situ})} = \exp\left(\frac{(R_w - 27)}{16}\right) \]  \hspace{1cm} (3)

Where

- \( R_w \) = rating for weathered zone in present classification
- \( \text{RMR} \) = rating for CSIR classification
- \( B_j \) and \( B_i \) = material constant for intact and jointed rocks in IITD criterion
- \( E_{(\text{in-situ})} \) = in-situ deformational modulus

APPLICABILITY AND LIMITATIONS

1. Classification can be very suitable for the subtropical regions where weathering profiles have considerable thickness.
2. It could be used for homogeneous as well as heterogeneous weathering profile.
3. Classification can be useful for the rapid and preliminary investigation of any rock engineering projects which involves evaluation of rock mass in weathering condition.
4. It may provide an engineering description of the location and zone which is represented by a sample for the detailed tests.
5. It can only be applied on the exposed profiles of rock and can not be used in borehole logging.
REFERENCES


Correlation exists between resistivity and rock lithology which can be used for preliminary rock characterisation.

Permeability increases with decrease in formation factor. However, further investigation is needed to establish a rational correlation between permeability and resistivity.

The effect of confining pressure on resistivity is observed to be a complex phenomenon and depends on various factors such as mineralogy, pore characteristics, moisture content, pore pressure etc.

Good correlation exists between resistivity and uniaxial compressive strength and modulus of elasticity particularly for saturated rocks.

Tensile strength and shear strength increase with increase in resistivity. Analysis of data indicates possibility of fair correlation between resistivity and tensile as well as shear strength.

In triaxial test resistivity decreases with increasing axial stress. At higher confining pressure a marginal increase in resistivity at low axial stress may be observed. At failure normalised resistivity nearly reaches the same value for a particular rock irrespective of magnitude of confining pressure.

Detail investigations are needed to understand the dependence of strength anisotropy on resistivity anisotropy.

REFERENCES


2. The in-situ stress in the rock mass significantly influences the deformations in both the elastic and plastic zones. The brittle-ductile transition occurs when $\sigma_3/\sigma_1 = 1/(B_1+1)$ as per the criterion adopted (Ramamurthy 1986). This transition coincides with $p_1/p_0 = 1/(B_1+1)$ below which large plastic deformations take place.

3. The radius of the broken zone is largely influenced by the in-situ stress and increases with the insitu stress, $p_0$.

4. The joint factor of the broken zone does influence its radius.

REFERENCES


Figure 9 Influence of Modulus, In-situ Stress and Joint Factors of In-situ and Broken Rocks Mass on Radius of Broken Zone


from bolting pump and in second step proper grouting is realised with low pressure from special injection pump.

- as a high pressure injection, when the expansion of bolt, breaking of the membrane and proper grouting is realised directly with grouting material from only one pump.

Hitherto experience from using of the combined bolting and grouting system in coal mines and underground structures are encouraging. It was proved, that bolts reinforce rock mass ring on the perimeter of excavation and prevent from the creation and increasing of fissures, so that the amount of waste grouting material flowing into the excavation decreases. So bolts can partly or fully replace the first 'packing' step in two-step injection procedure. The combined technology make it possible to use higher injection pressure. Due to immediately anchored bolts the total amount of grouting material also decreases in 20 - 30%. This type of bolts can be anchored in weak rocks too.

Practice confirms, that combined bolting and grouting system Boltex brings not only economical effect due to cancellation of certain operations, but it is advantageous from the geomechanical point of view too. The important advantages of the combined bolting and grouting technology can be summarized:

- very quick and easy installation
- low requirements on the accuracy of the borehole diameter
- immediate bearing capacity of bolts
- prestressing between pad and rock
- large range of injection pressure
- unlimited length of bolt column
- arbitrary position of packers along the length of bolt
- brick-box character of the Boltex make it possible to adapt the system to the real conditions of using.

The research was provided with the financial aids from the Grant Agency of Czech Republic - project No. 103/94/0777.

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62
A survey of a major part of the existing Norwegian sub sea road tunnels (a total of almost 100 km), may illustrate the point [4]. The average tunnel has two lanes, a cross section of about 50 m² and the average deepest point is about 125 m below sea level. Under average rock conditions (typically a granitic gneiss with local shear zones), the total cost, ready for traffic is about USD 6'000.- per meter.

This figure alone is probably difficult to transfer to other regions, but the split into cost elements is very illustrative, as shown in figure 4.

### Cost Example

<table>
<thead>
<tr>
<th>Cost elements (% of total)</th>
<th>0%</th>
<th>5%</th>
<th>10%</th>
<th>15%</th>
<th>20%</th>
<th>25%</th>
<th>30%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock support</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavation (D&amp;B)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Probe-drilling, injection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drip shields</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plann., investig., superv.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road base, pavement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Electr. equip., ventilation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drain., pumping equip.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4. Typical cost elements of sub sea road tunnels (Norway)

The cost of probe drilling and pre-injection in this investigation is typically 7% of the total project cost. This is probably for many tunnelling professionals a surprisingly low figure. It is in comparison to the excavation cost (D&B), about 25% and in comparison to the rock support cost, about 30%. One reason for this favourable situation, is that the whole tunnelling approach, including single shell permanent lining and pre-injection, was specified in the bidding documents.

With the latest technology improvements (like micro cements and GroutAid), it is now possible to specify a target leakage rate as required, down to around 1 l/min. and 100 m tunnel, with quite acceptable cost of pre-injection. Properly utilised as part of an overall tunnelling concept, the savings potential is quite substantial.

**REFERENCES**

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7. CONCLUSIONS

A brief review of the background to the Hvalfjordur sub-sea Tunnel project, from the geological investigations through to design and construction, has been presented in this case-history paper. The main aspects covered include:

- the unusual ground conditions comprising mainly basalt, scoria and sedimentary (volcanic) rocks
- a summary of the geological and geophysical investigatory work carried out along the tunnel route
- the presence of mixed face conditions along the majority of the tunnel route and a summary of the expected rock types to be encountered
- a summary of the typical properties of Icelandic rocks
- the application of the Q-system in the rock mass classification and rock support class selection
- the use of Scandinavian practice using the excavation-observation-support approach
- additional corrosion protection to rock bolts in this hydrothermally active environment
- details of the seismic hazard and risk assessment carried out and measures taken in the design to mitigate the effects of vibration induced damage.

8. REFERENCES

BLAST PERFORMANCE

Blast designs optimised for driving the cross drifts in several stages and the widening operations in two parts i.e. column section and side wall section resulted satisfactorily. The average progress i.e. pull per round was 85-90% of the drilled depth. The disturbance to the adjoining rock mass was the minimum. This was evident from the fact that the hole impressions (half cast drilled hole) observed in the roof after blast were more than 80% of the actual drilled length. The overbreak was practically nil.

CONCLUSIONS AND RECOMMENDATIONS

The results of vibration depend on the total charge and the maximum charge fired in a blasting round. The excavation done in stages helped in keeping the charge to the minimum. This is evident from the charging pattern discussed in the blast designs for widening operations. The charge/delay (6.50 kg) and the total charge (41.34 Kg) in a round was the maximum in case of full section taken in single operation, whereas the charges are the minimum (4.22 kg & 13.39 Kg) while taking side wall section separately after column section. Thus, it is concluded that minimum disturbance is caused to the adjoining roof rock when the widening operation is carried out in two parts in comparison to a single operation combining the two sections in one round of blast.

Thus authors are of the view that wherever such excavations are taken up it is imperative to take up in stages to control the charge which in turn will insure the safety and efficiency of a blast. Besides this strict supervision is essential for correct positioning of perimeter holes and maintaining the precision of drilling accuracy to achieve the objective.

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typically 15 to 25 mm; angular distortion for precise levelling/electro-levels.

Where the aim is to reduce permeability: purpose of treatment; zone to be treated; pre-treatment permeability (if available); acceptable limits to post treatment permeability; and acceptance test regime.

Where the aim is to increase shear strength or stiffness: purpose of treatment; zone to be treated; pre-treatment strength or stiffness or acceptable limit to post-treatment strength or stiffness; and acceptance test regime.

13.5 Method of Measurement and Payment

Grouting contracts fall essentially into two phases, the drilling for grout holes or driving injection pipes for grout holes and the grout materials and injections. Grouting contracts may be paid on a lump sum or per unit volume of grout injected or on cost-plus basis, a schedule of rates with measurement or Bill of Quantities. Lump sum contracts should only be contemplated for small and demonstrably straight forward contracts. Cost plus is probably the most equitable basis; but most clients would be reluctant to accept this form from the start.

14. CONCLUSIONS

A brief review of the types of grouting for ground improvement in tunnelling has been given for permeation, hydrofracture, compaction, compensation and jet grouting. The key aspects of any grouting program are the use of a flexible approach in the design and planning as well as the use of the observational approach during the grouting works. Grouting tests and field grouting trials are useful to optimise a grouting method and materials. The preference amongst practitioners is for trials to form part of an intermediate separate contract stage. The equipment used for grouting should be designed to safely withstand the anticipated maximum grouting pressures and be compatible with the grouting technique(s) being used. If grouting is to achieve its objectives in a cost effective manner, it is important at all stages that technical decisions are taken by those with the necessary knowledge and experience, that the contractual framework encourages this and that narrow commercial advantage does not subvert it.

15. ACKNOWLEDGEMENTS

This paper has been based on the work carried out by WS Atkins Consultants Limited for the Construction Industry Research and Information Association (CIRIA) for the project report - Grouting in ground engineering: principles, methods and applications which will shortly be published in a CIRIA report. This paper summarises the findings of this report with respect to grouting for ground improvement in tunnelling. WS Atkins gratefully acknowledges the support given by CIRIA as well as the steering group members and other individuals in the grouting industry. This paper is published with the permission of the Director General of CIRIA. Any views expressed in it are of the authors and not necessarily those of CIRIA.

16. REFERENCES

have the combination PE foam/shotcrete on the roof and concrete elements on the walls. The competition situation in the market changes from one project to the next and it is difficult to say which systems will dominate. Quality requirements are growing more stringent, and the struggle is going to be between qualitatively equivalent products.

9. FURTHER WORK

Due to the escalating rate of road-tunnel construction in recent years, an increasing number of designs have been developed. New guidelines for fundamental requirements and design have been prepared by the Norwegian Public Roads Administration (NPRA). Intensive work is in progress by the industry, as well as the NPRA, to achieve realistic adjustment to the market. Additionally, the NPRA has launched a programme designed to rapidly produce means to simplify evaluation and tendering work by an approval arrangement for lining designs.

REFERENCES


6. DRAINAGE MEASURES IN THE CONCRETE LINING

With drainage measures, such as drainage holes the loading of concrete lining by external water pressure can be prevented totally. The permeability of the concrete lining is increased by the drainage holes in such a way, that all the pressure head of the seepage flow is taken by the relatively tight rock mass. Drainage holes in the concrete lining can be accepted only if the following criteria are fulfilled:

1) Rock mass is rather tight; the leakage from the tunnel into the surrounding rock mass under internal pressure is acceptable.
2) There is no danger of washing out of joint fillings or fine materials from the rock matrix.

The last criteria is most important, since a gradual process of erosion of fine rock materials behind the concrete lining not only increases the permeability of the rock mass but also endangers the stability of the rock itself. In the worst case, failure of the rock mass and the concrete lining could occur.

Drainage holes should be drilled 0.5 m to 1.0 m deep into the rock only. Longer drainage holes would increase seepage flow towards the tunnel or shaft. Looking at a section of the tunnel, at least 3 to 4 drainage holes are required (clockwise at 9, 12, 3 hours; at the invert at 5 or 7 hours because of danger of clogging by mud). In the longitudinal direction the distance between the drainage holes should not exceed 3 m to 6 m depending on the rock mass permeability.

7. CONCLUSIONS

If uncracked and cracked concrete linings are considered absolutely tight, the design for external water pressure will lead to very conservative and uneconomical lining thickness. Even if the lining is rather tight with few small cracks spaced widely apart, the effective external water pressure acting on the lining is still diminished considerably, if one considers seepage flow conditions. When carrying out consolidation grouting, the acting external pressure on the lining is further strongly reduced. In the case of loosened, more permeable rock zones in the vicinity of the concrete lining, the external water pressure effectively acting on the lining increases. Therefore it is in such situations even more important to carry out consolidation grouting.

REFERENCES

The effect of steel fibers on crack control depends on steel fiber dosage and aspect ratio. Specifications have to mention a minimum fiber length and a maximum fiber spacing. Based on this value a minimum dosage can be specified for each fiber type.

Toughness should be specified based on
- French slab test energy absorption values (Joule) for outer linings;
- beam test equivalent flexural strengths (N/mm²) for designed inner linings.

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Fig. 6 - Rebound of particles of different sizes

ACKNOWLEDGEMENT

The authors are grateful to the authorities of Koyna Hydro-electric Project, stage-IV for sponsoring this study and for their help and cooperation during the field work. The authors are also thankful to Shri S. Gopalakrishnan and his co-investigators at the Structural Engineering Research Centre, Madras for agreeing to conduct the test in the presence of the first author and for sharing the results with CMRI, their sister Laboratory. The views expressed in the paper are those of the authors and not necessarily of the Organisations they represent.

REFERENCES


CONCLUSIONS:

Braced Excavations

The braced subway excavations for the Los Angeles Metro Segment 2 performed well, and no apparent signs of distress or instability were evident. The support pressures recorded at individual bracing levels and struts varied widely and in some instance exceeded the design equivalent pressure; however, the conservative design stress levels and the inherent flexibility of the support systems themselves prevented any significant problems. The use of a specified design equivalent lateral pressure diagram and strict design criteria minimized lateral soil movements adjacent to the excavations and forestalled problems of opening stability that could have slowed construction progress and extended construction durations. Wall movements generally were less than general practice would dictate, most probably due to the placement of tightly wedged upper decking beams that provided early support of the openings and restricted wall upper movements. Examination of records where large wall movements occurred showed that almost without exception, those cases could be traced to instances where bracing was not installed in a timely manner or where overexcavation below the bracing level exceeded the design span.

Performance of Tunnels

For tunnels constructed within the Puente bedrock, surface settlements generally remained below one in. (2.5 cm) for the twin tunnels. Generally, the tunneling in Alluvium resulted in somewhat larger volume losses and associated settlements. Movements from 1 to 2 in. (2.5 to 5 cm) total settlement were not uncommon. In some locally isolated reaches where tunneling encountered the less dense Young Alluvium, settlements exceed 8 in. (20 cm). Much of this settlement was attributed to hydrocompaction and consolidation occurring when water mains and other connections broke during tunnel driving. Other ground losses were generally attributed to losses behind the shields that developed because of the use of overcutters, inefficient and incomplete expansion of the segmental lining and the lack of consistent contact grouting in the annular void between the liners and the ground. In several instances compaction grouting was applied to minimize the ground movements and subsidence.

REFERENCES:


ACKNOWLEDGEMENTS:

The authors thank the Los Angeles County Metropolitan Transportation Authority and the Construction Manager, Parsons-Dillingham for sharing information and field data used in this paper.
- the verifications performed with the characteristic line method result in percentage deformations completely coherent with the ones obtained by convergency measurements. Magnitude order appears to be the same taking the design geomechanical parameters,
- despite of the unavoidable discordance between the real excavation conditions and the reference model underlying the characteristic lines method, such a method results in safety factor values, for the temporary facing, fully reflecting the good stability conditions really detected in the tunnel section considered.

CONCLUSIONS

The comparison between the results obtained with different methodologies for the calculation of the stability of an excavation made in a rock mass composed by “Flysh” and the convergency measurements collected during the excavation monitoring allowed some interesting considerations. First of all, the comparison of the values obtained with different procedures, considered in their limiting hypothesis, proved a correspondence between the design typical parameters of the rock mass and its real geomechanical behaviour. The use of different procedures and the agreement of results, validate the initial forecasting.

Secondly, the interpretation of the strain-static evolution of the excavation with the characteristic lines method, made clear that this method can give useful and reliable information, provided that the geomechanical behaviour of the rock mass is known in detail. The analysis has shown how this method can be employed with reliable outcomes even in the design phase.

Finally, for a correct forecast of the strain-static evolution of an underground excavation and for its correct interpretation is important the execution of an in-depth study of geologic and geomechanical characterisation that minimise the onset of unexpected, but in some way predictable phenomena.

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part of the tunnelling process, it will of course never be possible to accurately decide the quantities in advance. This fact should not be used as an argument to throw away a method offering a substantially lower cost, with some limited cost uncertainty.

It is furthermore extremely important that a built in contractual flexibility is linked to a motivation factor leading the Contractor to assume the same interest as the Owner. By saving construction time, the Contractor will save money, and the Owner will in many cases save an order of magnitude more. In the case of a hydro power development, it is often so that the Owner will save some hundred USD per day and MW installed production capacity. By offering only a small part of this as a premium to the Contractor, as part of the terms of contract, it is sometimes amazing what positive effect can be recorded [3].

CONCLUSION

Tunnelling and support technology is now available, allowing substantial project cost savings, applicable to a major part of tunnelling conditions and tunnelling purposes. The party suffering the negative cost consequences of not taking advantage of this technology, is the Owner. The fortunate situation, is that the Owner is also the party with the keys to preventing excess unnecessary cost. By requiring a project development on the basis of well documented technology, design, contract basis and construction methods, the full cost saving potential is well within reach.

The main elements of a cost saving solution are:

• Support design based on the Observational Method
• Tunnel support and lining carried out as a single shell structure, all shotcrete applied satisfying the requirements of a permanent lining and becoming part of the permanent lining.
• Ground water control by pre-injection and/or drainage, to avoid the problems of ground water in-leakage damaging the fresh shotcrete.
• A contract model providing the flexibility needed to adapt the support to the actual rock conditions as they are exposed in the tunnel, more or less of pre-injection as needed and a clear economic incitement for the contractor to save time and quantities with maintained quality.
• Construction management procedures allowing necessary delegation of authority to the Site for decision making according to the pre-planned rock support classes (prognosis) and actual rock conditions.

A key element of the whole approach, clearly is to ensure a sufficient quality of the shotcrete. It is therefore a requirement that the necessary focus and resources are put into actually reaching the design quality parameters. In comparison to a temporary and wasted shotcrete, this may call for a better cement, use of wet mix shotcrete, utilisation of modern admixture technology and alkali free accelerators. Additives like micro silica, fly ash and steel fibres are also often possibilities for evaluation. What during a first time evaluation may appear as a major cost increase and complication, leading to rejection of the whole idea, is only a marginal investment compared to the generated project savings.

REFERENCES


By calculation of the maximum and minimum principal stresses and by studying rock joint orientations, critical pore pressure curves can be created. These are specific for every project.

The theory has been verified in different projects associated with stimulations of deep HDR reservoirs. The material presented in [9] has been helpful in formulating the problem deal with in this paper.

REFERENCES


which can be used without any further processing in high strength shotcrete and possibly even in 0/32 mm frost resistant pump concrete.

Curves developed in Fig. 10 are based at cutter spacings of 86 mm (100 percent), 129 mm (150 percent) and 172 mm (200 percent).

Analysis Conclusions:-

1. Increasing the cutter spacing by 50 percent increase chip thickness by 60 percent.
2. Thickness of the flat chip produced during boring is the most critical dimensions in determining muck recyclability, with a minimum of 0/32 mm required for concrete aggregate.
3. Increased cutter spacing reduce the amount of fine material in the muck sample. This results in a sand-gravel ratio after crushing which is more favourable to concrete production.
4. Increased thickness of relatively small debris, an important feature to proper grain size distribution in the final material.

The benefits of recycling muck are both environmental and economic. Reuse of excavated material results in improved tunnel resource management, and the decreased need for muck transport and dumping areas reduces project costs and damage to the environment.

This study also shows that increased cutter spacing does not affect the rock boreability in a negative way.

An important feature.

Tunnel bored by TBM usually requires rock support during excavation and lining of the finished tunnel. Both procedures include the use of shotcrete and concrete. The potential for muck produced during TBM tunnelling to be used in support and lining operations is both economical, and an important development in tunnelling.

TBM boring with greatly increased cutter spacing even on hard rock such as granite is a significant breakthrough in hard rock tunnelling.

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