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GOVERNMENT OF INDIA
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MINISTRY OF COAL & MINES
खान विभाग
DEPARTMENT OF MINES
भारतीय खान ब्यूरो
INDIAN BUREAU OF MINES

# Cable Bolting Practices in Underground Mines

Prepared by
Publication Cell
Mineral Economics Division

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GOVERNMENT OF INDIA
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**INDIAN BUREAU OF MINES**
Nagpur

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# Preface

This Publication "Cable Bolting Practices in Underground Mines" is the 42nd in the series of bulletins brought out so far by Publication Cell of Mineral Economics Division, Indian Bureau of Mines on the various topics related to mining and geology under the "Advancement in Mining Technology Information" series.

In this bulletin, the technical aspects of special support system of cable bolting has been extensively dealt. The technical and economical benefits as against the conventional methods also have been critically analysed. The advantages of using this method as against the conventional support system in active mining conditions also has been explained in simple and effective manner.

Cable bolts are generally used as pre-support in open stope and cut-and-fill mining methods. Various aspects have been covered in the bulletin in order to have better understanding of cable bolting which includes several devices, load mechanism, stressing on tensile forces, practices adopted in anchoring in Indian underground mines and experiences abroad. Besides, potential application of cable bolting in mining industry and the philosophy behind reinforcement scheme to overcome difficulties with great success have also been explained.

All details included in this bulletin have been abstracted from various recent journals and magazines both Indian and Foreign, as well as from field study reports of IBM and are presented for the benefit of the mining industry and researchers in the mining institutions. It is hoped that this bulletin will serve as very useful publication in the field of Cable Bolting Practices.

Nagpur

Dated : 04/06/2002

**(K. S. RAJU)**
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# Contents

# Chapter 1

# Introduction

Rock reinforcement improve the shear and tensile strength of the rock mass adjacent to surface and underground excavations. Pre-reinforcement refers to the support prior to excavating the adjacent rock. The principal effect aimed through such measures is to limit the displacement of the rock mass to small values. This minimises the shear and dilation along existing geological structures and preserves their in situ cohesion and friction as a result that the rock mass becomes self-supporting.

Limited headroom available in underground operations has favoured the development of cable bolting. Cable bolt is a long steel wire stand of high modulus and strength, grouted into a hole to reinforce the rock mass, with the purpose to knit individual blocks of the rock mass together to form a thicker beam, or for anchoring weak strata to competent strata above. It also provides preplaced support under difficult ground conditions and ensures safety and economy in hard rock mining, and are generally used as pre-support in open stope and cut-and-fill methods in underground mining. They can be either tensioned (active support) or untensioned (passive support) and may be of any length, though for underground mining applications, lengths less than 20 m are preferred.

Caving of thick seams containing nearly 55% of the total coal reserve of India is posing a challenge to the mining community since the very inception. The coal left along the roof failed with time, adversely affecting the conservation, inviting danger of spontaneous heating and endangering the safety of the workers and the workings. Cable bolting techniques are now adopted in coal mines.

A brief history of cable bolting its materials, mechanism, design and applications, construction, pretesting, impact of corrosion and time are discussed in this bulletin, with special reference to the work carried out in various mines all over the country. Based on the rock mechanics investigations of pre-reinforced stope backs, some guidelines concerning their effectiveness in maintaining back stability are discussed in the form of a few important case studies.

# Chapter 2

# Theory and Principles of Bolting

Cable bolting and rock bolting are techniques of rock reinforcement in mining. Rock reinforcement is a method to improve the rock mass quality against any type of undesired deformation. This chapter deals with the technique of rock-reinforcement through bolting with respect to mining excavations only.

## 2.1 BOLTING TECHNIQUES — THEORY

In mining operations the technique of rock reinforcement is considered as a method of support. It is very important to understand the difference between support and reinforcement. Support means installation of a structural element at the boundary of the excavation as a restraint. Whereas, rock reinforcement is achieved by installing structural elements into the rock mass itself with the help of boreholes and bolts grouted/reinforced into the rock mass. These reinforcing elements enable the rock to withstand forces of deformation by improving the interior of the rock mass. Rock support and rock reinforcement are explicitly different by the action with which they stabilise the rock near the excavation boundary. The techniques of rock support devices include fill, timber, steel, or precast sets, shotcrete and props. All the devices installed in boreholes to improve the overall rock mass properties from within the rock mass fall under the catagory of rock reinforcement. The combination of supporting and reinforcing devices are used for safe mining operations.

### Important Definitions

Definitions of some of the terms relating to cable bolting.

**Anchor :** An anchor or a bolt is a device with a static function, which transfers the force to the desired region in the rock medium. An anchor is composed of three parts — anchor head, anchor root and tendon. Anchor may comprise bars, wires of strands, depending upon the material used.

*a) Anchor head* - Anchor head is the opposite end to the root which, generally remains outside the borehole.

*b) Anchor root* - Anchor root is the subterranean end of the anchor which remains fixed inside the borehole.

| | |
|---|---|
| *c) Anchor tendon* - | Tendon is the portion, connecting the root and head of an anchor. |
| **Fixed length :** | This is the length of the anchor along which the forces within the anchor are transferred to the ground. This is the portion of the anchor which remains in continuous contact with the rock mass either directly or through a grouting medium. |
| **Free length :** | This is the portion of the tendon from anchor head fixing point to starting point of the fixed length. |
| **Temporary anchor :** | These are the anchors having a service life upto two years. |
| **Permanent anchor :** | These are anchors with a service life in excess of two years. |
| **Pre-stressed anchor :** | These are permanently tensioned due to the elastic extension of the tendon over its free length. |
| **Anchoring force :** | This is the force, which is continuously transmitted by the anchor to the ground. |
| **Admissible load :** | This is the maximum permissible limit of the applied load which the anchor can sustain without any damage. It is determined by ascertaining the upper limit of its bearing capacity and then subtracting a safety margin from it. |
| **Bearing capacity :** | Bearing capacity is defined as a maximum load after which any of the functional part of the system fails and the anchor ceases to function. |

## 2.2 MECHANICS OF BOLTING

Bolting reinforces the rock by transferring the load to a stable zone from an unstable region within the rock mass.

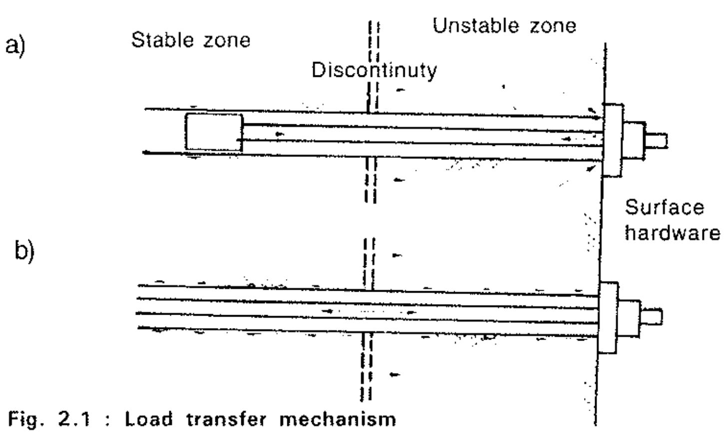



Fig. 2.1 : Load transfer mechanism
a) Discretely Coupled Bolt
b) Continuously Coupled Bolt

Figure 2.1 depicts the concept of load transfer and the mode of load transfer within a rock mass through a bolt. It comprises three stages.

(i) Rock movement in the unstable zone transfers the load to the bolt in this zone.

(ii) The load is transferred to the portion of the bolt falling in stable zone of the rock mass.

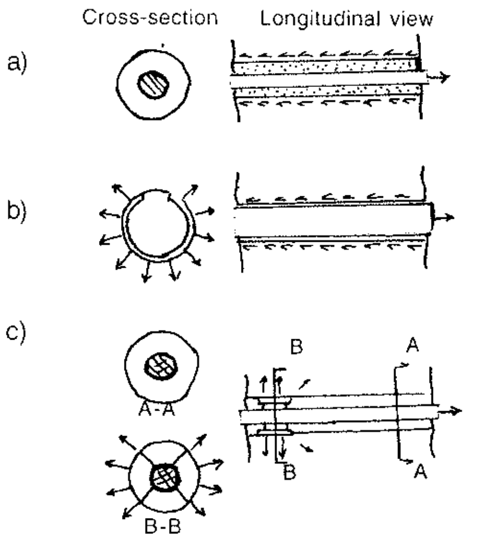



Fig. 2.2 : Categories for achieving load transfer

a) Continuous Mechanically Coupled

b) Continuous Frictionally coupled

c) Discrete Mechanically and Frictionally Coupled

(iii) This load from bolt is now transferred to the stable rock mass.

There are various ways of achieving load transfer between a bolt and rock. Almost all of them can be placed in one of the following three categories.

(i) Continuous Mechanically Coupled (CMC)

(ii) Continuous Frictionally Coupled (CFC)

(iii) Discrete Mechanically and Frictionally Coupled (DMFC).

**(i) Continuous Mechanically Coupled Bolts :** Such type of bolts are kept in continuous coupling with the borehole walls over its entire length through a cementitious/resinous medium known as grout. The bolt is secured by filling this grout in fluid state between borehole wall and bolt annulus. The grout takes some time for setting ranging from few minutes to a number of hours depending upon the type of grout selected. The basic function of the grout is to facilitate the load transfer between bolt and rock mass (Fig. 2.2 a). The bolts, which are used with the grout are made with variable cross section over their entire length. These variations serve as a mechanical key.

**(ii) Continuous Frictionally Coupled Bolts :** This type of bolts also remain in continuous contact with the borehole walls over its entire length like the previous one but the contact is direct in this case with no grouting. The load transfer between bolt and rock mass takes place through the friction between bolt and borehole walls, (Fig. 2.2 b) which is limited by the radial stresses set up during the installation. In such type of installations, there is hardly any mechanical key functioning. Whatever, little effect comes that is due to the irregularities of borehole walls. Such bolts are installed either through the expansion of an undersized section in a larger borehole (like Swellex) or contraction of an oversized section into a smaller borehole (like split set).

**(iii) Discrete Mechanically and Frictionally Coupled Bolts :** These bolts provide point anchorage in the form of a limited fixed length. These bolts are fixed through grouting the root or nearby portion of tendon or a deformed end with an expanding shell provided at the root. Hence, the load transfer takes place over the short length through a combination of geometrical interference or frictional interaction. The wedge bolt and expansion shell bolt fall under this category (Fig. 2.2 c). This portion of bolt length over which the load transfer takes place is known as anchorage length and is less than 500 mm for grouted bolts and below 200 mm for expansion shell type.

A system of classification showing the different types of bolts being used in different mines in the world is presented in Table 2. 1.

**Table - 2.1 : Classification of Reinforcement Types**

| Basic type | Subset | Discrimination |
|---|---|---|
| Continuous mechanically coupled | Short cement/resin encapsulation | Wooden dowel |
| | | Plain bar |
| | | Deformed bar |
| | | Thread bar |
| | | Pigtail bolt |
| | | Paddle bolt |
| | | Yielding bolt |
| | | Perfobolt |
| | | Sigbolt |
| | | Fibreglass bolt |
| | | Injection polymer bolt |
| | | Birdcage strand |
| | Long cement encapsulation | Multiwire cable |
| | | Prestressing strand |
| | | Destranded hoist rope |
| | | Shear key |
| Continuous frictionally coupled | Friction | Split Set |
| | | GD Rock Nail |
| | | Swellex bolt |
| | | Wedge-Pipe bolt |
| | | Ramp bolt |
| | | Pipe anchor |
| Discrete mechanically and frictionally coupled | Friction | Slot and wedge anchor |
| | | Expansion shell anchor |
| | | Plastic expansion anchor |
| | | Swellex |
| | Resin encapsulation | Plain bar |
| | | Deformed bar |
| | | Thread bar |
| | | Pigtail bolt |
| | | Paddle bolt |
| | | Fibreglass bolt |
| | | Tube anchor |
| | | Long tendons |

## 2.3 REINFORCEMENT : MODE OF ACTION

The interaction between the rock mass and reinforcing bolt is very complex. The constitution of rock mass, its complicated failure mechanism and load transfer between load and the rock mass are the processes involved. Any bolt installed is subjected to pure axial, pure shear or combined effect of axial and shear stresses, when rock mass tends to displace. The ultimate bolting action is a matter of close study of all the factors indicated above. However, to define the reinforcement behaviour in a simplified way the following principles are considered.

## Suspension

It is assumed that the bolts anchor the nearby strata of an excavation to a solid stable ground above. This is based on the assumption that the bolts are long enough to connect the potentially unstable zone around an excavation to a remote stable zone, thus transferring potentially dangerous stresses to the solid interior rock capable of withstanding these stresses well.

## Beam Building

This theory is most suitable for the rectangular openings in a stratified rock mass. The bolting clubs several individual layers into a thick and strong composite beam and is well capable of withstanding the stresses which otherwise would have induced the failure of an individual layer. The reinforcement action of a fully grouted bolt in a layered strata can be explained very well by this theory.

The maximum bending stress in tension ($\sigma x$) max, in case of a clamped beam is experienced at the upper surface, just above the clamped edge, which is given by

$$(\sigma\, x)\max = \frac{vb^2}{2t} \quad \ldots\ldots\ldots\ldots\ldots\ldots 1$$

where v = unit weight of rock
b = span
t = beam thickness

Maximum deflection $\delta$ max experienced at the centre is given as

$$\delta\max = \frac{vb^4}{32\,Et^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots 2$$

E being the modulus of elasticity of the beam

From the equations (1 & 2) it is observed that, if five individual layers of equal thickness are bolted to form a composite beam, the magnitude of maximum bending stress ($\sigma x$) max and that of maximum deflection $\delta$ max will be reduced to 1/5th and 1/25th respectively of what would have been experienced by the individual layer.

## Arching

This theory is applicable to all types of compact and fissured rocks, to soft rocks. When the rock beam fails, a natural arch develops over the excavation, even in stratified rock mass.

This natural arch corresponds to a zone of increased stress within the rock mass and is unaffected directly by the excavation. The pressure of overlying strata is supported by this natural arch and transferred on the sides of the opening and into the substrata. Therefore, it is the weight of the loosened rock, below this natural arch, which may load the supports. In order to calculate pressures to determine a supporting system, knowledge of atleast approximate natural arch position is very important. The extent (thickness) of this loosened rock below the natural arch can be given by the empirical formula based on M.M. Protodjakonov Procedure. Terzaghi also tabulated it for different kinds of rocks and spane. Details are discussed in the section on Rock mass classification.

Once the approximate height of this arch is known, the optimum length and spacing of a bolt can be assessed. The bolts suspend this loosened rock with the load bearing zone of natural arch or strengthen the rock by imparting forces of compression so that it becomes self-supporting.

The length of a bolt is given as

$$L = h + lf$$

where $h$ is the distance of lower extremity of natural arch from the immediate roof of an excavation, i.e., loading height and $lf$ is the fixing length of bolt in the zone of natural arch.

The spacing of the bolts $lr$ is given as

$$lr = \sqrt{\frac{Fs \cdot xt}{h \cdot v}}$$

Fs = area of cross-section of the bolt

xt = Permissible tensile stress of bolt material

v = Unit weight of the rock

h = loading height

Even if the bolt has not been fixed in the load bearing zone of natural arch, the rock in the loosened zone is reinforced and pre-stressed by bolt anchorage to form a load bearing arch, as indicated by $d$ in the Fig. 2.3.

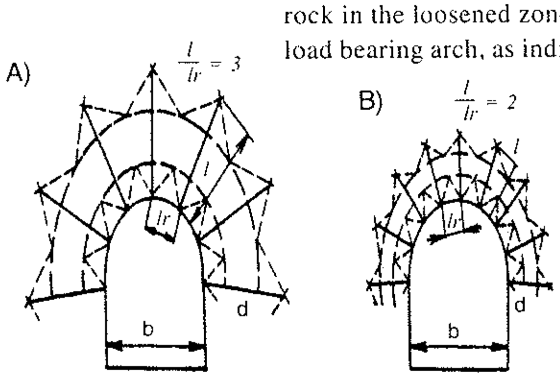



**Fig. 2.3 : Artificial arch formed by bolts**

A - Width of arch formed where $\frac{l}{lr} = 3$

B - Width of arch formed where $\frac{l}{lr} = 2$

Let the apex angle of the pressure cones emerging from both ends of a bolt of length $l$ is $2\alpha$. If the bolt is pre-stressed with a tensile force $p$, each bolt will exert a radial stress $\sigma p$ in the zone of compression $d$ which is given as

$$\sigma p = \frac{8p}{\pi l^2 \tan \alpha}$$

This radial stress induces a peripheral stress in the rock mass, which acts in a direction perpendicular to the axis of the bolt, thereby, increasing the strength of the rock in this compression zone *d*.

It has been established by triaxial tests that a rock having very low uniaxial compressive strength acquired a strength of 2 to 8 MPa under a lateral compression load of 0.2 MPa. Similarly, an arch formed from rock which when pre-stressed acquires a strength of 5 MPa in a zone of 100 cm width and can take a peripheral load of upto 50 kN/cm, and is thus equivalent to a concrete arch 20 cm thick.

In this way, a continuous compressed zone of sufficient thickness over an excavation is created to transfer the dead weight of the loosened section of the rock mass.

## Keying

Rock mass consists of several discontinuities like slips, joints, faults, bedding planes, whereas, formation and propagation of new discontinuities takes place at the time of excavation. Rock mass tends to move through these discontinuities. The bolts installed across such planes of weaknesses responds to prevent such movements by stitching them together or in other words it can be said that they form keystones which oppose the rock movements.

As already indicated, the actual mode of action of a bolt is a resultant of complex interactions between rock mass and bolt and structural nature of rock mass, type of bolt used and orientation of bolt with respect to discontinuity. In fact, some reinforcement designs work, only when rock mass attempts to fail. Thus it may be concluded that, the mode of displacement at a discontinuity define the mode of action of a reinforcing device. In practice, displacement at discontinuity may consist of translation and rotation combined in a three dimensional manner. It may also be path dependant including reversals in direction. The orientation of reinforcing device in relation to discontinuity and displacement makes it further complicated. Adhesion, friction and mechanical interlock are the basic mechanisms involved in load transfer between reinforcement device and rock mass and its combined effect is termed as bond.

Adhesion is relevant in case of continuously and discretely coupled devices, since they use some kind of bonding agents, while installed in a borehole. However, bonding is of little significance in real practice. First thing required for optimum adhesion is thoroughly clean surfaces, which is highly unlikely in day to day practice. Secondly, simple analysis suggests that, the shear strength of grout is quickly exceeded by the stresses developed at the reinforcement/grout interface at low load levels.

Mechanical interlock, described as the keying effect caused by having a reinforcement surface profile that keys into the rock in case of frictionally coupled devices or keys into the grout in case of mechanically coupled devices, is the relevant component to all classes of reinforcement. This interlock is provided by the borehole surface irregularities in frictional devices and by the bolt surface geometry (ribs, helical grooves, threads) in the mechanically coupled devices. This phenomenon of mechanical interlocking ensures that weakest material failure must occur rather than a simple sliding mechanism to take place at interface.

The component of friction is the dominant one and relevant to all classes of reinforcing devices, specially after a small displacement has taken place. When the reinforcement is loaded the induced shear stress at grout/reinforcement or rock/grout interface exceeds the shear strength of grout or rock at small displacement. Hence, at reasonable loads some failure of material at interface is evident which suggests, the interface material strength and friction become the most important factors to calculate the anchorage length for a designed load.

Friction depends on coefficient of friction and radial stress acting at failure interface. Microroughness of reinforcing device and grout particle size affect the coefficient of friction. Factors which affect the level of radial stress include installation process, dilation/contraction/removal of material during interface shear failure and radial contraction of reinforcing device under load as a result of Poisson effect.

Traditionally, reinforcing schemes are devised based on the axial strength and stiffness of the reinforcement devices, but the shear performance is also important. The mechanics of shear must include all the factors mentioned above in addition to the crushing of rock due to bearing stresses and bending of reinforcing device. In case of grouting, the crushing of grout must also be included. The important parameters that affect the behaviour in shear are

(i) axial properties of the reinforcing device material
(ii) bending properties is of the reinforcing device material
(iii) axial load transfer at interface and
(iv) Properties of grout and rock against crushing

## 2.4 DESIGN METHODS FOR REINFORCEMENT

The mechanics of load transfer between an anchor(bolt) and rock (ground) is fairly complex and rock structures, pre-existing discontinuities and their orientation with respect to the excavation, type of reinforcing device used and further its orientation with regard to the discontinuities play a major role in it. All these factors are required to be well accounted for a proper design of a reinforcement scheme besides the others. There are different designing approaches based upon the response of rock mass under pressure and the reaction required in the form of a reinforcement element. The rock mass response could be either of the two types.

(i) Continuous Rock Response
(ii) Discontinuous Rock Response

The rock response is of continuous type in all types of rocks, i.e., massive, stratified or jointed till the initiation of a fracture or failure along a pre-existing discontinuity. There exists a continuous distribution of normal and shear stresses within the rock mass which can be defined by continuous mathematical expressions. The discontinuous response occurs in stratified or jointed rock mass as soon as displacement takes place at pre-existing discontinuities and with the creation and propagation of the crack in case of massive rock. The stress is redistributed away from the discontinuity and is born by the more competent portion leaving this portion destressed and sometimes unstable also. This redistribution creates an irregularly distributed stress and strain field which cannot be defined by continuous mathematical expressions.

The most common approach is to assume the rock mass as homogeneous and isotropic, while designing a reinforcement scheme, showing a linear elastic relationship between stress and strain for a rock model. All the methods of design for a reinforcement scheme can be put into three categories.

(i) Analytical Methods

(ii) Computational Methods

(iii) Empirical Methods

All these methods have their field of application and limitations as well. It is upto the designers to choose the method most suitable taking into account the site related factors, since it is very difficult to precisely predict the behaviour of a deforming rock mass from continuous to discontinuous response region. Sometimes, the mechanism is simple and the shape of excavation also, hence it is possible to analyse the zone of possible failure and directions of displacement.

The computational methods are intended to identify the requirements for quantifying the effects of reinforcement on excavation behaviour and the methods which can potentially achieve these. The following table gives a general idea about the field of application pertaining to the different computational methods.

**Table - 2.2 : Computational Methods**

| Method | Rock mass type | Reinforcement model | Application |
|---|---|---|---|
| Boundary element | Continuum<br>Discontinuum | Equivalent material<br>Limited explicit | Stress analysis<br>Stress analysis |
| Finite difference | Continuum<br>Discontinuum | Equivalent material<br>Explicit | Stress analysis<br>Stress analysis |
| Finite difference | Continuum<br>Discontinuum | Equivalent material<br>Explicit | Stress analysis<br>Stress analysis<br>Modes of displacement |
| Distinct element | Continuum<br>Discontinuum | Equivalent material<br>Explicit | Stress analysis<br>Stress analysis<br>Modes of displacement |

Design evolutions based on precedents come under the category of empirical methods. It involves repeated review of reinforcement schemes applied in the past for similar type of conditions, over the years. The best example is the development of different Rock-mass classification systems. These systems require calculation of an indices or rating based on different parameters of rock mass such as, R Q D, number of joint sets, joint strength, orientation, stress reduction factors, etc. An advantage of this system is that we need little information in the form of input data and quickly arrive at a conclusion. However, it may not be particularly reliable but can very well serve as a guide.

## 2.5 ROCK MASS CLASSIFICATION

The rock mass classifications form the backbone of the empirical design approach. Most of the tunnels, mines, slopes and foundations constructed currently make use of a classification system. The purposes of classifying rock masses are given as under :

(i) to identify the most significant parameters influencing the behaviour of a rock mass,

(ii) to divide a particular rock mass formation into a number of rock mass classes of varying quality,

(iii) to provide a basis for understanding the characteristics of each rock mass class,

(iv) to derive quantitative data for engineering design,

(v) to recommend support guidelines for tunnels and mines,

(vi) to provide a common basis for communication between engineers and geologists, and

(vii) to relate the experience on rock conditions at one site to the conditions and experience encountered at others.

The system of rock mass classification is based on past experiences applied for design purposes and therefore the input data is of vital importance which forms the base for the accuracy. Following parameters are considered to be the most important for rock mass classification :

(i) the strength of rock material, which constitutes the upper strength limit of the rock mass, (Uniaxial compressive strength),

(ii) the rock quality designation (RQD),

(iii) basic geological parameters such as spacing, orientation and condition (roughness, separation, continuity, weathering and infilling) of discontinuities,

(iv) ground water conditions,

(v) Stress field and,

(vi) major faults and folds.

## Terzaghi's Rock Load Classification

In 1946, Terzaghi proposed a simple rock classification system for use in estimating the loads to be supported by steel arches in tunnels. He described various types of ground and, based upon his experience in steel-supported railroad tunnels in the Alps, he assigned ranges of rock load for various ground conditions.

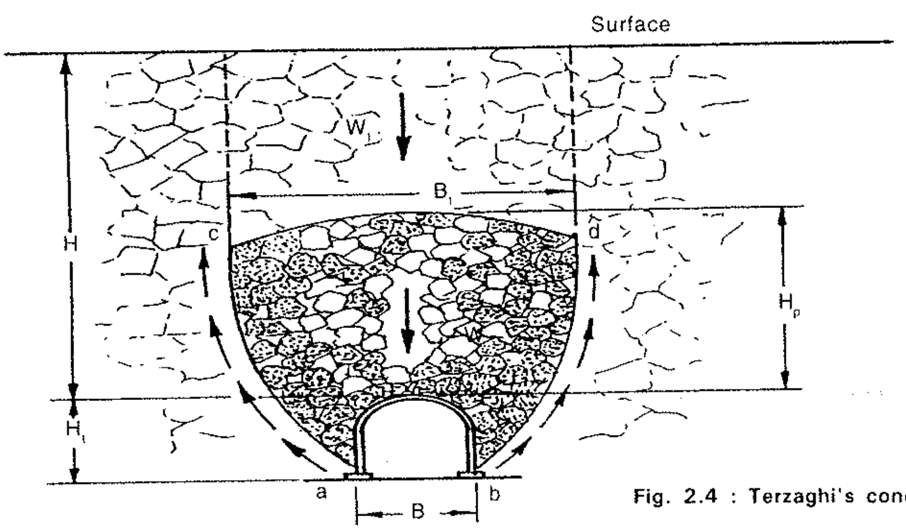



Fig. 2.4 : Terzaghi's concept of natural arch

According to Terzaghi, any excavation, if left unsupported, creates a zone of high stress concentration. This zone is formed in the shape of an arch over the roof of the excavation. The form and position of this arch, as shown in Figure 2.4 depends on the characteristics of the rock mass, excavation span and height. This arched zone is well capable to withstand and transfer the load to the sides of the excavation. The loosened rock below this arch, within the area *a,b,c,d* (Fig. 2.4) may require support. Hence, the designed support is required to negotiate the load of this portion of loosened rock, which will further take into account the frictional forces acting along the paths *ac* and *bd*. Based upon his experience with the steel supported tunnels, Terzaghi proposed the range of rock load values as listed below in Table 2.3.

**Table - 2.3 : Terzaghi's Rock Load Classification for Steel Arch Supported Tunnels**

Rock load $H_p$ (feet) of rock on roof of the tunnel with width B (feet) and height $H_t$ (feet) at a depth of more than $1.5(B + H_t)$.

| Rock condition | Rock load Hp in feet | Remarks |
|---|---|---|
| 1. Hard and intact | Zero | Light lining required only if spalling or popping occurs |
| 2. Hard stratified or schistose ** | 0 to 0.5 B | Light support, mainly for protection against spalls |
| 3. Massive, moderately jointed | 0 to 0.25 B | Load may change erratically from point to point |
| 4. Moderately blocky | $0.25B$ to $0.35(B + H_t)$ | No side pressure |
| 5. Very blocky and seamy | $(0.35$ to $1.10)(B + H_t)$ | Little or no side pressure |
| 6. Completely crushed but chemically intact | $1.10\ (B + H_t)$ | Considerable side pressure. Softening effects of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs. |
| 7. Squeezing rock, moderate depth | $(1.10$ to $2.10)(B + H_t)$ | Heavy side pressure, invert struts required. Circular ribs are recommended. |
| 8. Squeezing rock, great depth | $(2.10$ to $4.50)(B + H_t)$ | |
| 9. Swelling rock | up to 250 feet, irrespective of the value of $(B + H_t)$ | Circular ribs are required. In extreme cases use yielding support. |

* *The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty percent.*

** *Some of the most common rock formations contain layers of shale. In a unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in a tunnel like squeezing or even selling rock.*

*If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and the rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence in such formations, the roof pressure may be as heavy as in very blocky and seamy rock.*

Terzaghi stressed the importance of the geological survey during estimation of rock loads. According to him "from an engineering point of view, knowledge of the type and intensity of the rock defects may be much more important than the rock which will be encountered. Therefore, during the survey, rock defects should receive special consideration. The geological report should contain a detailed description of the observed defects in geological terms. It should also contain a tentative classification of the defective rock in the tunnel man's terms, such as blocky and seamy, squeezing or swelling rock". Such report would be of immense help in designing bolts for rock reinforcement.

Rock conditions mentioned by Terzaghi in his Classification are

**Intact rock :** It contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock.

**Stratified rock :** It consists of individual layers with little or no resistance against separation along the joint plane between strata.

**Moderately jointed rock :** It contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support.

**Blocky and Seamy :** This type of rocks consists of chemically intact or almost intact rock fragments, which are entirely separated from each other and imperfectly interlocked. In such rocks, vertical walls may require lateral support.

**Crushed rock :** It is a chemically intact rock where most or all of the fragments are as small as fine sand grains.

**Squeezing rock :** This advances slowly into the tunnel underground opening without perceptible volume increase. It contains a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

**Swelling rock :** It advances into the tunnel/underground opening mainly on account of expansion. It contains clay minerals with a high swelling capacity.

## Classification by Stini and Lauffer

A rock mass classification lays emphasis on the importance of structural defects in the rock mass and the need to avoid tunnelling parallel to the strike of steeply dipping discontinuities. It was Lauffer who emphasised the importance of the 'stand-up' time for the 'active span' in a tunnel. The stand-up time is the duration of time which an underground opening will withstand, unsupported. 'Active span' is the span between face and the supports. Lauffer suggested that the stand-up time for any given active span depends upon the rock mass characteristics. This is illustrated in the Figure 2.5.

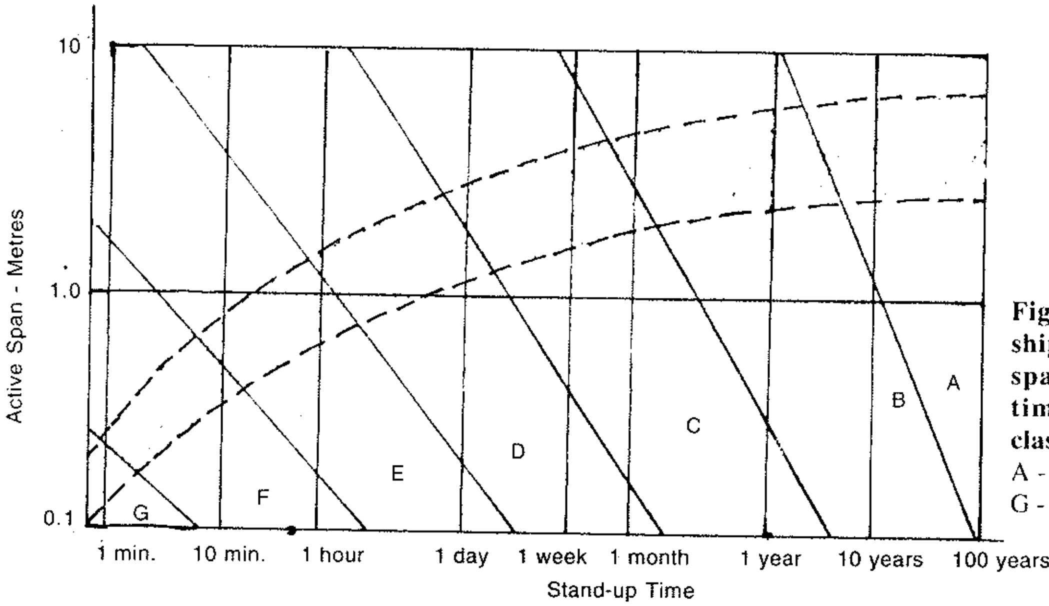



**Fig. 2.5 : Relationship between active span and stand-up time for different classes of rock mass**
A - very good rock,
G - Very poor rock

## Deere's Rock Quality Designation (RQD)

In 1964, Deere proposed a quantitative index of rock mass quality based upon core recovery by diamond drilling. The RQD is defined as the percentage of core recovered in intact pieces of 100 mm or more in length in the total length of a borehole. Therefore,

$$\text{RQD}(\%) = 100 \times \frac{\text{Length of Core in pieces} > 100\text{ mm}}{\text{Length of borehole}}$$

The core should be drilled with double barrel diamond drilling equipment and should have a diameter of at least 50 mm. The relationship between the RQD and type of rock, as proposed by Deere is given below.

| RQD | Rock Quality |
|---|---|
| < 25% | Very poor |
| 25 - 50% | Poor |
| 50 - 75% | Fair |
| 75 - 90% | Good |
| 90 - 100% | Very good |

Merritt tried to extend the range of applicability of RQD for estimating tunnel support requirements. As a result of which he pointed out that "The RQD support criteria system has limitations in areas where the joints contain thin clay fillings or weathered material. This would result in unstable rock despite RQD being high". The efforts of Merritt are illustrated in the Figure 2.6.

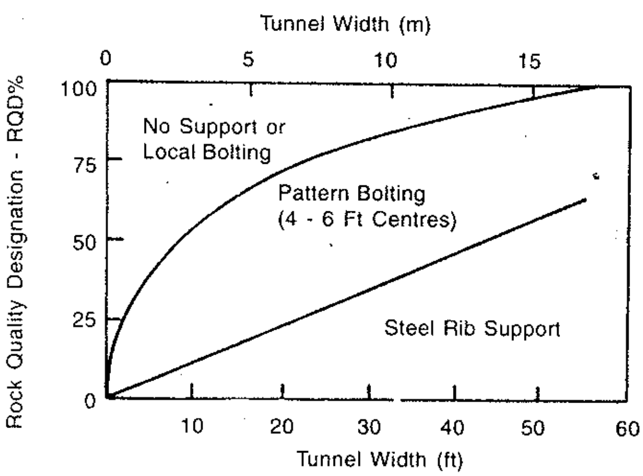



**Fig. 2.6 : Proposed use of RQD for choice of rock support system suggested by Merritt**

In addition to this limitation, the RQD does not take into account the factors like joint orientation.

Brekke and Howard pointed out that it is equally important to classify the discontinuities as well, which has a very vital role in the ultimate behaviour of rock mass under load. This behaviour classification is presented in Table 2.4.

**Table - 2.4 : Influence of Discontinuity Infilling Upon the Behaviour of Tunnels**

| Dominant material in gouge | Potential behaviour of gouge material At face | Later |
|---|---|---|
| Swelling clay | Free swelling, sloughing. Swelling pressure and squeeze on shield. | Swelling pressure and squeeze against support or lining, free swell with down-fall or wash-in if lining inadequate. |
| Inactive clay | Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions. | Squeeze on supports of lining where unprotected, slaking and sloughing due to environmental changes. |
| Chlorite, talc, graphite or serpentine. | Ravelling | Heavy loads may develop due to low strength, particularly when wet |
| Crushed rock fragments of sand-like gouge. | Ravelling or running. Stand-up time may be extremely short. | Loosening loads on lining, running and ravelling if unconfined. |
| Porous or flaky calcite, gypsum | Favourable conditions. | May dissolve, leading to instability of rock mass. |

## Geomechanics Classification

In 1972-73, Bieniawski developed a system to assign the importance ratings based on the various parameters of a rock mass. This is known as Geomechanics classification or more commonly known as Rock Mass Rating(RMR) system. The system originally involved 49 case histories, followed by 62 case histories added by Newman and Bieniawski and 78 more tunnelling and mining case histories between 1984 and 1989. With over three hundred and fifty case studies, the RMR system has very well stood the test of time. The system has been modified repeatedly over the years depending upon the specific uses but the basics remains the same. The present modified form of the system is summarised below :

There are six parameters used in RMR system.

1. Uniaxial compressive strength of rock material
2. Rock Quality Designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Ground Water State
6. Orientation of discontinuities

All the above parameters have been assigned importance rating based on their nature (Table 2.5).

**Table - 2.5 : The Rock Mass Rating System (Geomechanics Classification)**

**(a) Classification Parameters and their Ratings**

| | Parameter | | Ranges of values | | | | |
|---|---|---|---|---|---|---|---|
| 1. | Strength of intact rock material | Point load strength index (MPa) | >10 | 4-10 | 2-4 | 1-2 | For this low range, uniaxial compressive test is preferred |
| | | Uniaxial compressive strength (MPa) | >250 | 100-250 | 50-100 | 25-50 | 5-25  1-5  <1 |
| | Rating | | 15 | 12 | 7 | 4 | 2  1  0 |
| 2. | Drill core quality RQD(%) | | 90-100 | 75-90 | 50-75 | 25-50 | <25 |
| | Rating | | 20 | 17 | 13 | 8 | 3 |
| 3. | Spacing of discontinuities | | >2m | 0.6-2 m | 200-600 mm | 60-200 mm | <60 mm |
| | Rating | | 20 | 15 | 10 | 8 | 5 |
| 4. | Condition of discontinuities | | Very rough surfaces Not continuous No separation Unweathered wall rock | Slightly rough surfaces Separation < 1 mm Slightly weathered walls | Slightly rough surfaces Separation < 1 mm Highly weathered wall | Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous | Soft gouge > 5mm thick or Separation > 5 mm Continuous |
| | Rating | | 30 | 25 | 20 | 10 | 0 |

(Table - 2.5 Contd.)

(Table - 2.5 Concld.)

| Parameter | | Ranges of values | | | | |
|---|---|---|---|---|---|---|
| 5. Groundwater | Inflow per 10 m tunnel length (L $min^{-1}$) | None or | <10 or | 10-25 or | 25-125 or | >125 or |
| | Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$ | 0 | <0.1 | 0.1-0.2 | 0.2-0.5 | >0.5 |
| Rating | General conditions | Completely dry | Damp | Wet | Dripping | Flowing |
| | | 15 | 10 | 7 | 4 | 0 |

**(b) Rating Adjustment for Discontinuity Orientations**

| Strike and dip orientations of discontinuities | | Very favourable | Favourable | Fair | Unfavourable | Very Unfavourable |
|---|---|---|---|---|---|---|
| Ratings | Tunnels and mines | 0 | - 2 | - 5 | - 10 | - 12 |
| | Foundations | 0 | - 2 | - 7 | - 15 | - 25 |
| | Slopes | 0 | - 5 | - 25 | - 50 | - 60 |

**(c) Rock Mass Classes Determined from Total Ratings**

| | | | | | |
|---|---|---|---|---|---|
| Rating | 100-81 | 80-61 | 60-41 | 40-21 | < 20 |
| Class | I | II | III | IV | V |
| Desc ription | Very good rock | Good rock | Fair rock | Poor rock | Very poor rock |

**(d) Meaning of Rock Mass Classes**

| Class | I | II | III | IV | V |
|---|---|---|---|---|---|
| Average stand-up time | 20 y for 15 m span | 1 y for 10 m span | 1 week for 5 m span | 10 h for 2.5 m span | 30 min for 1 m span |
| Cohesion of the rock mass(kPa) | > 400 | 300-400 | 200-300 | 100-200 | < 100 |
| Friction angle of the rock mass(deg) | > 45 | 35-45 | 25-35 | 15-25 | < 15 |

The above table has four sections a,b,c, and d. Section 'a' contains importance rating for first five parameters excepting the orientation of discontinuities. All the different ratings assigned to the above five parameters are added to obtain an unadjusted Rock Mass Rating.

The sixth parameter is discontinuity conditions which are categorised based on the classification of discontinuities originally suggested by Wickham and later modified. This classification is given in Table 2.6

**Table 2.6 : The RMR System : Guidelines for Classification of Discontinuity Condition**

| Parameter | Ratings | | | | |
|---|---|---|---|---|---|
| Discontinuity length (persistence/continuity) | < 1 m | 1-3 m | 3-10 m | 10-20 m | > 20 m |
| | 6 | 4 | 2 | 1 | 0 |
| Separation (aperture) | None | < 0.1 mm | 0.1-1.0 mm | 1-5 mm | > 5 mm |
| | 6 | 4 | 2 | 1 | 0 |
| Roughness | Very rough | Rough | Slightly rough | Smooth | Slickensided |
| | 6 | 5 | 3 | 1 | 0 |
| | | | Hard filling | | |
| Infilling(gouge) | None | < 5 mm | > 5 mm | < 5 mm | > 5 mm |
| | 6 | 4 | 2 | 2 | 0 |
| Weathering | Unweathered | Slightly weathered | Moderately weathered | Highly weathered | Decomposed |
| | 6 | 5 | 3 | 1 | 0 |

*These conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge .In such cases, use Table 2.5 directly.*

After knowing the nature (i.e. favourable, unfavourable etc.) of discontinuities, the unadjusted rock mass rating is adjusted for the sixth parameter as per the section 'b' of Table 2.5. Section 'c' of the Table defines the type of rock mass based on the final rock mass ratings. The rating holds good for civil engineering purposes but, for mining applications or deep tunnels, some minor adjustments are to be made and these are given below.

(i) Blasting damage adjustment ........ (a)
rating range (0.8 - 1.0)

(ii) Adjustment for in situ stress and change in stress ........ (b)
rating range (0.6 - 1.2)

(iii) Adjustment for major fault and fracture ........ (c)
rating range (0.7 - 1.0)

The final Mining Rock Mass Rating is RMR x (a) x(b) x(c)
Where maximum (a) x (b) x (c) = 0.5

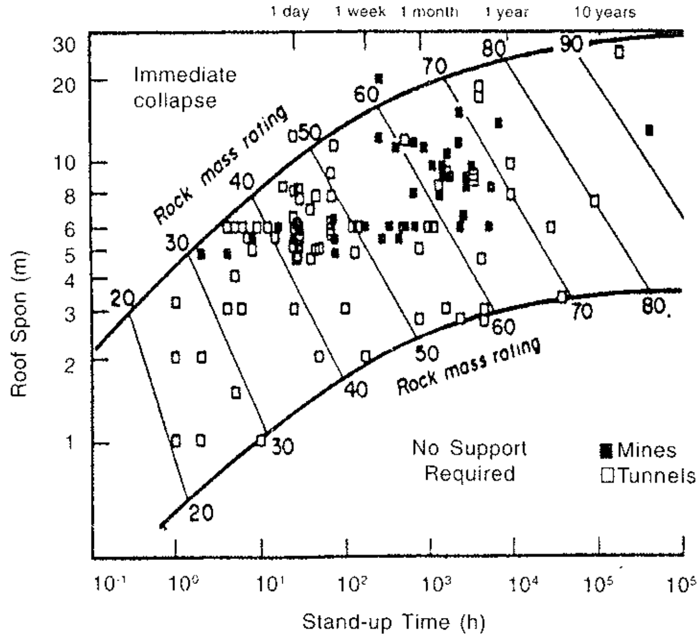



**Fig. 2.7 : Use of RMR System output**

The use of the output from RMR system in the form of stand-up time and maximum stable rock span for a given RMR is shown in Figure 2.7.

There may be a typical condition in mining or tunnelling application when an excavation face contains mixed variety of rocks. In such a situation it is essential to judge the RMR based upon the most crucial feature in the stability, such as, a fault or dyke which play a significant role even if the rock mass is very strong. In case, two distinct zones of rocks are available, the RMR for each zone should be calculated separately and then the weighted average based on the corresponding area or the influence may be taken as actual RMR. The final use of RMR system for selection of reinforcements for tunnels is given in Table 2.7.

**Table - 2.7 : Use of RMR System for Selection of Support**

| Rock mass class | Excavation | Rockbolts(20 mm diameter, fully grouted) | Support Shotcrete | Steel sets |
|---|---|---|---|---|
| Very good rock, I RMR:81-100 | Full face, 3 m advance | Generally no support required except for occasional spot bolting | | |
| Good rock, II RMR:61-80 | Full face, 1.0-1.5 m advance. Complete support 20 m from face | Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh | 50 mm in crown where required | None |
| Fair rock, III RMR:41-60 | Top heading and bench, 1.5-3m advance in top heading. Commence support after each blast. Complete support 10m from face | Systematic bolts 4 m long, spaced 1.5-2m in crown and walls with wire mesh in crown | 50-100 mm in crown and 30 mm in sides | None |
| Poor rock, IV RMR: 21-40 | Top heading and bench, 1.0-1.5 m advance in top heading. Install support concurrently with excavation 10 m from face | Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh | 100-150 mm in and 100 mm in | Light to medium ribs spaced 1.5 m where required |
| Very poor rock, V RMR:_ 20 | Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting | Systematic bolts 5-6 m long, spaced 1-1.5m in crown and walls with wire mesh. Bolt invert | 150-200 mm in crom 150 mm in sides & 50 mm on face | Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert |

* *Shape : horsehoe: width: 10m; vertical stress: 25 MPa; construction: drilling and blasting.*

## NGI Tunnelling Quality Index

Barton, Lieh and Lunde of the Norwegian Geotechnical Institute (NGI) proposed a tunnelling quality index 'Q' based upon a number of case studies related to underground excavation stability. The index Q is defined as

$$Q = \left(\frac{RQD}{Jn}\right) \times \left(\frac{Jr}{Ja}\right) \times \left(\frac{Jw}{SRF}\right)$$

Where RQD = Deere's Rock Quality Designation
Jn = Joint set number
Jr = Joint roughness number
Ja = Joint alteration number
Jw = Joint water reduction factor
SRF = Stress Reduction Factor

Tunnelling quality index Q depends on a combination of three parameters.

(i) Block size (RQD/Jn)
(ii) Inter-block shear strength (Jr/Ja)
(iii) Active stress (Jw/SRF)

The first parameter RQD/Jn represents the structure of the rock mass. It is a crude measure of block or particle size. The second parameter (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling material. This is loaded in favour of rough unaltered joints in direct contact. The third parameter (Jw/SRF) is a complex empirical factor describing 'active stresses', where SRF is a measure of loosening loads through a shear zone, rock stress in competent rock and squeezing loads in plastic incompetent rocks. Jw is a measure of adverse effect on shear strength of joints due to water pressure. The system as a whole is summarised in Table 2.8.

**Table - 2.8 : Parameters and values of Tunnelling Quality Index**

| Description | Value | Notes |
|---|---|---|
| 1. ROCK QUALITY DESIGNATION (RQD) | | |
| A. Very Poor | 0 - 25 | 1. Were RQD is reported or measured as 10 (including 0), a nominal value of 10 is used to evaluate Q. |
| B. Poor | 25 - 50 | |
| C. Fair | 50 - 75 | |
| D. Good | 75 - 90 | 2. RQD intervals of 5, i.e. 100, 95, 90...etc are sufficiently accurate. |
| E. Excellent | 90 - 100 | |
| 2. JOINT SET NUMBER (Jn) | | |
| A. Massive, no. of few joints | 0.5 - 1.0 | |
| B. One joint set | 2 | |
| C. One joint set plus random | 3 | |
| D. Two joint sets | 4 | |
| E. Two joint set plus random | 6 | |
| F. Three joint sets | 9 | 1. For intersections use (3.0 x Jn) |
| G. Three joint set plus random | 12 | |
| H. Four or more joint sets, random, heavily jointed 'Sugar cube', etc. | 15 | 2. For portals use (2.0 x Jn) |
| I. Crushed rock, earthlike | 20 | |

(Contd.)

(Table - 2.8 Contd.)

| Description | Value | Notes |
|---|---|---|
| 3. JOINT ROUGHNESS NUMBER (Jr) | | |
| a) Rock wall contact | | |
| b) Rock wall contact before 10 cm shear | | |
| A. Discontinuous joints | 4 | |
| B. Rough or irregular, undulating | 3 | |
| C. Smooth, undulating | 2 | |
| D. Slickensided, undulating | 1.5 | 1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. |
| E. Rough or irregular, planar | 1.5 | |
| F. Smooth, planar | 1.0 | 2. Jn = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength. |
| G. Slickensided, planar | 0.5 | |
| c) No rock wall contact when sheared. | | |
| H. Zone containing clay minerals thick enough to prevent rock wall contact. | 1.0 | |
| J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact. | 1.0 | |
| 4. JOINT ALTERATION NUMBER (Ja) (a) Rock wall contact. | 0r (approx) | |
| A. Tightly healed, hard, non-softening, impermeable filling | 0.75 | |
| B. Unaltered joint walls, surface staining only | 1.0 (25°-35°) | |
| C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc. | 2.0 (25° -30°) | 1. Values of 0r the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present. |
| D. Silty, or sandy-clay coatings, small clay-fraction (non-softening) | 3.0 (20° - 25°) | |
| E. Softening or low friction clay mineral coatings, i.e.kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness) | 4.0 (8° - 16°) | |
| b) Rock wall contact before 10 cm shear. | | |
| F. Sandy particles, clay-free disintegrated rock etc. | 4.0 (25° - 30°) | |
| G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5 mm thick) | 6.0 (16° - 24°) | |

(Contd.)

(Table - 2.8 Contd.)

| Description | Value | Notes |
|---|---|---|
| H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick) | 8.0 (12° - 16°) | |
| I. Swelling clay fillings, i.e. montmorillonite (continuous, 5 5 mm thick). Values of Ja depend on percent of swe lling clay-size particles and access to water | 8.0 - 12.0 (6° - 12°) | |
| c) No rock wall contact when sheared. | | |
| J. Zones or bands of disintegrated | 6.0 | |
| K. or crushed rock and clay (see | 8.0 | |
| L. G.H and J for clay conditions) | 8.0 - 12.0 (6° - 24°) | |
| M. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening) | 5.0 | |
| N. Thick, continuous zones or | | |
| O. bands of clay (see G, H and | 10.0 - 13.0 | |
| P. J for clay conditions) | 13.0 - 20.0 (6° - 24°) | |

| 5. JOINT WATER REDUCTION FACTOR | Jw | approx. water (pressure (kgf/cm²)) | Notes |
|---|---|---|---|
| A. Dry excavations or minor inflow i.e. < 5 lit/min. locally | 1.0 | < 1.0 | |
| B. Medium inflow or pressure, occasional outwash of joint fillings | 0.66 | 1.0 - 2.5 | |
| C. Large inflow or high pressure in competent rock with unfilled joints | 0.5 | 2.5 - 10.0 | 1 Factors C to F are crude estimates. Increase Jw if drainage measures are installed. |
| D. Large inflow or high pressure considerable outwash of fillings | 0.33 | 2.5 - 10.0 | |
| E. Exceptionally high inflow or pressure at blasting, decaying with time | 0.2 - 0.1 | > 10 | 2. Special problems caused by ice formation are not considered. |
| F. Exceptionally high inflow or pressure continuing without decay | 0.1 - 0.05 | > 10 | |

6. STRESS REDUCTION FACTOR (SRF)

a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

| | |
|---|---|
| A. Multiple occurrences of weakness zones | 10.0 |

(Contd.)

(Table - 2.8 Contd.)

| Description | Value | Notes |
|---|---|---|
| containing clay or chemically disintegrated rock, very loose surrounding rock (any depth) | | 1. Reduce these values of SRF by 25-50%, if the relevant shear zones only influence but do not intersect the excavation. |
| B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m) | 5.0 | |
| C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m) | 2.5 | |
| D. Multiple shear zones in competent rock (clay free, loose surro unding rock (any depth) | 7.5 | |
| E. Single shear zones in competent rock (clay free), (depth of excavation < 50 m) | 5.0 | |
| F. Single shear zones in competent rock (clay free) (depth of excavation > 50 m) | 2.5 | 2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma 1/\sigma 3 \leq 10$, reduce $o_c$ to $0.8o_c$ and $0_t$ to $0.8o_t$, when $\sigma 1/\sigma 3 > 10$, reduce $o_c$ and $\sigma 1/\sigma 3 > 10$, reduce $\sigma c$ and $\sigma t$ to $0.6\sigma c$ and $0.6\sigma t$, where $\sigma c$ = unconfined compressive strength, and $\sigma t$ = tensile strength (point load) and $\sigma 1$ and $\sigma 3$ are the major and minor principal stresses. |
| G. Loose open joints, heavily jointedor 'sugar cube' (depth of excavation > 50 m) | 5.0 | |
| b) Competent rock, rock stress problems | | |
| H. Low stress, near surface | >200 >13 2.5 | |
| I. Medium stress | 200-10 13-0.66 1.0 | |
| J. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability) | 10-5 0.66-0.33 0.5-2 | 3. Few case records available where depth of crown than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H). |
| K. Mild rock burst (massive rock) | 5-2.5 0.33-0.16 5-10 | |
| L. Heavy rock burt (massive rock) | < 2.5 <0.16 10-20 | |
| c) Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure | | |
| M. Mild squeezing rock pressure | 5-10 | |
| N. Heavy squeezing rock pressure | 10-20 | |

| | | |
|---|---|---|
| | d) Swelling rock, chemical swelling activity depending upon presence of water | |
| O. | Mild swelling rock pressure | 5-10 |
| P. | Heavy swelling rock pressure | 10-20 |

## Additional Notes on the Use of These Tables

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables :

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :

   RQD = 115 - 3.3Jv (approx.), where Jv = total number of joints per $m^3$

   (RQD = 100 for Jv < 4.5)

2. The parameter Jn representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible,or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating Jn.

3. The parameters Jr and Ja (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (Jr/Ja) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of (Jr/Ja) should be used when evaluating Q. The value of (Jr/Ja) should in fact relate to the surface most likely to allow failure to initiate.

4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.

5. The compressive and tensile strengths ($\sigma c$ and $\sigma t$) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A conservative estimate or strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

To make use of tunnelling quality index Q to describe the support requirements of an underground excavation, Barton, Lien and Lunde proposed another factor known as equivalent dimension De of the excavation.

$$De = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation support ratio}}$$

The excavation support ratio (ESR) for different kind of excavations is given as under :

| | Excavation | ERS |
|---|---|---|
| (i) | Temporary mine openings | 3-5 |
| (ii) | Permanet mine openings, water tunnels for hydro-power (excluding high pressure penstocks) pilot tunnels, drifts and headings for large excavations. | 1.6 |
| (iii) | Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels | 1.3 |
| (iv) | Power stations, major road and railway tunnels, civil defence chambers, portals, intersections | 1.0 |
| (v) | Underground nuclear power stations, railway stations, sports and public facilities, factories | 0.8 |

The relationship between the tunnelling quality index Q and the equivalent dimension De, which will stand unsupported is given in the Figure 2.8.

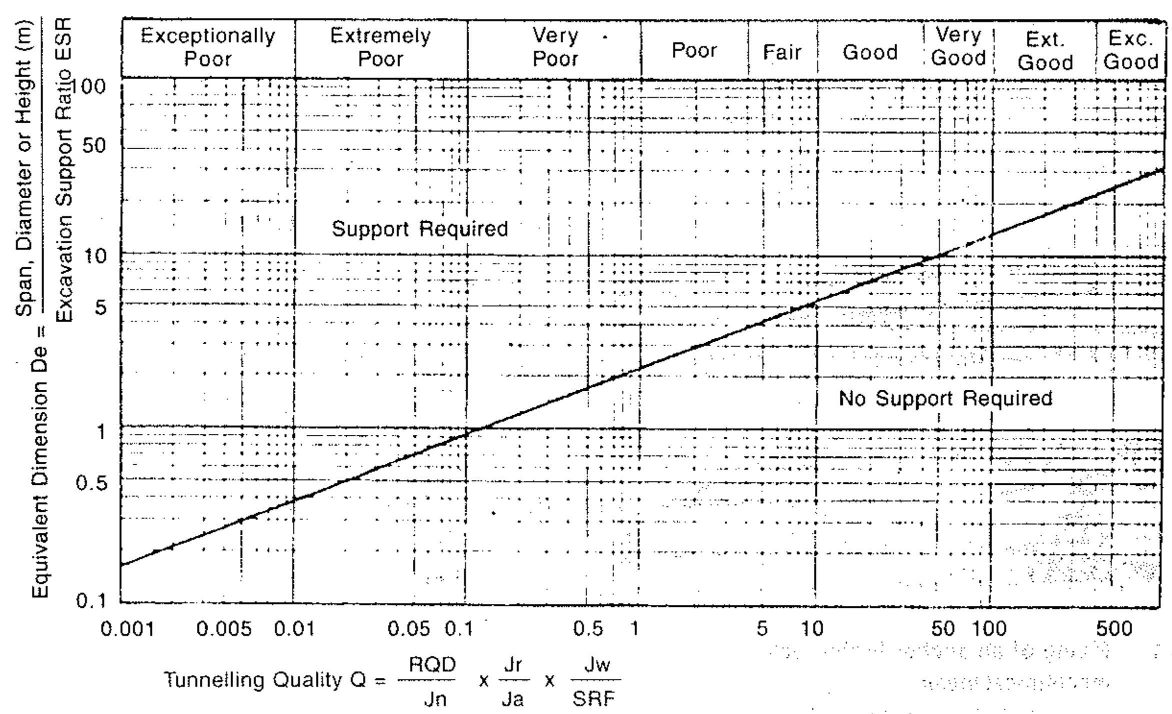



**Fig. 2.8 : Relationship between the maximum equivalent dimension De of an unsupported underground excavation and the NGI tunnelling quality Index Q**

Chapter 3

# Installation of Bolts

Bolt installation technique is a simple and critical activity undertaken to treat the quantum of load mechanism. A properly installed bolt will give the desired end results, whereas, a badly installed bolt may not work at all and may result in rock failure. There are various ways, through which different types of bolts can be installed for different purposes. The phenomenon of load transfer is a common feature in bolt installation. It is most appropriate to classify the various modes of bolt installation based on the manner in which they transfer the load. The methods of bolt installation are under three categories :

(i) Mechanical fixing

(ii) Fixing with Cement & resins

(iii) Fixing with abutting bases.

## 3.1 MECHANICAL FIXING

Bracing the anchor against the walls of the borehole stresses the rock upto 10 MPa usually in the short sections of the length over which the anchor is fixed. Thus the use of this method is limited to strong rocks where relatively small tensile forces (upto 0.2 MN) are involved. The fixing of the anchor in the borehole is achieved by a mechanical expanding or bracing device (base), which works on the principle of a wedge and which is fixed at the anchor foot in the borehole as shown in Figure 3.1 (a) and (b).

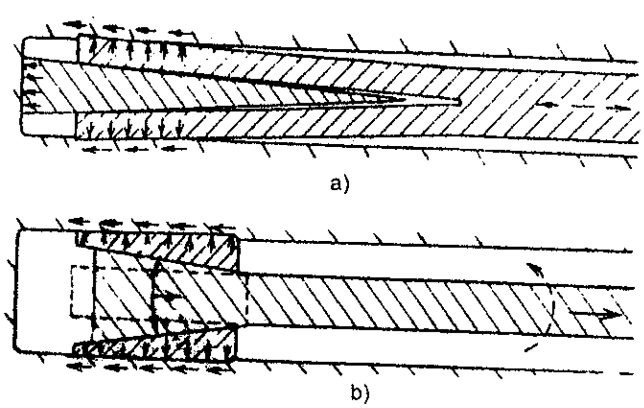



**Fig. 3.1 : Fixing of an anchor in the rock by mechanical means**
a) by ramming a wedge into the split rod end
b) by pulling a cone into the expanded sleeve

The device is expanded in the borehole by driving in, pulling out, or turning the anchor rod, depending on the mechanism employed. The tensile force is transferred from the anchor to the rock by friction at those points where the root has been forced into the borehole wall. The advantage of mechanical anchoring is its immediacy, as soon as the anchor is inserted and fixed, it may be loaded or pre-stressed.

## Fixing by Thrust

The bolts in this case are installed in a borehole by pushing hard up the hole with the help of a hammering action. The most common type is wedge-bolt. This is made of a circular section of steel having a longitudinal cut at its base (root) thus forming a cleft. It is provided with a wedge to be pushed into this cleft. During the installation the bolt is pushed into the hole with wedge inside the clefts. Now the bolt is hammered up the hole thus forcing the wedge deep into the cleft. Since driving the bolt with the help of a mallet has little effect, the best way is to use a medium size pneumatic pick fitted with a cut away bit and guide pipe. This type of bolt can be used successfully in all hard rocks. 70 to 120 kN load bearing capacities can be achieved and if installed properly there is hardly any decrease in the fixing strength, even after several years.

## Fixing by Tension

The bolts with tensile bases are also made of steel sections and carry a cuneiform of conical support at their base/root. These bolts are installed by pulling this cuneiform initially by hand and then with the help of a nut provided at the external end. The basic action remains the same as that of wedge-bolt since installation is achieved by expanding the root elements of the bolt assembly in both the cases but in this case it is achieved by applying tension in the form of pull whereas in wedge-bolt it is thrust in the form of hammering. The Goldenberg bolt (Figure 3.2), GD anchor and Worley bolt (Figure 3.3) are the bolts with tensile bases

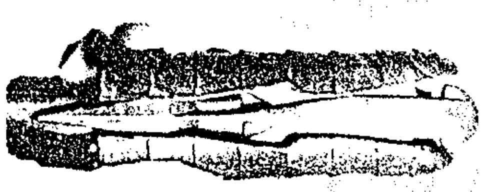


**Fig. 3.2 : Tensile anchor base of Goldenberg bolt**

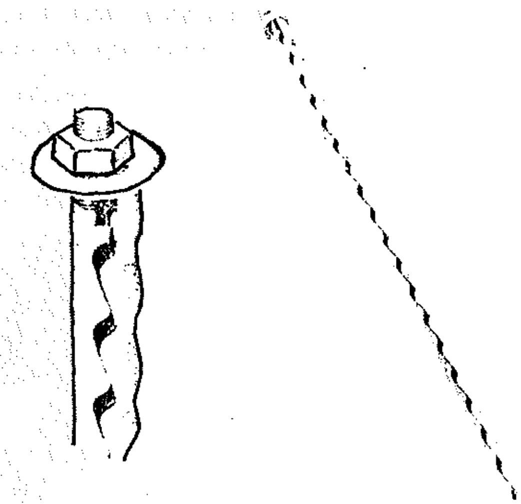


**Fig. 3.3 : Worley bolt with anchoring effect along entire borehole length**

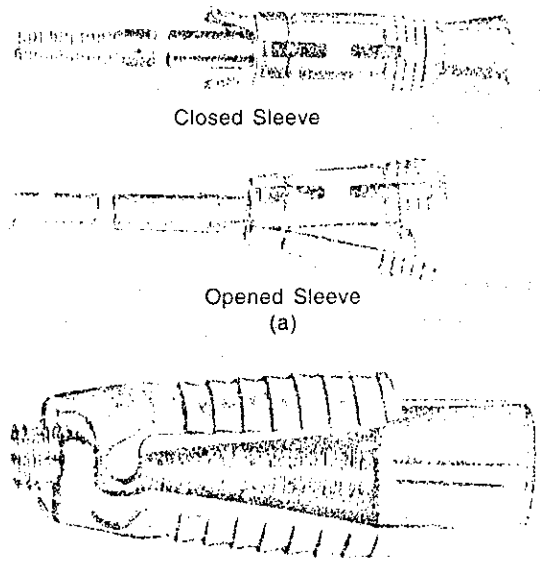



**Fig. 3.4 : Bolts with threaded base**
a) Bayliss bolt
b) Bail bolt

## Fixing by Screwing

These bolts are usually made out of circular steel sections with their base/root threaded to draw a cuneiform piece provided at the root between the jaws. A number of expansion shell bolts are provided with threaded base. The bolt is installed by first inserting in the borehole and then by rotating it to expand the jaws against the borehole walls. The initial tightening may be carried out by hand turning using 50 cm lever or a torque wrench for appropriate tightening. A torsional moment of 350 Nm induces a pre-stress of about 70 kN in the bolt. A few such bolts are shown in Figure 3.4. The load bearing capacity of such bolts can be from 170 to 220 kN in strong rocks.

## Fixing by Expansion of Contraction

This type of bolts are made of steel tubes and are fixed either by expanding an undersized section or by contracting an oversized section all along the length of the borehole. These bolts are also known as friction bolts—since resistance against any displacement of such bolts is achieved due to friction between rock and bolts.

Swellex bolts (Figure 3.5) consists of a mechanically reshaped tabular section with an original diameter of 41 mm but assume a diameter of 28 mm carrying two sleeves fitted and sealed at both the ends. The sleeve at outer end of the bolt has a provision of water inlet. This is installed in a borehole of 35 mm by simply inserting in and then pumping water at high pressure through the inlet port at outer sleeve. High pressure water expands it immediately thus providing a quick installation.

Contrary to this an oversized tabular section is of around 38 mm diameter and is provided with a slot of about half inch cut all along the length. This is pushed in an undersized hole of 35 mm diameter during installation. It is shown in Figure 3.6 and is known by its trade name split set stabilizer.

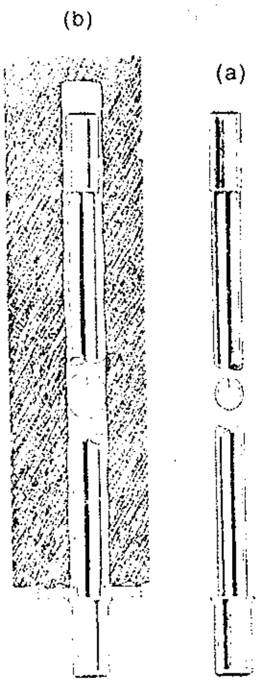



**Fig. 3.5 : Swellex bolt**
a) Bolting tube before expansion
b) Bolting tube fixed in the borehole by expansion

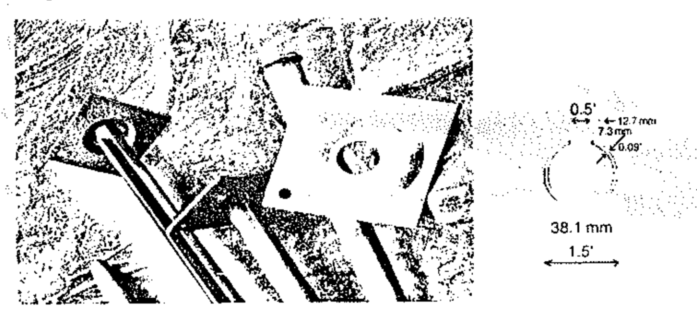



**Fig. 3.6 : Splitset Stabilizer**
a) A view of the bolting tube with rings and washers
b) Cross-section of the slotted tube

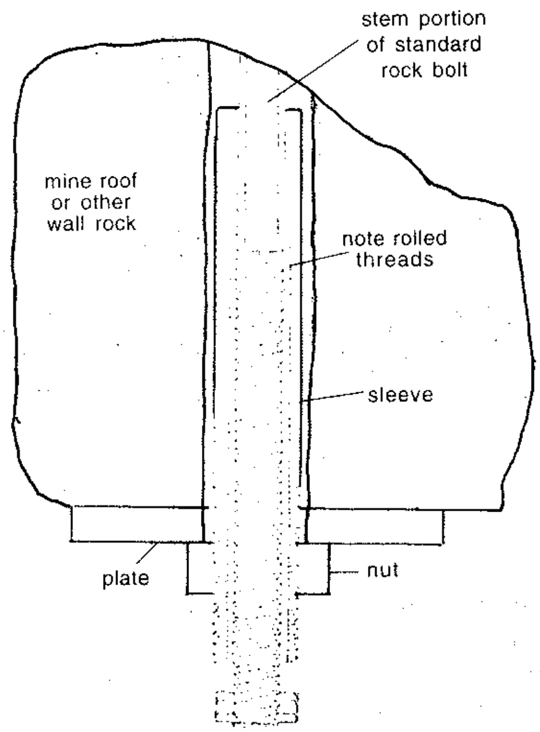



**Fig. 3.7 : Controlled yielding coupling for standard rock bolts**

## Controlled Yielding Bolts

Slight yielding of reinforcing elements and supports is very beneficial in mining applications. Ordinary rock bolt yield about 18% before failure. A little bit more yielding will be helpful in providing a new state of equilibrium to the rock mass without making the reinforcement scheme to fail. With this concept a controlled yielding coupling was developed in the USA. This elongation coupling of the bolt, as shown in Figure 3.7 consists of a sleeve around the outer part of the stem of a standard bolt, the latter being provided with rolled threads with an over all diameter greater than that of the stem. Because the inside diameter of the inner end of the sleeve is less than the diameter of the threads on the bolt, the bolt may be pre-stressed to the permitted load by tightening the nut threaded on the external section of the sleeve. When the loading of the bolt reaches a predetermined limit value, the threads on the bolt stem are partially stripped with the stem moving into the sleeve to some extent. In this way tension in the bolt automatically drops below the admissible limit, which is determined by the strength of the threads. This type of bolt allows a yielding upto 50 cm while transferring a constant tensile force of about 214 kN.

## 3.2 FIXING WITH CEMENT AND RESINS

Bolt installation with cement grouting is the most popular and simple in nature. It is based on the cohesive bonding of a suitable cement ( or synthetic resin) with the tendon as well as rock over a long section of the borehole. The cement grout is pumped in the borehole and it starts transferring the load only after hardening. Hence, it takes some time before offering any resistance. This is the cheapest mode of reinforcement. Usually cement, sand and water mixture in the ratio 1:1:0.5 is used to prepare a grout. Sometime accelerators are added for early setting of the mixture. The optimum dose may be around 2% of cement by weight.

In certain applications, where quick setting is required, synthetic resins are used in place of a grout mix. The cohesion developed between synthetic resin and strong rocks is two to three times greater than that between grout and rock. Resins also exhibit excellent resistance to the corrosive effects of the rock medium and the dynamic effects of shocks. The disadvantage is the high cost as compared to the cement grout. Since the mode of installation requires resin capsules, usually short bar anchors are used in this method.

## Installation Procedure for Cable Bolts

(i) The drilled borehole is cleaned thoroughly.

(ii) The cable to be grouted is inserted in the borehole to reach the bottom of the borehole. The breather tube, grouting pipe and sealing plug is arranged with it.

(iii) Grouting mixture is pumped in the hole through grouting pipe gradually by allowing the air to vent out through the breather tube.

(iv) As soon as the complete filling of hole is indicated grouting is stopped.

(v) Some times specially designed cable is provided with a hole in the centre all along the length which serves as a breather tube. As soon as the hole is filled completely the grouting mixture oozes out from this hollow portion indicating the termination of grouting.

## Fixing with Abutting Bases

Anchor roots which transmit large tensile forces especially in soils, should be designed as bases abutting on to load distribution structures built at or sunk to, an appropriate depth of the ground. The load distribution structures are usually sunk in to reinforced concrete parapet walls, trench walls etc. Since this type of reinforcement is usually associated with large surface structures and falls out of context, further details are not discussed.

## 3.4 BOREHOLE DRILLING

The drilling of boreholes is usually the costliest operation in bolting. Hence, the most efficient drilling methods have to be selected along with careful estimation of time schedule. Borehole drilling can be broadly classified into two categories.

(i) Small diameter boreholes for short anchors

(ii) Large diameter boreholes for long anchors.

Ordinary hand-operated percussion drills are suitable for boreholes upto 3 to 4 m in length and 45 mm in diameter. For this purpose a number of multi-purpose pneumatic hammer drills are available.

For long boreholes, percussion drilling, rotary drilling or the combination of the two may be used. A rotary drill transmits axial thrust and rotational torque to the rock. In general, circular three-cone (roller) or auger bit is used.

Multi-function drill jumbos and rock bolters are available for quick drilling and installation of bolts. The selection of most suitable drilling equipment depends upon the following factors :

a) Type and quality of rock.

b) The diameter and length of the borehole.

c) The accessibility of the anchoring site.

d) The type of flushing medium to be used.

e) Type of anchor to be installed.

f) The required drilling rate of the machine.

Mc Gregor laid down general guidelines for selection of the most suitable drilling method based on the diameter of hole and type of rock. It is illustrated in Figure 3.8.

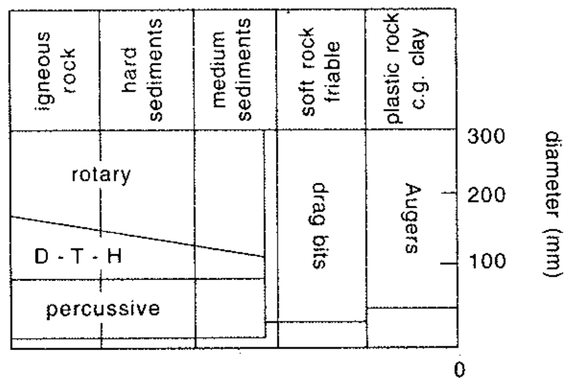



**Fig. 3.8 : Preferred methods of drilling according to class of rock and hole diameter**
(D - T - H = down-the-hole hammer)

Some of the important factors associated with borehole drilling for anchor installation are described as under :

## Work Rate of Drilling Equipment

The work rate of a drilling machine is measured in terms of the length of a borehole of pre-determined diameter drilled per unit time. Usually it is determined in a direct test on rock of a defined type. The work rate of machine depends upon :

a) Condition of the machine and drill bits,

b) the flushing method employed,

c) the air pressure in case of pneumatic machine,

d) the torque developed in case of rotary machines,

e) rock type and bore hole diameter.

## Drillability of Rock

The drillability of a particular rock is the power output required to drill a borehole in it. It depends upon the following factors :

a) Hardness of the rock,

b) Mineral composition,

c) Strength of the rock,

d) Grain size,

e) Porosity,

f) Stratification,

g) Density,

h) Direction of joints.

## Borehole Defects

The most common defects of anchor boreholes arise from incorrect positioning and alignment of the boreholes and deviation or curvature of the axis away from the intended direction and inclination. The correct alignment of a borehole is a matter of careful preparation and the use of appropriate survey instruments. The straightness of the borehole direction depends on the drilling set and drilling technology. The borehole become curved when the drilling rods are too slender, excessive thrust is applied or there is a tendency for the bit to follow joints or other planer rock features which cross the borehole obliquely specially when harder rock follows a softer rock oblique to the direction of hole. Boreholes drilled by the percussion down-the-hole method show the least tendency to curving. If the curvature of the borehole is sufficiently great, the drill rods damage the borehole walls and cause fragments of rock to fall into the borehole.

## Permeability of Anchor Boreholes

The pressure tests in strong rock are carried out in borehole sections from 1 to 5 m long, using one or two scale to observe the loss of water and to establish permeability of hole. This test defines the most suitable type of grout for fixing and its volume, pressure and anticorrosive protection. If the water losses are considerable during the test, the permeable ground it is sealed with thick grout and the hole is rebored.

Chapter 4

# Construction, Prestressing and Testing of Anchors

## 4.1 CONSTRUCTION OF ANCHORS

The material for construction of anchor tendons in most of the cases is steel which is in the form of bars, steel pipes (seldom), straight patented wires or strands, etc. The short bar anchors are used to distribute small tensile forces (upto 10 kN) among a number of units like face of an underground opening. Cable anchors are more suitable for transfer of considerable tensile forces to the deeper rock zone. For longer lengths tendons of wire or strands are preferable as they are flexible, easy to transport and several tens of metres length can be easily inserted regardless of borehole direction. Functionally, prestressed high quality steel tendons are the most suitable reinforcement devices, as any kind of reduction in prestressing due to relaxation and rock creep can be effectively negotiated. The prestressing of high quality steel wire upto the yield point, produces an elongation several times greater than that of a similar stress in a steel bar. Considering modulus of elasticity E = 190,000 MPa for all types of steel the prestressing upto yield point gives following results :

(i) For steel bars

$$\in_a = \frac{0.2}{E} = \frac{210}{1,90,000} = 0.0011$$

(ii) For Steel Wires

$$\in_b = \frac{0.2}{E} = \frac{800}{1,90,000} = 0.0042$$

(iii) For Steel Strands

$$\in_s = \frac{0.2}{E} = \frac{1400}{1,90,000} = 0.0076$$

$\in$ = relative elongation

Thus it is very evident that when strands are used for reinforcement, almost seven times more ground creep is allowed without loosing the prestressing effect.

Steel wires for anchor tendons are made of cold-drawn plain carbon steel. The basic heat treatment known as patenting is carried out in furnaces with automatic thermal control. In the pickling process, followed by heat treatment, the metal is descaled with the help of chemical processing. The wire cross-section is gradually reduced as the wire is extended during cold drawing. The tensile strength, elastic limit and the field strength also increase, a low temperature tempering on patented wires and cables at 400 °C further increases the yield strength by about 80 percent.

To utilise the mechanical properties so produced, these wires are stranded together to form seven-wire, twelve-wire and nineteen wire cables. Such cables, designed to be used as tendon material never contain any compressible component such as a hemp core, nor they are greased. Seven-wire strands have the simplest structure and its load bearing capacity is limited by the largest practicable diameter of the wire. Using 7 mm wire, a strand composed of 7 wires would have a diameter of 21 mm, and cross-sectional area of 269.25 mm$^2$. Considering a nominal wire strength of 1400 MPa, the load bearing capacity of such a strand would be 370 kN.

Large anchors are constructed using multiple cables which are arranged in the same way as the multi-wire cable. The manipulation of bundled cable is more difficult than the handling of compound cable comprising 7 strands, each of 7 wires of 6 mm diameter which can be suitably used for 1.1 MN anchor, whereas the nominal bearing capacity of an anchor comprised of 37 strands each 19 wires of 2.9 mm diameter is 7.6 MN. The disadvantage of stranded cables compared to the multi-wire cables is their larger tendency to creep and lower tensile strength. However, effect of this creep is almost negligible as compared to creep in rock. The difference between the actual cable strength and the nominal cable strength (sum of strengths of all individual wires) is due to state of stress distribution in the loaded cable. A lower actual strength is well compensated by the advantages like greater ease of handling and transport on site, greater cost effectiveness, easier fixing and improved resistance to corrosion.

## 4.2 PRESTRESSING OF ANCHORS

A prestressed anchor is one where tension is imparted by elastic extension of its free tendon length, with the help of instruments. The force is transferred to rock-mass for the purpose of reinforcement. Apart from this, the prestressing also serves as a small test which confirms suitability and type to some extent. The different methods of prestressing, testing and checking of anchors are standardised in different countries with the objective to maintain better characteristics of material used and to achieve proper safety demands.

The equipment used for prestressing of anchors are similar to those used for testing purpose. The basic elements are hydraulic set, stressing head and anchoring head. The hydraulic set essentially contains a pump, a jack, connecting hose and appropriate pressure gauges. A load sensor for measurement of tensile forces and dial gauge for measurement of tendon displacement are also provided with the equipment. It is desirable that a prestressing equipment inducing tensile forces must be capable of maintaining a constant value. This value is usually given by more than one pressure gauges installed between hydraulic jack and pump. For accurate measurement of stressing force, dynamometers mounted on anchor tendon is used. The approximate displacement of the tendon can be measured with the help of a scale fitted with the piston of jack. However, precise measurement is obtained through a dial gauge mounted on an independent structure, not related to the object under prestressing. To obtain the relationship between stressing force and displacement, the tendon displacement is measured nearest to 0.1 mm and for displacement–time relationship it is required to be measured nearest to 0.01 mm. Regular calibration of measuring instruments is very important.

The design of prestressing equipment depends upon the type of anchor and in particular, the head of the anchor available for the purpose of prestressing. It is very important that the tensile stress should be applied axially without inducing any kind of bending. Suitable mountings in the form of distribution plates may be used for this purpose.

In mines and tunnels, bolts are often prestressed by hand using a flat spanner with an arm of 80 to 100 cm. A force application of approximately 0.3 kN will result in a torque of 235 Nm thus inducing a tension of 39 kN in a 24 mm diameter bar with metric threads. For more accuracy torque spanners are used.

Torque spanners equipped with ratchet adaptors producing moments of upto 200 Nm are often used for prestressing bolts. Pneumatic hammer drills or pneumatic impact tools equipped with wrench adaptors can be used to tighten bolts quickly. Torque upto 800 Nm is achieved through them. Though the torque wrenches and pneumatic tools provide relatively accurate and controlled torque the tension in the bolt is less predictable due to variable resistance offered by the nut to the revolutions. To avoid this uncertainty, hydraulic pumps with simple gauges are used.

The tensioning of heads provided with a screw thread on the outside is simple, the stressed state is maintained by a ring nut resting on a load distribution headplate. These anchors are stressed by means of a hydraulic jack with hollow cylinder (Fig. 4.1). The threaded rod passes through this cylinder and is fixed into the centre of the anchoring head at one end and onto the cylinder face by a screwed-on nut at the other end (Fig. 4.2). The required prestressing is maintained by tightening the nut on the outside of the head, or by fixing the tendon in position with a nut bearing on the surface of the anchored structure.

Most anchors with locking heads holding cables made up of individual wires or strands are prestressed with special prestressing guns. The wires or strands are fixed either around the perimeter of the gun on the gun cover (Fig. 4.3) or at one end of the hollow cylinder in many cases (Fig. 4.1) or inside the hollow cylinder (Fig. 4.4) or around the perimeter of the jack as well as at the end of the hollow cylinder.

Sufficiently long ends of the wires or ropes must be left projecting from the borehole for fixing to the stressing equipment.

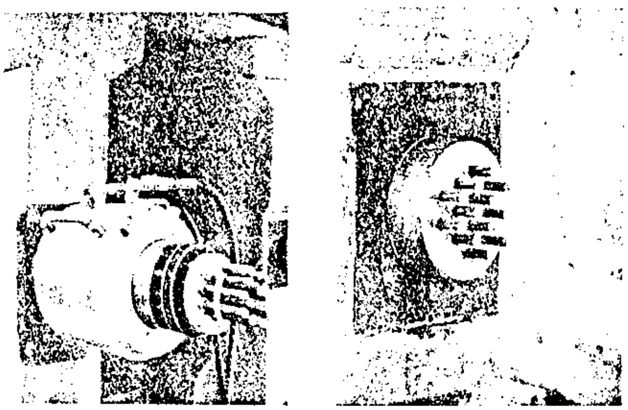


Fig. 4.1 : Prestressing with the help of hydraulic jack

Fig. 4.3 : Prestressing of individual wires

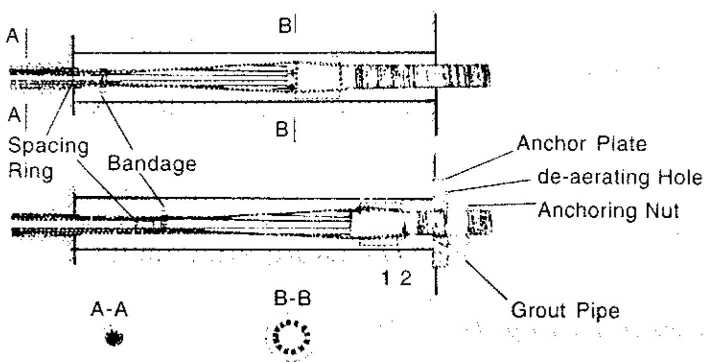



Fig. 4. 2 : Prestressing an anchor provided with screw-on nut
a) before prestressing b) after prestressing and tightening of the nut;
1- fixing sleeve, 2 - conically expanded end of the anchor head

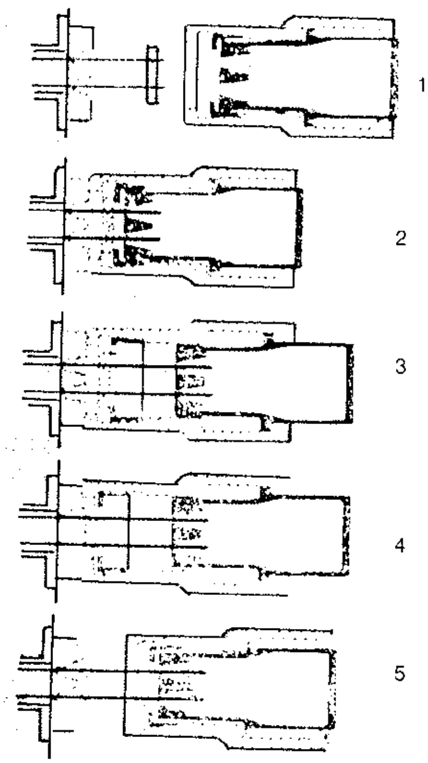



Fig. 4.4 : Prestressing of anchors inside the hollow cylinder

## Procedure of prestressing of cable anchors with locking head

(i) The individual parts of the tendon, anchoring head and bearing plate are thoroughly cleaned.

(ii) The anchoring head is slipped on to the tendon, and moved along until it touches the distribution plate, by pulling the individual ropes through the openings in the head, the gripping wedges are inserted in the openings beside the strands. Care must be taken that the strands remain parallel and do not cross.

(iii) The resting chair, stressing jack, and stressing head are mounted on the tendon, and lightly greased wedges are inserted into the opening in the stressing head. Prior to stressing short anchors, it is important to make sure that the displacement of the anchor head will exceed 30 mm under the highest loading. Otherwise, it becomes impossible to remove the gripping wedges from the stressing head following the release of the jack. Wherever a displacement of 30 mm or less is expected, the jack piston must be advanced 30 mm before the stressing and mounting of the stressing head are carried out.

(iv) The oil hose from the high-pressure power or hand driven pump is connected to the jack and the stressing is begun.

(v) With the initial movement of the jack piston, the tendon ropes become fixed in the stressing head. The anchor head and its free wedges are held in position by the resting chair.

(vi) The pressure of the jack is progressively raised to the maximum value required. The magnitude of the pressure and the advance of the piston are carefully observed and recorded.

(vii) When the prescribed tensile force, or the maximum advance of the piston has been reached, the pressure in the jack is released and the ropes are automatically fixed in the anchor head by the forcing of the wedges into piston. The stressing head is freed by light topping.

(vii) The jack can be disconnected, or after retraction of the piston to its original position, stressing can be continued to the next stage by repeating the procedure. To obtain the final tensile force in the tendon or in compensating for losses of prestress, a higher resting chair and head plates are positioned under the anchoring head.

## 4.3 TESTING OF ANCHORS

Anchors may be stressed to the production load or the testing load. The 'production load' of an anchor is given by the working force, Pw as given by static analysis. The anchor must be able to sustain this force throughout its service life. This working force must be exceeded by some safety margin before the anchor's ultimate state is reached, as indicated by the point of failure of one of its main components (breaking strength failure or the yielding failure of the tendon steel). Some of the safety factor values as standard recommendations is as follows :

| | | |
|---|---|---|
| (i) Ultimate strength of anchor steel | = | (1.65-2.0) Pw |
| (ii) Yield strength or elastic limit of steel | = | (1.33-1.65) Pw |
| (iii) Ultimate strength of the anchor root in rock of soil | = | (1.60-1.70) Pw |

The testing load is a short term loading applied to the anchor in order to test the integrity of the whole installation and to check the factor of safety chosen, and make sure that anchor has a capacity to transmit the working force Pw permanently. The maximum testing force Ptmax for production anchor established as a function of working force Pw. Ptmax is the largest force experienced by the anchor during its service life, with the exception that the ground is overloaded by forces not anticipated during the static analysis. In special tests on anchors, Ptmax is determined by the ultimate load of the anchor, until failure occurs.

The stressing of all production anchors is carried out in the form of an acceptance test, in which the anchor is subjected to a test load greater than the working force, Pw for a predetermined time. The purpose of the short term loading of an anchor by a larger force is to obtain measurable safety coefficients relating to the designed production load Pw or to discover defects in the design or installation of the anchor in good time.

The standard values of Ptmax for production anchor are as follows -

| | | | |
|---|---|---|---|
| (i) | Permanent Anchors Ptmax | = | (1.20-1.50) Pw |
| | | | (0.70-0.85) Ps |
| | | | (0.90-0.95) Py |
| (ii) | Temporary Anchors Ptmax | = | (1.15-1.25) Pw |
| | | | (1.70-0.85) Ps |
| | | | (0.90-1.00) Py |

Where

Ps - Ultimate tensile force for steel

Py - Force at elastic limit of steel.

## Simple Acceptance Test

The anchor is stressed by an initial force Po = 0.1 Pw to 0.2 Pw, and the initial reading at the anchor ring head is registered. The loading of the anchor is progressively increased to Pt max = 1.15 Pw to 1.5 Pw and the displacement of the anchoring head is registered. Subsequent displacement of head is measured under this load for minimum test period of 5 to 15 minutes depending upon the type of strata. For strong rocks it is 5 minutes and for cohesive soils 15 minutes. Usually stabilisation take place at the end of the test period, i.e., no further displacement or a displacement within the allowable limits (1-2 mm). Now the anchor load is decreased to Po and the drop in the displacement at the anchoring head is recorded. Then the anchor is loaded by a load little over working force Pw with anchoring head fixed.

A load displacement diagram based upon the above text gives important parameters such as elastic and permanent components. This diagram is divided into permanent and elastic components. The two points say $B_1$ and $B_2$ represent the calculated elastic deformation of the tendon free length, reduced by 20% (point B and increased by a half of the root length (point $B_2$).

The elongation of the steel tendon is calculated as under

$$S = \frac{P.L}{E.A.}$$

Where P = tensile force

L = free length of the tendon

E = modules of elasticity

A = area of cross-section.

## Detailed Acceptance Test

Most of the standards stipulates that at least 5% of the total production anchors must undergo a detailed acceptance test at site. In this test the anchor is subjected to an initial force Po which is gradually increased to maximum testing force Ptmax.

This is usually done in 4–5 steps like 0.4 Pw, 0.8 Pw, 1.0 Pw, 1.2 Pw and 1.5 Pw. At each step anchor is relieved back to Po for measurement of residual deformation. At each step the force is maintained and the displacement of anchor head is recorded. This is done till the stability is reached. Stabilisation is achieved when the displacement increments measured at the anchor head remain unchanged over three successive time intervals or it change by no more than 0.1 to 0.2 mm. It is recommended that the displacement increments be measured at progressively increasing time intervals like 1.25, 1.45, 2.5, 3.5, 5, 7, 10, 14, 20, 28, 40, 56, 80, 112, 160 minutes etc. Besides the load–displacement diagram the displacement–time diagram is also drawn. If time values are plotted on a logarithmic scale, a displacement movement that is approaching stability will be represented by a nearly straight line (Fig. 4.5).

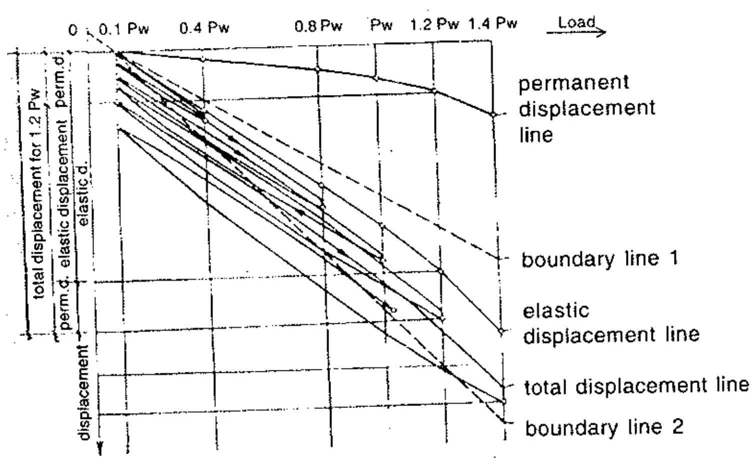



**Fig. 4.5 Evaluation of detailed acceptance test**

# Chapter 5

# Impact of Time and Corrosion on Bolts

## 5.1 IMPACT OF TIME

The longterm observation of anchors provide important data supplementary to that obtained in short-term anchor tests. The long-term observation is one which continues for more than twenty-four hours after the installation. The basic purpose of such an observation is to record any changes taking place in the prestressing or any displacement as a result of temperature fluctuations, shocks, changes in the state of stress of the rock, etc.

It has been observed that there is a certain drop in the initial prestressing with time. Primarily, it can be attributed to the combined effect of relaxation of steel and ground creep. The relaxation cause the drop of prestress without deformation, whereas, deformation under permanent load is known as creep. The losses as a result of creep can be considerable.

### Relaxation of Steel

Usually the information about steel relaxation is supplied by the manufacturer alongwith the other data. It has been tested and found that relaxation losses in prestressed steel under long-term loading remains within the range of 5–10 percent. The tests reveal that relaxation losses after 100 hours of loading were about double the losses occurring after 1 hour of loading. The above losses were found to be about 80 percent of the losses after 1000 hours of loading and 40 percent of the losses after 30 years of loading. In relation to the amount of loading of the steel, the relaxation loss is negligible at loading 50 percent of its strength but the losses increase sharply with higher loads. The losses are significant at temperatures in excess of 20 °C. The use of stabilised wires and strands brings down the losses to 1.5 percent from the range of 5–10 percent at 20 °C temperature and loading at about 75 percent of tensile strength.

## Ground Creep

The phenomenon of creep in the ground under loading takes place as a result of plastic compression or failure of the ground under stress. In most cases of prestressed anchors, the creep takes place at places of high stress concentration like near the anchor root and below the anchor head near the surface.

In case of hard rock the loading stress of prestressed anchors is well accommodated by the strength of the rock. Still, the pre-existing/natural planes of separation/ discontinuity are further compressed under load. This is indicated by the comparatively quick loss of prestressing with time during the initial phase. This loss diminishes with time to become almost negligible by the time a state of equilibrium is reached. In case of bar anchors the total loss can be as high as 20 percent of the original prestressing but, in case of long strands with greater ductility, the loss is significantly low. Moreover, the loss in prestressing can be easily restrored with the help of equipment. In hard rocks the additional prestressing is usually done after 24 hours and then after a week or two. It is observed that the creep in strong rocks is very small even under high loading over prolonged periods. While refering the recommendations of Post-Tensioning Committee, Chicago, 1974, Hobst & Zejic indicated that the prestressing losses in hard rock reach 3 percent after 7 days, entirely due to steel relaxation. It is further reported by them that the long-term observations of anchored dams also indicate a maximum loss of 10 percent as a result of steel relaxation and creep in concrete. The observations carried out at the Cheurtas Dam in Algeria indicated that the losses were found to be 4 percent after one year and only 5.5 percent after 18 years.

In soft rocks and soils the compression of ground under loading is quite significant. The gradual compression results in deformation. Hobst and Zejic reports that well marked and long lasting deformations have been found in cohesive clayey soils and in fine uniformly grained sands. In these soils large creep displacements of the anchor root, and plastic flow of the soil around the root, take place at the ultimate load. The displacements continue to increase with time and the required tension in the anchor can not be maintained permanently.

The results of many tests show that anchors 20–25 m long with long grouted roots of 10–15 cm diameter register a prestressing loss of about 6 percent in hard clays and 12 percent in stiff clays due to creep. These losses are usually registered within the first 2–4 months following the prestressing of the anchors and do not increase thereafter.

Losses (prestressing) caused by creep in soils and soft rocks may be compensated by repeated additional stressing at increasingly longer time intervals (upto one year), provided the load carrying capacity of the root is not exceeded. Anchors can not be installed in highly compressible soils with large amounts of organic matter, or in very soft ground of low consistency or high liquid limit (in excess of 50%) because of large creep deformations, which may ultimately result in failure.

## 5.2 OTHER FACTORS AFFECTING PRESTRESSING IN ANCHORS

Besides ground creep and steel relaxation, there are other factors which may cause changes in prestressing. For example, shocks in the anchoring medium, fluctuations in load, changes in temperature, changes in the state of stress field etc. These can induce significant changes in prestressing, even sometimes impairing the function for which the anchor is intended.

### Shocks in Anchoring Media

Shocks imparted to anchoring medium result in losses to prestressing. The high intensity shocks may even affect the load bearing capacity of anchors. Usually, the highest intensity shocks experienced by the anchors are those triggered by blasting in the mines. It is believed that shocks account for more than the long term static loading as far as prestressing loss in anchors is concerned. Apart from blasting, seismic shocks and vibrations from operation of heavy machinery may also cause damage.

According to Hobst and Zejic, when blasting was carried out to see the impact on anchors in a horizontally stratified dolomite mine, a significant drop in prestressing was observed in case of the anchors within 3 m of the blast. The drop in prestressing was about 36 times than that recorded in case of static-load over the same duration after installation. The effect was found to be almost negligible on the anchors/block falling beyond a distance of 5 m from the blast.

It was also observed that the anchors with mechanical fixing suffer the most due to blast vibrations. At the same time, the long cable anchors are less affected as compared to short ones. The reason behind it is the tendon length of the anchor. Since any change in the prestressing is due to variation between the distance of anchor root and head. This variation in longer cable anchors, as a result of blast shock, is very insignificant with respect to the total tendon length. For example, most of the vibrations in the ground, as a result of blasting occurs in the frequency range of 5 to 50 Hz. If a practical situation is considered where permissible maximum velocity Vmax = 80 mm/s, the ground suffers a harmonic vibration with maximum displacement of 2.5 mm at 5 Hz and only 0.25 mm at 50 Hz. If the free length of anchor tendon is taken as 10 m which is prestressed to give a relative elongation of 0.006, the elastic elongation over 10 m length will be 60 mm. Hence the resultant variation experienced in the anchoric force will be 4% at 5 Hz and as low as 0.40% at 50 Hz.

### Variable Loading

In the anchor load continuous variations of loading over a long period not only affect the prestressing adversely but it also threatens the strength of fixing of root.

### Changes in State of Stress and Temperature

The temperature changes bring about expansion or contraction in anchors. These dimensional changes depends upon the coefficient of thermal expansion of the material of anchors. The impact of temperature in the mine atmosphere is normally negligible since for all practical purposes most of the anchors are deep into the rock, i.e., not exposed to the mine atmosphere. However, in some cases the changes in temperature and state of stress of anchoring medium may decrease or increase the stress in anchors.

## 5.3 CORROSION IN ANCHORS

The term corrosion mean the damage caused to metals by their chemical or electro-chemical reaction with the surrounding medium. Typically hetrogeneous reactions take place at the boundaries between solid, liquid and in gaseous phases. The mechanisms of corrosion are governed by many factors (which cannot be defined easily) and changes in the course of reactions themselves. It may be generalised that steel embedded in ground mainly suffer the type of corrosion caused by electrochemical reaction. This entails a transformation of the metal into free ions and electrons as a result of the interaction of the metal surface with an electrolyte (soil moisture in most of the cases). If ions of only one metal take part in this reaction, the process is anodic. The ions passing from the metal into solution as free hydrated ions, when metal ions separate out from the solution and recombine with the metal, the process is cathodic.

Corrosive process involve both anodic and cathodic reactions. They are usually limited to definite areas on the metal surface owing to the heterogeneous nature of the metal or because of the differences in the composition of the ground in which the metal is placed. It is the electric current passing between these specific areas which causes the corrosion. The cathodic reaction reinforces the anodic reaction by drawing off the electrons released by the latter. After some time, however, the primary anodic and cathodic products mutually combine to produce insoluble substances which prevent further corrosion, unless oxygen or hydrogen arising from the reduction of hydrogen ions penetrates the metal at the site of corrosion. These products disturb the anodic and cathodic processes and facilitate further transformation of the metal into the free ion form.

The ground consists of solid, fluid, and gaseous matter, and nearly always provides suitable conditions for electrochemical corrosion. Even the slightest moisture within the ground capillaries can act as an electrolyte, and it is not necessary for potential anodic and cathodic sites to be completely surrounded by electrolyte. In fact, metal corrosion tends to be depressed in saturated rocks. The degree of aeration and its local differences have a considerable influence on the progress of corrosion.

Under some conditions, the metal of anchors is also attacked by aerobic and anaerobic bacteria. This biocorrosion can be caused, for example, by an irregular distribution of bacteria on the metal surface, which produces varying degrees of aeration. Bacteria also accelerate chemical processes, such as the formation of sulphuric acid from pyrites. The chemical composition of the soil has an important bearing on the development of corrosion. The presence of salts, particularly those of sodium calcium and magnesium, encourages corrosion. Because of their high solubility, these salts are mobilised readily by the soil moisture.

In order to select the best method for protecting anchors against corrosion, the corrosive activity of the ground in which the anchor will be embedded must be known. The activity is determined by the following aspects :

i) the composition of the ground and the ground water level,

ii) the virtual ground resistivity,

iii) the specific conductivity of the ground water and surface water,

iv) the chemical composition and moisture content of the individual geological beds of the ground,

v) the chemical composition of the ground and surface water,

vi) physical and chemical properties (pH etc.),

vii) the possible existence of extraneous electric fields.

The cables from which anchors are made, consist exclusively of wires created by patenting and cold drawing. The high mechanical stresses in the individual strands of a rope can contribute to the development of severe and very dangerous corrosion under stress including corrosive cracking and hydrogen brittleness. These corrosive effects develop into intercrystalline and transcrystalline corrosive attack. It is in the nature of these types of corrosion that they are hardly noticeable at the outset. Subsequent stages characterised by very fine cracks without any visible products of corrosion are detectable only with the help of a microscope. Failure occurs at once, without any prefailure drop in strength.

Patented wires are less susceptible to corrosive cracking than wires heat-treated by other methods. The danger of corrosion under stress is much reduced by a further treatment of low-temperature tempering and curing of the wires. This process removes a substantial part of the non-uniform internal stress, which in turn reduces the total mechanical stresses developing within the wire as a result of the summed internal stresses and those stresses induced by external forces like prestressing. Great care is therefore required to be taken to protect anchors against corrosion.

The anti-corrosive protection for anchors are considered, with respect to three main criteria.

## a) Method of providing anti-corrosive protection

(i) Direct protection by coating/wrapping or sheathing with water proof material which keeps out the aggressive external medium. This is often termed as passive protection.

(ii) Electric cathodic protection by creating an electric circuit, the anchor surface thus being cathodically polarised and maintained at a potential which prevents occurrence of the corrosive process. This is known as active protection.

### b) Anchor Type

(i) Permanent protection throughout service life for permanent anchors.

(ii) Short-term protection for anchors having a service life of less than five years.

### c) Static-functional Requirements

(i) Anti-corrosive treatment which allows for static cooperation of the anchor with the ground along its entire length or a part of it, which usually includes root.

(ii) Anti-corrosive treatment which prevents adhesion of the anchor to the ground within a predetermined section of the anchor. Usually the tendon receives this type of protection.

# Chapter 6

# Case Studies

Cable bolting in its broad terms is a method of reinforcing loose or fractured in situ rock to prevent from caving or spalling.

A major advantage of cable bolts is that they are flexible and can be coiled to approximately 1m diameter coils. This allows relatively long lengths to be installed in a limited head room (1.8m).

The earliest known use of cable bolt reinforcement in underground mining was at the Willrog Mine in Canada (Marshal, 1963) and at the Free State Geduld Mine Ltd in South Africa. Extensive development of cable bolt reinforcement technology occurred during the 1960s using discarded hoisting ropes which was relatively inexpensive, however, excessive labour was involved in unwinding and degreasing the used rope. In addition, the limited availability of used hoist rope for cable bolting reduced its attractiveness. Multi-wire prestressed steel tendons were introduced as an alternative. However, the load-transfer characteristics of the plain wire is poor due to its smooth profile. Subsequently, multi-strands cable bolts were introduced in early 1970s (Windsor, 1992).

During the late 1970s and early 1980s, cable bolt technology developed for cut-and-fill mining was used to support and reinforce large spans commonly encountered in open stoping. The review of ground failures, which used cable bolts, revealed that in most instances the steel cables were still intact, instigating several modified cable designs. These designs focused on trying to increase the cable's pull out resistance by altering its configuration through the addition of buttons, coatings and unwinding the individual strands of the cable. During this time, research into using composites for the construction of cable bolts was initiated. Though satisfactory performance was experienced, the cost of composite cables prohibited commercial acceptance.

In Canada during the early 1990s, a cable bolt made up of polystal, a patented glass fibre for continuous mining technology in hard rock (a cuttable bolt), was introduced for ground support. It exhibited higher pullout loads than single steel strand cable bolts when loaded axially but the cost of the cable was approximately seven times higher than conventional steel.

In India, in early 1970s application of cable bolting in Hydro-Electric projects in Chipro underground power house and Sardar Sarovar Projects and in metal mines were reported. Now a days it has been extended to coal mines ( also for mining of thick coal seams) and a few metalliferous mines. Some of the applications are listed below [1]:

(i) Rajpura Dariba Lead & Zinc Mine, M/s, HZL – for hanging wall.

(ii) Mosabani and Surda Copper Mines of M/s HCL – for the mining of wide ore bodies in jointed rock.

(iii) Balaghat Manganese Mine, M/s. MOIL – for jointed orebody.

(iv) Gumgaon Manganese Mine, M/s, MOIL – for the presence of clay bands along the joint planes.

(v) NCPH (Chirimiri) underground Coal Mine of M/s. SECL – for working of 6.5 to 11 m thick No.1 seam.

(vi) RK New-tech Mine in Shreerampur, Asansol area – for adverse ground condition with clay band.

(vii) Madhusudanpur Colliery of M/s. ECL – for thick seam mining.

(viii) Saoner Coal Mine No.1 WCL – for thick seam mining of 5-6 m.

## 6.1 INDIAN EXPERIENCES

Cable bolting has widely been practiced during the past decade, in hard rock weak ground conditions in metal as well coal mines for effective ground control while working on wide orebodies.

### 6.1.1 Cable Bolting as High Roof Support for Depillaring Thick Coal Seam

Cable bolting at two coal mines are discussed below as case studies :

**NCPH Mine of Chirimiri Area**

The limiting height for conventional depillaring in Indian mines was about 4.8 m in view of poor resistance of high support system even with the use of steel props in place of timber supports. As a result, the level of recovery from thick seams was below 40 percent in case of 5 to 8 m thick coal seams. The left over coal within the goaf has been responsible for spontaneous heating. The importance of good quality coal in developed pillars of thick seams has drawn attention from mining industry as India has 56 per cent of its total reserve in seams, more than 5m thick. So the need of a cost effective, safe and productive mining method with optimal recovery was realised. CMRI, took the responsibility of developing such a method wherein the height became irrelevant so far as the efficacy and the resistance of the support was concerned. The method of mining so developed culminated into a technology based on cable bolting, wherein old haulage ropes are used in reinforcing the immediate coal roof and the nether roof well in advance of the depillaring operation. To ensure safety and effectiveness, the efficacy of the cable bolts was tested after cycles of blasting, winning of roof coal and in the process of depillaring. The technology was first experimented at 6–8 m thick in No III seam of Chirimiri Area, NCPH mine where the extraction was partial in view of poor coal roof cohesion, non-availability of long props and other strata control problems. The seam was developed upto 3 m height and the final extraction by way of slicing was approved upto 4.8 m height. The overall recovery within the panel by this method was envisaged to be 40% only. Studies were undertaken to maximise the safe recovery of coal from the thick seam standing on the pillars and improve the production and productivity by the way of modified method of mining. Under the geo-mining condition of the mine, several options were reviewed in depth for their suitability and the final choice was put to depillaring with cable bolt as high roof support. The roof coal band of 3.5–5 m thickness was cable bolted such that the bolt remained anchored at least 1.5 m within the sandstone roof. The top section coal was thereafter recovered by systematic slicing in one or two passes with recovery around 75%. The hanging type support helped in keeping the floor free of obstruction for the movement and maneuvering of SDL.

A full-scale trial was undertaken at NCPH where the efficacy of cable bolting was studied in the mine, wherein 19-22 mm dia. old ropes of 5 – 6.5 m length were used as cable bolts. The cable bolted roof coal was subsequently blasted in single or double passes. The experiment was repeated in 4 panels of 22-52 pillars and under 33-256 m depth with the financial support of the Sanding Scientific Research Committee, Ministry of Coal, Government of India. The concept was given full-scale trial with the active cooperation of DGMS officials, SECL management in general and the officials of the Chirimiri area in particular. The method first of its kind capable to facilitate depillaring of seams of 6 m to 8 m thickness improved the strata condition, safety and achievement in respect of conservation, production and productivity. The trial improved the level of recovery from 40% to 75%, productivity from 1.43 to 2.8 and production from 500 to 13,000 tons/month with marginal investment towards mechanisation for coal loading, drilling and grouting of the cable bolts. The production cost decreased in the subsequent panels by Rs 80 per ton of coal. The anchorage of the cable bolts, invariably over 10 t even after blasting of the roof coal was adequate to support the nether roof. All the development headings were supported in advance by the cable bolts before the start of depillaring. The technology proved to be techno-economically successful as a support system.

The said method was successfully experimented at Madhusudanpur Colliery of M/s ECL for 7.2 m thick seams. Number of proposals are in the pipeline which includes depillaring of 6.5 m thick Lower Bachra seam at Churi Colliery of M/s CCL, 6 to 11 m thick Maher seam at Rajpur Colliery, 4.3 to 6 m thick MEC No. III (Top) seam of Nandan Colliery of M/s WCL, 7.2 m thick Gopinathpur Bottam seam at Basantimata Colliery of M/s Bharat Coking Coal Ltd by this method.

In panel no 15,16 & 17 of NCPH colliery of SECL, depillaring with cable bolting was tried successfully. Salient indices of the above trial are tabulated as under[5]

| | | |
|---|---|---|
| Size of development gallery | : | 4.8 m x 2.4 m |
| Depillaring mode | : | Splitting and slicing |
| Roof coal extraction | : | On retreat with cable bolt |

The level of recovery improved to 75% over the conventional depillaring.

**Underpinning :** Since no suitable technology is available for extraction of coal in the presence of fragile formation, extraction of clean coal from contiguous seam or sections of a seam is a major problem in Indian coal sector.

In this method two sections of the seam would be developed along the floor with superimposed pillars. The weak parting and the seam section both in the roof will be reinforced by underpinning using cable bolts to strengthen the parting as well as to facilitate extraction of bottom seam/section roof coal. Top section coal developed with roof bolting would be extracted by splitting and slicing. The width of the split and height would be maintained equal to that of the development headings supported by roof bolts and props and the parting by underpinning from top section. The roof during this operation would be supported in the identical manner. Once the slice development reaches the extreme end – with simultaneous support by conventional props, roof bolting/ cable bolting and underpinning – the top and bottom section roof coal was blasted down leaving no coal parting. In a case SDL handled, the coal from the slice were moved under the cable bolted roof upto the extreme end. During this stage the roof remained supported by the bolts or cable bolts only. The height at this junction was maintained to development height only. The rib was reduced judiciously during the roof coal winning to facilitate roof settlement in the goaf. The coal remained clean because of poor mixing of the waste roof mass.

The technology was experimented at zero seam of Chirimiri Mine underneath Bartunga Hill of South Eastern Coalfields Ltd with encouraging results. The seam over 12 m thickness was developed in two sections leaving a parting of 6.0 to 6.5 m including 3 m coal in the bottom section under flat mountain cap. The thickness of the clean coal of the top and bottom sections were 3.0 to 3.5 m and 5.0 to 6.0 m respectively. The 3 to 3.5 m parting was weak fragile shale and mudstone and roof was almost identical in nature.

**Revindra Khani-Newtech Colliery, SCCL**

The cable bolting was applied at RK Newtech mine in development roadways. Roof conditions became adverse due to the presence of 1.8 m thick weak clay bed in the roof. The seam thickness was 4.04 m and gradient is 1 in 4.2 and the gallery height was 2.5 m. The RMR of the roof was 46 and the geo-technical classification of the strata was very poor to poor. Rock load was estimated by interrupting the RMR and was 2.88 t/sq. m Support density was calculated as 5.1 t./sqm. The length of cable bolt was 4.4 m which can cross the clay bed and enter into the main sandstone roof. Experiences in coal mines may be useful for metalliferous mines also[1].

## 6.1.2 Hindustan Copper Limited

Mosabani Copper Mine of Indian Copper Complex, Hindustan Copper Limited lies south of the Subernrekha river in Singhbhum district of Bihar. Mosabani mine was the oldest running mine in this region and the deepest non-auriferous mine in India. Depth of its working extended well below 1000 m. Host rock is quartz-chiorite-biotite schist. Although the overall dip of the load in this mine is 30 degrees, local variations in width and dip of orebody are very common.

Thinner parts of orebody (upto 4 m) were mined by incline room-and-pillar method. Thicker portions of the deposits were worked by horizontal cut-and-fill (HCF) stoping without post pillars if the thickness is within 6 m and with 4 m x 4 m post pillars at skin to skin interval of 9 m along dip and 13 m along strike if orebody is further thicker. These stopes were worked usually between two consecutive level drives through the orebody at a vertical interval of 37.5 m. A 4.5 m thick crown and 8 m thick sill were left at the top and bottom of a HCF stope and was worked by top slicing taking 2.5 to 3.0 m horizontal slice at a time, followed by hydraulic filling of the earlier slice by deslimed mill tailings. Working areas in the mine were hot, humid and located far off from the shafts. Unstable ground condition had resulted at places due to great depth, large areas of exposure and prolonged period of excavation.

Though R & P and HCF methods were in practice in this mine for two decades, with the increase in depth of working over a period there has been a drastic change in in situ stress configuration at mining sites consequently leading to the deterioration in ground stability in stoping areas at deeper levels. It is more pronounced in and around HCF stopes as they are long and wide, extending from 100 to 200 m along strike and 15 m in average horizontal width. As a result, the stability of the HCF stoping areas lying at a depth beyond 800 m, had to be freshly assessed and a suitable rock reinforcement guidelines was to be worked out to ensure the stability of excavations in these areas.

The investigations were conducted by CMRI, Dhanbad, in collaboration with the mine management. The exercise was carried out in detail to have a precise idea of the states of ground stability in and around a HCF stope at any given stage of excavation.

The investigations revealed that the back and hangwall of the stopes under investigation namely 27L and 28L and corresponding ore drives are required to be supported regularly and systematically to ensure ground stability during stoping. A support design guideline was prepared to overcome the ground control problem.[3]

**Rock Bolts :** Back and hangwall of any such stope was recommended, to be supported by 20 mm diameter full column cement grouted rock bolts, upto 1.5 m thickness, till the crown thickness remains 14.5 m or more. The spacing recommended was 1.5 m x 1.5 m. The rock bolts are required to be placed preferably normal to the foliation planes as soon as a fresh rock surface is exposed. When crown thickness is below 14.5 m, 1.8 m thick back and hangwall must be supported by rockbolts. For, crown height 10.5 m or less, rockbolting must be continued with pre-placed reinforcement. All the ore drives were recommended, to be supported by 2.0 m full column cement grouted rock bolts.

**Cable Bolting :** The right hand lay cables of single strand, 210 mm long, prestress 17 mm diameter was proposed for Mosabani mine for grouting with cement : water mixture in 2:1 ratio by weight.

Cable-grout strength for the above cable was determined by pull-out tests on cable samples grouted to known lengths in G.I. pipes of 43 mm diameter by cement :

water mixture in 2:1 ratio by weight and cured for 8 to 10 days. The results were as follow :

| Test No. | Grout length | Curing Time | Pull-out Load | Bond Strength |
|---|---|---|---|---|
| 1. | 39.2 cm | 8 days | 5.5 t | 14.0 t /m |
| 2. | 58.7 cm | 8 days | 7.2 t | 12.3 t/ m |
| 3. | 45.2 cm | 9 days | 6.0 t | 13.3 t/ m |
| 4. | 60.5 cm | 9 days | 7.9 t | 13.0 t/ m |
| 5. | 72.7 cm | 10 days | 9.8 t | 13.5 t./m |

Had more curing time been allowed, greater values for able-grout bond strength might have been obtained, because cement-water grout attains only 80 to 90% of its full strength in 8 to 10 days (Stillborg, 1984). In mines, cable bolt get much longer time for curing. However in present design, cable-bolt bond strength has been taken as 11.89 t/m (10% less than the average value).

The design of cable bolting pattern was worked out keeping the following factors into consideration :

a) Major failure in the back of 27L/28L stope may occur when its crown thickness is 4.5 m or less. In this situation, the height of failure zone may be as much as 4.0 m. Possibility of puncture of crown at location immediately below the ore drive cannot be ruled out.

b) Cable bolts should be so installed that at any stage the dead load of failed rock does not exceed either the tensile strength of the cable or the strength of cable-grout bond.

c) A cable bolt must extend 2 m beyond the failure zone.

d) To avoid deviation in drilling in placement of cables, length of cable bolt should preferably be limited to 15 m.

e) The design must take care of the joint pattern in rock mass.

For cable bolting, stope was with uniform true width and dip of 7.5 m and 30 degrees respectively. Each set of the cable bolts consisted of 22 cables, installed in equal number of boreholes drilled from three different locations, one in the upper ore drive and the other two in stope when its crown was 10.5 m thick. Drill centres have been considered 1 m above the floor or fill level. Six holes were drilled in fan shape from the hangwall side drill centre in the stope, located 10 m away from hangwall.

Four holes were drilled from the other drill centre in the stope, located 3 m away from the footwall. Other twelve bolts were placed in holes drilled from the upper ore drive. The cable bolts to be placed from the ore drive must be put prior to any deformation or stress development in the horizontal pillar in which the drive is situated. In other words, they must be placed before any mining in the stope immediately above it. A summary of the design details is given below :

| | | |
|---|---|---|
| Total length of cable bolts per set | : | 177.0 m |
| Number of cable bolts per set | : | 21 |
| Number of drill centres for each set | : | 3 |
| Spacing of cable bolt sets along strike | : | 2.5 cm. |

The proposed design should hold even the largest rock wedge that may form in the stope back, though only in rare cases, due to intersection of discontinuity sets. The wedge is triangular in section, 9.8 m wide at base, 4.3 m high, 20.58 sq. m in x-sectional area and 57.21 t/m in weight. Six cable bolts with a total grout length of 14 m would hold the wedge. Considering the tensile strength of cable as 25 t (about 11% less than the prescribed tensile strength), the total load bearing capacity of these cables was calculated to be 150 t, assuming no failure at cable-grout interface. Hence, the maximum spacing possible, required between the sets of cable bolts in different directions of orebody strike to hold the above wedge was 2.62 m.

To further simplify the cable bolting design parameters and also to increase the margin of safety to take care of other uncertain and unknown factors, if any, the spacing of each set of cable bolts have been designed to be 2.5 m, in such a situation:

| | |
|---|---|
| Max. possible tensile load/cable | 23.84 t |
| Max. possible load on cable-grout interface | 21 t |
| Max possible stress at cable-grout interface | 19.13 kg/cm$^2$ |

**Preliminary Cost Analysis :** It was estimated that installation of cable bolt in these stopes would yield an additional recovery of 2 m thick slice per stope. Approximate cost of cable bolting in Mosabani mine during 1992-93 was Rs 290 per metre. Taking into account the post pillars of 4 m x 4m x-section left at skin to skin spacing of 9 m (along dip) x 13 m (along strike), it was estimated that around 6280 m of cable bolt is required per 100 m of stope length along the strike, costing about Rs 26,20,000/-. This would lead to an extra recovery of 7,500 tonne of copper ore.

Total, mineable reserves from every 100 m of strike length of an HCF stope with 15 m horizontal width, 8.0 m thick sill and 4.5 m thick ultimate crown, is around 93,800 tonnes. Hence, by adopting the cable bolting in HCF stopes, approximate increase in overall cost of production of copper ore from these stopes will be Rs 20.12 per tonne.[3]

### 6.1.3 Hindustan Zinc Ltd.

At Rajpura Dariba mine of HZL, cable bolting was introduced for ground re-enforcement to minimise movement along the weak joint plane of hanging wall. The stope back was supported by pre-placed cables of 12 m length at 2 m x 2 m spacing in 57 mm dia. holes. The cables were of 16 mm dia. with bricking load of 250 kN. Cable bolting was also done in the high wall of the VRM primary stopes to control the slabbing of the weak hang wall. The cable bolts were installed from a hang wall drive made for the purpose. In Rajpura Dariba mine the graphite mica-schists form the immediate hanging wall of the deposit. The orebody is composed of calbiotite schist, cal-quartzite and sil-dolomite. Orebody width is upto 70 m.

Numerical model analysis indicated upto 5.7 MPa induced tangential stresses at mid-span of the roof. Frequent block failures from the back and the hangwall may result.

Flat back cut-and-fill with post pillar method, initially envisaging 91% recovery reduced to 70% recovery and 2.5 to 3 m long-bolts failed to contain the roof block failures. This was attributed to the time lag between blasting and effective rock bolt reinforcement. This factor brought the concept of cable bolting into practice.

The mechanics of failure indicated the opening of shallow dipping joint planes in the roof supported by insufficient number of rock bolts. The earlier instrumentation had revealed maximum displacement of the roof occurring within 1.5 to 2 m height. The cable bolts are installed at angles such that they penetrate the hangwall at least 5 m inside and are placed at 3 m intervals in columns and rows[4].

The overlapping between two consecutive sets of cable bolts is 5 m. This pattern of support has proved to be effective through ground monitoring instrumentation.

### 6.1.4 Manganese Ore India Limited (MOIL)

In different mines of MOIL, considering the structural geology of orebody, hangwall and footwall, a flat back cut-and-fill method of mining is continued since inception of underground workings. Level intervals are kept at 30 m for speedy extraction of ore because of slow process of all related activities in this method. The length of each stope is kept from 30 to 45 m depending upon the thickness of the orebody.

In the beginning of the underground operations, the stopes were supported by conventional timber support such as chock and prop etc. At greater depth, as strata control problems increased due to multiple joints these supports were replaced by square sets. In the past the timber specifically brought from Burma and treated chemically to increase the life span were used for square sets and were costly.

Central Mining Research Station (CMRS), Dhanbad recommended pre-mining support with cable bolts at a grid pattern of 3m x 3m with a chock in between. In the meantime National Institute of Rock Mechanics (NIRM) was given an assignment to design the stopes at lower levels in 1992. NIRM submitted its recommendations in July 1995[2]

(i) Providing cable bolts as pre support in 2 m x 2 m grid pattern.

(ii) 6 m wide rib pillars to be left from hang wall to foot wall to divide the load from block to block.

(iii) Post pillars of 5 m x 5 m to be kept at an interval of 20 m to sustain the load of stope block.

(iv) Providing cable bolts as a pre support in 2 m x 2 m grid and rock bolts to be provided in the centre of four cable bolts.

(v) Reduction of crown pillar from 6 m to 5 m.

Since January 1996 stope blocks having above parameters with some modifications have been developed.

| S.No. | Level | Block | Grid of Cable bolts |
|---|---|---|---|
| 1. | 6th | ch. 1200 to 800 | 3 m x 3 m |
| | | ch. 800 to 500 | 2 m x 2 m |
| 2. | 7th | ch. 2400 to 2300 | 3 m x 3 m |
| | | ch. 2300 to 700 | 2 m x 2 m |
| 3. | 8th | ch. 2800 to 2600 | 3 m x 3 m |
| | | ch. 2600 to 1900 | 2 m x 2 m |

The share of production from cable bolted area as received for the years 1995-96, 1996-97, 1997-98, 1998-99 (until October) were recorded at 43.33%, 63.70%, 65.27%, 73.89% respectively and from North section 21.0%, 88.3%, 90.5%, 94.5% respectively.

After having a great success in solving the strata problem through application of cable bolting the system was popularised in other mines too besides Balaghat Mine.

## Cost Economy

a) A crew of three persons drill on an average of 12 m/day

b) Two persons are required for transportation of material and one for maintenance

| | | |
|---|---|---|
| c) | Total salary/month | Rs 22000 |
| | Indirect cost (60%) | Rs 13200 |
| | Supervision cost | Rs 2000 |
| | **Total** | **Rs 37200** |
| | Cost per metre | Rs 124 |
| d) | Cost of material | |
| | Cost of spares/metre | Rs 61 |
| | Cost of accessories/metre | Rs 85 |
| | Cost of cable, cement, pipe/ metre | Rs 73 |
| | Total cost/m | Rs 219 |
| e) | Power cost/m | Rs 177 |
| f) | Interest & depreciation/m | Rs 87 |
| | Total cost/m of cable | Rs 607 |
| | Net cost/m of cable (considering 75% utilisation of cable length) | Rs 809 |
| | Cost of cable bolting/tonne of ROM | Rs 57.78 |
| | Cost of square set/ tonne of ROM | Rs 95 |

Besides this huge saving on per tonne of mineral output, other direct and indirect benefits achieved are as under :

1. Face OMS increased from 2.5 tonnes to 3.5 tonnes.
2. The reduction in the cases of mine accidents was to the tune of 50% over a period of 4 years from 1995 to 1999.
3. The expenditure incurred on procurement of timber came down to 27.08 lakhs in the year 1997-98 from 58.53 lakhs in 1992-93.
4. Open stope is suitable for mechanisation as timber supports are eliminated.
5. The dilution of ore is minimised considerably.

## 6.2 EXPERIENCES ABROAD

One of the earliest trials of pre-reinforcement was referred to by Gramoli in Canada, in which pre-tensioned hoist ropes were used at the Geco Division of Noranda Mines to reinforce backs and walls of large, open stopes. The necessity to install the cables from the upper drilling access before mining commenced meant that the cables acted to pre-reinforce the back and haunch sections of the large stopes. The technique developed at Geco involved a two stage grouting process. First, a plug of cement was placed at the top of up-holes to give a 0.9 m bond length. The hoise rope was then pushed into the plug before it had set to form an anchor. After 5 or 6 days the rope was pre-tensioned to approximately 350 kN and the remaining annulus was then filled with a low density cement-based grout. The technique proved to be successful and an economical method of controlling overbreak and dilution in the stope was adopted.

The application of long, resin-bonded wooden dowels to pre-reinforce unstable roof areas near the face in coal mines was developed to alleviate problems of grout loss through broken ground and production stoppage to bolt unstable roof. Pre-placement of the dowels allowed mining to proceed for a full shift without interruption. Since the dowels were not pre-tensioned, when the roof was exposed and the rock mass relaxed, the dowels developed a passive reaction. A trial in 1970 to pre-reinforce the back of a cut-and-fill stope was described by Cliffod. Coupled, hollow centre, deformed rock-bolts similar to those mentioned by Underwood and Distefanco were successful in controlling instability in the backs. This system was, however, expensive, heavy to handle and difficult to anchor by mechanical means.[6]

Cut-and-fill stope walls pre-reinforced with full column, resin-bonded steel dowels proved to be effective method of minimising damage to the stope wall due to stope firings. Following its success in pre-reinforcing stope walls, the technique was tried in the backs of the cut and fill stopes and proved unsuccessful.

Cable bolts, each 15 m long and comprising six 12.5 mm dia. strands, were first used in the late 1960s to pre-reinforce a bridge pillar region of cut-and-fill stopes at New Broken Hill Ltd. Cables were installed from a sill above the pillar. Because of the rate of stoping in this area, it has not been possible to assess the effectiveness of these pre-reinforcement measures. Clifford used a similar approach at the New Broken Hill Consolidated mine (NBHC) in 1973, except that the cables were installed from within the cut-and-fill stopes. The cables comprised seven 7 mm dia. high tensile steel wires with an expansion shell anchor. Discarded hoist ropes were considered for this application, but the limited quantity available and the necessity to degrease these ropes made the use of pre-stressing cables, common in Civil Engineering applications. Cable of 20 m length were placed in 64 mm dia. holes drilled on a 1.8 m sq. grid, tensioned to 180 kN and fully bonded with a low viscosity, cement-based grout. Clifford showed that in trial areas pre-reinforced by this method a dramatic improvement in stope back conditions was achieved. In a similar trial in the MICAF stopes at Mount Isa mine cables of the same configuration were equally successful in improving back stability and minimising overbreak.

Following the initial trial, installation of pre-placed cables in cut-and-fill stope backs, many improvements have been made to both cables and installation methods. It soon became apparent that the cable in the lower part of the hole was superfluous because the back had already been stabilised by conventional rock bolting prior to drilling the long holes for cables. A technique known as countersinking evolved in which a shorter cable was pushed up beyond the hole collar with extension rod equipment. The lower end of the cable was located so that the stope back on the next lift would be pre-reinforced when exposed. This development was only made possible when cable pre-tensioning was no longer considered necessary for effective pre-reinforcement.

Following the successful trials with no tensioned cables, less expensive steel cables were taken up for investigation. Reinforcing strand of the type used in the first trial at New Broken Hill Ltd. was found to have bonding properties superior to that of, smooth wire and is now used in the majority of cable installations.

### 6.2.1 New Broken Hill Consolidated Mine

The initial trials with cable pre-reinforcement proved to be very successful. There were, however, no information with regard to how the cables performed. The magnitude and distribution of the load on the cables due to mining successive lifts were not known. There was also no basis for deciding how close the stope backs could be from the uppermost ends of cables to ensure continuity of the pre-reinforcing action. In 1974 a research investigation was commenced to study the action of cables in improving stope back stability and to develop guidelines for the design and use of pre-reinforcement method. As part of that investigation, techniques were developed to instrument cables installed in stope backs and to monitor the instrument output during stoping. Details of one field based exercise are given as under [6].

The site for the investigation was stope 5N, one of the series of 10 m wide cut-and-fill stope advancing vertically from 14 level at the NBHC mine at Broken Hill. Access about the stope back was from the sill drive on the 13th level. The stopes had advanced approximately 37 m from the 14 level so that the stope back was 10.7 m below the floor of the sill drive on 13 level when the investigation commenced. The position of the drive on 13 level with respect to stope 5N is evident from the localised 13 level plan.

Initially, six non-tensioned cables, each having seven 7mm dia. wire configuration, were installed on a 4 m grid from the 13 level sill drive in holes drilled to within 0.9 m of the stope back. One wire of each cable was instrumented with fully encapsulated, 4 mm long, linear strain gauges at locations to detect any localised strain changes as the cable became loaded. All gauges were sealed against the ingress of water and grout and covered with a thick bituminous material to dissipate any shear between the wire and grout in the localised region of each gauge. The standard three wire configuration was used to connect each gauge to a bridge completion network, which, in turn, was wired to a multi-channel data logger for long-term monitoring. For 2 to 3 hours periods during stope firings, selected channels were connected to an FM instrumentation recorder with a frequency response of 0-1.2 kHz to continuously monitor selected strain changes at and immediately after stope firing.

A prototype, soft stress monitoring gauge, based on the cantilever principle, was grouted into the separate borehole at the vertical location. The cell was oriented to respond to stress changes in the horizontal plane caused by mining the next lift in the stope.

The first lift to be blasted in the instrumented region of stope 5N (blast A) raised the back to the level approximately 9 m below the floor of the 13 level drive and approximately 0.7 m below gauge No. 0 on each cable. Five strain gauges on cable 1.3 were monitored with the FM recorder and the output from each for a 3 hour period after the blast shows that immediately after the blast all gauges indicated that the cable was in tension, with the maximum recorded strain of only 330 micro-strain (5% of yield strain) in gauge No.3. Gauge No.7 which was 4.5 m above the new back, showed an immediate response of only 40 micro-soft strains in tension. These conditions were maintained for a period of 69 minutes, when a rapid unloading occurred and the strain in all gauges reverted to the pre-blast values. Apart from a minor deviation, these strains remained at the low levels for the period of continuous monitoring. The immediate effect of the blast was to generate tensile load in the cable 5 m above the new back. After the re-adjustment approximately 1 hour after the blast, the tensile load was confined to a 2 m zone above the back. One possible explanation for the strain re-adjustment in the cable is that the grout-wire zone failed over the 5 m length of cable above the back. The stress monitoring cell showed that a compressive stress change of approximately 20 MPa had occurred in the direction N 20 deg. W along the long axis of the stope, and it is likely that the re-adjustment of the reinforced mass was stable apart from a thin layer in the immediate back which loosened during the blast.

Strains of low magnitude were also measured on other cables in this pattern, further confirming the overall tight and self-supporting conditions of the reinforced rock mass above the stope. Manual readings indicated that no significant deviations in cable strains had occurred.

The spacing of cables was reduced to 2 m to enable its influence to be studied in relation to the performance of cables as the next lift was mined. In comparing strain responses of the cables for the 4 m and 2 m spacings it must be assumed that the initial conditions in the stope back were identical. In stope 5 N such an assumption is reasonable because a similar self supporting condition had developed after mining previous lifts. For the last lift (blast A) it is likely that the 4 m grid of cables assisted in the development of the stable condition.

Mining the next lift in stopes 5N commenced at its eastern extremity and progressed in a westerly direction by mining a succession of blocks approximately 4 m long and 2 m thick. With the exception of cable 3.1, strain changes were mainly tensile and of very small magnitude. Cable 3.1 was subjected to a substantial tension and a visual observation of the brow after blast B confirmed that a block of rock had been dislodged by the blast. Because of confinement in the haunch region of the stope, however, it had not been able to break completely free. Thus, cable 3.1 performing a direct support function and the shape of the strain response curve indicates that the load from the block is transferred to the cable over a bond transfer length of approximately 1.2 m.

Of particular note is the localized strain change for cable 1.3. This response is typical of dilatation of a joint intersecting the cable. The frequent occurrence of such local loading along cables indicates that their reinforcing action at the joints is a significant factor in stabilising the rock mass.

The block immediately under the cables was mined with blast C, which raised the stope back to the level. During blast C a continuous recording of the output from strain gauges 5 to 9 on cable 1.3 shows that each gauge except No.9 detected a pulse of very short duration. Within 1 m of this response all gauge outputs had stabilised to small compressor strain values, which remained essentially stable for the remaining 5 hour of continuous recording. This pattern of behaviour represent a significant contrast from that caused by blast A. After blast C the rock mass above the back reached stable, self supporting condition within 1 min. of the blast. In contrast, when blast A was fired through the rock mass reinforced with a 4 m grid of cables this condition did not develop until 69 min. after the blast. This marked difference in behaviour is most likely to be due to increasing cable density prior to blast C, all the different stress and the structure conditions may have been contributory factors. It is significant to note that all strain gauge is located more than 1.2 m above the new back. The strain changes were either less than or equal to 70 micro soft strain or 1.2% of the yield strain. Within the 1.2 m thick zone above the new back, however, many of the cables in the central region of the slope were subjected to tensions of upto 25% of yield indicating that the cables provide support to loosen material in a manner similar to that observed in cable 3.1. After blast A in cables that are located in the haunch areas of the stope indicate that high compressive forces develop above the back. This is consistent with the haunches and pillars between the stopes acting as abutment zones for the rock mass above the stope back. In summary, the study of cable reinforcement in the back of stope 5N at NBHC has confirmed that the principal function of fully bonded cables is to reinforce and stabilise the rock mass immediately after stope blasts until the rock mass become stable. The study has shown that this condition developed with both the 4 and 3 m grid of cables, although the 2 m grid cable did not loosen before the stable condition developed. Within the 1 to 2 m loosened zone forming the new stope back the fully bonded cables can provide some support but minor spalling can occur because of grout-steel bond failure over short bond lands. When the first 1 to 2 m of rock above the stope is supported the majority of the cable length is redundant until the next blast. It is implied from the measured performance of cables that to preserve the benefits of pre reinforcing stope backs there should be an overlap between successive cable installations. If the back behaviour in stope 5N can be taken as a guide to the general behaviour in cut-and-fill stoping, for successive installations of long cables, there should be a length overlap of at least 1.2 m for cables on a 2 m grid and 2 m for cables installed on a 4 m grid. Pre- placement of the cables obviates the need for them to be pre-tensioned, provided that compressive stress acts across the back.

### 6.2.2 Outokumpu's Viscaria Copper Mine in Kiruna, Sweden

The salient features of the mine are as follows [4]:

*Annual ore production* – about 1.3 mt

| | | |
|---|---|---|
| *Depth* | – | more than 400 m from surface |
| *Dip* | – | Steel dip of 70 to 90$^0$ near surface. Below 200 m the dip declines to 45$^0$. |
| *Mean width* | – | 10 m ranging from 5 to 30 m. |
| *Orebody* | – | Sedimentary. Interbedded layers of tuffite, albitefels, copper rock graphite and limestone as the main component. |
| *Country Rock* | – | Limestone is host to magnetite and copper graphite schists. |
| *Stresses* | – | Horizontal stresses 5 MPa near surface, rose to 50 MPa below 400 m. |

**Mining Methods**

The mine employs large-scale mining methods with open stopes, such as raise mining and sub-level open stoping. Hanging wall failure is a problem. Flatter dip stopes, 60$^0$ or less, are continued to be hazardous. Stocks dipping 70$^0$ or steeper are sufficiently stable to be exploited without extra reinforcements. Intermediate stopes are excavated either by employing cable blots or by special designs. The deteriorating effect of the low cohesion graphite schist existing in the vicinity of the hang wall is further aggravated by the high horizontal stresses whereas, large-scale longitudinal sub-level stoping methods were fairly straight forward above the 400 m level, below that point the conditions deteriorated sharply. The orebody has a mean width of 10 m and ranges from 10 m to 30 m. The ore is contained in interbedded layers of tuffite, albitefels, copper rich graphite and limestone. Raise mining is practiced in relatively narrow orebody and the stopes are 55 m long and 50 to 60 m high. Three variations of sub-level open stopings are practiced in different ground conditions.

| | | |
|---|---|---|
| a) *Sub-level III Method* | – | Favourable ground conditions, where no graphite schist occurs and the dip is steeper than 70$^0$. |
| b) *Sub-level II Method* | – | Normal method for stopes with graphite schist located in the hang-wall with dip of 60-70$^0$. |
| c) *Sub-level I Method* | – | For stopes classified as harzadous with dip less than 60$^0$. |

All 20 extracted stopes were investigated for the following parameters :

(i) Stope dip

(ii) Ore width

(iii) Separation between ore and graphite schist in hanging-wall

(iv) Rock quality

(v) Width of graphite schist

(vi) Height of stope when caving occurred.

The correlation between dip and caving was undisputedly established. Flatter the ore deteriorated the ground conditions with dip less than 55$^0$. Every stope caved and dip with 60 to 70$^0$. Caving occurred after mining in several slopes.

Two alternative explanations as suggested to the hang-wall failure are :

I. Importance of horizontal stress

II Based on toppling and inadequate cohesion of graphite schists.

**Suitable Methods of Stoping**

i) *Low copper price* - Copper less than 2%, cable bolting cost makes it non-profitable. Hence, internal support of rock-bridges only can be left-temporary pillars, they are good stabilizers.

ii) *Dip less than* 60$^0$ - Copper content 2 to 3.5%, the cable bolting cost and development cost of sublevel is economic, therefore, sublevel open stoping with cable bolting is practiced for copper content between 2.5 and 4%.

iii) Copper more than 4%, using rock/concrete fill is more profitable than the price of ground failure - adopt sublevel stoping I with rock concrete fill. One truck load of concrete : 4 truck loads of rock.

Cable bolting is not found advantageous in poor conditions and also in good ground conditions. In ground conditions between these two, cable bolting fulfills the expectations and has proved to be an economic way to prevent ground failure. Stopes with dip 60 to 70$^0$ are considered suitable for cable bolt reinforcement. In stopes with rock quality of 5 and more, no reinforcement is considered necessary. To the contrary, the stope height has been increased to 85 m.

Now taking cue from the Finnish Pyhasalmi Mine, Viscaria has introduced"Mandolin" and "Hedgehog" (also called Flexible Skin) pattern of reinforcement. The aim is not that of permanently stable stope but merely to keep the weak walls of the stopes in reasonable conditions to prevent cave-ins until the stope has been emptied. Without these it would not be possible to achieve the production of about 7,20,000 t of copper at average 2.8% copper in 1990.

The support and reinforcement cost was running at about $ 1 million a year. Viscaria had calculated that every 1% reduction in waste rock dilution represented an annual saving of $300,000 and that the dilution need be only 2 to 3% lower to recover the costs of cable bolting.

## 6.2.3 Outokumpu's Pyhasalmi Mine Finland

The 10-40 m thick orebody extends to over 800 m depth. Though the ore itself is stiff and quite strong, it is enveloped by an altered rock formation (sericite quartzite and mica rock), which is very schistose and weak. High horizontal stress field, of 40 to 70 MPa at 400 m depth, was about three times as high as the vertical gravity load. hence, sub-level stoping without excessive waste rock dilution had been a challenging problem. The stability problems due to excessive stresses had been eliminated in sub-level stoping by using :

a) pre-reinforcement with cable bolting

b) yielding pillars and stope shape design

c) rock destressing; "Slot stopes" technique

d) fast stoping progress from opening to back filling

From previous 30% level, the waste rock dilution had been forced down to 12% by these methods. Thus, the profitability had been increased dramatically. 1% decrease in dilution corresponded to an annual saving of $30,00,000.

The mine used about 25,000 cement grouted steel bar bolts, 10,000 friction bolts, 6000 cu.m. of shotcrete and 40 km of cable bolts.

A fully mechanised electro-hydraulic cable bolting rig performed the entire bolting procedure, drilling, grouting and cable installation by one operator under a safety canopy. About 50 to 80 m of cable bolting (4 to 40 m lengths) was achieved per manshift, totalling 25 km per year in a two shift system at less than $20 per metre.

## 6.2.4 "Placer Dome's Campbel Gold Mine, Ontario, Canada

The annual production was 4,90,000 t i.e., 1,350 t/day with 27 working levels, 45 m a part extending to 1280 m depth below surface. The majority of production was between 440 m and 950 m.

The three mining methods practiced are

Shrinkage (2%)

Long-hole(33%)

Overhand Cut-and-fill (55%)

With 10% Development

### Important Features

Steeply dipping gold bearing quartz-carbonate fracture filled veins; narrow 0.2 to 1 m fracture filled along prominent cleavage within strong brittle andesite, or 0.6 m to 11 m wide veins in flexures along the andesite and altered chloritic and talc schist rock units.

### Reinforcement Practice

Un-tensioned, fully grouted cable bolts, as secondary support system in the backs of the CAF stopes and as a primary support (pre-reinforement) system in the hanging walls of the long-hole stopes. The pre-reinforcement gave immediate support to the stope boundaries upon excavation and minimum over-break from blasting.

### Cut-and-Fill (CAF) Stopes

Cable bolts 15 m long, 16 mm dia., single wire with 7 strands were installed first with the back off the fill floor without interrupting the production cycle. Cable bolt spacing varied from 1.8 m x 2.4 m to 2.4 m x 2.4 m depending on joint spacing, joint orientation and overall ground conditions. The cable bolts supplemented the primary supporting weld-mesh screen and 2.4 m mechanical rock bolts placed on 1.2 m x 1.2 m array. The cable bolts were effective for 3 lifts after which new set of cable bolts were installed in between the remainder of the previous cables. This ground support procedure was practiced until a 13.5 m sill remained whereupon long-hole drilling and blasting was adopted.

**Long-hole Stopes**

Drilling and installation were accomplished from within the stope; support installation from a suitable located bypass drift, if available was preferred since more favourable placement was achieved and grouting down-holes ensured a better quality support. Bolting densities ranged from 1.2 m x 2.1 m to 2.4 m x 2.4 m.

**Installation Techniques**

The stiffness of the medium surrounding the cable controls the integrity of the steel grout bond interface, considered to be the weakest link of the support system. The stiffness of this medium is both a function of the grout column and the rock mass strengths. In order to mobilise the frictional resistance at the cable grout interface, the grout column must be free of air voids and sufficiently stiff.

Air voids are trapped air pockets in the grout column or empty breather tubes. Weak grout mixtures may be due to water : cement ratios exceeding 0.4 by weight. Other problems are avoiding high pumping rates to ensure batch mixing rather than continuous mixing methods and packaging and storage of cement.

## 6.2.5 William's Gold Mine, Ontario, Canada

Average width of the orebody 25 m (6 m to 45 m) dip 70$^0$. Extends from surface to 1,300 m depth. Mining method employed is blasthole stoping with delayed rock-fill. Object-highest possible extraction without affecting the ground stability. The orebody is concordantly intruded by felsic dykes and may also contact muscovite schist bands. Orebody is overlain by banded metasedimentary rocks and underlying felsic volcanic rocks. A weak muscovite schist band, 1 to 15 m thick, separates the orebody and the meta-sediments.

The observations and remedies are as under :

(i) The field stresses are isotropic. Horizontal and vertical stresses 0.9 to 1.1. However, the numerical modeling predicted that at depth and at high extraction ratios the induced stresses may exceed the measured intact rock strength.

(ii) The sub-level interval was reduced to accommodate a reduction in blast-hole size from 152 mm to 115 mm. This change improved drilling accuracy and increased the hanging wall stability by reducing its area and shortening the time required for backfilling.

(iii) Double lift stopes required longer stand-up mine. Mucking time and backfilling time were about double those of a single stope. Despite the substantially greater availability of broken ore the double lift stoping was abandoned in favour of greater stability and lower dilution.

(iv) Stope strike length of 25 m was considered too prone to excessive dilution due to weak hang-wall schists. A 15 m strike length would not allow an adequate pillar between adjacent draw points. So, 20 m strike length was adopted. Any backs more than about 8 m wide, opened more than 30 m along the strike were found prone to failure.

In order to keep the PPV within limits so as not to cause damage to the rock mass, the blast-hole diameter and the resulting reduced charge weight per delay, the blast hole diameter was reduced. However, this affected the drilling accuracy with increased deviation. During multiple lifts mining the hanging wall support in every over-cut consisted arrays of 5 inclined 10 m long cable bolts at 2 m horizontal spacings. The cables were linked by steel straps and tensioned to about 2 tonnes.

Cable bolting is applied routinely in the backs of all stopes using rows of four 7 m, fully grouted, slightly fanned cables on 2 m x 2 m pattern at the toes. The cables are plated with 150 mm sq. steel plates and tensioned to 4 tonnes.

# Chapter 7

# Conclusions

In India cable bolting was introduced in metalliferous mines in late 1970s as reinforcement in cut-and-fill mining. Today cable bolting is being widely practised in the copper, lead-zinc and manganese mines, especially for the mining of wide ore-bodies in jointed rock and as pre-support for blasting of successive slice in cut-and-fill stopes.

The use of cable bolting in Indian coal mines is also gaining popularity. Potential application of cable bolting in coal mining is to support weak strata where the roof deformations extend to greater height and conventional roof supports and roof bolts cannot provide the reinforcement.

Cable bolt support was introduced in the mining industry about 20 years ago. The first use of strand cable bolts was in the early 70s at New Broken Hill Consolidated mines in Australia. Fariseau (1988) and Windsor (1992) made extensive review of the cable bolting experiences in various countries. In recent years Australian coal mining industry has been successfully using cable supports to reinforce the roof strata particularly in longwall gate-roads.

Cable bolt is a long steel wire strand of high modulus and strength, grouted into a borehole to reinforce the rock mass. The need for placing a longer reinforcing element and the constraints of limited headroom available underground favoured the development of cable bolting. Similar to rock bolting, the purpose of cable bolting is to knit individual blocks of the rock mass together to form a thicker beam, or for anchoring weak strata to competent strata high above. Cable bolts provide the advantage of preplaced support under difficult ground conditions. Since its introduction, it contributed to significant improvement in safety and economy in hard rock mining worldwide.

The two main advantages of the cable bolting are that they can be installed in openings with very low headroom because they are flexible, and they can accommodate relatively larger amounts of strain without failure. Cable bolts provide reinforcement of the rock mass by building a considerably thick beam, and also by tying up the potential rock blocks to a higher stable horizon. They provide the advantages of pre-placed support under difficult ground conditions, of secondary support to supplement rock bolting, and also of permanent support. Long cables can hold large blocks of rock masses formed by the intersection of joints/slips, and the resulting zone of reinforcement will inhibit fracture dilation by providing a more stable rock mass extending beyond the zone of influence of the conventional ground support systems.

The cable bolts can be placed at any angle. They can be grouted with or without tensioning; the tensioning can be both before or after grouting. Cables are inexpensive, offer a high corrosion resistance in permanent installations, and have a very high load bearing capacity.

Bolting technique includes the reinforcing elements like cables, glass fibre bolts, anchors, dowels, bolts, tendons and bonding element being cement grout or polyester resin. The installation procedures could be pre and post-tensioning, grouted and ungrouted, bonded and debonded, coupled and uncoupled, permanent and temporary reinforcement. The philosophy behind the reinforcement scheme design is strata reinforcement, rock support, cable doweling, rock anchoring, pattern reinforcement and spot bolting. The cable bolting is a special reinforcing technique for long column or thick formation using flexible steel ropes. In coal mines, where the strength requirement is less, degreased old haulage ropes of 20-25 mm dia. were tried in the first experiment.

There are generally four possible modes of failure of a cable bolt support system. These are, (1) Failure within the rock mass, (2) Failure of the cable bolt, (3) Failure of the rock-grout interface and (4) Failure of the cable grout interface,.

Only a few failure of cable bolting due to breaking of steel strands have been reported. Most reported failure have been due to large blocks slipping, mostly when the cables or the strands tend to be left with a curled and twisted shape and totally stripped off the grout. With recent developments in cable bolting hardware worldover, it is possible to overcome this remedies with great success.

From time to time Directorate General of Mines Safety (DGMS) has also reviewed and analysed the bolting practices in use, in Indian underground mines based upon the feedback available from the mines. DGMS has come out with set guidelines in this regard recommended for Indian coal/metal mines and modified them with latest advances in the field bolting through their technical circulars. One such circular issued in the year 1993 states that "In general, extended use of bolting as a method of support would have to become an integral part of future mining systems. This can not only be installed early to support the green roof but also as an active support and has a distinctive edge vis-a-vis passive supports currently used in mines. Full column grouted bolts using quick setting cement capsules appear ideal for most of the conditions prevailing in Indian mines. In thick coal seems, if extraction is proposed using multiple slices in ascending order, use of cable bolts should be undertaken while working the bottom slice." The circular also indicates that among the different rock mass classification systems the CMRS - ISM classification system would be the most useful in Indian conditions. The five parameters in this classification and their importance rating are as under :

| Parameter | | Range of values | | | | |
|---|---|---|---|---|---|---|
| 1. Layer thickness | (cm) | < 2.5 | 2.5-7.5 | 7.5-20 | 20-50 | <50 |
| | Rating | < 0.5 | 6-12 | 13-20 | 21-26 | 27-30 |
| 2. Structural features | (index) | <14 | 11-14 | 7-11 | 4-7 | 0-4 |
| | Rating | 0-4 | 5-10 | 11-16 | 17-21 | 22-25 |
| 3. Weatherability | (%) | <60 | 60-85 | 85-97 | 97-99 | >99 |
| | Rating | 0-3 | 4-8 | 9-13 | 14-17 | 18-20 |
| 4. Strength of the rock | ($kg/cm^2$) | <100 | 100-300 | 300-600 | 600-900 | >900 |
| | Rating | 0-2 | 3-6 | 7-10 | 11-13 | 14-15 |
| 5. Groundwater seepage rate | (ml/min.) | <2000 | 200-2000 | 20-200 | 0-20 | dry |
| | Rating | 0-1 | 2-4 | 5-7 | 8-9 | 10 |

Above five parameter values for the classification should be determined individually for all the rock types in the roof, upto a height of atleast 2 m.

## 1. Layer thickness

Spacing between the bedding planes or planes of discontinuities should be measured using borehole stratascope in a 2.0 m long drill hole made in the roof. Alternately, all bedding planes or fissile (weak) planes within the roof strata can be measured in any roof exposure like a roof fall area, shaft section or cross measure drift. Core drilling should be attempted wherever feasible, and the core log can be used to evaluate RQD and layer thickness. Average of five values should be taken and layer thickness should be expressed in cm.

## 2. Structural features

Random geological mapping should be carried out and all the geological features (discontinuities, like joints, faults and slips, and sedimentary features like cross bedding, sandstone channels) should be carefully recorded. The relative orientation, spacing and degree of abundance for all these features should be noted. Their influence on gallery stability should be assessed, and the structural index for each feature should be determined from tables below.

**Indices for Paramneter Structural Features**

(i) *Presence of Major faults*

| | |
|---|---|
| Net displacement more than 10 m. | 15 |
| Displacement 2-10 m. | 8 |
| Displacement less than 2 m. | 5 |

(ii) *Presence of minor faults/slips*

Spacing less than 5 m.

| | |
|---|---|
| a) Orientation unfavourable | 10 |
| b) Orientation not unfavourable | 5 |

*Spacing more than 5 m*

| | | | |
|---|---|---|---|
| a) Orientation unfavourable | 7 | | |
| b) Orientation not unfavourable | 3 | | |

(iii) *Occurrence of joints and cleats*

| | Orientation Unfavourable | Orientation not unfavourable |
|---|---|---|
| a) *Minimum spacing less than 30 cm* | | |
| Single set | 6 | 4 |
| Two sets | 7 | 6 |
| More than 2 sets | 8 | 8 |
| b) *Minimum spacing more than 30 cm* | | |
| Single set | 5 | 2 |
| Two sets | 6 | 4 |
| More than 2 sets | 6 | 6 |

(iv) *Sedimentary features*

| | |
|---|---|
| Lateral thickness variations | 3 |
| Sandstone channels | 6 |
| Kettle bottoms | 4 |
| Plant impressions | 3 |
| Ball coal | 4 |

Index for structural features will be the sum of indices for individual features.

## 3. Weatherability

ISRM standard Slake durability test should be conducted on fresh samples of rock collected from the mine to determine the susceptibility of rocks to weathering failure on contact with water or the atmospheric moisture. For this test, weigh correctly any ten irregular pieces of the sample (the total weight should be between 450 and 500 g). Place them in the test drum immersed in water, and rotate it for 10 minutes at 20 rpm. Dry the material retained in the drum and weigh it again. Weight percentage of material remaining after the test is the first cycle slake durability index, expressed in percentage. Mean of three such first cycle values should be taken. Core may be broken to obtain the samples.

## 4. Rock strength

Point load test is the standard index test for measuring the strength of rocks in the field. Irregular samples having a ratio of 2:1 for longer axis to shorter axis can be used for the test. The sample is kept between the pointed platens, and the load is applied gently but steadily. The load at failure (in kg) divided by the square of the distance between the platens (in cm) gives the point load index[1]. The mean of the highest five values out of at least 10 samples tests should be taken. The compressive strength of the rocks can be obtained from the irregular point load index Ci for the irregular lump point load index for coal measure rocks by the relation.

$$Ci = 14 \times I \text{ (in kg / cm}^2\text{)}$$

## 5. Groundwater

A 2 m long vertical hole should be drilled in the immediate roof, and the water seeping through the hole after half an hour should be collected in a measuring cylinder. The average of three values from three different holes should be taken and expressed in ml per minute.

The ratings for the five parameters has already been given in the Table (in page 66). Rock mass rating (RMR) is the sum of the five parameter ratings. If there are more than one rock type in the roof, RMR is evaluated separately for each rock type, and the combined RMR is obtained as :

$$RMR = \frac{\text{RMR of each bed} \times \text{bed thickness}}{\text{Thickness of each bed.}}$$

The RMR so obtained may be adjusted, if necessary to account for some special situations in the mine, like greater depth, stresses and method of work. Chart-1 shows the way for deriving RMR.

The department has further stressed the necessity to be observed through its circular dated 7.8.1996 that

1. Roof bolts should be installed as soon as the roof has been exposed, incorporating such steps under the direct supervision of manager/overman.
2. As per design, the holes should be drilled to the correct diameter and length, with a suitable drill. The hole diameter should be more than 8 mm to 12 mm larger than the bolt diameter, for full column cement grouted bolts. Due care should be taken to maintain the verticality/inclination of the holes.
3. Where fast and slow bonding materials are used together (i.e. in case of pre-tensioned grouted bolts) it should be ensured that fast capsules are inserted first.

4. Atleast 10 % of installed bolts should invariably be subjected to encourage testing under the direct supervision of competent person and the results of such tests should be maintained in prescribed format. The minimum anchorage strength of the bolts should not be less than that of the value, which has been considered for design of support system.

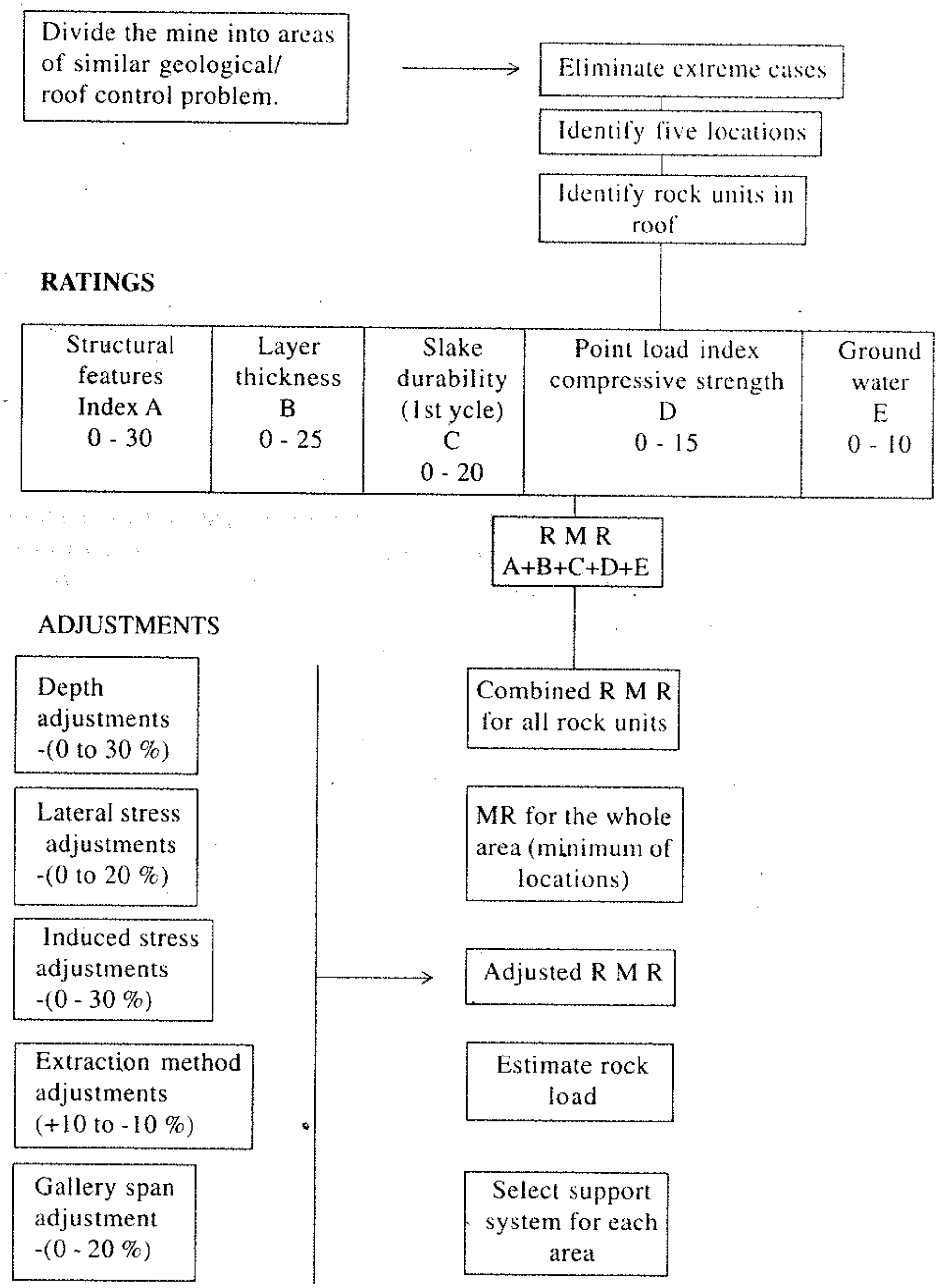
5. During the anchorage testing the bolt load should be increased smoothly and gradually. About 9% bolts should be tested upto the designed capacity and rest 1 % may be subjected to destructive testing. The tested bolts should be identified with tags bearing the record of load applied on them.

6. The anchorage testing equipment should be subjected to regular maintenance and calibration.

7. Measurement of roof-floor convergence should be an essential part of monitoring strata behaviour scheme. A suitable approach may be used for estimating the critical value of the convergence vis-a-vis span and rock mass rating, in a particular geo-mining condition.

Hence, it is amply clear that the system of rock reinforcement with the help of cable bolting is going to play significant role in development and exploitation of coal and non coal mineral deposits in future mining. There is every likelihood that with infusion of the latest technology in development of better, suitable and commercially acceptable anchorage systems the future mining operation will become economical and safe.

## CHART NO. 1 : PROCEDURAL FLOW CHART FOR DERIVING RMR



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