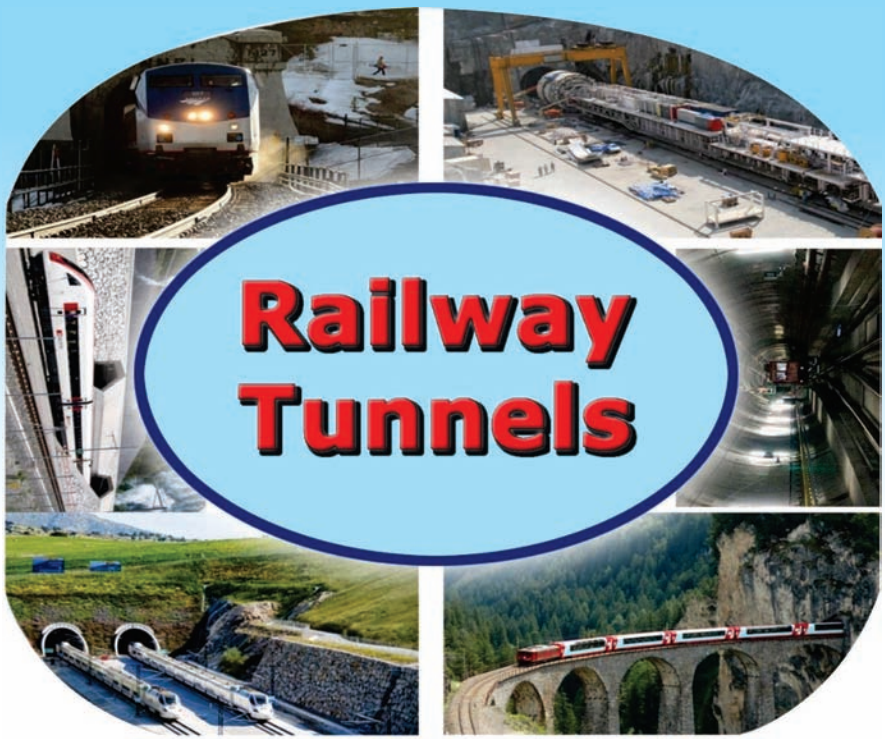

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FOREWORD

Indian Railway Institute of Civil Engineering (IRICEN) regularly publishes text books and monographs on various subjects dealt by and relevant to the Civil Engineers of Indian Railways.

In last few years, lot of tunneling works are being executed by Indian Railways and many of them in Himalayan geology, one of the toughest geology in the world to construct or maintain any underground structures. Considering this, IRICEN has started special course on “Tunneling” from year 2018 onwards for the Indian Railway officials dealing with tunnels. During these courses, most of the trainee officers had lot of doubts and queries, with a desire to have answers to all these doubts and queries in one common document or publication. To address this, Shri R. K. Shekhawat, Senior Professor/IRICEN, has authored this book. The contents of the books are based on various Text Books/Research Papers/ Publications and lectures notes of eminent teachers on the subject, as listed in the Bibliography and References at the end of book. This book covers all technical aspects about Planning of tunnels, Construction of tunnels and Inspection & Maintenance of tunnels. It is expected that this publication will be useful for all the engineers dealing with tunnels in general, and engineers of Indian Railway in particular.

Suggestions for improvement, including need for addition/ deletion of any specific topics, may be forwarded to IRICEN. These suggestions will be considered in future revision of this book.

November, 2018

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PREFACE

Before early 1990s, when construction of Konkan Railway commenced, most of the tunnels constructed in Indian Railways network were in the areas where the rocks were strong “igneous” type, not having much of geological discontinuities or other problems (like joints, folds, faults, shear zones, water charged zones, high tectonic stresses etc.). This is reflected in the fact that these tunnels are generally unlined and even after so many years there is not much of structural problems in these tunnels, except some instances of water seepage (due to no drainage system being provided in these tunnels) or problem of stone pieces falling at isolated places.

In Konkan Railway network, out of total 92 tunnels of total length 84.8 km, only 6 - 7 tunnels (Pernem Tunnel, Old Goa Tunnel, Verna Tunnel, Honnaver Tunnels and part of Byndoor Tunnel) were located in soft or weak grounds. All other tunnels were located in Basalt, a competent igneous rock. But these few tunnels in soft ground (of total length less than 5 km and maximum length of any tunnel being 1.5 km only) presented peculiar problems and contributed to delay in completion of project in targeted time.

With the Indian Railway network getting expanded to all parts of the country, tunnels are being/have to be excavated in Jammu & Kashmir, Himachal Pradesh, Uttarakhand and the Northeastern States, mostly in the Himalayas. The Himalayan Geology, being one of the youngest geology of the world, contains many geological discontinuities and surprises in the form of joints, faults, folds, shear zones, water charged areas, high tectonic stresses, vulnerable slopes etc. This presents a great challenge to tunnel engineers in excavation and maintenance of tunnels in these areas. In fact, some of the railway projects in these areas have been delayed due to problem related with tunnels. Therefore, understanding of the all the issues relevant to construction and maintenance of tunnels has become further important for engineers of Indian Railways.

There are many books and research papers on the subject of tunnels, but they are mostly written by foreign authors, and they may not be sufficient to tackle the difficulties in Himalayan region. Dr. Bhawani Singh, and some other authors with him, have written about tunneling in weak rocks (relevant to Himalayan region). Thus, for understanding the subject of tunneling in entirety, one has to refer to a number of Text Books, Research Papers and other Publications. This may not be possible and practicable for every field engineer. While organizing training courses on Tunnels at RDSO and teaching the subject of Tunnels at IRICEN, it was observed that the trainee officers always expressed need for a common document/ publication, containing technical material covering all activities related to railway tunnels. This book is an effort in that direction. It contains all the relevant aspect of tunneling i.e. Planning, Construction, Inspection and Maintenance of Railway Tunnels.

The contents of the books are based on various Text Books/ Research Papers/Publications (listed in the Bibliography) and learning/lectures notes on the subject, which the author had good fortune of learning from stalwarts on the subject especially Dr. Bhawani Singh (Retired Professor, IIT/ Roorkee) and Dr. K. G. Sharma (Emeritus Professor, IIT/ Delhi). The author expresses his indebtedness to them. The technical material collated for publishing Indian Railway Tunnel Manual, during posting of the author as Director/ GE/RDSO, has also been referred at some place, which is acknowledged.

The support and help rendered by Shri. Ghansham Bansal (Chief Vigilance Officer/Delhi Metro), Shri B. Ravi Kumar (Senior Instructor) and Shri Sabyasachi Roy (Senior Instructor), in proof reading of the book and offering valuable suggestions, is also appreciated.

It is felt that this book will be useful to engineers of Indian Railways. However, there is always scope for improvement in any publication. Therefore, suggestions for improvement

are welcome from all the readers and the same may please be forwarded for incorporation in the future editions.

November, 2018

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CHAPTER-1

ROCK MATERIAL AND ROCK MASS

Some basic concepts about rocks and rock mass are being presented in this chapter, which will be helpful in appreciating the other issues related with tunnelling.

1. Rocks: A mineral is solid inorganic material of the earth having known chemical properties, crystal structure (Fig. 1.01), hardness and cleavages pattern. A rock is solid aggregate of one or more minerals that have been cohesively brought together by a rock forming process (Fig. 1.02). Various properties of minerals combined with their arrangement in the rock inter-alia influence the properties of rocks.

Fig. 1.01: Quartz Crystals

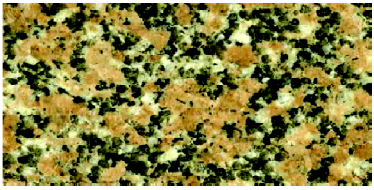


Fig 1.02: Quartz Crystals in Granite

2. Origin of Rocks: Rocks are as dynamic as rest of the nature, but they change very slowly. The oldest rocks were formed with formation of planet earth, about 4.6 billon years ago. But they were built, eroded and rebuilt in the process called “rock cycle” (Fig. 1.03).

Igneous rocks are formed by volcanic activity. As the magma cools under earth's surface in magma chambers, intrusive igneous rocks are formed (common example is Granite). As the lava gets out to the surface, it cools, and extrusive igneous rocks are formed. Common examples of igneous rocks are Granite, Diorite, Gabbro, Rhyolite, Andesite and Basalt (Fig. 1.04 & Fig. 1.05).

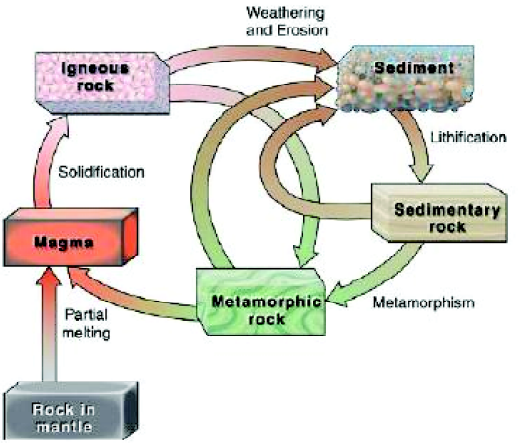


Fig. 1.03: Rock Cycle

Granite

Diorite

Gabbro

Fig 1.04

Rhyolite

Andesite

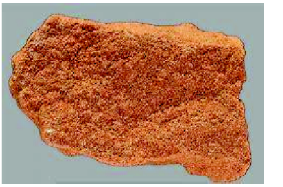
Basalt

Fig 1.05

These rocks erode mainly through wind and rain and the eroded material ends up on the ground or finds its way to waterways. In the bottom of oceans, lakes and rivers, this material accumulates in the form of layers,

known as lithification, with older layers overlain by younger layers. The older layers buried deep turn into Sedimentary rocks under the pressure from younger layers. Some sedimentary rocks end up on the surface, by uplifting, and/or erosion. Common examples of sedimentary rocks are Shale, Sandstone, Rock salt and Conglomerate (Fig. 1.06 & Fig. 1.07).

Fig 1.06: Shale



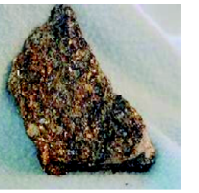
Sand Stone

Fig 1.07: Rock Salt

Conglomerate

Pre-existing rocks metamorphose into **Metamorphic rocks,** by pressures and temperatures, wherein it is stretched and compressed but not melted. Some metamorphic rock ends up on the surface by uplifting forces and erosion. Others can get buried even deeper, where the temperatures are so hot that the rock starts melting - it turns into magma melt. Different forces may bring it closer to the surface again, where the temperatures may get so low that it "freezes" (this is still at a temperature of thousands of degree Celsius) and turns into an intrusive igneous rock. Or, it may end up exploding in a volcano and freeze on the surface of the earth to an extrusive igneous rock. Common examples of metamorphic rocks are Slate, Phyllite, Schist, Gneiss, Marble and Quartzite (Fig. 1.08 & Fig. 1.09).





Slate **Phyllite** **Schist**

Fig 1.08



Gneiss **Marble** **Quartzite**

Fig 1.09

And the cycle continues by erosion, burial, uplift and so on. All the rocks at any time on the Earth, are in some stage of that cycle.

3. Rock Discontinuities: The rocks are not found in the form of monolithic continua but they consist of intact rock material and number of discontinuities, also known as geological structures, which are planes/sources of weaknesses. These discontinuities are introduced either during the rock formation stage or later on due to various forces acting on it over a prolonged period of time. These discontinuities play a significant role at all stages in tunneling i.e. deciding alignment of tunnels, design of tunnel supports, and construction & maintenance of tunnels. Some of the common discontinuities are as following.

3.1 Rock Joints: Joints are most common rock discontinuity. A joint is a break in the continuity of the rock material without any movement along the joint surface (Fig. 1.10). A series of parallel joints is called as “Joint Set” and two or more joint sets

intersecting each other produce a “Joint System”. Joints are normally in parallel sets and their spacing can vary from a few to few ten centimeters.

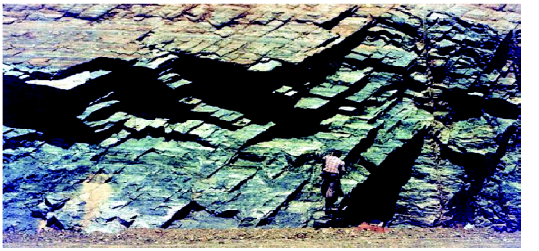


Fig. 1.10: Rock Joints

3.2 Faults: Fault is a discontinuity in rock, across which there has been significant displacement as a result of rock mass movement (Fig. 1.11 & Fig. 1.12). A fault plane is the plane that represents the fracture surface of a fault. A fault trace or fault line is the place where the fault can be seen or mapped on the surface. Since most of the time faults do not consist of a single clean fracture, term fault zone is also used to refer to zone of fault plane. The fault zone may contain crushed parent rock material or other material with or without water entrapped in it.

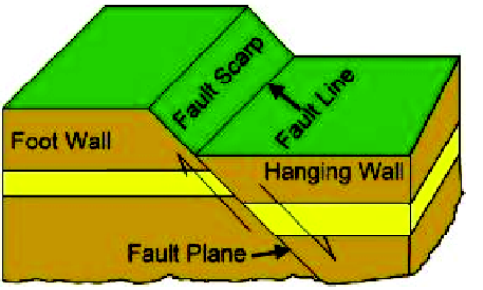


Fig. 1.11: Components of a Fault

Fig. 1.12: Faults

3.3 Folds: Fold is a bended and planar rock strata, as a result of tectonic forces or movements. They are often associated with high degree of fracture and relatively weak and soft rock. An anticline is a fold that is convex upwards and a syncline is a fold that is concave upwards (Fig. 1.13). Any underground excavation located at the bottom zone of a syncline fold is expected to have horizontal stresses of compressive in nature and any underground excavation located at the top zone of an anticline fold is expected to have less horizontal compressive stresses or tensile stresses. A **symmetrical fold** is one in which the axial plane is vertical and **asymmetrical fold** is one in which axial plane is inclined.

Anticline Fold **Syncline Fold**

Fig. 1.13

In most of the cases, the folds may be a combination of many types of folds as shown in Fig. 1.14.

3.4 Bedding Planes: Bedding plane is an interface between sedimentary rock layers (Fig. 1.15). Some bedding planes could also become potential weathered zones and ground water pockets.

Fig. 1.14: A Fault in Katra - Banihal Section of USBRL Project

Fig. 1.15: Bedding Planes

4. Rock Material and Rock Mass: For civil engineering works e.g. Foundations, Slopes and Tunnels, the scale of projects may vary from a few tens to a few hundred metres. In this extent, the rock encountered, often termed as rock mass, will contain the "rock material" in the form of intact rock blocks of various sizes and "rock discontinuities" (liken Joints, Faults, Folds, Bedding planes etc.). The "rock material" will be strong, stiff and brittle material, very strong in compression but weak in tension; with even weak rock material having compressive strength of order of 40-50 MPa which is more than strength of high strength concrete (M40 or M50 Grade Concrete). But behaviour of "rock mass" is controlled by the discontinuities and due to this, the rock mass strength may be 1/2 to 1/10 of the rock material strength. The discontinuities reduce rock mass strength, reduce rock mass quality and increase deformation under load. Therefore, "**Rock Mass = Rock Material + Rock Discontinuities".**

5. Scale Effect: In case of underground excavation like tunnel, the behaviour of rock mass, to a large extent, also gets governed by the spacing/extent of discontinuities vis-à-vis the dimension of excavation. In case, the rock mass is having very few discontinuities or the excavation dimension is less than the spacing of the discontinuities, then rock mass will behave like "Massive Rock", without much of influence due to discontinuities (Fig. 1.16).

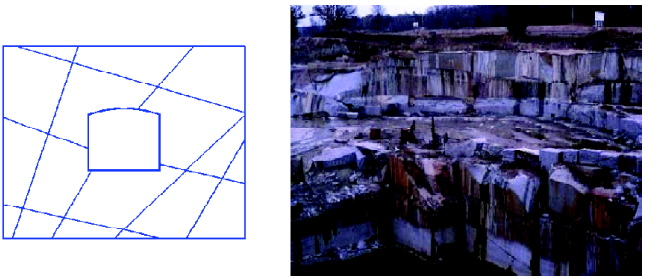


Fig. 1.16: Scale Effect - Massive Rock

When the rock mass is having moderate number of discontinuities or the excavation dimension is bigger than the spacing of the discontinuities, then rock mass will behave like "Jointed or Blocky Rock", with some influence due to discontinuities (Fig. 1.17).

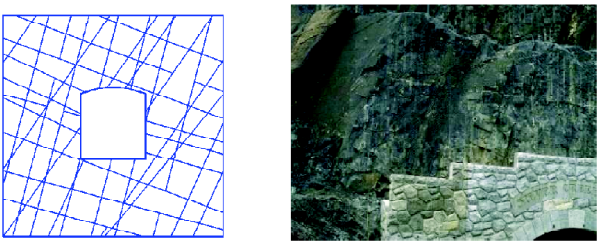


Fig. 1.17: Scale Effect – Jointed or Blocky Rock

In case rock mass is having large number of discontinuities or the excavation dimension is significantly higher than the spacing of the discontinuities, then rock mass will behave like "Heavily

Jointed Rock", with significant influence due to discontinuities (Fig. 1.18).

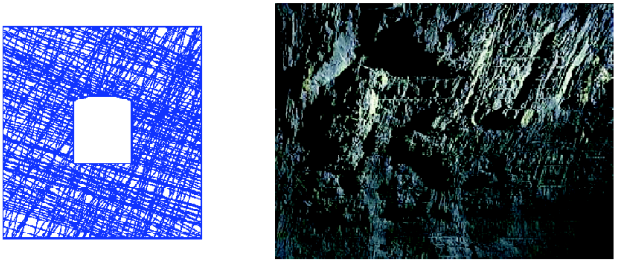


Fig. 1.18: Scale Effect – Heavily Jointed Rock

6. Groundwater: Groundwater is a very important factor for consideration in any underground excavation because:

(i) Water pressure contributes to the stresses on the tunnel supports.

(ii) Presence of water alters the properties of rock mass.

(iii) Presence of water increases the complexity in construction and maintenance of tunnels.

Most of the igneous and metamorphic rocks are very dense with interlocked texture and, therefore, have extremely low permeability and porosity. But some clastic sedimentary rocks, typically sandstones, can be porous and permeable. Weathered rocks can also be porous and permeable.

Fig. 1.19: Water inflow in Tunnel T-3 of Udhampur – Katra Section (measured peak Discharge of 600–1200 lit./Sec)

Therefore, the “Rock Mass Behaviour” is influenced by “Rock Material”, “Discontinuities” and “Ground Water”. (Fig. 1.20)

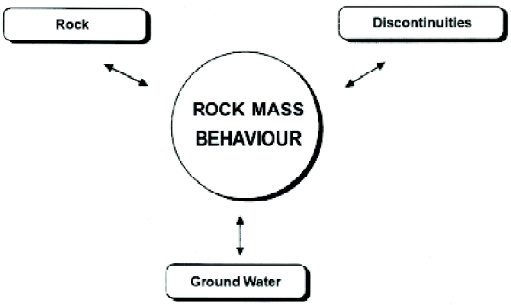


Fig. 1.20: Behaviour of Rock Mass

7. Inhomogeneity and Anisotropy:
Inhomogeneity represents property varying with locations. Rocks exhibit great inhomogeneity, due to:

(i) Different minerals in a rock,
(ii) Different bonding between minerals, and
(iii) Existence of discontinuities.

Anisotropy is defined as properties being different in different direction. It occurs in both rock materials and rock mass.
Inhomogeneity and Anisotropy present challenge in design and construction of tunnels.

8. In-situ Stresses: In-situ stresses are another complexity in design and construction of tunnels. In-situ stresses can be of two types;

8.1 Vertical Stress and Overburden: Vertical stress in rock is the overburden stress, generated by weight of the overlying material. The average specific gravity of rocks is about 2.7. The vertical stress at depth can be estimated as:

σ_{ZZ} (in MPa) $\approx$ 0.027 z, Where: z is overburden (in m)

8.2 Horizontal Stress (Tectonic Stresses): Horizontal stresses in rock are primarily tectonic stresses. Tectonic stresses have huge variations in magnitude, and can be exceptionally large. From large scale experimental observations (Fig. 1.23), it was observed that ratio of average horizontal stresses $(\sigma_{XX} + \sigma_{YY})/2$ to vertical stress is between 0.5 to 3.0, mostly bounded between (100/z + 0.3) and (1500/z + 0.5).

From Fig. 1.22, it can be seen that at common depths for Civil Engineering applications (<1000 m), the variation of horizontal stress is quite wide, which becomes narrow at depths of 2000m or more.

9. Special Rocks: There are many special types of rocks which present a difficulty in design of tunnel supports, construction of tunnel and inspection & maintenance of tunnels.

9.1 Weathered Rocks: All rocks disintegrate slowly as a result of Mechanical and Chemical weathering (Fig. 1.21). Some weathered rocks show structure and texture as normal rock, but due to weathering, rock material strength is significantly reduced.

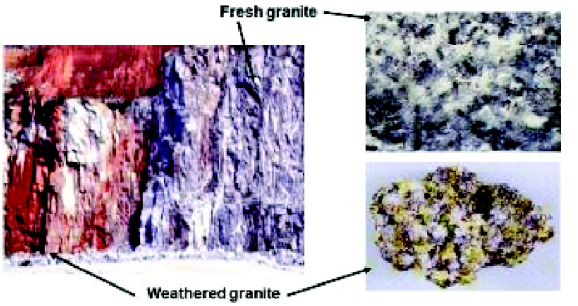


Fig. 1.21: Weathered Rocks

9.2 Soft Rocks and Hard Soils: Sedim-entary rocks are formed by sediments (soils) through long process of compaction and cementation. Many times, this process is stopped before the sediments are completely solidified. The materials then could be highly consolidated but not fully solidified. Typically, these materials have low strength and high deformability, and when placed in water, they often can be dissolved, but in dry condition, they behave as weak rock.

9.3 Swelling Rocks: Some rocks have the characteristics of swelling, when exposed to water. Rock and soil containing considerable amount of montmorillonite minerals will exhibit swelling and shrinkage characteristics. Such rocks can cause excessive pressures on the tunnel supports and can lead to their collapse, in worst case, even after completion of tunnel construction.

9.4 Crushed Rock: Characteristics of highly fractured and crushed rocks (Fig. 1.22) are quite different from the massive rock mass. When such materials are encountered, they need to be addressed separately.

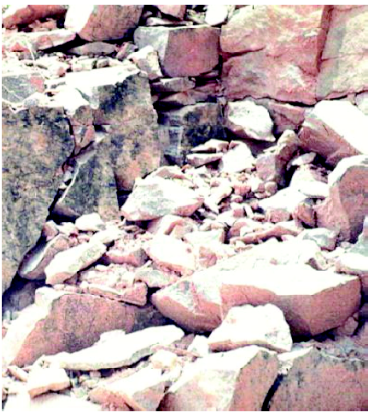


Fig. 1.22: Crushed Rock

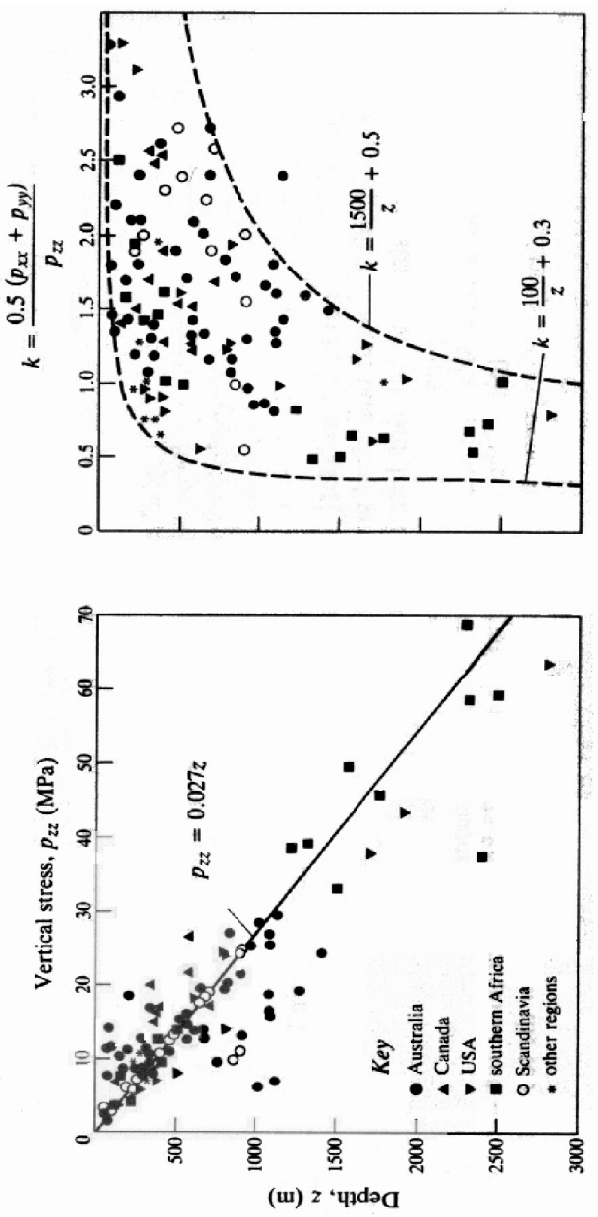


Fig. 1.23: Vertical and Horizontal Stresses

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CHAPTER-2

ENGINEERING PROPERTIES OF ROCKS

Due to various complexities, it is difficult to describe any single value for engineering properties of the rock as well as rock mass (Fig. 2.01). It has to be seen and understood in the overall context of the its' usage, duly considering effect of discontinuities, scale effect, in-situ stresses etc.

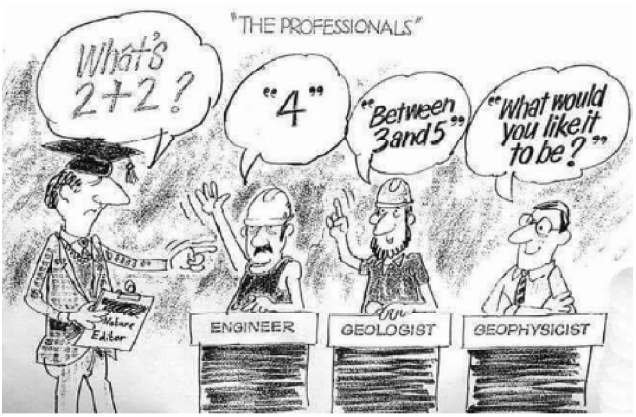


Fig: 2.01: Complexities in Rock Mass Properties

1. Determination of Rock Properties: The engineering properties of intact rock material or rock mass can be determined by two methods:

1.1 Direct Methods: Direct methods consist of Laboratory tests and Field (in-situ) tests. In-situ tests are normally time consuming and conducting them on the scale required for underground excavation sometimes becomes very costly. The data obtained by direct tests, especially for rock mass, is many times not consistent.

1.2 Indirect Methods: These methods are based on empirical or theoretical correlations. Current practices for determination of rock mass properties rely heavily on indirect methods. Indirect methods can also be used for verifying the results obtained from direct methods.

2. Test Methods: Though the tests to be carried out on Rock Material or Rock mass will get dictated by the intended use of the test results, type of rock mass and other relevant factors, International Society of Rock Mechanics (ISRM) has suggested some categories of tests (after Brown, 1981) which are as following:

2.1 Laboratory Tests

(A) For Characterization

(i) Porosity, Density and Water content

(ii) Absorption

(iii) Hardness - Schmidt Rebound Hammer and Shore Scleroscope

(iv) Resistance to Abrasion

(v) Point Load Strength Index

(vi) Uniaxial Compressive Strength and Deformability

(vii) Swelling and Slake Durability

(viii) Sound Velocity

(ix) Petrographic description

(B) For Engineering Design

(i) Tri-axial Strength and Deformability Test

(ii) Direct Shear Test

(iii) Tensile Strength Test

(iv) Permeability

(v) Time Dependent and Plastic Properties

2.2 In-situ Tests

(A) For Characterization

(i) Discontinuity Orientation, Spacing and Persistence, Roughness, Wall Strength, Aperture Filling, Seepage, Number of Joint Sets and Block Size

(ii) Drill Core Recovery and RQD

(iii) Geophysical Borehole Logging

(iv) In-situ Sound Velocity

(B) For Engineering Design

(i) Plate and Borehole Deformability Tests

(ii) In-situ Uniaxial and Tri-axial Strength and Deformability Test

(iii) Shear Strength - Direct and Torsional Shear

(iv) Field Permeability Measurement

(v) In-situ Stress Determination

3. Lab Test Samples: For carrying out various tests, test samples (mostly cylindrical in shape) are prepared, as per the size specified in the relevant code or specification (Fig. 2.02 & Fig. 2.03). The commonly used L/D (Length/Diameter) ratio for lab test specimen is as under:

Table-2.01: L/D Ratio for Lab Test Samples

Description of Test	L/D Ratio
Uniaxial or Tri-axial Compression	2 - 3
Brazilian Test	0.5
Point Load Index	1.0
Punch Test	0.2 – 0.25
Bending Test	3 - 7
Single Shear	2
Double Shear	3
Oblique Shear	1
Permeability	2

Fig. 2.02: Specimen for different Tests

Ends (top and bottom) should be flat to 0.05mm. Ends should be perpendicular to the axis of specimen within 0.001 radian (3.5 minutes). Sides should be smooth and free of abrupt irregularities, straight to within 0.3mm over full length of specimen.

Fig. 2.03: Strain Gauging of Test specimen

4. Strength and Deformation Tests: Some of the most commonly performed strength and deformations tests are covered in brief.

4.1 Uniaxial Compressive Strength (UCS) Test: UCS is the ultimate stress a cylindrical rock specimen can take under axial load (Fig. 2.04). It is important mechanical property of rock material, used in design, analysis and modelling.

Diameter of specimen must be greater than 10 times the diameter of largest grain size. Preferable dia. is 45mm. But in no case the diameter of core sample should be less than 35mm. Specimen is kept in a loading frame and axial load is continuously increased till the specimen fails

(crumbles). At least 5 specimens are required to give a representative value. Along with axial load, the axial and lateral deformations (strains) are also measured. *(Ref. 42, IS:9143-1979).*

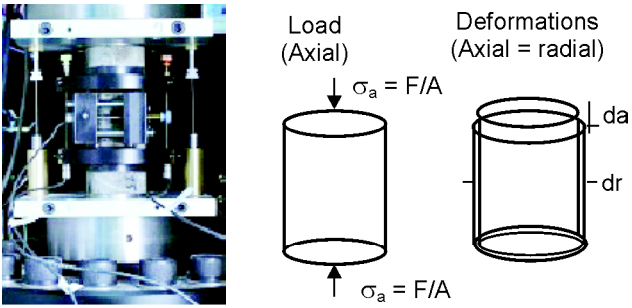


Fig. 2.04: UCS Test for Rock

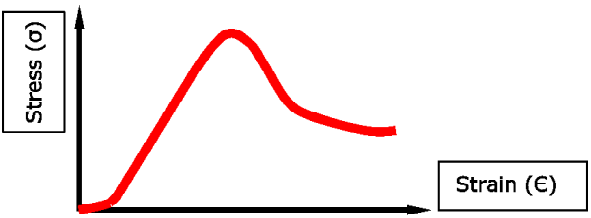


Fig. 2.05: Stress-Strain curve for UCS Test

4.2 Tri-axial Compression Test: True tri-axial compression means three different principal stresses. But it is often simplified by conducting axisymmetric tri-axial test wherein two lateral stresses equal to minor principal stress σ_3 are applied by confining pressure around the sample in tri-axial cell and the major principal stress is applied as deviator stress ($\sigma_1 - \sigma_3$) required to take sample up to failure for the given confining pressure *(Ref. 43, IS:13047-1991).*

For various confining pressures/stresses, which is minor principal stress (σ_3), values of deviator

stress ($\sigma_1 - \sigma_3$) required to take sample up to failure are measured, which in-turn gives the major principal stress (σ_1). By this approach, a set of σ_1 and σ_3 values are obtained. By continuous measurements of deviator stress and axial strain, the values of Axial Stress and Axial Strain can be worked out/plotted for any given value of confining stress (σ_3) (Fig. 2.06).

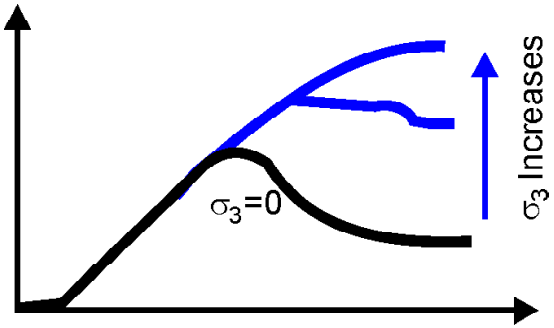


Fig. 2.06: Stress-Strain curve for Tri-axial Tests

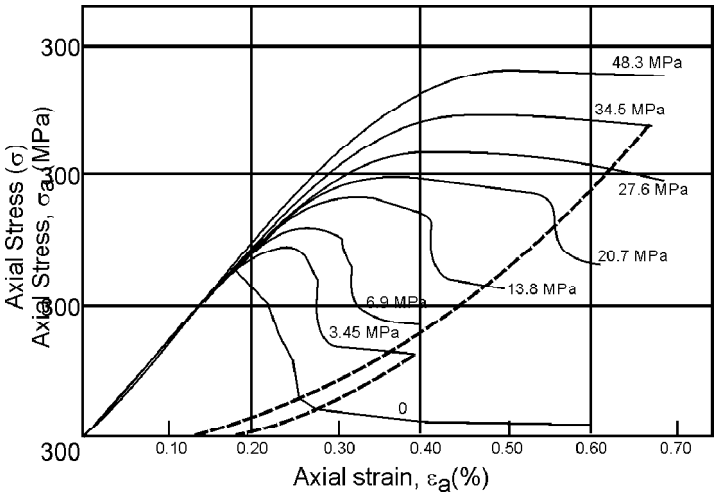


Fig. 2.07: Sample of Tri-axial Test Results

The behaviour of rock in tri-axial compression changes with increasing confining pressure, with the stress-strain behaviour in elastic region remaining same as in uniaxial compression (Fig. 2.06 & 2.07).

4.3 Young's Modulus and Poisson's Ratio: Young's Modulus and Poisson's Ratio can be experimentally determined from the Stress-Strain curve. They seem to be unaffected by change of confining pressure. High strength rocks also tend to have high Young's Modulus, depending on rock type and other factors. For most rocks, the Poisson's ratio is between 0.15 and 0.4. Typical values/range of values for Young's Modulus for various types of rocks are given in Fig. 2.08.

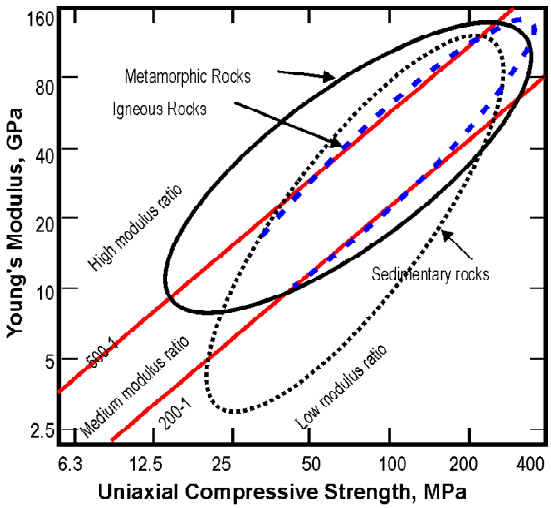


Fig. 2.08: Typical values of Young's Modulus

4.4 Brazilian Tensile Strength Test: Rock material generally has a low tensile strength, due to the pre-existing micro-cracks in the rock material.

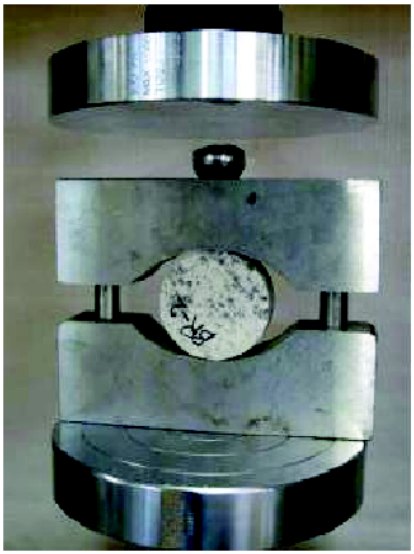


Fig. 2.09: Brazilian Test

Rock material tensile strength can be obtained from several types of indirect tests and the most common tensile test is “Brazilian test” (Fig. 2.09). When using the Brazilian test to determine the indirect tensile strength of rock *(Ref. 44, IS:10082-1981)*, it is usually assumed that failure is the result of the uniform tensile stress normal to the splitting diameter and that the tensile strength, T_0, is given by the formula:

$$T_0 = 2*P / \pi*D*L$$

Where:

“P” is the applied load, “D” is the core diameter and “L” the core thickness (length of specimen)

Table 2.02: Typical Values of UCS and Tensile Strength

Rock	UC Strength (MPa)	Tensile Strength (MPa)
Granite	100 -300	7 – 25
Dolerite	100 -350	7 – 30
Gabbro	150 -250	7 - 30
Basalt	100 – 350	10 -30
Sandstone	20 - 170	4 - 25
Shale	5 - 100	2 - 10
Dolomite	20 - 120	6 – 15
Limestone	30 - 250	6 – 25
Gneiss	100 - 250	7 - 20
Slate	50 - 180	7 – 20
Marble	50 - 200	7 – 20
Quartzite	150 - 300	5 - 20

4.5 Shear Strength: Rocks resist shear stress by two internal mechanisms, Cohesion (c) and Internal Friction (Ø). Like soil, shear strength of rock material can be determined by direct shear test and by tri-axial compression test.

In Direct Shear Test, specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample (Fig. 2.10). A normal stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails. The normal stress applied and shear stress at failure is recorded. Several specimens are tested at varying normal stresses, to plot stress–strain curve, with peak shear stress on the y-axis and the normal stress on the x-axis (Fig. 2.11). The y-intercept of the curve which fits the test results is the cohesion (c), and the slope

of the line or curve is the friction angle (Ø) *(Ref. 45, IS:12634-1989).*

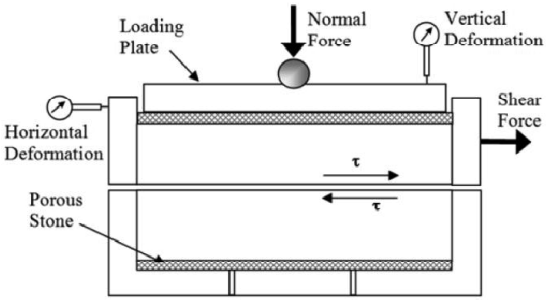


Fig. 2.10: Direct Shear Test Apparatus

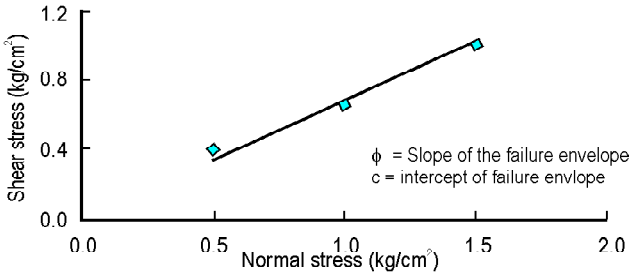


Fig. 2.11: Plot of Shear Stress v/s Normal Stress

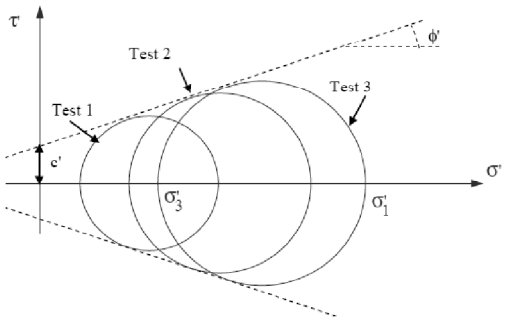


Fig. 2.12: Mohr Circles based on Tri-axial Tests

In Tri-axial Shear Test, from a series of tri-axial tests, peak stresses (σ_1) are obtained at various confining stresses (σ_3). Using each set of σ_1 and σ_3 values, numbers of Mohr Circles are drawn (Fig. 2.12). The slope of best fit tangent on these circles is friction angle (Ø) and y-intercept of this tangent is the cohesion (c).

4.6 Point Load Index Test: Point load test is a simple index test for rock material. It is an attractive alternative to UCS as it can provide similar data at a lower cost, as block or irregular lump specimen can also be tested in this test. The apparatus for this test consists of a rigid frame, two point load platens, a hydraulically activated ram with pressure gauge and a device for measuring the distance between the loading points (Fig. 2.13 & Fig. 2.14).

This test gives the standard Point Load Index $Is_{(50)}$ for a 50mm diameter sample. Minimum of 10 test specimens are required to find out the average value of Point Load Index *(Ref. 46, ASTM D5731-2016).*

$$Is_{(50)} = (P*1000)/(D^{1.5} * \sqrt{50}) \text{ in MPa}$$

Where,

"P" is Breaking Load in KN and "D" is distance between platens in mm.

Point Load Index can be correlated to Strengths by various empirical formulas, as given below, and it can be used as an independent strength index also.

Compressive Strength: $\sigma_c \approx 22 * Is_{(50)}$

(The multiplication factor of 22 can vary from 10 to 30)

Tensile Strength: $\sigma_t \approx 1.25\ I_{s(50)}$

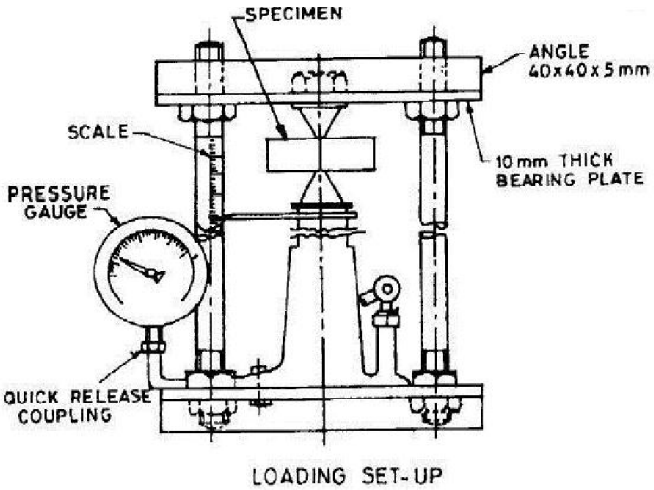


Fig. 2.13: Test Setup for Point Load Testing

Fig. 2.14: Point Load Testing Machine

5. Physical and Engineering Properties Tests: Some of the most commonly performed tests for Physical and Engineering properties of rocks are described in brief.

5.1 Density, Porosity and Water Content:

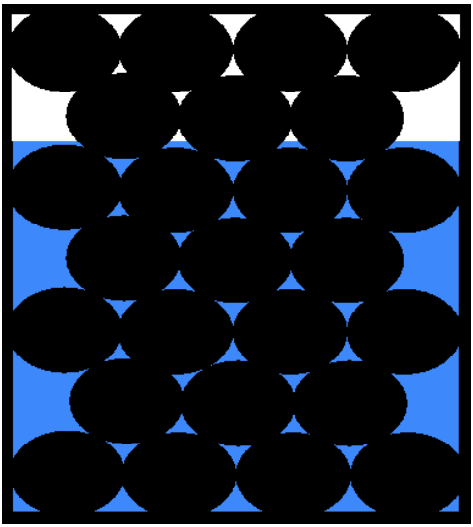


Fig. 2.15: Rock Mass Matrix

Density=Bulk Mass/Bulk Volume

Porosity=Non-solid Volume/Bulk Volume

Water Content=Volume of Water/Bulk Volume

Dry density of rock material is generally between 2.5-2.8 g/cm^3. High density generally means low porosity.

Porosity is generally low for crystalline rocks, e.g. Granite (<5%) and can be high for clastic Sedimentary rocks, e.g. Sandstone (up to 50%). Porosity effects permeability.

Water content depends on saturation. Wet rock tends to have slightly lower strength.

5.2 Hardness: Hardness of rock/rock mass is used for predicting cutting/boring rates for tunnel boring machines and determination of rock quality for construction purposes etc. The rebound hammer test provides a means for rapid determination of hardness.

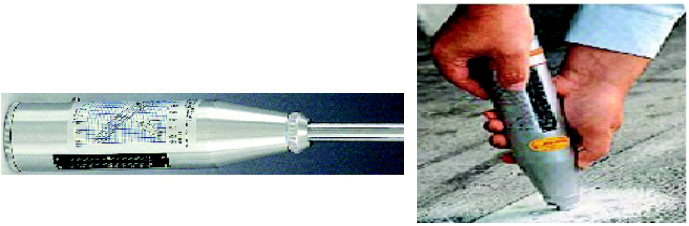
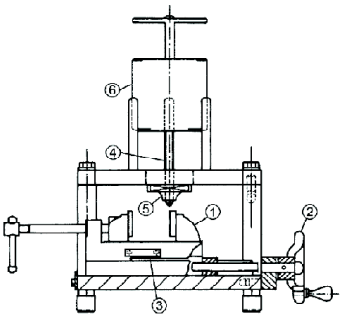


Fig. 2.16: Rebound Hammer Test

Using Rebound Hammer Values, the hardness is arrived at using the standard charts supplied with the equipment.

5.3 Abrasivity: Abrasivity of the rocks is used to design the cutting tools of Tunnel Boring Machine or Road Headers or to predict wear of excavator buckets. It is determined most commonly in terms of Cerchar Abrasivity Index (CAI) *(Ref. 47, ASTM D7625-2010).*



(1) Rock sample holder
(2) Hand wheel for base movement
(3) Displacement scale
(4) Steel stylus
(5) Stylus holder
(6) Dead weight

Fig. 2.17: CERCHAR Abrasivity Test setup

The test is performed on a small freshly broken rock sample, less than 25mm in size, which is held in position by sample holder. The sample is scratched by a hardened sharp heat-treated alloy steel needle of a defined geometry, over a length of 10mm in 1 second, under a static load of 70kN (Fig. 2.17). The CAI is then calculated as 1/10th of d_i (average measured worn flat diameter of testing needle, in tenth of mm).

Table 2.03: Typical values of CAI

Granite	4.5 – 5.3
Diorite	4.2 – 5.0
Andesite	2.7 – 3.8
Basalt	2.0 – 3.5
Sandstone	2.8 – 4.2
Shale	0.6 – 1.8
Limestone	1.0 – 2.5
Gneiss	3.5 – 5.3
Slate	2.3 – 4.2
Quartzite	4.3 – 5.9

5.4 Permeability: It is a measure of ability of the material to transmit fluids. It is of importance only for porous rock masses, as it can be used to predict the amount and pressure of water likely come through rock material, which has to be tackled during construction phase and for design of waterproofing arrangements. However, in rock mass the flow is concentrated in fractures.

Coefficient of Permeability (k) is measured by Darcy's law, using following formula (Fig. 2.18) *(Ref. 48, IS:4348-1973):*

$$Q = A * k * (h_1 - h_2) / L$$

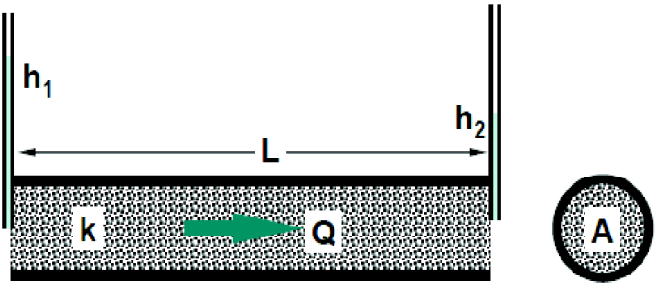


Fig. 2.18: Darcy's Law Setup

Where:

Q	=	Flow rate
k	=	Coefficient of Permeability
A	=	Cross section area
h_1 & h_2	=	Hydraulic head
L	=	Length of specimen

5.5 Wave Velocity: Two types of waves are normally used in velocity measurements i.e. Longitudinal (P) wave and Shear (S) wave. P wave is the fastest travelling wave and is most commonly used in wave velocity measurements.

The waves generated by a pulse generator are transmitted into the sample of known length by a transmitter at one end of sample and received at other end of sample by a receiver. By using the time of travel, the transmission velocity of the wave in the sample is calculated.

A well compacted rock mass will generally have high velocity as the grains are in good contact and wave travels faster through solid grains.

P wave velocity (V_p) of Gneiss and Quartzite is 5000-7000 m/s and of Shale, Sandstone and Conglomerate is 3000-5000 m/s.

S wave velocity (V_s) of Gneiss and Quartzite 3000-4000 m/s and of Shale, Sandstone and Conglomerate is 2000-3000 m/s.

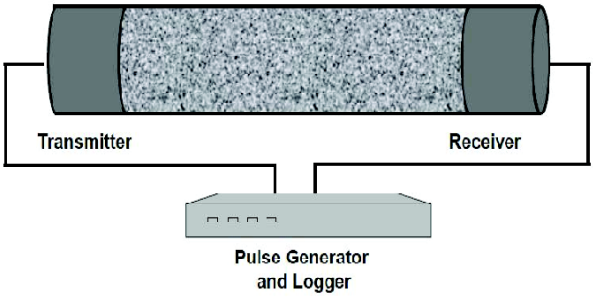


Fig. 2.19: Wave velocity measurement

Wave velocities are used to estimate the modulus of rock material, by using following equations:

Elastic Modulus $E_s = \rho\ Vp^2 (GP_a), (g/cm^3), (km/s)$

Shear Modulus $G_s = \rho\ Vs^2 (GP_a), (g/cm3), (km/s)$

Poisson's Ratio $\nu_S = [1-2(V_s/V_p)^2]/[2\{1-(V_s/V_p)^2\}]$

Where:

ρ = Density of material

ν_S = S Wave velocity in the material

ν_S = P Wave velocity in the material

The modulus values arrived at by using the above equations is slightly higher than the modulus determined from static tests.

6. Failure Criteria of Rock Material: There are two failure criteria normally used for rock material.

6.1 Mohr-Coulomb Criterion is a mathematical model describing the response of brittle materials to shear stress as well as normal stress (Fig.

2.20). The failure envelope, in a plot between Normal Stress (σ_n) and Shear Stress (τ) is represented by a circle. Any point within the circle is stable and any point outside the circle is unstable, with the points on the periphery of the circle being on the verge of instability.

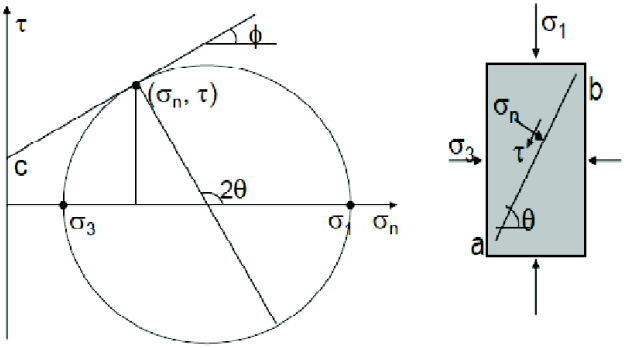


Fig. 2.20: Mohr-Coulomb Failure Criterion

$$\sigma_n = \tfrac{1}{2}(\sigma_1 + \sigma_3) + \tfrac{1}{2}(\sigma_1 - \sigma_3)\cos 2\theta$$

$$\tau = \tfrac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta$$

$$\theta = \tfrac{1}{4}\pi + \tfrac{1}{2}\varnothing$$

Where: σ_n = Normal Stress on the failure plane
τ = Shear Stress on the failure plane
σ_1 = Major Principal Stress
σ_3 = Minor Principal Stress
θ = Inclination of the failure plane

Shear strength is made up of two parts, a constant Cohesion (c) and a normal stress dependent Angle of Internal Friction (Ø):

$$\zeta = c + \sigma_n \tan\varnothing$$

Actual tensile strength of rock material is lower than the criterion. A tensile cut-off is usually

applied at a selected value of uniaxial tensile stress, σ_t', at about 1/10 of σ_c (Fig. 2.21).

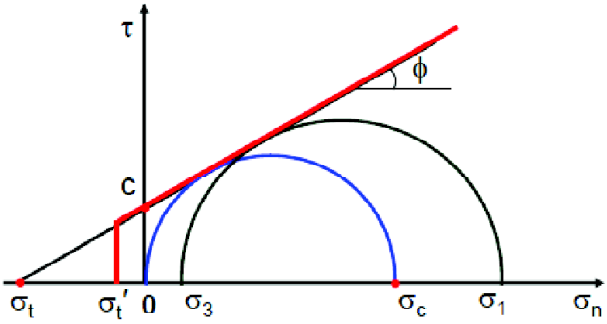


Fig. 2.21: Compressive & Tensile Strength

The compressive strength is given by:

$$\sigma_c = (2c \cos Ø) / (1 - \sin Ø)$$

The tensile strength is given by:

$$\sigma_t = (2c \cos Ø) / (1 + \sin Ø)$$

6.2 Hoek-Brown Criterion: Mohr-Coulomb criterion is suitable for the low range of confining stress only and at high confining stress, it overestimates the strength. It also overestimates tensile strength. Since in most cases, rock engineering deals with shallow depth problems and low confining stress, so Mohr-coulomb criterion is widely used, due to its simplicity and popularity.

To cater for a wide range of compressive stress conditions, number of empirical strength criteria have been introduced for practical use and one of the most widely used criteria is the Hoek-Brown (H-B) criterion for isotropic rock materials and rock masses. The classic Hoek–Brown failure criterion, empirical in its formulation and based on numerous experimental data, has been widely used to predict the failure of rocks.

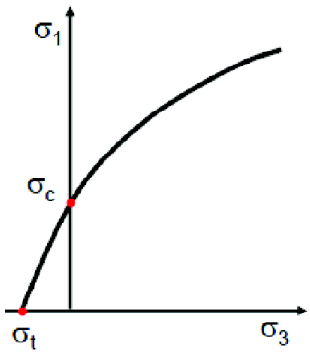


Fig. 2.22: Generalized H-B Criterion

The generalized Hoek-Brown criterion is non-linear (parabolic) in form (Fig. 2.22) and is given by the following equation:

$$\sigma_1 = \sigma_3 + (m_b \sigma_3 \sigma_{ci} + s \sigma_{ci}^2)^a$$

Where,

σ_1 = Major Principal Stress

σ_3 = Minor principal Stress

σ_{ci} = Compressive strength of rock material

m_b, s & a = Constants depending on type/condition of rock mass

Hock-Brown criterion for rock material is a special form of the generalised equation when s =1, a = 0.5, $m_b = m_i$, thereby giving the following equation:

$$\sigma_1 = \sigma_3 + (m_i \sigma_3 \sigma_{ci} + \sigma_{ci}^2)^{0.5}$$

Values of constants m_b, s & a are calculated using equations given by Hoek and Brown with GSI (Geological Strength Index) and Disturbance Factor (D). Hoek and Brown have given detailed methodology for estimating GSI *(discussed in detail in Chapter-3)* and D.

Using suitable software also (e.g. RocLab), values of these constants can be easily estimated with results of tri-axial shear test (number of σ_1 and σ_3 values) as the input.

Typical values of m_b and s, depending upon the quality of rock mass are given in Table 2.04 *(values of RMR and Q are discussed in Chapter-3).*

As shown in Fig. 2.23, exact type of failure envelope depends on the spacing of the discontinuities in the rock mas vis-à-vis the size of the openings or excavation (i.e. size affect).

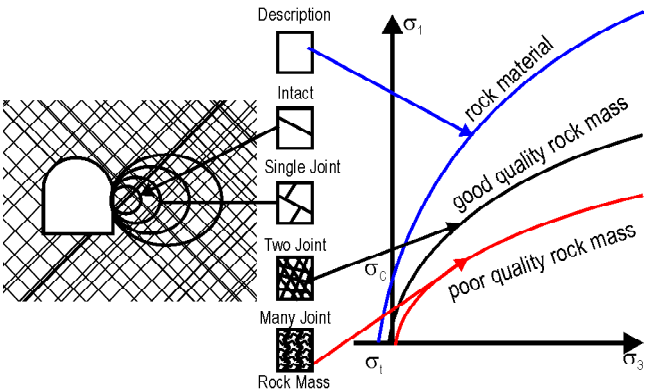


Fig. 2.23: Types of H-B Failure Envelopes

Table 2.04: Typical values of m_b and s

Hoek-Brown Failure Criterion $\sigma_1/\sigma_c = \sigma_3/\sigma_c + (m_b \sigma_3/\sigma_c + s)^{0.5}$	Carbonate rocks - *dolomite, limestone, marble*	Argillaceous rocks - *mudstone, siltstone, shale, slate*	Arenaceous rocks - *sandstone, quartzite*	Fine grained igneous - *andesite, dolerite, basalt, rhyolite*	Coarse metamorphic & igneous - *gabbro, gneiss, granite*
Intact rock material RMR = 100 , Q=500	$m_i = 7.0$ $s = 1.0$	$m_i = 10.0$ $s = 1.0$	$m_i = 15.0$ $s = 1.0$	$m_i = 17.0$ $s = 1.0$	$m_i = 25.0$ $s = 1.0$
Very good quality rock mass RMR = 85, Q = 100	$m_b = 3.5$ $s = 0.1$	$m_b = 5.0$ $s = 0.1$	$m_b = 7.5$ $s = 0.1$	$m_b = 8.5$ $s = 0.1$	$m_b = 12.5$ $s = 0.1$
Good quality rock mass RMR = 65, Q = 10	$m_b = 0.7$ $s = 0.004$	$m_b = 1.0$ $s = 0.004$	$m_b = 1.5$ $s = 0.004$	$m_b = 1.7$ $s = 0.004$	$m_b = 2.5$ $s = 0.004$
Fair quality rock mass RMR = 44, Q = 1.0	$m_b = 0.14$ $s = 0.0001$	$m_b = 0.20$ $s = 0.0001$	$m_b = 0.30$ $s = 0.0001$	$m_b = 0.34$ $s = 0.0001$	$m_b = 0.50$ $s = 0.0001$
Poor quality rock mass *RMR = 23, Q = 0.1*	$m_b = 0.04$ $s = 0.00001$	$m_b = 0.05$ $s = 0.00001$	$m_b = 0.08$ $s = 0.00001$	$m_b = 0.09$ $s = 0.00001$	$m_b = 0.13$ $s = 0.00001$
Very poor quality rock mass *RMR = 3, Q = 0.01*	$m_b = 0.007$ $s = 0$	$m_b = 0.01$ $s = 0$	$m_b = 0.015$ $s = 0$	$m_b = 0.017$ $s = 0$	$m_b = 0.025$ $s = 0$

CHAPTER-3

ROCK MASS CLASSIFICATION SYSTEMS

Rock mass classification systems form the back bone of the empirical design approach and are widely employed in rock engineering. The rock mass classifications have been quite popular and being used in feasibility designs. It has been experienced repeatedly that when used correctly, a rock mass classification can be a powerful tool in designs. In fact, on many projects, the classification approach serves as the only practical basis for the design of complex underground structures. The Gjovik Underground Ice Hockey Stadium of 60m width in Norway was also designed by the classification approach. *(Ref. 4: Bhawani Singh & R. K. Goel).*

Rock mass classifications systems have improvised with time, to take into account various developments in geological investigations as well as underground excavations support systems (like rock bolts, shotcrete, steel fiber reinforced shotcrete etc.) and they have been widely used due to following reasons:

(i) It provides a common acceptable platform to geologist, designers, contractors and engineers, in understanding and expressing quality of rock for the intended purpose.

(ii) Rather than expressing the rock quality in subjective terms, it expresses rock quality in terms of a numerical value, thereby enabling better understanding of rock quality without any element of subjectivity.

In all rock mass classification systems, minimum rating is assigned to poorest rock mass and maximum rating is assigned to excellent rock mass. It must be noted that no single classification system may cater to all types of rock masses from poorest to excellent. Therefore, it requires experience to select proper

classification system for the given type of rock mass. A sound engineering judgment emerges out of working in the field for a long time and no formula or rating system can be a substitute for this.

Precaution should be taken that joint parameters are not accounted twice, once for analysis and once for classification. Any isolated variation or aberration in rock parameters should not be accounted, once allowance has been given for such uncertainties. It is better to give a range of rating for each parameter. In linear classification systems like RMR and GSI, average of rock mass rating is taken for design of supports, whereas in non-linear classification systems like Q the geometrical mean of maximum and minimum values are considered for the design.

Empirical, numerical/analytical and observational approaches are various tools for engineering design of tunnel supports. The empirical approach, based on rock mass classification systems, is very popular because of simplicity and ability to manage uncertainties. However, in present day, a mix of all approaches is employed and "Design as you go" approach is adopted. It should be noted that Rock mass classification based design procedures have inherent limitations and their use does not (and cannot) replace more elaborate design procedures. The approach generally adopted is as under:

(i) In feasibility studies, empirical correlations may be used to estimate rock parameters.

(ii) At design stage, in-situ tests should be conducted for the major projects to determine actual rock parameters as well in-situ stresses.

(iii) At the initial construction stage, instrumentation should be carried out on the supported excavated surface and within the rock mass, for getting field data about actual displacements/stresses etc. and comparing them with predicted values. Instrumentation is very

critical for a safe and steady tunneling rate. The instrumentation data should be utilized in computer modelling for back analysis of the model and its' improvisation for further tunneling.

(iv) At construction stage, forward analysis of rock structures should be carried out using the back analyzed model and parameters of rock mass. Repeated cycles of back analysis and forward analysis (BAFA) may eliminate many inherent uncertainties in geological mapping and knowledge of engineering behaviour of rock masses.

(v) In case of non-homogenous and complex geological environment, slightly conservative values of rock parameters may be used for the purpose of designing site specific remedial measures.

(vi) Be prepared for the worst and hope for the best. *(Ref. 4: Bhawani Singh & R. K. Goel).*

Some of the important Rock Mass Classification Systems are discussed as under:

1. Rock Quality Designation (RQD): The RQD was initially proposed by Deere (1963) as an index of assessing rock quality quantitatively and it has since then been the topic of various assessments for civil engineering projects. It has become a fundamental parameter or property of rock mass, mainly due to its simple definition.

$$RQD\ (\%) = \sum x_i / L \times 100$$

Where:

x_i = Lengths of individual pieces of core $\geq$ 10 cm (obtained using NX-Size Core 54.7mm Dia., with double tube core barrel using a diamond bit).

L = Total length of the drill run

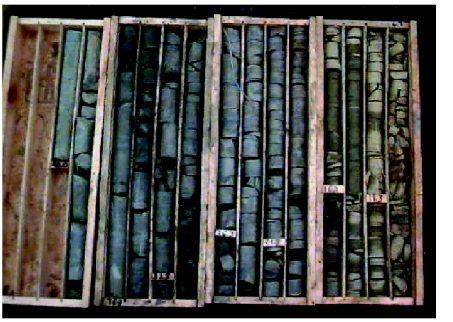
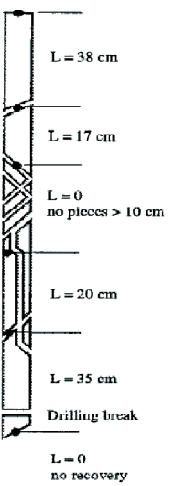


Fig. 3.01: Rock fragments obtained in drill run

RQD represents degree of fracturing of rock mass. It partially reflects the in-situ rock mass quality. It is a directionally dependant parameter and is used as a component in most of the "Rock Mass Classification Systems".

Calculation of RQD is explained by an example, as under:



Total length of core run=200cm

RQD = [(x_i > 10cm)/L] * 100

(x_i > 10cm) = 38+17+20+35
= 110 cm

L = 200 cm

Hence, RQD = (110/200)*100
= 55 %

Table 3.01: Rockmass Quality based on RQD

Description	RQD
A. Very Poor	< 25
B. poor	25 - 50
C. Fair	50 - 75
D. Good	75 - 90
E. Excellent	90 - 100

2. Terzaghi's Rock Mass Classification: This was one of the earliest attempts (in 1946) to classify rock mass for engineering purpose. Terzaghi classified rock masses into 9 categories, based on structural discontinuities, as detailed in Table 3.02.

Table 3.02: Terzaghi Rock Mass Classification

Rock	Rock Class	Description Condition
I	Hard and intact	The rock is un-weathered. It contains neither joints nor hair cracks. If fractured, it breaks across intact rock. After excavation, the rock may have some popping and spalling failures from roof. At high stresses, spontaneous and violent spalling of rock slabs may occur from the side or the roof. The unconfined compressive strength is equal to or more than 100 MPa.
II	Hard stratified and schistose	The rock is hard and layered. The layers are usually widely separated. The rock may or may not have planes of weakness. In such rocks, spalling is quite common.

III	Massive, moderately jointed	A jointed rock, the joints are widely spaced. The joints may or may not be cemented. It may also contain hair cracks but the huge blocks between the joints are intimately interlocked so that vertical walls do not require lateral support. Spalling may occur.
IV	Moderately blocky and seamy	Joints are less spaced. Blocks are about 1m in size. The rock may or may not be hard. The joints may or may not be healed but the interlocking is so intimate that no side pressure is exerted or expected.
V	Very blocky and seamy	Closely spaced joints. Block size is less than 1m. It consists of almost chemically intact rock fragments which are entirely separated from each other and imperfectly interlocked. Some side pressure of low magnitude is expected. Vertical walls may require supports.
VI	Completely crushed but chemically intact	Comprises chemically intact rock having the character of a crusher-run aggregate. There is no interlocking. Considerable side pressure is expected on tunnel supports. The block size could be few centimeters to 30cm.

VII	Squeezing rock – Moderate depth	Squeezing is a mechanical process in which the rock advances into the tunnel opening without perceptible increase in volume. Moderate depth is a relative term and could be from 150 to 1000m.
VIII	Squeezing rock – Great depth	The depth may be more than 150m. The maximum recommended tunnel depth is 1000m.
IX	Swelling rock	Swelling is associated with volume change and is due to chemical change of the rock, usually in presence of moisture or water. Some shales absorb moisture from air and swell. Rocks containing swelling minerals such as montmorillonite, illite, kaolinite and others can swell and exert heavy pressure on rock supports.

3. Rock Mass Rating (RMR): The geo-mechanics classification or Rock mass rating (RMR) was developed by Bieniawski in 1973 in South Africa *(Ref. 49, Bieniawski Z. T.)* with significant evolutions later on, last being in 1984 in USA. To apply this system, the site should be divided into a number of geological structural units having their boundaries usually coinciding with a major structural feature such as a fault or a change in rock type. Each unit should then be classified separately. In some cases, within the same rock type, division of the rock mass into a number of small structural units may be required due to significant changes in discontinuity spacing or characteristics. Each type of rock mass shall be represented by a separate

geological structural unit. Following six parameters are considered for each of the geological structural unit:

(i) Strength of Intact Rock Material
(ii) Rock Quality Designation (RQD)
(iii) Spacing of discontinuities
(iv) Condition of discontinuities
(v) Groundwater conditions
(vi) Orientation of discontinuities

For each geological structural unit, ratings are assigned for first five parameters, as detailed in Appendix 3.1. After summing up the ratings for these five parameters, a correction is applied for the sixth parameter (i.e. orientation of discontinuity vis-à-vis direction of excavation) as detailed in Appendix 3.1. The resultant rating, so obtained, is RMR.

While assigning the ratings, the typical rather than the worst conditions are evaluated, since this classification being based on case histories, has a built-in safety factor.

This classification is more applicable to hard rock situations and not found to be very reliable in very poor rock mass.

Based on RMR value, the rock mass quality is described as given in Table-3 of Appendix 3.1.

4. Rock Mass Quality (Q): Barton, Lien and Lunde (1974) of the Norwegian Geotechnical Institute (NGI), on the basis of about 200 case histories of tunnels and caverns, proposed a Tunnelling Quality Index (Q) *(Ref. 16, Barton N., Lien R. and Lunde J.).* The numerical value of Q varies on a logarithmic scale from 0.001 to a maximum of 1000, but its value typically varies from 0.01 to 100. The Rock Mass Quality (Q) is defined as:

$$Q = \left(\frac{\mathrm{RQD}}{\mathrm{J_n}}\right) \times \left(\frac{\mathrm{J_r}}{\mathrm{J_a}}\right) \times \left(\frac{\mathrm{J_w}}{\mathrm{SRF}}\right)$$

Where:

RQD	=	Rock Quality Designation ≥ 10
	=	$115 - 3.3\ J_v \leq 100$
J_n	=	Joint Set Number
J_r	=	Joint Roughness Number for critically oriented joint set
J_a	=	Joint Alteration Number for critically oriented joint set
J_w	=	Joint Water Reduction Factor
SRF	=	Stress Reduction Factor to consider in-situstresses
J_v	=	Joint Volume Count

Numerical ratings are assigned to above six parameters, based on rock conditions. The ratings do not include joint orientation to make the classification more general and orientation of joints/discontinuities is considered by taking lowest value of second quotient for computing Q for Most unfavourable joint.

The parameter J_n, representing the number of joint sets, is often affected by foliations, schistocity, slaty cleavages or beddings etc. If strongly developed, these parallel discontinuities should be counted as a complete joint set. If there are few joints visible or only occasional breaks in rock core due to these features, then one should count them as “a random joint set” while evaluating Jn.

The parameters J_r and J_a, represent roughness and degree of alteration of joint walls or filling materials. These parameters should be obtained for the weakest critical joint set or clay-filled discontinuity in a given zone. If the joint set or the discontinuity with the minimum value of (J_r/J_a) is favorably oriented for stability, then a second less favorably oriented joint set or discontinuity may be of greater significance, and its value (J_r / J_a) should be used when evaluating Q.

The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints. This is due to reduction in the effective normal stress across joints. Water in addition may cause softening and possible wash-out in the case of clay-filled joints. The value of J_w should correspond to the future ground water condition where seepage erosion or leaching of chemical can alter permeability of rock mass significantly.

The parameter SRF is a measure of (i) loosening pressure in the case of an excavation through shear zones and clay bearing rock masses, (ii) rock stress σ_c/σ_1 in a competent rock mass, where σ_c is uniaxial compressive strength of rock material and σ_1 is major principal stress before excavation, and (iii) squeezing or swelling pressures in incompetent rock masses. SRF can also be regarded as a total stress parameter. Some of the ratings for SRF have been altered by Grimstad and Barton (1993) *(Ref. 50).*

Tables for ratings for various parameters, including alterations by Grimstad and Barton (1993), are given in Appendix 3.2.

Table 3.03: Rock Mass Classification v/s Q

Group	Q	Classification
1	1000-400	Exceptionally good
	400-100	Extremely good
	100-40	Very good
	40-10	Good
2	10-4	Fair
	4-1	Poor
	1-0.1	Very poor
3	0.1-0.01	Extremely poor
	0.01-0.001	Exceptionally poor

Many empirical correlations have been developed between RMR and Q, as given in the Table 3.04 below:

Table 3.04: Empirical correlations

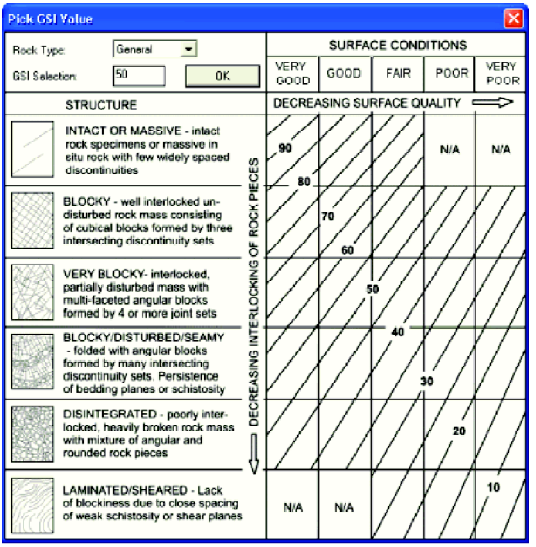
Correlation	Reference
RMR = 9.0 ln Q + 44	Bieniaswki (1976), Jethwa et al. (1982)
RMR= 5.9 ln Q + 43	Rutledge & Preston (1978)
RMR = 5.4 ln Q + 55	Moreno (1980)
RMR = 4.6 ln Q + 56 (drill core) RMR= 5.0 ln Q + 61 (In-situ results)	Cameron-Clarke and Budavari (1981)
RMR = 10.5 ln Q + 42	Abad et al. (1984)
RMR = 8.7 ln Q + 38	Kaiser et al. (1986)
RMR = 9.1 ln Q + 45 RMR= 7.0 ln Q +41 (Bore cores)	Trunk & Homisch (1990) El-Naqa (1994)
RMR = 7.0 ln Q + 44 (Scan lines)	
RMR = 15 ln Q + 50	Barton (1995)

RMR and Q system or their variants are the most widely used rock mass classification systems. Both incorporate geological, geometric and design/engineering parameters to obtain a "value" of rock mass quality. But, they are empirical and require subjective assessment.

5. Geological Strength Index (GSI): Hoek and brown (1980) proposed a method for estimating the strength of jointed rock masses, based upon assessment of interlocking of rock blocks and condition of the surfaces between these blocks. This method was modified over the years and it was eventually extended recently for heterogeneous rock masses *(Marinos and Hoek, 2000)* in the form of evaluating a parameter, Geological Strength Index (GSI) *(Ref. 51, Marinos, Marinos and Hoek).*

The basic input required are uniaxial compressive strength (σ_{ci}) and a material constant (m_i) that is related to the frictional properties of the rock. Ideally, these basic properties should be determined by laboratory testing as described by Hoek and Brown (1997) but, in many cases, the information is required before laboratory tests have been completed. To meet this need, tables that can be used to estimate values for these parameters are reproduced in Table 3.05.

Table 3.05: Estimation of GSI



For better quality rock masses (GSI>25), the value of GSI can be estimated directly from the 1976 version of Bieniawski's Rock Mass Rating, with the Groundwater rating set to 10 (dry) and the Adjustment for Joint Orientation set to 0 (very favourable) (Bieniawski 1976), as under:

$$GSI = 10 + \sum_{i=1}^{4} R_i$$

For very poor quality rock masses, the value of RMR is very difficult to estimate and the balance between the

ratings no longer gives a reliable basis for estimating rock mass strength. Consequently, Bieniawski's RMR classification should not be used for estimating the GSI values for poor quality rock masses.

If 1989 version of Bieniawski's RMR classification is used, then GSI = (RMR_{89} – 5) where RMR_{89} has the Groundwater rating set to 15 and the Adjustment for Joint Orientation set to zero.

GSI can also be estimated using Q system, as under:

$$GSI = 9\log + \left[\left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\right] + 44$$

GSI can be used to describe rock quality as given in Table 3.06

Table 3.06: GSI and Rock Mass Quality

GSI Value	76-95	56-75	41-55	21-40	<20
Rock Mass Quality	Very good	Good	Fair	Poor	Very poor

6. Tunnelman's Ground Classification for Soils: Anticipated ground behavior in soft ground tunnels was first defined by Terzaghi (1950) by means of the Tunnelman's Ground Classification, a classification system of the reaction of soil to tunneling operation. Heuer (1974) modified the Tunnelman's round Classification, as shown in Table 3.07 below:

Table 3.07: Tunnelman's Ground Classification

Classification	Behavior	Typical Soil Types
Firm	Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.

Classification		Behavior	Typical Soil Types
Raveling	Slow raveling	Chunks or flakes of material begin to drop out of the arch or walls sometimes after the ground has been exposed, due to loosening or over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground).	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
	Fast raveling	In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excav-ation surface and squeezing at depth behind surface.
Running	Cohesive running	Granular materials without cohesion are unstable at a slope greater than their	Clean, dry granular materials. Apparent cohesion in moist sand, or weak

Classification	Behavior	Typical Soil Types
	angle of repose (approx. 30°-35°). When exposed at steeper slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing	A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, com-pletely filling the tunnel in some cases.	Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling	Ground absorbs water, increases in volume and expands slowly into the tunnel.	Highly pre-consolidated clay with plasticity index in excess of about 30, generally conta-ining significant per-centages of mont-morillonite.

Anticipated ground behavior has been further expanded by various researchers/authors for various soil conditions (clays to silty sands, cohesive soils, silty sands, sands, gravels) above/below the water table.

7. Examples of estimating RMR, Q and GSI

7.1 Example - 1

(A) Input values

- Granite rock mass,
- Contains 3 joint sets,
- Average RQD is 88%,
- Average joint spacing is 0.24m,
- Joint surfaces are generally stepped and rough, tightly closed and un-weathered with occasional stains observed,
- The excavation surface is wet but not dripping,
- Average rock material uniaxial compressive strength is 160 MPa, and
- The tunnel is excavated 150m below the ground where no abnormal high in situ stress is expected.

(B) Calculation of RMR *(Ref. Appendix-3.1)*

Sl. No.	Parameter	Rating
(i)	Rock Material Strength = 160 MPa	12
(ii)	RQD (%) = 88%	17
(iii)	Joint Spacing (m) = 0.24 m	10
(iv)	Condition of Joints: Very Rough, un-weathered and no separation	30
(v)	Ground Water: Wet	7
	Total = RMR =	76

Rock Mass Class - II : Good

(C) Calculation of Q *(Ref. Appendix-3.2)*

Sl. No.	Parameter	Value of
(i)	RQD (%) = 88	-
(ii)	Joint Set Number – 3 sets	J_n = 9
(iii)	Joint Roughness Number – Roughly stepped (undulating)	J_r = 3

(iv)	Joint Alteration Number – Unaltered, Some Stains	$J_a = 1$
(v)	Joint Water Factor – Wet only (Dry excavation or Minor flow)	$J_w = 1$
(vi)	Stress Reduction Factor: σ_c / σ_1 = 160/(150x0.027) = 39.5	SRF = 1

Q = (88/9) * (3/1) * (1/1) = 29

Rock Mass Class : Good *(Ref. Table-3.03)*

(D) Calculation of GSI

Rock Mass Structure: Blocky

Joint Surface Condition: Very good

GSI = 75 $\pm$ 5 (Ref. Table-3.05)

Rock Mass Quality : Good *(Ref. Table-3.06)*

7.2 Example - 2

(A) Input values

- A sandstone rock mass,
- Fractured by 2 joint sets plus random fractures,
- Average RQD is 70%,
- Average joint spacing is 0.11m,
- Joint surfaces are slightly rough, highly weathered with stains but no clay found on surface,
- Joints are generally in contact with apertures generally less than 1mm,
- Average rock material uniaxial compressive strength is 85 MPa,
- Tunnel is to be excavated at 80m below ground level, and
- The groundwater table is 10m below the ground surface.

(B) Calculation of RMR *(Ref. Appendix-3.1)*

Sl. No.	Parameter	Rating
(i)	Rock Material Strength = 85 MPa	7
(ii)	RQD (%) = 70%	13
(iii)	Joint Spacing (m) = 0.11 m	8
(iv)	Condition of Joints: Slightly Rough, Highly weathered and Separation <1mm	20
(v)	Ground Water: Water Pressure/Stress = 0.32	4

Total = RMR = 49

Rock Mass Class - III : Fair

(C) Calculation of Q *(Ref. Appendix-3.2)*

Sl. No.	Parameter	Value of
(i)	RQD (%) = 70	-
(ii)	Joint Set Number – 2 sets plus random	J_n = 6
(iii)	Joint Roughness Number – Slightly rough (Rough Planar)	J_r = 1.5
(iv)	Joint Alteration Number – Highly Weathered, Only Stain (altered non-softened mineral coating)	J_a = 2
(v)	Joint Water Factor – 70m Water Head = 70 kg/m2 = 700 kPa	J_w = 0.5
(vi)	Stress Reduction Factor: σ_c / σ_1 = 85/(80x0.027) = 39.3	SRF = 1

Q = (70/6) * (1.5/2) * (0.5/1) = 4.4

Rock Mass Class : Fair *(Ref. Table-3.03)*

(D) Calculation of GSI

Rock Mass Structure: Blocky

Joint Surface Condition: Poor (Highly Weathered)

GSI = 45 $\pm$ 5 (Ref. Table-3.05)

Rock Mass Quality : Fair *(Ref. Table-3.06)*

7.3 Example - 3

(A) Input values

- A highly fractured siltstone rock mass,
- Has 2 joint sets and many random fractures
- Average RQD is 41%,
- Average joint spacing 5cm,
- Joints appears continuous,
- Joint surfaces are slickensided & undulating, and are highly weathered,
- Joints are separated by about 3-5mm,
- Filled with clay,
- Average rock material uniaxial compressive strength is 65MPa,
- Inflow per 10m tunnel length is approximately 50 litre/minute, with considerable outwash of joint fillings, and
- Tunnel is at 220m below ground.

(B) Calculation of RMR *(Ref. Appendix-3.1)*

Sl. No.	Parameter	Rating
(i)	Rock Material Strength = 65 MPa	7
(ii)	RQD (%) = 41%	8
(iii)	Joint Spacing (m) = 0.05 m	5
(iv)	Condition of Joints: Continuous, Slickensided, Separation 1-5 mm	10
(v)	Ground Water: Inflow = 50 lit/min	4

Total = RMR = 34

Rock Mass Class - IV : Poor

(C) Calculation of Q *(Ref. Appendix-3.2)*

Sl. No.	Parameter	Value of
(i)	RQD (%) = 41	-
(ii)	Joint Set Number – 2 sets plus random	J_n = 6

(iii)	Joint Roughness Number – Slightly rough (Rough Planar)	J_r = 1.5
(iv)	Joint Alteration Number – Slickensided, Undulating	J_a = 2
(v)	Joint Water Factor: Large inflow with considerable outwash	J_w = 0.33
(vi)	Stress Reduction Factor: σ_c/σ_1 = 65/(220x0.027) = 11	SRF = 1

Q = (41/6) * (1.5/4) * (0.33/1) = 0.85

Rock Mass Class : Very Poor (Quite near to Poor)

(Ref. Table-3.03)

(D) Calculation of GSI

Rock Mass Structure: Blocky

Joint Surface Condition: Very Poor

GSI = 35 ± 5 (Ref. Table-3.05)

Rock Mass Quality : Poor *(Ref. Table-3.06)*

7.4 Comparison of Rock Class/Quality, as estimated from different classifications systems, is summarized in Table 3.08; which shows that the rock class/quality obtained by using three different/independent classification systems is almost same.

Table 3.08: Summary of Rock Class / Quality

Rock Type	RMR System		Q System		GSI System	
	RMR	Quality	Q	Quality	GSI	Quality
Granite	76	Good	29	Good	75	Good
Sand stone	49	Fair	4.4	Fair	45	Fair
Silt stone	34	Poor	0.85	Very Poor	35	Poor

8. Correlation between RMR, Q and GSI: Researchers have developed many empirical correlation between these values, as given under and as shown in Fig. 3.02:

$$RMR = 9 \ln Q + (44 \pm 18)$$

$$RMR = 13.5 \log Q + 43$$

$$GSI = RMR - 5 \quad (\text{for } GSI > 25)$$

9. Squeezing behaviour of Rock Mass: Squeezing is time dependent large deformation, which occurs around an opening, and is essentially associated with creep caused by stress exceeding shear strength. The degree of squeezing is classified as under:

(i) Mild squeezing: Closure 1-3% of opening D
(ii) Moderate squeezing: Closure 3-5% of D
(iii) High squeezing: Closure > 5% of D

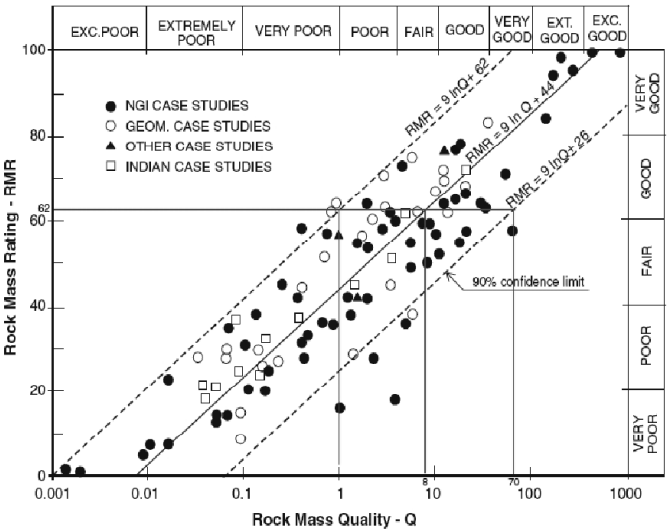


Fig. 3.02: Correlation between RMR & Q

Rate of squeezing is time and stress dependent. Usually the rate is high at initial stage, say, several cm/day, and it reduces with time. Squeezing may continue for a long period.

Squeezing may occur at shallow depths in weak and poor rock masses. Poor rock masses with moderate strength at great depth may also suffer from squeezing.

Based on the analysis of several case studies in various parts of the world, Barton et al. (1974) and Singh et al. (1992) have established that it is possible to predict the squeezing condition or otherwise, in any underground excavation, based on the value of "Q" and "Height of Overburden". The dividing line between the squeezing and non-squeezing line is given by the equation, as shown in Fig. 3.03 also:

$$H = 350\ Q^{1/3}$$

Where:

H = Overburden (in m)
Q = Rock Mass Quality

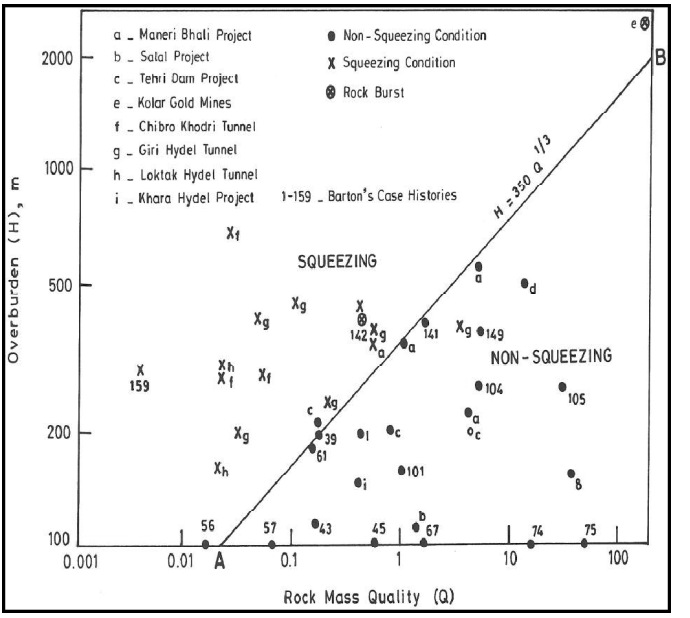


Fig. 3.03: Boundary line for Squeezing

10. Empirical relations for rock mass strengths: It is difficult to determine the strength (UCS, Strength parameters) of jointed rock masses in the laboratory as the samples need be undisturbed and sufficiently large to be representative of the discontinuity conditions.

To estimate strength of rock masses, empirical correlations developed by various researchers are usually used, some of which are as under:

10.1 Compressive Strength of Rock Mass

$$\sigma_{cm} / \sigma_c = e^{[(RMR-100)/24]}$$

$$\sigma_{cm} / \sigma_c = e^{[(RMR-100)/18]}$$

$$\sigma_{cm} = 5\gamma \ (Q \ \sigma_c / 100)^{1/3}$$

Where:

σ_{cm} = Compressive Strength of rock mass

σ_c = Compressive Strength of rock material

γ = Unit weight of rock mass (g/cc)

10.2 Tensile Strength of Rock Mass

By Singh & Goel:

$$\sigma_{tm} = 0.029 \ \gamma \ fc \ Q^{0.3}$$

Where:

$f_c = \sigma_c/100$ for Q>10 and σ_c > 100 Mpa

Otherwise $f_c = 1$

γ = Unit weight of rock mass in g/cm^3.

By Hoek & Brown:

$$\sigma_{tm} = 0.5 \ \sigma_c \ [m_b - \sqrt{m_b^2 + 4s} \ ]$$

Where: m_b & s are material constants.

Appendix – 3.1

Table 1: Classification parameters in RMR and their ratings

Parameter			Range of Values						
1	Strength of Intact Rock	Point-load strength index (MPa)	>10	4-10	2-4	1-2	-		
		UCS (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 (m)		>2	0.6-2	0.2-0.6	0.06-0.2	<0.06		
	Rating		20	15	10	8	5		
4	Conditions of discontinuities		Very rough surfaces, Not continuous, No separation, Unweathered wall rock	Slightly rough surfaces, separation <1mm, Slightly weathered walls	Slightly rough surfaces, separation <1mm, Highly weathered walls	Slicken sided surfaces or Gouge < 5 mm thick or Separation 1-5 mm continuous	Soft gouge > 5 mm thick or Separation > 5 mm continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/min)	None or	<10 or	10-25 or	25-125 or	>125 or		
		Ratio of joint water pressure to major principal stress	0 or	<0.1 or	0.1-0.2 or	0.2-0.5 or	>0.5 or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Table 2: Rating adjustment for joint orientations

Strike and dip orientations of discontinuities		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnel and mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 3: Rock mass classes and corresponding design parameters and engineering properties

Class No.	I	II	III	IV	V
RMR	100-81	80-61	60-41	40-21	<20
Description	Very good	Good	Fair	Poor	Very poor
Average stand-up time	20 years for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of rock mass (MPa)	>0.4	0.3-0.4	0.2-0.3	0.1-0.2	<0.1
Internal friction angle of rock mass (°)	>45	35-45	25-35	15-25	<15
Deformation Modulus (GPa) [a)]	>56	56-18	18-5.6	5.6-1.8	<1.8

[a)] Deformation modulus values from Serafim and Pereira (1983)

Table 4: Guidelines for classifying discontinuity condition (after Bieniawski, 1989)

Parameter		Range of values				
Discontinuity length (persistence/ continuity)	Rating	6	4	2	1	0
	Measurement (m)	<1	1-3	3-10	10-20	>20
Separation (aperture)	Rating	6	5	4	1	0
	Measurement (mm)	None	<0.1	0.1-1	1-5	>5
Roughness	Rating	6	5	3	1	0
	Description	Very rough	Rough	Slightly rough	Smooth	Slickensided

Infilling (gouge)	Rating	6	4	2	2	0
	Description and Measurement (mm)	None	Hard filling <5	Hard filling >5	Soft filling <5	Soft filling >5
Degree of weathering	Rating	6	5	3	1	0
	Description	None	Slight	Moderate	High	Decomposed

Table 5.Ratings for discontinuity orientations (after ASCE, 1996)

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Irrespective of strike
Drive with dip		Drive against dip		Dip	Dip	Dip
Dip	Dip	Dip	Dip			
45°-90°	20°-45°	45°-90°	20°-45°	45°-90°	20°-45°	0°-20°
Very favorable	Favorable	Fair	Unfavorable	Very unfavorable	Fair	Fair

Appendix – 3.2

Table for Assessment of Rock Mass Quality (Q) (after Barton et al., 1974)

1. Rock Quality Designation RQD (%)		
A. Very poor	0-25	Note:
B. Poor	25-50	(i) Where RQD is reported or measured to be <10, a nominal value of 10 is used to evaluate Q
C. Fair	50-75	
D. Good	75-90	
E. Excellent	90-100	(ii) Take RQD to be nearest 5%
2. Joint Set Number (J_n)		
A. Massive, none or few joints	0.5-1.0	Note:
B. One joint set	2	(i) For intersections use (3.0 x J_n)
C. One joint set plus random	3	(ii) For portals use (2.0 x J_n)
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube' etc.	15	
J. Crushed rock, earthlike	20	
3. Joint Roughness Number (J_r)		
(a) Rock wall contact and		Notes:
(b) Rock wall contact before 10cm shear		(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
A. Discontinuous joint	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	(ii) Jr=0.5 can be used for planar slickensided joints having lineations, provided the lineations are favorably oriented
D. Slickensided, undulating	1.5	
E. Rough and irregular, planar	1.5	
F. Smooth or irregular	1	
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	(nominal)
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1	(nominal)
4. Joint Alteration Number (J_a)		$Ø_r$ (Approx.)
(a) Rock wall contact and		
A. Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote	0.75	-
B. Unaltered joint walls, surface staining only	1	25-35
C. Unaltered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2	25-30
D. Silty or sandy clay coatings, small clay fraction (non-softening)	3	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	8-16
(b) Rock wall contact before 10cm shear		
F. Sandy particles, clay free disintegrated rock, etc.	4	25-30
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5 mm in thickness)	6	16-24
H. Medium or low over-consolidation, softening, clay mineral fillings (continuous, < 5 mm in thickness)	8	12-16
J. Swelling clay fillings, i.e. montmorillonite	8-12	6-12

(continuous, < 5 mm in thickness). Value of Ja depends on percentage of swelling clay-sized particles and access to water, etc.		
(c) No rock wall contact when sheared		
K, L, M. Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6, 8 or 8-12	6-24
N. Zones or bands of silty - or sandy clay, small clay fraction (non-softening)	5	
O, P, R Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10, 13 or 13-20	6-24
5. Joint Water Reduction Factor (J_w)		Approximate water pressure (kPa)
A. Dry excavations or minor inflow, i.e. <5 1/min locally	1	<100
B. Medium inflow or pressure occasional outwash of joint fillings	0.66	100-250
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	250-1000
D. Large inflow or high pressure, considerable occasional outwash of joint fillings	0.33	250-1000
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.1	>1000
F. Exceptionally high inflow or water pressure continuing without decay	0.1-0.05	>1000
Note: (i) Factors C-F are crude estimates. Increase J_w if drainage measures are installed (ii) Special problems caused by ice formation are not considered		

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 containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)		10		Note: (i) 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 (depth of excavation < 50 m)		5		
C. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)		2.5		
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5		
E. Single shear zones in competent rock (clay free, depth of excavation < 50 m)		5		
F. Single shear zones in competent rock (clay free, depth of excavation > 50 m)		2.5		
G. Loose open joints, heavily jointed, or "sugar cube" etc. (any depth)		5		
(b) Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		
H. Low stress, near surface open joints	>200	>13	2.5	(ii) If stress field is strongly anisotropic: when $5<\sigma_1/\sigma_3<10$, reduce σ_c and σ_t to 0.8 σ_c and 0.8 σ_t; when $\sigma_1/\sigma_3>10$, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t. Where σ_c = unconfined compressive
J. Medium stress, favourable stress condition	200-10	13-0.66	1	
K. High stress, very tight structure (usually favorable to stability, maybe unfavorable to wall stability)	10-5	0.66-0.33	0.5-2.0	

L. Moderate slabbing after >1h in massive rock	5-3	0.5-0.65	5-50	strength, σ_t = tensile strength, σ_1 and σ_3 = major and minor principal stresses.
M. Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1.0	50-200	
N. Heavy rock burst (strain-burst) and immediate deformations in massive rock	<2	>1	200-400	
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressure				(iii)Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)
N. Mild squeezing rock pressure	5-10			
O. Heavy squeezing rock pressure	10-20			
(d) Swelling rock; chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure	5-10			
R. Heavy swelling rock pressure	10-15			

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CHAPTER-4

GEOTECHNICAL INVESTIGATIONS

Successful planning, design, construction and maintenance of a tunnel requires various geotechnical investigations to obtain a broad spectrum of pertinent topographic, geologic, geo-hydrological and structural information and data.

Exploration programs for tunneling must be planned by construction engineers in close cooperation with engineering geologists, geotechnical engineers & designers.

1. Phases of Geotechnical Investigation Program: The geotechnical investigation is generally carried out in following phases to obtain the information necessary at each stage of the project:

1.1 Preliminary Geotechnical investigations for Feasibility Studies: In this phase, the emphasis is on defining the regional geology and the basic issues of design and construction. Investigations at this stage are largely confined to:

(A) Collection, organization & study of available data: Available information should be collected and reviewed to identify existing conditions and features that may impact the design and construction of the proposed tunnel, and can guide in planning the scope and details of the subsurface investigation program to address these issues. Some possible sources of available information are:

- Topographical maps of Survey of India
- Geological maps and reports of GSI
- Geological maps and reports from agencies other than GSI
- Geotechnical investigation reports from other agencies, working in that area

- Case histories of other underground works in the region
- Details of land ownership (Government, Private, Forest), access routes, hydrological information and environmental sensitivity from State Govt./ Local bodies
- Satellite images and aerial photographs
- Seismic Records
- Records of landslides (caused by earthquakes or other reasons), documented by the other agencies: can be useful to avoid locating tunnel portals and shafts at these potentially unstable areas.

(B) Preliminary Survey: The information studies should be followed by a preliminary survey. Initial on-site studies should start with a careful reconnaissance over the tunnel alignment, paying particular attention to the potential portal and shaft locations. Features identified on maps and aerial photographs should be verified. Rock outcrops, often exposed in highway and railway cuts, provide a source of information about rock mass fracturing and bedding and the location of rock type boundaries, faults, and other geologic features. Features identified during the site reconnaissance should be photographed and documented.

The reconnaissance should cover the immediate project vicinity, as well as a larger regional area so that regional geologic, hydrologic and seismic influences can be accounted for. A preliminary horizontal and vertical control survey may be required to obtain general site data for route selection and for design. This survey should be expanded from existing records, alignment posts and benchmarks that are based on the same horizontal and vertical datum that will be used for final design of the structures. Additional alignment posts and benchmarks can be established, as needed, to support field investigations and mapping.

(C) Conducting investigations to compare alternative alignments and for arriving at a conceptual preliminary design: Carrying out following investigations at this stage would help in comparing alternative alignments and for arriving at a conceptual preliminary design:

(i) Preliminary Geological field mapping with particular attention to features that could signify difficulties like slides (particularly in portal areas), major faults, thrusts etc. The mapping should identify major components of the stratigraphy and the geologic structure, which form the framework for zonation of the alignment and for the planning of the explorations.

(ii) Selected exploratory borings in critical locations

(iii) Geophysical explorations: Geophysical methods of exploration are often useful at the earlier stages of a project because they are relatively inexpensive and can cover relatively large volumes of geologic material in a short time. The most commonly used techniques are Seismic Refraction Survey and Electric Resistivity Survey.

(a) <u>Seismic Refraction Survey</u>

In seismic refraction survey, pulses of low frequency seismic energy are emitted by a seismic source such as a hammer-plate or weight-drop (Fig. 4.01). The type of source is dependent on local ground conditions and required depth of penetration. Explosives are best for deeper applications but are constrained by environmental regulations.

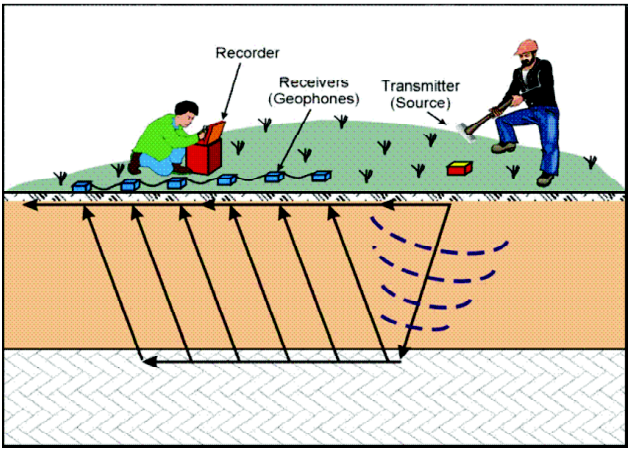

Fig. 4.01: Seismic Refraction Survey

The seismic waves propagate downward through the ground until they are reflected or refracted off subsurface layers. Refracted waves are detected by arrays of geophones spaced at regular intervals of 1-10m, depending on the desired depth of penetration.

Geophones output data is time traces which are compiled and processed by the seismograph. Interpretation techniques are applied to the first arrival times to calculate the seismic velocities of the layers and the depths of individual refracting interfaces. The interfaces are correlated with real physical boundaries in the ground, such as the soil-bedrock interface and other lithological boundaries, to produce a model of the subsurface ground structure. The final interpretation is presented in a format that is easily understood by engineers (Fig. 4.02).

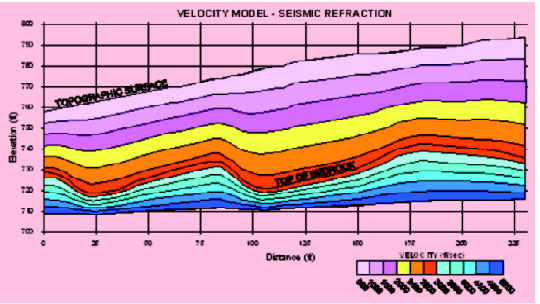

Fig. 4.02: Seismic Refraction Survey Output

(b) <u>Electrical Resistivity Survey:</u> Resistivity geophysical surveys measure variations in the electrical resistivity of the ground, by applying small electric currents across arrays of ground electrodes. The survey data is processed to produce graphic depth sections of the thickness and resistivity of subsurface electrical layers (Fig. 4.03). The resistivity sections are correlated with ground interfaces such as soil and fill layers or soil-bedrock interfaces, to provide engineers with detailed information on subsurface ground conditions.

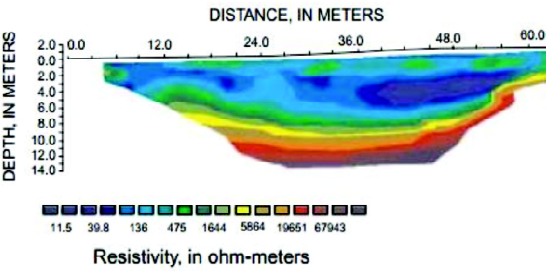

Fig. 4.03: Electric Resistivity Survey Results

(iv) Aerial Photography: To supplement existing data.

(v) Hydrological Survey: To define the ground-water regime, aquifers, sources of water etc. As a part of the hydro geological survey, all existing water wells in the area should be located, their history & condition assessed and groundwater levels taken. Mapping of permanent or ephemeral streams and other water bodies and the flows and levels in these bodies at various times of the year is usually required.

Additional hydro-geological work to be carried out at a later stage includes measurements of groundwater levels or pressures in boreholes, permeability testing using packers in boreholes and sometimes pumping tests.

1.2 Preconstruction Planning and Engineering Phase: More detailed geotechnical investigation should be carried out in this phase to refine the tunnel alignment and profile once the general corridor is selected, and to provide the detailed information needed for design of tunnel & selection of appropriate tunneling methodology.

As the final design progresses, additional geo-technical investigations might be required for fuller coverage of the final alignment and for selected shaft and portal locations. Investigations carried out during this phase are:

(A) <u>Topographical Surveys:</u> Detailed topographic maps, plans and profiles should be developed to establish primary control for final design and construction based on a high order horizontal and vertical control field survey.

Accurate topographic mapping is also required to support surface geology mapping and the layout of exploratory borings. The principal survey techniques include:

- Conventional Survey

- Global Positioning System (GPS)
- Electronic Distance Measuring (EDM) with Total Station
- Remote Sensing
- Laser Scanning

(B) <u>Subsurface Investigations:</u> Subsurface investigation is the most important type of investigations to obtain ground conditions, as it is the principal means for:

- Defining the subsurface profile (i.e. stratigraphy, structure, and principal soil/rock types)
- Determining soil/rock material properties and mass characteristics
- Identifying geological anomalies, fault zones and other hazards (squeezing soils, etc.)
- Defining hydro geological conditions (groundwater levels, aquifers, hydrostatic pressures, etc.)
- Identifying potential construction risks

Subsurface investigations typically consist of:

(i) Borings to identify the subsurface str-atigraphy, and to obtain disturbed and undisturbed samples for visual classification and laboratory testing.

The number, location, depth, sample types and sampling intervals for each test boring must be selected to match specific project requirements, topographic setting and anticipated geological conditions. When deciding boring locations, knowledge of the geology of the area is necessary to determine fault locations.

The layout of borings should take into consideration various factors like the structure of the material, stratigraphy, strike and dip of the rock, consistency of strata etc. For example, in steeply dipping rock, vertical borings from the surface may be of little use. In such cases, it is important to angle the holes to cross the strata.

Table 4.01 presents general guidelines from AASHTO (1988) for determining the spacing of boreholes for tunnel projects:

Table 4.01: Guidelines for Borehole Spacing

Ground Conditions	Typical Borehole Spacing (feet)
Cut-Cover Tunnels	100 to 300
Rock Tunneling	
Adverse Conditions	50 to 200
Favourable Conditions	500 to 1000
Soft Ground Tunneling	
Adverse Conditions	50 to 100
Favourable Conditions	300 to 500
Mixed Face Tunneling	
Adverse Conditions	25 to 50
Favourable Conditions	50 to 75

The above guideline can be used as a starting point for determining the number and locations of borings. However, for a long tunnel through a mountainous area, it may not be economically feasible or the time sufficient to perform borings accordingly. Therefore, engineering judgment need to be applied by experienced geotechnical professionals to adapt the investigation program.

For a tunnel having length of 1 km or more, borings may be initially spaced at about 100–300m (depending on anticipated geological condition) along the tunnel alignment. If the borings or surface features indicate differences in the material, then additional borings should be done between the borings. This should be done until the alignment between the borings can be characterized with a high degree of certainty. In addition to the alignment, portal locations and shaft locations should be thoroughly investigated.

In general, borings should extend to at least 1.5 times tunnel diameter below the proposed tunnel invert. However, if there is uncertainty regarding the final profile of the tunnel, the borings should extend at least two or three times the tunnel diameter below the preliminary tunnel invert level.

In some cases, especially for a long high cover tunnel through a mountainous area, it may not be economically feasible or the time sufficient to perform borings in accordance with above provisions. In such cases recourse may have to be taken for few borings in combination with advance probe holes or pilot tunnel. Notwithstanding above provisions, engineering judgment need to be applied in complex geological and/or topographical conditions in consultation with experienced geotechnical professionals to plan and adapt suitable investigation program.

Horizontal boreholes along tunnel alignments provide a continuous record of ground conditions and information which is directly relevant to the tunnel alignment. Although the horizontal drilling and coring cost per m may be much higher than the conventional vertical/inclined borings, a horizontal borings can be more economical, especially for investigating a deep mountainous alignment, since one horizontal boring can replace many deep vertical conventional boreholes.

A deep horizontal boring will need some distance of inclined drilling through the overburden and upper materials to reach to the depth of the tunnel alignment. Typically the inclined section is stabilized using drilling fluid and casing and no samples are obtained.

All borings should be properly sealed at the completion of the field exploration, if not

intended to be used as monitoring wells.

(ii) In situ testing to obtain useful engineering and index properties by testing the material in-place to avoid the disturbance caused by sampling, transportation and handling of samples. In-situ tests can also aid in defining stratigraphy.

The parameters to be tested would depend on the nature of underground strata viz. Rock or Soil, and they should be finalized in advance in consultation with design engineers. Appropriate test methods should then be used to obtain those parameters with an acceptable degree of validity & reliability.

(iii) Geophysical investigations to quickly and economically obtain subsurface information over a large area to help define stratigraphy and to identify appropriate locations for performing borings.

Geophysical method to be used should be finalized in advance in consultation with design engineer. Typical techniques of geophysical tests for various geological conditions are tabulated below:

Table 4.02: Geophysical Investigations

Geological Conditions to be Investigated	Useful Geophysical Techniques	
	SURFACE	SUBSURFACE
Depth and thickness of layers	Seismic Refraction	Seismic Wave Propagation
Depth to Bedrock	Seismic Refraction, Electrical Resistivity, Ground Penetrating Radar	Seismic Wave Propagation
Depth to Groundwater Table	Seismic Refraction, Electrical Resistivity, Ground Penetrating Radar	
Location of Highly Fractured Rock and/or Fault Zone	Electrical Resistivity	Borehole TV Camera
Solution Cavities	Electrical Resistivity Ground Penetrating Radar Gravity	Borehole TV Camera

The data from geophysical exploration must always be correlated with information from direct methods of exploration that allow visual examination of the subsurface materials, direct measurement of groundwater levels and testing of physical samples of soil and rock.

Direct methods of exploration provide valuable information that can assist not only in the interpretation of the geophysical data, but also for extrapolating the inferred ground conditions to areas not investigated by borings. Conversely, the geophysical data can help determine appropriate locations for borings and test pits to further investigate any anomalies that are found.

(iv) Laboratory testing provides a wide variety of engineering properties and index properties from representative soil samples and rock core retrieved from the borings.

Soil Testing: Testing is done on selected representative samples (disturbed and undisturbed) in accordance with relevant standards. Table 4.03 shows common soil laboratory testing for tunnel design purposes.

Table 4.03: Common Lab Tests for Soil

Parameter	Test Method
Soil Identification/ Classification	Particle size distribution, Atterberg Limits (Liquid Limit & Plastic Limit), Moisture Content, Unit Weights and Coefficient of Permeability.
Mechanical Properties	Unconfined compressive strength, Tri-axial compressive test for determination of Friction angle Φ and Cohesion C, Consolidation Test for determination of Compressibility (m_v and c_v).
Mechanical properties determined by field testing	Shear strength (Vane test), Standard Penetration Test, and Deformability (Plate bearing and Dilatometer).

Parameters to be tested should be finalized in advance in consultation with designers. However, as a general guideline tests for Particle

size distribution, Atterberg limits, moisture content, unit weight, unconfined compressive strength and Tri-axial compressive strength must be carried out on the selected representative samples. In case undisturbed samples are difficult to obtain, strength properties can alternatively be obtained from empirical correlations using N-values from standard penetration tests. Consolidation test should be carried out in case of clayey soils

Rock Testing: Table 4.04 summarizes common laboratory testing for rocks, for tunnel design purposes.

Table 4.04: Common Lab Tests for Rock

Parameter	Test Method
Index Properties	Density, Porosity, Moisture Content, Slake Durability, Swelling Index, Point Load Index, Hardness and Abrasivity.
Strength	Uniaxial compressive strength, Tri-axial compressive strength, Tensile strength (Brazilian) and Shear strength of joints.
Deformability	Young's modulus and Poisson's ratio.
Time dependence	Creep characteristics
Permeability	Coefficient of permeability
Mineralogy and grain sizes	Thin-sections analysis, Differential thermal analysis and X-ray diffraction.

Parameters to be tested should be finalized in advance in consultation with designers. However, as a general guideline tests for Density, Porosity, Moisture Content, Point Load Index, Uniaxial compressive strength, Tri-axial compressive strength, Tensile strength (Brazilian), Young's modulus, Poisson's ratio & Coefficient of permeability must be carried out on the selected representative samples. When the rock contains clay minerals, then swelling index should be determined. Hardness & Abrasivity tests would be additionally required in case mechanized tunneling is planned to be adopted.

It is desirable to preserve the rock cores retrieved from the field properly until the construction is completed and disputes/claims are settled. Photographs of rock cores (in core boxes) should be kept for review by designers.

In addition to typical geotechnical, geological, and geo-hydrological data, subsurface investigation for a tunnel project must consider the unique needs for different tunneling methods, i.e. cut-and-cover, drill and blast, NATM. Table 4.05 shows special considerations for various tunneling methods.

Table 4.05

Construction Method	Special Requirements
Drill and blast	Data needed to predict stand–up time for the size and orientation of tunnel.
NATM	Generally requires comprehensive geotechnical data and analysis to predict behavior and to classify the ground conditions and ground support systems into various categories based on the behavior.
Road header	Data on jointing & hardness of rock.
Tunnel Boring Machine	While data required would depend on kind of machine being deployed, the following information is useful: Data required to determine cutter costs and penetration rate Data to predict stand-up time to determine if open-type machine will be ok or full shield is necessary. Data for assessing face stability Data for full characterization of all potential mixed-face conditions. Reliable estimate of groundwater pressures and strength & permeability of soil to be tunneled.
Portal Construction	Need reliable data to determine most cost-effective location of portal and to design temporary and final portal structure.
Construction Shafts	Should be at least one boring at every proposed shaft location.

Various geological conditions demand special considerations for subsurface investigations as summarized in the Table 4.06.

Table 4.06

Geological condition	Special Requirements
Hard or Abrasive Rock	Difficult and expensive for TBM or road header. Investigate, obtain samples and conduct lab tests to provide parameters needed to predict rate of advance and cutter costs.
Mixed Face	Should be characterized carefully to determine nature and behavior of mixed-face and approximately length of tunnel likely to be affected for each mixed-face condition.
Karst	Potentially large cavities along joints.
High stress	Could strongly affect standup time and deformation patterns both in soil and rock tunnels. Should evaluate for rock burst for popping rock in particularly in deep tunnels.
Adverse Geological Features	1. Faults: ● Known or suspected active faults. Investigate to determine location and estimate likely ground motion. ● Inactive faults but still sources of difficult tunneling conditions. ● Fault gouge sometimes a problem for strength and modulus. 2. Groundwater: Groundwater Is one of most difficult and costly problems to control. Must investigate to predict groundwater as reliably as possible. 3. Thrust Zones & Shear Zones. 4. Adverse Bedding & jointing.

(C) Detailed Geological Mapping: Detailed geologic mapping should be carried out which includes mapping & plotting of joints, faults, and bedding planes. The geologist must then project the geological conditions to the elevation of the proposed tunnel so that tunneling conditions can be assessed. Geologic mapping should characterize and document the condition of rock mass such as:

- Discontinuity type
- Discontinuity orientation
- Discontinuity infilling
- Discontinuity spacing

- Discontinuity persistence
- Weathering

In addition, following surface features should also be observed and documented during the geologic mapping program:

- Slides, new or old, particularly in proposed portal and shaft areas
- Faults
- Rock weathering
- Sinkholes and karstic terrain
- Groundwater springs

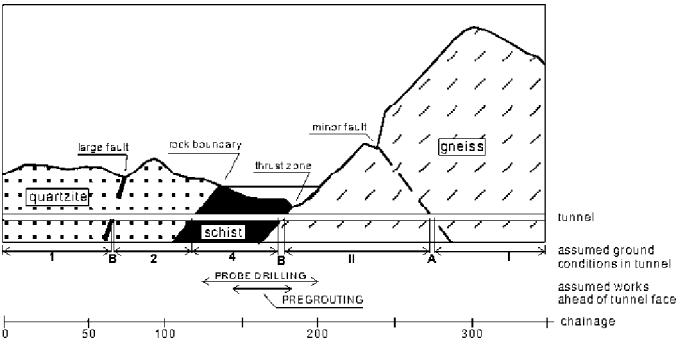

Fig. 4.04: Tunnel alignment zones

Based on detailed geological mapping, it should be possible to divide the tunnel alignment into zones of consistent rock mass condition (Fig. 4.04). Criteria for zonation would be site specific, but factors involving intact rock, rock mass and excavation system characteristics should be considered. Each zone should be characterized in terms of average expected condition as well as extreme conditions likely to be encountered.

(D) Groundwater investigation: Since groundwater is a critical factor for tunnels, special attention must be given to defining the groundwater

regime, aquifers, and sources of water, any perched or artesian conditions, depth to groundwater, and the permeability of the various materials that may be encountered during tunneling.

Related considerations include the potential impact of groundwater lowering on settlement of overlying and nearby structures, utilities and other facilities; other influences of dewatering on existing structures; pumping volumes during construction; the potential impact on water supply aquifers; and seepage into the completed tunnel etc.

Groundwater investigations typically include most or all of the following elements:

- Observation of groundwater levels in boreholes
- Assessment of soil moisture changes in the boreholes
- Installation of groundwater observation wells and piezometers
- Borehole permeability tests
- Pumping tests

During subsurface investigation i.e. drilling and coring, it is particularly important to note and document any groundwater related observations made during drilling or during interruptions to the work when the borehole has been left undisturbed. Even seemingly minor observations may have an important influence on tunnel design and ground behavior during construction.

Observation wells and piezometers should be monitored periodically over a prolonged period of time to provide information on seasonal variations in groundwater levels. Monitoring during construction provides important information on the influence of tunneling on groundwater levels, forming an essential component of construction control and any protection program for existing structures and facilities.

(E) Structure & Utility pre-construction survey: Structures located within the zone of potential influence may experience disturbance due to soil movement caused by tunnel excavation and construction in close proximity (e.g. cut-and-cover excavation, shallow soft ground tunneling, etc.). If the anticipated movement can induce potential damage to a structure, some protection measures will be required, and a detailed preconstruction survey of the structure should be performed. Preconstruction survey should ascertain all pertinent facts of pre-existing conditions, and identify features and locations for further monitoring.

The requirement for utility survey varies with tunneling methods and site conditions. Cut-and-cover tunnel and shallow soft ground tunnel constructions, particularly in urban areas, extensively impacts overlying and adjacent utilities. Water, sewerage, storm water, electrical, telephone, fiber optic and other utility mains and distribution systems may require excavation, re-routing, strengthening, reconstruction and/or temporary support, and may also require monitoring during construction.

1.3 Geo-technical Investigations during Construction phase: It sometimes becomes necessary to perform additional subsurface investigations and ground characterization during construction. Such construction phase investigations serve a number of important functions like:

- Verify initial ground support selection and for design/re-design.
- Documenting existing ground conditions for reference, in case of contractual claims.
- Assessing ground and groundwater conditions ahead of the advancing face, to reduce risks and improve the efficiency of tunneling operations. This enables forewarning of

adverse tunneling conditions like potential high water inflow, very poor ground etc.

- Verification of conditions assumed for final tunnel lining design, including choice of unlined tunnel.
- Mapping for the record, to aid in future operations, inspections, and maintenance work.

A typical construction phase investigation program would include some or all of the following elements:

- Subsurface investigation (borings and geophysical) from the ground surface.
- Additional groundwater observation wells and/ or piezometers.
- Additional laboratory testing of soil and rock samples.
- Geologic mapping of the exposed tunnel face: with due safety precautions.
- Geotechnical instrumentation.
- Probing in advance of the tunnel heading from the face of the tunnel: It typically consists of drilling horizontally from the tunnel heading by percussion drilling or rotary drilling methods.
- Pilot Tunnels are small size tunnels (typically at least 2mx2m in size) that are occasionally used for large size tunnels in complex geological conditions.

Pilot tunnel may also be located adjacent to the proposed tunnel, using the pilot tunnel for emergency exit, tunnel drainage, tunnel ventilation, or other purposes for the completed project.

Tunnel Seismic Prediction: Himalayas is the one of the youngest geologies on earth having rock mass with fault zones, sheared zones, fractured rock mass and being permeable and water bearing. Even with the best of geotechnical exploration, all these features cannot be captured in advance. Therefore, geological prediction during construction becomes inevitable in such cases. Tunnel Seismic Prediction (TSP) is one of such modern techniques, which has been used in many tunnels world over including tunnels in Himalayas region. TSP can also explore water bearing formations in 3D image. It takes less time as compared to probe drilling method. Once the geological risk is identified and mapped properly, risk management becomes easy, cheaper and predictable to large extent.

This is adopted as predictive method during excavation process, for both drill & blast and TBM techniques, and no access face is required to perform the measurements. The TSP system is an underground seismic reflection package comprising measurement instrumentation and its own interpretation software. By employing the principle of echo sounding, it serves to predict changes in rock physical properties ahead of and around spatially very restricted underground excavations such as tunnel tubes. TSP-3D is one such patented technique developed by Amberg Technologies AG of Switzerland. In this method, acoustic signals are produced by a series of 24 shots of usually 50 to 100 grams of detonation cords aligned along one tunnel wall side and having additional shot line along opposite tunnel wall side in case of more complex geology (Fig. 4.05). The 3 component receiver picks up the seismic signals which were being reflected from any kind of discontinuity in rock mass ahead (Fig. 4.06). The capability of system to record full wave field of compressional and shear wave in conjunction with analysis

software enables determination of rock mechanical properties such as Poisson's Ratio and Young's Modulus within the prediction area. The final 2D and 3D result produced by the system software presents boundary planes crossing the tunnel axis coordinates ahead (Fig. 4.07).

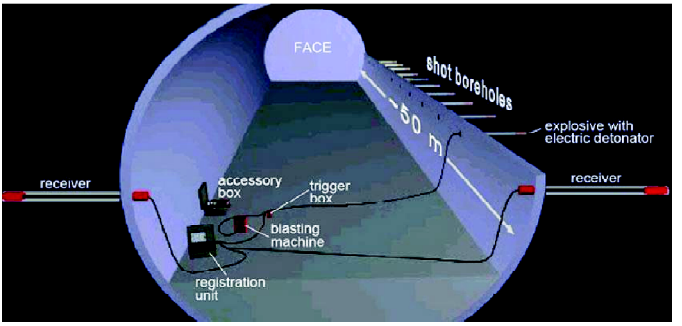

Fig. 4.05: TSP Survey

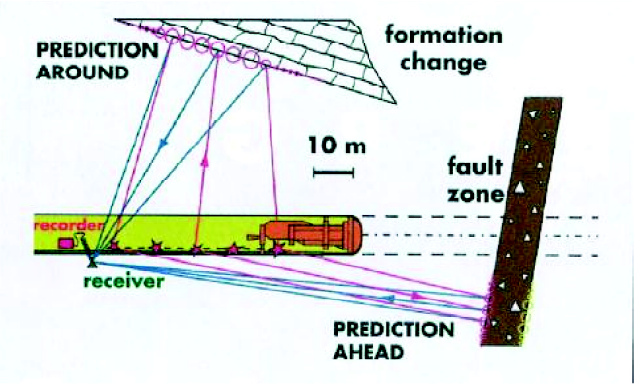

Fig. 4.06: Principle of TSP

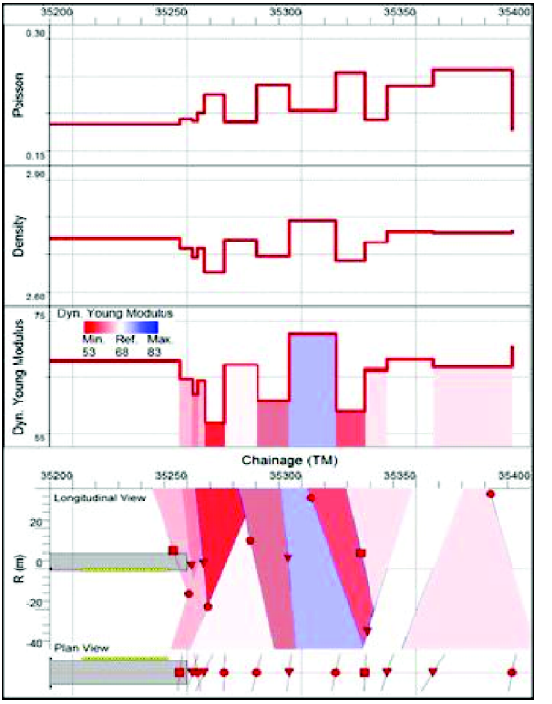

Fig. 4.07: TSP Output

2. Assessing exploration requirements of a Tunneling Project:

(1) Because of the complexities of geology and the variety of functional demands, no two tunnels are alike. It is therefore difficult to give hard and fast rules about the required intensity of explorations or the most appropriate types of exploration. Nonetheless, following can help in the planning of explorations:

(a) Plan explorations to define the best, worst, and average conditions for the construction of the underground works; locate and define conditions that can pose hazards or

great difficulty during construction.

(b) Use qualified geologists to produce the most accurate geologic interpretation so as to form a geological model that can be used as a framework to organize data and to extrapolate conditions to the locations of the underground structures.

(c) Determine and use the most cost-effective methods to discover the information sought.

(d) Anticipate methods of construction and obtaining data required to select construction methods and estimate costs.

(e) Anticipate potential failure modes for the completed structures and required types of analysis, and obtain the necessary data to analyze them (e.g., in situ stress, strength, and modulus data for numerical modeling).

(f) Drill at least one boring at each shaft location and at each portal.

(g) Special problems may require additional explorations.

(2) Frequently, even the most thorough explorations will not provide sufficient information to anticipate all relevant design and construction conditions. Here, the variation from point to point may be impossible to discover with any reasonable exploration efforts. In such instances, the design strategy should deal with the average or most commonly occurring condition in a cost-effective manner and provide means and methods to overcome the worst anticipated condition, regardless of where it is encountered.

(3) The specific scope and extent of the investigation must be appropriate for the size of the project and the complexity of the existing geologic conditions; must consider budgetary

constraints; and must be consistent with the level of risk considered acceptable.

(4) Since unanticipated ground conditions are most often the reason for costly delays, claims and disputes during tunnel construction, a project with a more thorough subsurface investigation program would likely have fewer problems and lower final cost.

3. Geotechnical investigation Program for tunnels should involve / include:

(a) Active consultation with experienced geotechnical engineers, geologists & designers.

(b) “What”, “Why”, “Where”, “How” & “How much” for each Geotechnical parameter to be tested/investigated.

(c) Phasing the investigations (Para-1 above).

(d) Keeping the investigation program and contract flexible enough so as to enable taking up of additional investigations as per unexpected requirements that emerge during course of work.

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CHAPTER-5

TUNNEL SURVEYING

Surveying plays an important role in construction of tunnels, right from the planning stage to the final completion. Surveying techniques are applied in tunnels for:

(i) Proper initial planning.

(ii) Integration of geo-technical and geographical data with topographical mapping (and utility mapping, if located in urban area).

(iii) Actual alignment and guidance of tunnel, adit and shaft construction.

1. Type of Surveys: Following surveys are required in any typical tunnel project:

1.1 Preliminary Surveys: Topographic maps, maintained by Survey of India (some of them in digital form), are generally sufficient for initial planning. However, in most cases, supplementary data are required, either due to inaccuracies in the data available or due to changes in land use or topography. Therefore, a horizontal and vertical control survey is required to obtain general site data for route selection and for design. This survey should be expanded from existing records and monuments, that are based on the same horizontal and vertical datum, that will be used for final design of the structures. Additional temporary monuments and benchmarks are placed as needed to support field investigations, mapping, environmental studies and route selection.

Typically, reasonably detailed mapping in corridors 100 to 1000m wide are required along all contemplated alignments. This mapping should be

sufficiently detailed to show natural and man-made constraints to the project. In urban areas, mapping of major utilities, that may affect the project, must also be done.

When the project corridor has been defined, new aerial photographs should be obtained and photogrammetric maps should be prepared to facilitate/obtain data on portal design, access, drainage, depth of cover, geology, seismic history etc.

1.1.1 Equipment and Techniques: Modern mapping equipment and techniques provide a wide range of products and services to support planning and design, and ongoing construction management, including:

(a) Digital Ortho Mapping, wherein the aerial photographic image is digitized in true plan position and scale (Example: Fig. 5.01), and can be inserted into the project Geographic Information System (GIS) or database.

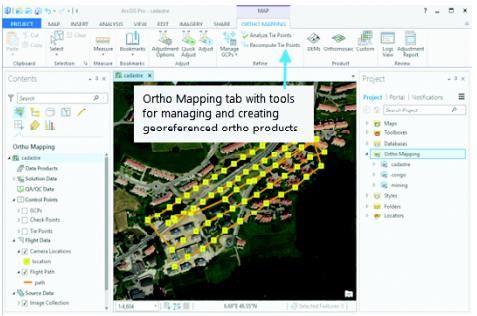

Fig. 5.01: Digital Ortho Mapping

(b) Digital topographic mapping, wherein contours and planimetric features are directly digitized during the map compilation process (Example: Fig. 5.02) and can be CAD-plotted and/ or inserted into the project GIS.

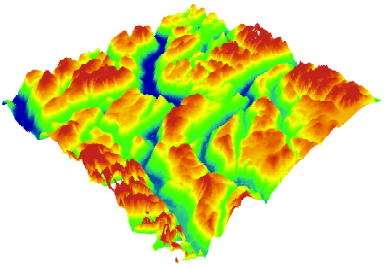

Fig. 5.02: Digital Topographic Mapping

(c) Software enabling manipulation of digital map and survey data to extract profiles, cross sections, spot elevations, etc., and to superimpose this data selectively with design, right of way, geologic, and other data sets that have been digitized into the GIS/database.

1.2 Utility Surveys: Utility surveys are mainly required for urban tunnels for information on preliminary and final route selection and to determine the type and extent of utility protection, relocation, reconstruction or monitoring needed. Deep tunnel construction may not pass through any utility systems, but vibration, blasting shock, and settlement may affect surface and underground utilities in the project corridor. Cut-and-cover construction, particularly in urban areas, extensively affects overlying and adjacent utilities. The information from utility surveys supplements existing utility maps and records.

Utility surveys, like all other surveys on the project, must be based on the primary horizontal and vertical control network, and must be sufficiently accurate to ensure that all utility features are located within required tolerances.

1.2.1 Equipment and Techniques: Instruments and systems available for locating utilities include:

(a) Photogrammetric mapping: Routinely used to document the location of pre-painted surface features such as manholes, valves, inlets, hydrants, etc. This is normally done during the photogrammetric mapping phase of the survey work.

(b) Magnetic surveys: Ferrous bodies such as iron and steel pipes, barrels, piles, etc., induce anomalies in the earth's magnetic field. Magnetometers detect the anomalies, whose amplitude is a function of the ferrous mass and the distance from the surface.

(c) Electromagnetic toning: A low-frequency AC current is conducted into linear metal features such as pipelines, cables, cable jackets etc., by connecting an AC tone generator to an exposed section of the feature. A handheld receiver detects the feature by electromagnetic signals whose magnitudes are a function of the strength of induced AC current, distance between tone generator and mobile receiver, depth of cover over the feature, electrical conductivity of the feature, and electrical insulation between the feature and its burial medium (earth, water). Operating AC electrical cables may also be detected by electromagnetic toning.

(d) Ground Penetrating Radar (GPR): A portable instrument that emits radar frequency signals vertically downward and plots energy pulses reflected by buried objects.

1.3 Primary Survey Network: Primary surveys are the basic positional reference for the project. These surveys must be founded on stable and accessible monuments, and they must be conducted to a high degree of accuracy to meet project needs. The survey work, computations, adjustment, and data recording must be accurate and reliable so that design and construction can

proceed with absolute confidence in the credibility of the survey data.

1.3.1 Survey Control: Primary horizontal surveys are conducted using Triangulation, Electronic Distance Meter (EDM) traverse, Global Positioning System (GPS) surveys or a combination of these methods. The GPS is helpful in providing precise references at low cost over long distances. When used in differential mode in establishing control networks, GPS gives relative positioning accuracies as good as two ppm. GPS is also flexible, because line-of-sight is not required between points.

After completion of route selection, a horizontal and vertical survey of high accuracy is conducted, with permanent monuments installed near portals, adits, and other selected locations in the project corridor. Design and execution of the survey must be done with the objective of establishing a singular and authoritative survey system that is based on securely founded monuments and meets the accuracy standards required for the project. All subsequent surveys and construction work must be based solely on the control survey network, and the project plans and specifications should contain specific statements affirming this.

(a) Electronic Distance Measuring: Modern EDM instruments (TotalStation) combine accurate measurement of angles and distances, computer processing of data and storage of observed angle & distance data. Range of distance measurement depends upon type of EDM used, number of reflective prisms and clarity of the air. Typical range is 2000-3000m, with some specialized instruments ranging in excess of 7000m. Standard deviation of angles and distance measurements vary with the various models and makes of EDMs available. EDMs with data collectors can download survey data for processing and plotting using specialized "field-to-finish" software.

Theodolites, TotalStation and EDMs cannot be adjusted or calibrated in the field. This work must be done in a competent service facility as recommended by the OEM. Level instruments, however, require regular testing to assure that the horizontal crosshair defines a true level plane.

(b) Global Positioning System (GPS): Coordinate positioning of widely spaced control monuments is usually accomplished by GPS surveys, which utilize the signal transit time from ground station to satellites to determine the relative position of monuments in a control network. The accuracy of measurement is dependent upon the number of satellites observed, configuration of the satellite group observed, elapsed time of observation, quality of transmission, type of GPS receiver, and other factors including network design and techniques used to process data. GPS surveying requires the simultaneous operation of several receiving instruments located at different stations throughout the survey network, and the success of an observing session depends upon each instrument being in place and operating at a predetermined time. This requires detailed advance planning.

Although GPS surveying is now increasingly becoming common, high-order GPS surveys entail extremely sophisticated procedures for both field and office work. Accordingly, the work should be planned and executed under the direction of a qualified GPS specialist with strong credentials in the application of advanced geodesy to design and construction.

2. Surveying steps in alignment control of Tunnels: Setting out centre line of tunnel at exact location and elevation is done in following steps:

(i) Establishment of temporary benchmarks and alignment posts, as required for work, and

ensuring their stability. In case stability is in danger or they have been disturbed, prompt measures shall be taken to transfer & re-establish these temporary benchmarks and alignment posts.

(ii) Surface Survey (setting out tunnel on ground surface).

(iii) Transferring the alignment underground (transfer of centre line from surface to underground).

(iv) Underground setting out, taking care to eliminate cumulative error.

(v) Transferring levels underground (underground leveling), taking care to eliminate cumulative error.

Modern Tunnel Boring Machines are normally equipped with semi-automated or fully automated guidance instrumentation that offers good advance rates with great precision.

3. Recommendations for framing contract documents: Except in rare instances, the contractor should be entrusted with responsibilities for all surveying, including control of line and grade and layout of all facilities and structures. Railway officials must conduct verification surveys at regular intervals and also ensure that the work is properly tied to adjacent existing or new construction. Contract documents should be framed to clearly stipulate the responsibilities of Railways & Contractor. Also, the contract documents must contain all reference material necessary to conduct surveying control during construction including specifications stating the accuracy requirements and the required quality control, quality assurance and surveyor qualification requirements. Minimum requirements to the types and general stability of construction benchmarks and alignment posts should also be stated.

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CHAPTER-6

CHOICE OF TUNNEL SYSTEM, ALIGNMENT AND SHAPE/SIZE OF CROSS-SECTION

The tunnel system, alignment of tunnel and shape/size of the tunnel cross-section are decided in early planning stage, based on the requisite geological/geotechnical exploration and a thorough understanding of the ground.

1. Choice of Tunnel System: The tunnel system comprises all underground works that are necessary to achieve the planned use of tunnel and ensure safety of persons & material assets. Besides main tunnel(s), the tunnel system may comprise of cross-passages, adits and shafts as escape routes or other ancillary structures such as ventilation shafts or caverns for technical equipment. The choice of the tunnel system is based mainly on operational, organizational and safety considerations. The ground conditions and the topography may also have an influence on the selection of the tunnel system.

Following factors should be considered for deciding provision of Escape Distances, Lateral Exits/Access and Parallel Service & Safety Tunnel:

1.1 Escape Distance: To reach a safe place in the event of fire is the central aspect of all rescue concepts including escape distances. Additionaly, arrangements used for providing shorter escape distances can also be used by maintenance staff for performance of their work. Maximum distance between two safe places (portal, emergency exit or cross passage) in the tunnel is defined in order to enable self rescue. UIC standard specifies it to be not more than 1000m as a general guideline, i.e. mean escape distance of 500m for self rescue. For double-bore

single-track tubes and parallel safety tunnels, the distance between safe places can be reduced to 500m. This distance can vary depending on the local conditions, operating parameters and total safety concept. Maximum escape distance should be decided after considering following factors:

- Cost-effectiveness.
- Expected situation in the tunnel: smoke spread etc.
- Topography, particularly availability of opportunities like construction shafts/adits or a place very close to the surface.
- Local situation (including tunnel length, daily traffic, rescue concept, availability of parallel tunnel etc.).
- Security & maintainability.

1.2 Lateral Exits/Access: Provision of Lateral exits / access in tunnels offers following advantages:

- Ensures escape to a safe place in the event of fire and smoke.
- Leads directly out of tunnel or to a safe place.
- Access for emergency services.
- Can also be used for maintenance purposes.

The disadvantage of providing Lateral exits/access is possibility of their misuse by miscreants for sabotage purpose and additional cost of maintenance.

A decision on providing Lateral exits/access should be taken based on following:

(i) Opportunities like Construction adits or a place close to surface may be utilized for construction of lateral exits/access with necessary precautions for their maintenance/ security.

(ii) Relevant factors including cost effectiveness, daily traffic, tunnel length, rescue concept, local situation, security and maintainability.

In case Lateral exits are provided, cross section of 2.25mx2.25m may be adopted as a guideline upto a length of about 150m. For longer lengths, the exits should be accessible with road vehicles. Lateral exits should have:

- Design or installation that prevents smoke from spreading into the safe place.
- Adequate lighting and communication means.
- Design or installation for preventing unauthorized access from outside.

1.3 Parallel Service & Safety Tunnel: Provision of Parallel service & safety tunnel offers following advantages:

- Provides a safe place in the event of any accident.
- Possibility of reducing the escape distance in the main tunnel with cross passages.
- Independent access for emergency services and possibility of arriving close to place of accident.
- Can be used for maintenance purposes also.

Disadvantages of providing parallel service & safety tunnel are:

- Passengers are not yet outside the tunnel.
- Possibility of use by miscreants for sabotage purpose.

A decision on providing parallel service & safety tunnel should take following aspects into consideration:

(i) It is not recommended as a general solution. A decision in this regard should be made after evaluation of the optimal system based on consideration of relevant factors like cost

effectiveness, daily traffic, tunnel length, rescue concept, geo-technical factors, security and maintainability etc.

(ii) In case an exploratory tunnel has been planned to be constructed, the possibility of using it as a service and safety tunnel may be considered, if it is cost effective.

(iii) It may cover only parts of the tunnel length (in combination with shafts or adits)

In case parallel service & safety tunnel is provided, cross section of 3.5m x 3.5m may be adopted as a guideline. It should have independent ventilation system to keep it free of smoke. It should be accessible by road vehicles, with facility to reverse and pass. It should be connected to main tunnel through appropriately designed cross passages of size approximately 2.25m x 2.25m spaced at about 500m. These cross passages should have:

- Design or installation that prevents smoke from spreading into the safe place.
- Adequate lighting and communication means.
- Design or installation for preventing unauthorized access from outside.

2. Choice of Alignment: The vertical and horizontal alignment of the tunnel(s) depends on several factors such as:

2.1 Maximum ruling gradient: Ruling gradient in tunnels is normally kept flatter than that in open air, owing to:

(i) Reduced rail-wheel adhesion due to presence of moisture in tunnels. This causes decrease in the traction force in tunnels.

(ii) Increased air resistance: The magnitude of air resistance depends on the relative velocities of wind & train, as well as on the relative cross-section areas of tunnel and train. Resistance is especially large in single

track tunnels which are comparatively narrow.

(iii) Decrease in efficiency of internal combustion engines, used in diesel traction, due to presence of less oxygen inside tunnels.

Therefore, maximum grades in straight tunnel should preferably not exceed 75% of the ruling gradient of the track outside tunnel. Grades in curved tunnels should be compensated for curvature in the same manner as for sections outside the tunnel.

2.2 Permitted maximum degree of curve.

2.3 Drainage considerations during constru-ction and operation.

2.4 The accessibility of and natural hazards in the portal areas.

2.5 The ground conditions.

If possible, the alignment should be adapted to the ground conditions in an early phase of the project, as hazards and the respective construction time and cost risks can be avoided or reduced by the choice of a different alignment.

Aspects of execution or operation and safety (such as the necessity of intermediate adits, ventilation shafts or escape adits) may also influence the choice of the alignment, especially in long tunnels.

3. Shape and Dimensions of the Cross-section: They are determined essentially by:

3.1 Serviceability requirements: The required dimensional/ clearance profile is a key factor in determination of cross section of tunnel. Other serviceability criteria relevant for choice of cross section are:

(a) Additional space requirements for operating and safety equipment (cable installations, signaling systems, signage, lighting, ventilation, etc.).

(b) Aerodynamic requirements, for the given speed and dimension profile of the vehicles which will use the tunnel.

(c) Drainage requirements.

(d) Maintenance requirements

(e) Requirements arising from the safety and rescue concepts (escape routes within the tunnel, availability of the facilities in emergencies)

3.2 Geological/geotechnical conditions: The shape and the size of the cross section also depend on the ground conditions, as the later determine the extent of required support measures in the construction stage (tunnel support) and in the service stage (permanent lining). Unacceptable reduction in size of the opening due to ground convergence is avoided by means of additional excavation to account for ground deformations and corresponding support measures.

3.3 Construction aspects:

(a) Economic considerations and availability of the necessary equipment may be decisive for the construction method and have, therefore, a considerable influence on the shape of the cross section.

(b) In determination of shape and dimensions of the cross section, attention must be given to tolerances with respect to driving accuracy, construction tolerances and surveying tolerances.

(c) In contrast to TBM or shield tunneling, the cross section of tunnels excavated by conventional methods can be freely chosen

within the constraints of the geological conditions.

4. Shapes of Railway Tunnels: Following shapes are commonly used in Railway tunnels:

4.1 D Shaped: This shape (Fig. 6.01) is commonly seen in railway tunnels constructed upto 15-20 years back. From structural/load distribution pint of view, this shape in not efficient. Therefore, it is suitable for "Hard Rock" only.

Most of the railway tunnels constructed earlier were of "D" shape, and mostly unlined, because they were located in central part of the India, passing through competent rocks. In Udhampur – Katra section of Indian Railways, one of the tunnels (T-1) collapsed after being constructed fully and one of the reasons pointed out by an international consultant for this collapse was "One of the main problem lies within the geometrical cross section of the tunnel. A horizontal side wall as executed basically only allows for transfer of vertical stresses. In case horizontal loads are occurring, the system is likely to fail as horizontal loads can only be coped with to a very limited extent".

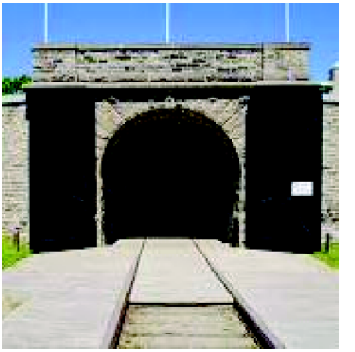


Fig. 6.01: D-Shaped Tunnel

Structural analysis of a typical railway tunnel, with D shape and Elliptical shape, has given following results:

Shape of Tunnel	Max. Axial Force (kN)	Max. Bending Moment (kN-m)	Max. Shear Force (kN)	Max. Displacement (m)
	With Top Pressure 15000 kg/m and Side Pressure 5000 kg/m			
D-Shaped	412.5	137	15129	0.083
Elliptical	567	35.80	9025	0.024
	With Top Pressure 15000 kg/m			
D-Shaped	412.5	48.10	8565	0.03
Elliptical	639	35.65	9635	0.028

The tabulation above clearly shows the structural inefficiency of D-shaped vis-a-vis elliptical shaped tunnels, especially when side pressure/horizontal loads are considered. Now-a-days this shape is very rarely used.

4.2 Horseshoe Shaped: The horse-shoe shape consists of a series of arcs, with no sharp corner (Fig. 6.02). There are horse-shoe shaped and modified horse-shoe shaped cross sections in use, most of them comprising of semi-circular roof along with arched sides and curved invert.

Fig. 6.02: Horse Shoe Shaped Tunnel

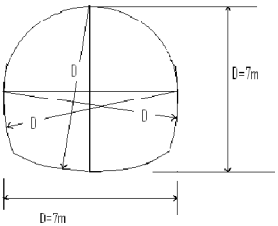


Fig. 6.03: Horse Shoe Shape Horse

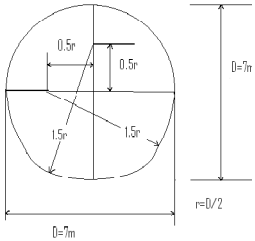


Fig. 6.04: Modified Shoe Shape

This cross-section offers a good resistance to external ground pressure and is suitable for soft rocks. This shape is most common now-a-days in tunnel constructed with “drill and blast technique”.

4.3 Elliptical/Circular Shape: These shapes are strong in offering resistance to external pressure caused by water, water bearing soils or soft grounds. This is the best theoretical section for resisting internal or external forces and it provides the greatest cross sectional area for the least perimeter. They are best suited shapes for tunnels in “soft grounds”. The circular section is often uneconomical as more filling will be required for obtaining flat base. However, the tunnels constructed by TBM are only circular in shape.

Fig. 6.05: Elliptical Shape

Fig. 6.06: Circular Shape

Weak rock zones, squeezing or swelling rock and soft ground (soils) require a circular cross section or at least a horseshoe-shaped cross section including an invert arch.

CHAPTER-7

TUNNEL DESIGN AND TUNNEL SUPPORTS

Understanding, designing and constructing any underground excavation (e.g. Tunnel) is conceptually quite different and complex as compared to other normal surface constructions (e.g. Building, Bridge etc.). In normal surface constructions, the loads on the structure can be estimated with reasonable accuracy (e.g. Dead Load, Live Load, Wind Load, Earthquake Load etc.) and the materials (e.g. Steel, Concrete, Timber etc.) of known strength/deformation properties are assembled in a known fashion in such a way that the strength/ deformation capability of the construction materials/ structure are lesser (with certain margin of safety, known as factor of safety) than the stresses/ deformations caused by the imposed loads. But in case of underground excavations, it is neither possible to predict the loads, the way it is done in surface constructions, nor the strength/deformation capability of the construction materials/structure can be defined due to the ground around excavation (rock mass) being a highly non-homogeneous and anisotropic matter.

1. Concept of Stabilization of a Cavity: For understanding the behaviour of underground excavation, it is necessary to understand this concept, which is many times called as “Arch effect” also.

To understand this, take easily understood example of flow lines/water current in a river, which get deviated/ disturbed due to construction of a pier in the water stream (Fig. 7.01). There is turbulence/increase in speed of water flow around the pier for some extent and beyond that there is no change in flow lines/water current. Similarly, in case of a rock mass, the

equilibrium state of ground stresses is disturbed due to excavation of an underground cavity like tunnel (Fig. 7.02).

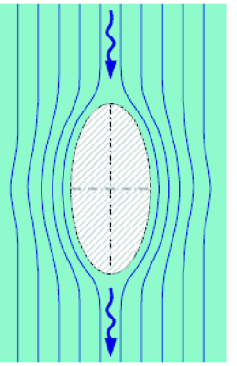


Fig. 7.01

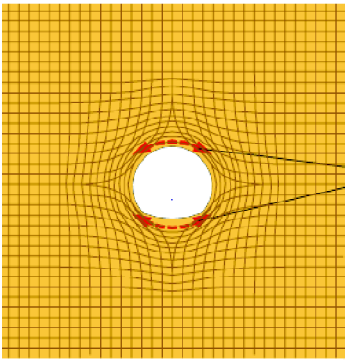


Fig. 7.02

The stresses are channeled/re-distributed around the cavity, creating a zone of higher stresses around the excavation boundary. This causes convergence of the cavity and some extent of rock mass just behind the excavation boundary getting badly affected/cracked (and may be going into plastic state), some rock mass further inwards in the ground getting moderatly affected (may be in elastoplastic state) and the remaining rock mass not getting affected at all (remaining in elastic state). This re-distribution of stresses is called “arch effect”. The magnitude of convergence and the extent of various states of rock mass, which is a “reaction” of the ground to the disturbance caused in it, depends upon the properties of “medium” (i.e. the ground in which cavity is made) and the “action” (i.e. shape/size of tunnel, method of tunnel excavation and tunnel advance rate).

Depending on various combinations of “medium” and “action”, the “reaction” can be broadly classified in two categories. In the first case, the ground around the cavity well withstands the re-distributed stresses with the convergence being limited (within permissible limits,

depending on the type of excavation) and the extent of plastic and elastoplastic zone not being excessive. In this case, the excavation will become self-stabilizing in a reasonable time and no support may be required. In second case, either the convergence may be progressing towards unacceptable limits (worst case being collapse of the cavity) or the extent of plastic & elastoplastic zone not getting stabilized (or becoming very high, which in-turn contributes to instability of the cavity) i.e. “arch action” not getting established. In this case, some “intervention” is required from within the cavity, to limit the convergence to permissible limits and also to limit extent of plastic/elastoplastic zones (i.e. arch effect getting formed). Such type of “intervention" is called “tunnel support” and design of tunnel is essentially design of “tunnel support system”.

Another peculiarity of underground excavations is that the stage at which the structure is subjected to most load is not when the tunnel is finished but it is the intermediate construction stage. This is very delicate situation because the effects of disturbance caused by excavation have not yet been completely countered by the final tunnel support system. Thus, the purpose of “tunnel supports” is to:

(i) Preserve the integrity of rock mass.

(ii) Provide safe and secure work area.

(iii) Maintain an opening in the rock mass.

Initial or primary ground support is installed shortly after excavation, to make the tunnel opening safe until permanent support is installed. The initial ground support may also function as permanent ground support or as a part of the permanent ground support system. Initial ground support systems are usually not subject to rigorous design but are selected on the basis of a variety of rules (viz. Empirical, Analytical & Numerical), and one or more of these approaches can be used.

2. Properties of Supports: Important properties of supports (including their material) are as under:

2.1 Stiffness: The correlation between various parameters, related to stiffness of supports, can be explained as under:

Table: 7.01

Stiffness of Support	Deformability	Resistance to	
		Bending Moments	Normal Forces
Highly Stiff	Rigid	High	Low
Stiff	Semi-rigid	High	Low
Weak	Highly Deformable	Low	High

The vertical and horizontal load on supports is important to determine the section and type of the support. But since this cannot be predicted accurately, the supports may have to be designed in such a way that they are suitable for many scenarios of loading. If a stiff support is installed initially, it prevents redistribution of stresses in rock mass and therefore, such supports are advisable to be installed only after stress distribution has largely taken place (i.e. as final lining). Some examples of locations, where highly stiff supports are provided, are:

(i) In cut and cover tunnels.

(ii) In or near portal zones, where earthquake forces are taken into account for design of supports.

(iii) Tunnels in heavily faulted zones.

(iv) Tunnels with very shallow cover.

(v) Tunnels designed with very low deformation and settlement.

Support systems, which are weak in bending, are not stable on their own but require interaction with surrounding rock mass. In such cases, their bond with the rock mass is very critical.

2.2 Bond: The bond between the support and rock mass ensures transfer of radial forces over the whole area and continuous transfer of tangential forces.

2.3 Time of installation: If the rock mass is unstable, temporary support should be provided during or before the excavation. In a stable/ competent rock mass, supports may not be needed also. However, the support is required to be installed before the convergence of the cavity and/ or extent of plastic zone in the rock mass around the cavity becomes more than permissible limits.

3. Type of Tunnel Supports: Following type of tunnel supports are commonly used:

3.1 Steel ribs/Steel sets: Such supports are made using rolled steel sections (e.g. ISMB, ISHB etc.) which are bent to the shape of the tunnel cross section (Fig. 7.03). For the purpose of handling, the supports are made in two or three parts and they are joined after erection in position. They are normally used with lagging (made of precast concrete, steel plates etc.) and backfill (of lean concrete or mortar or tunnel muck) (Fig. 7.04).

Fig. 7.03: Steel ribs/sets

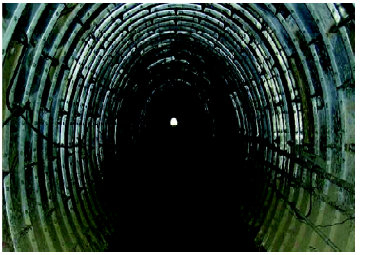


Fig. 7.04: Lagging

Many types of yielding supports are also available, in which the support can change its' length within a given limit, to account for some convergence in

the cavity. Advantages of such supports are – possible to pre-fabricate and feasibility to install vertically/inclined according to the form of the face. Disadvantages of such supports are - difficult to handle, being very heavy, costly and poor flexibility.

Till few years back, use of steel ribs with backfilling by lean concrete or tunnel muck was most common method of tunnel supports. But due to their poor flexibility and uncertainties in lack of interaction with rock mass, they are very rarely being used as primary supports, except at the locations warranting very rigid supports (listed in Para 2.1 above). In Udhampur – Katra section of Indian Railways (Fig. 7.05 to Fig. 7.07), one of the tunnels (T-1) where such supports were installed and which collapsed after being constructed fully, one of the reasons pointed out by an international consultant for this collapse was *"Due to installation process of support (concreting and utilization of sandbags for excess excavation) a complete sound contact of primary support with the surrounding ground is unlikely. Therefore, gravitational loads caused by loosening of the rock mass due to cavities will occur which are loading the lining additionally".*

Fig.7.05: Tunnel T-1 in Udhampur - Katra Section

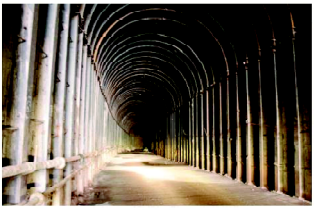


Fig. 7.06: Finished Tunnel

Fig. 7.07: Tunnel Collapse

3.2 Lattice Girders: They are fabricated at site, to the required cross section of tunnel, by welding of reinforcement bars (Fig. 7.08).

These types of supports are relatively light weight (hence economical), flexible and easy to handle. They are normally installed with shotcrete (Fig. 7.09). Use of such supports is very common now-a-days and their flexibility is an added advantage, as will be discussed subsequently in "Rock Structure Interaction".

Fig. 7.08: Lattice Girder

3.3 Rock Bolts: Rock reinforcement in tunnels (of which rock bolt is one of the types) is used for many purposes. There are many types of rock bolts, with many of them being patented products, generally made of steel (mostly reinforcement bars of suitable diameter). A borehole of required diameter is drilled in the rock mass and the bolt is

inserted in the borehole. The bolt is anchored near the tip using suitable mechanism, then it is stressed and the bore hole mouth is covered by using a face plate and face nut. Afterwards, grouting is done (in most of the cases) to fill up the annular space between the bolt and the walls of the borehole.

Fig. 7.09: Installation of Lattice Girder

In case bore hole does not remain stable until withdrawal of drill rod and insertion of bolt, Self-Drilling Rock bolts (SDR) or Self-Drilling Anchors (SDA) are used, wherein the drill bit is located at the end of bolt and borehole is drilled using the bolt and drill bit. In such a system, the drill bit is not re-used and one drill bit gets scarified in each borehole. SDAs or SDRs are costly as compared to normal rock bolts, but their use is necessary in case of weak ground tunnelling. When the required length of rock bolts becomes excessive (say more than 10m or so), rock anchors (which are made of woven high tensile steel ropes) are used in pace of steel bolts.

Rock bolts serve many purposes in the tunnels. First purpose of rock bolts is to support individual rock block(s), which has become loose due to creation of cavity and may have eventually fallen (Fig. 7.10). The rock bolt anchors/stitches the

loose block(s) to the competent rock mass away from the excavation boundary, thereby, providing stability to such potentially unstable blocks.

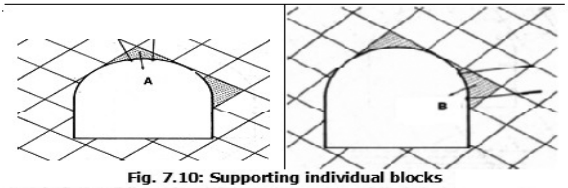


Fig. 7.10: Supporting individual blocks
A: Block may fall. Rock Reinforcement prevents fall of Block.
B: Block may slide. Rock reinforcement increases the resistance to sliding

Fig. 7.10: Supporting individual blocks

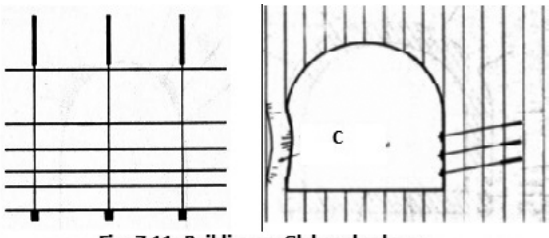


Fig. 7.11: Building up Slab and column
C: Inward deformation (buckling) of side wall because of high vertical in-sity stress.

Fig. 7.11: Building up Slab and column

Strength in bending or as a column, of a number of layers joined together and working as one layer is multiple fold higher as compared to strength when these layers act individually. Using this principle, the rock bolts are used as a joining or stitching mechanism for various layers of rock mass, to increase their strength in bending as well as a column (Fig. 7.11).

Use of rock bolts limits plastic zone in a localized manner (Fig. 7.12) as it helps in re-distribution of stresses at that point and when used all around cavity they help in controlling yield (development of plastic zone around the periphery of the excavation) by building a structurally active arch/

ring of the reinforced zone around the periphery of the excavation (Fig. 7.13 & Fig. 7.14). After the cavity is created, the stress in rock mass is re-distributed. The tri-axial stress state changes to bi-axial state, since the radial stress is no longer present. The tangential stresses increase and can exceed the compressive strength of the rock, if the rim is not reinforced. Due to formation of a structurally active ring of the rock, the stresses are transferred to a competent/unaffected rock mass at deeper depths. However, to achieve this, the rock bolts/anchors length should be more than the extent of plastic/affected zone and they should always be anchored in competent/unaffected rock mass.

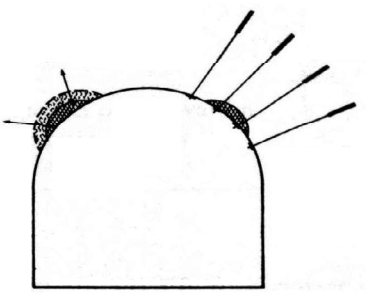


Fig. 7.12: Limiting Plastic Zone locally

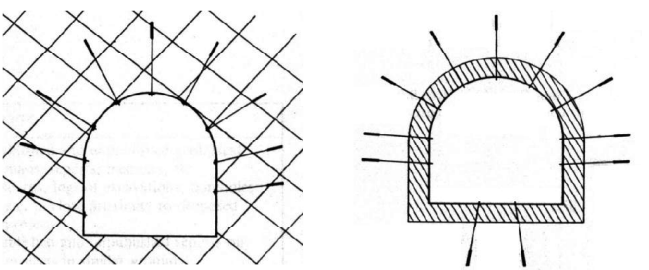


Fig. 7.13: Building up Arch/Ring and Controlling Yielding

When pattern bolting is installed quickly with spraying of shotcrete, this acts to provide support resistance and a near tri-axial stress condition is restored at the periphery of the excavation. This acts against further weakening of rock mass and prevents further build-up of plastic/disturbed zone.

The rock bolts should be positioned at an angle of 45^0 to 90^0, but never less than 30^0, to the bedding planes/joining surface. Rock bolts installed parallel to the joining surfaces can only bear little or no longitudinal load and are of no use.

Owing to their ease of installation, faster installation after excavation and flexibility provided by them in construction process; rock bolts have become essential part in all tunneling works. They are normally used to secure the profiled excavation boundary till installation of final lining. But depending on the type of rock mass and dimension of the excavation, rock bolts can be used as permanent support also.

The load bearing capacity of the rock bolt depends on its' shaft and its' head, and furthermore on the anchor force in the rock mass. The load bearing capacity of the shaft is determined by its' permissible load, which is force at the yield limit divided by a factor of safety, normally 1.5 to 2.

Common types of rock bolts are as under:

(A) Mechanical/Expanding/End Anchor bolts: In these bolts, an expansion mechanism is provided at the end of shaft, which expands when the shaft is rotated. Due to this expansion, the bolt end gets anchored with the wall of the bore hole (Fig. 7.14). Such kind of anchors are used mostly for temporary supports only and are not very popular in tunnelling.

(B) Grouted/Friction Bolts: In these types of rock bolts, steel bar (mostly steel reinforcement bars of required diameter) is used in the bore hole. If the

anchor is to be installed as “unstressed”, the bolt is simply kept in the bore hole and then grouted or alternatively the borehole is filled with grout and then bolt is pushed in it. If the anchor is to be installed as “stressed”, then the farthest end of bolt is anchored in borehole using fast-setting mortar or

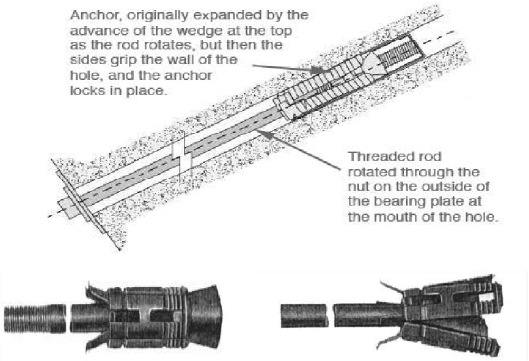


Fig. 7.14: Mechanical/Expanding Rock bolts

epoxy resin at the end portion (Fig. 7.15). After this portion gets anchored, the bolt is tensioned, borehole is closed using face plate/face nut and then the annular space between the bolt shank and borehole boundary is filled by pumping cement grout through a hole in the face plate. Some anti-shrink agent (e.g. aluminum powder) is added to cement grout to prevent reduction in adhesion with rock surface due to shrinkage during its’ drying. These bolts are most commonly used in tunnel constructions.

(C) Friction Tube/Swellex Bolts: This is a patented product of Atlas-Copco, consisting of steel tube of diameter of 41mm, which is reduced to a diameter of 28mm by double folding (Fig. 7.16). The hollow folded tube, which is closed at both ends with a bush, is installed in drilled bore hole. Then it is hydraulically expanded by the water pressure of

up to 300 bars, into the borehole periphery, to provide the anchoring effect (Fig. 7.17). The

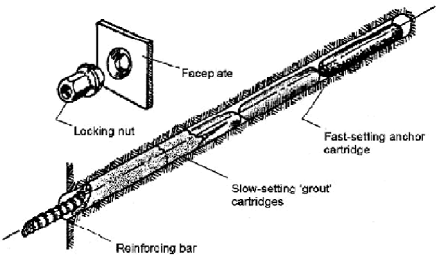


Fig. 7.15: Grouted / Friction bolt

anchoring action is provided by the friction between the hydraulically expanded anchor and the rock mass. These anchors can be installed in a very short time, they immediately give full load bearing capacity over the full length, are not sensitive to vibrations from blasting and have good adaptability for various diameter holes. Thus, they are useful as immediate supports after excavation, as part of initial support, especially in soft rocks/grounds.

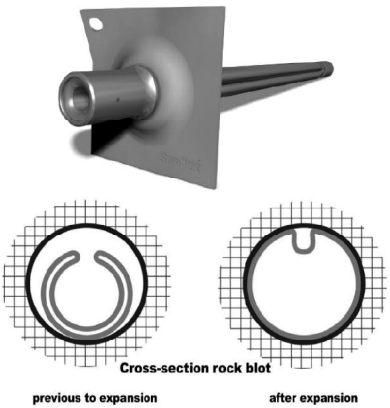


Fig. 7.16: Swellex Bolts

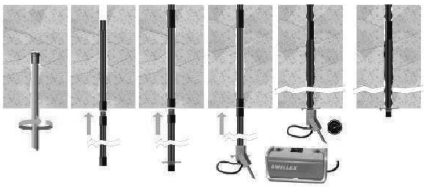


Fig. 7.17: Swellex Bolt Installation

(D) GRP Anchors: They consist of one or more bars of glass-fiber reinforced plastic (GRP), consisting of textile glass and polyester resin (Fig. 7.18).

Fig. 7.18: Glass Fiber Tubes

This results in weight reduction of 60-70% compared with other steel bolts of same permissible load. They also have High tensile strength, High elasticity, Low weight, better corrosion resistance and better electrical insulation properties vis-a-vis conventional steel bolts. Use of such anchors, are commonly used for advance face support in “ADECO-RS Tunneling” methodology.

3.3.1 Reinforcement by Rock Bolts: The rock bolts reinforce the rock mass in following ways:

3.3.1.1 Reinforced beam: According to Lang (1961), due to tensile pre-tension stress of the bolt, compressive stress is developed in the rock mass , in the direction normal to bolt axis, due to Poisson’s effect. This pre-stress can stabilize the rock beam effectively as in the case of pre-stressed concrete beam.

A two-dimensional photo-elastic study showed that the pre-tension of bolts form a zone of uniform compression between the ends of the bolts (Fig. 7.19). The only condition is that the ratio between length (l) and spacing (s) of bolts is more than 2. At this ratio, the zone is relatively narrow whereas for l/s equal to 3, it is approximately equal to two-third of the bolt length (i.e. equal to l-s). The normal stress (σ_v) within the zone may be estimated as ratio of pre-tension to the area per bolt. The horizontal stress (σ_h) equal to $k_0\sigma_v$ would be induced within this zone provided that the bolted beam is clamped laterally. The total horizontal force is the sum of axial pre-stress (P_h) and the thrust (T) due to the arch action. Higher horizontal force means greater frictional resistance to sliding of the beam downwards.

The photo-elastic model further indicated that zones of tensile stresses develop between bolts and so it may require an additional support in the form of wire-netting.

3.3.1.2 Reinforced rock arch: As can be seen from the Fig. 7.20 that radial bolting pattern creates a reinforced rock arch over the tunnels. The thickness of arch can be increased by employing supplementary bolts of shorter length. The most common practice is (Lang, 1996; Barton et al., 1974):

(i) Rock bolts be pre-tensioned to give required ultimate support capacity (p_{roof} or p_{wall}) which is equal to P/b.s where P = pre-tension, b = bolt spacing along tunnel axis and s = bolt spacing perpendicular to tunnel axis. The pre-tensioned bolts are suitable for temporary support of openings in the hard rocks.

(ii) Grouted bolt anchors should be designed

to provide ultimate support pressure (P_{roof} or P_{wall}) equal to P/b.s where P is the tensile strength of bolts, provided bolts are adequately grouted. The bolt length should be greater than $\frac{1}{4}$ to $\frac{1}{3}$ of span of tunnel.

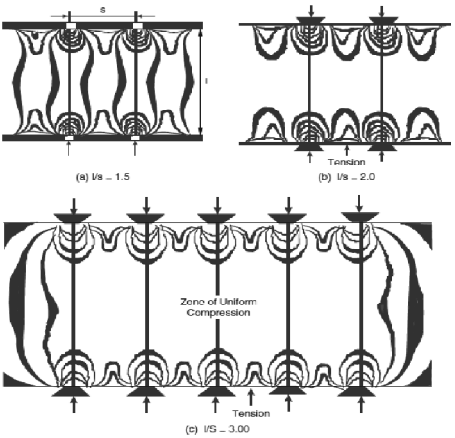


Fig. 7.19: Rock bolt – Photoelastic stress pattern

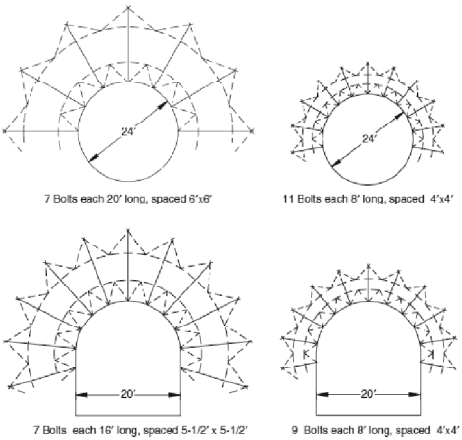


Fig. 7.20: Arch concept of rock reinforcement

(iii) The l`ength of bolts (L in meters) should be calculated from the following simple relationship given by Barton et al. (1974),

L = 2 + (0.15 B/ESR) for roof
= 2 + (0.15 H/ESR) for wall

Where : B = span or width of opening in meters,
H = height of opening wall in meters,
ESR = excavation support ratio (Table-7.02).

(iv) The adequate length of grouted anchors be obtained similarly as follows:

L = 0.40 B/ESR for roof
= 0.35 H/ESR for wall

Table 7.02: Values of ESR *(Barton et al., 1974)*

S. No.	Type of excavation	ESR
1	Temporary mine opening, etc.	3 - 5
2	Vertical shafts: (i) circular section (ii) rectangular/square section	 2.5 2.0
3	Permanent mine opening, water tunnels for hydropower (excluding high pressure penstock), pilot tunnels, drifts and headings for large excavations, etc.	1.6
4	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (Cylindrical cavern?)	1.3
5	Oil storage caverns, power stations, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
6	Underground nuclear power stations, railway stations, sports and public facilities, factories, etc.	0.8

Note: ESR should be increased by 1.5 times for temporary supports

(v) When single (2-3 cm thick) or double (5 cm thick) layers of shotcrete are applied usually in combination with systematic bolting, the function of shotcrete is to prevent

loosening, especially in the zone between bolts. The capacity of shotcrete lining is, therefore, neglected. The application of shotcrete is essential to make grouted bolt-anchor system as permanent support.

(vi) Clear spacing between bolts should not be more than three times the average fracture spacing otherwise use wire mesh and guniting or shotcreting. Further center to center spacing must be less than one-half of the bolt length.

(vii) Bolts are installed on a selected pattern except near weak zones that would require special treatment. Spot bolting should be discouraged.

(viii) Bolts should be oriented to make an angle of 0 to Ø to the normal on the critical joint sets in order to develop maximum resistance along joints (Fig. 7.21).

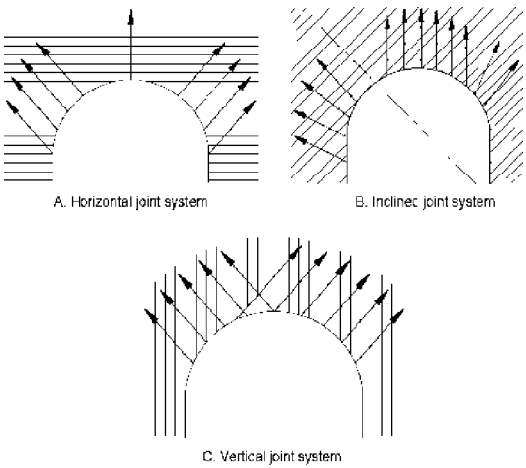


Fig. 7.21: Rock bolting with different dip angles

(ix) Bolts must be installed as early as possible within the "stand up time" and close to the excavated face.

However, a tunnel is always unsupported in a certain length "t" between the last row of bolting and the newly excavated/blasted face. According to Rabcewicz (1955), the zone of rock mass of thickness t/2 may be fractured and loosened due to blasting as shown in Fig. 7.22. Thus, the bolt length must be at least equal to the thickness of loosened zone (t/2), so that the loose zone may be suspended by competent rock mass.

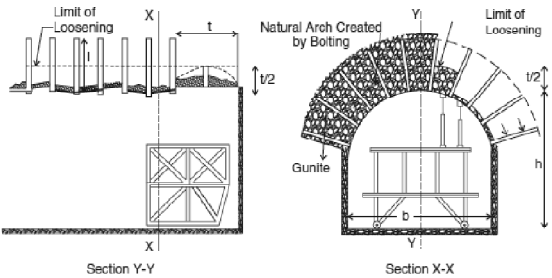


Fig. 7.22: Principle of rock bolting

Rock bolts/anchors should be designed to absorb high longitudinal strains in the cases of weak rock masses. So the bolts of high tensile strength are failure in caverns and tunnels in weak rocks under high tectonic stresses.

3.3.2 Rock bolting pattern: It is generally agreed that pattern bolting should be preferred over spot bolting because unknown conditions behind the surface of an excavation may be more critical than those visible at the surface. In addition, pattern bolting is advantageous from construction point of view.

3.3.3 Floor bolting: Floor bolting is required to prevent floor of a tunnel heaving in order to maintain the track properly. Attempts to chop off squeezed rock mass are fruitless and may damage the wall support. However, there is no standard practice.

3.4 Concrete Lining: The concrete lining can be pre-cast or cast in-situ. The lining can be in single layer or double layer. Even sprayed concrete (shotcrete), sprayed in one layer or multiple layers, is also used as lining system. In case of tunnelling by Tunnel Boring Machine (TBM) normally pre-cast lining segments (Fig. 7.23) are used. They are cast at a casting yard, transported to site and then placed in position in the form of lining rings, using erector arm in the TBM. In case of tunnels excavated by drill and blast or road header method, normally one/multiple layer of shotcrete or cast in-situ concrete lining is used. The in-situ concrete lining is done using a lining shutter (Fig. 7.24) of the shape of the tunnel profile, which moves on the temporary track provided in the tunnel invert. After placing the shutter in the position, the concrete is pumped into the annular space between the shutter and the tunnel periphery. After the concrete sets in, the shutter is moved forward and the process is repeated till whole length of the tunnel is covered.

Fig. 7.23: Pre-cast Lining Segments

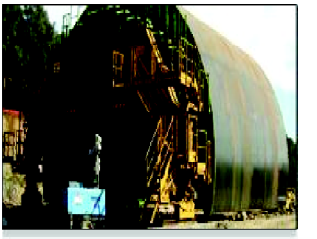


Fig. 7.24: Lining Gantry for in-situ casting

3.4.1 Cast in-situ Concrete Lining: These are generally installed sometime after the initial ground support. These can be used in both soft ground and hard rock tunnels and can be constructed of either reinforced or plain concrete. While the lining may generally remain unreinforced, structural design considerations and design criteria will dictate the need and amount of reinforcement.

To ensure contact between the initial and final linings, contact grouting is performed as early as final linings have achieved its 28 days strength. This ensures contact between initial and final tunnel support and any deterioration or weakening of initial support will lead to an increased loading of the final support by the increment not being supported by initial lining.

Cast in-situ final lining pour length is normally restricted to limit surface cracking and is mandatorily followed in unreinforced lining. Adjacent concrete pours feature construction joints and continuous reinforcement in joints is not desired to allow relative movement.

Cast in-situ concrete lining is having following advantages:

- Suitable for use with any excavation and initial ground support method.
- Corrects irregularities in the excavation.
- Can be constructed to any shape.
- Provides a regular sound foundation for tunnel finishes.
- Provides a durable, low maintenance structure.

Cast in-situ concrete lining is having following disadvantages:

- Concrete placement, especially around rein-forcement may be difficult. The nature of the construction of the lining

restricts the ability to vibrate the concrete. This can result in incomplete consolidation of the concrete around the reinforcing steel.

- Reinforcement is subject to corrosion resulting into deterioration of concrete. This is a problem common to all concrete structures, however underground structures can also be subjected to corrosive chemicals in the groundwater that could potentially accelerate the deterioration of reinforcing steel.
- Cracking that allows water infiltration can reduce the life of the lining.
- Chemical attack in certain soils can reduce lining life.
- Construction requires a second operation after excavation to complete the lining.

In order to maximize flexibility and ductility, a cast in-situ concrete lining should be as thin as possible. There are, however, practical limits on how thin a section can be placed and still obtain proper consolidation. Thickness of 25cm is considered practical minimum thickness for cast in-situ concrete lining.

Reinforcing a thin section can also be problematic. If two layers of reinforcement are used, then staggering the bars may be required to obtain the required concrete cover over the bars. This can make the formwork congested and concrete placement more difficult.

Cast in-situ concrete is used as the final lining. In many cases a waterproofing system is placed over the initial ground support prior to placing the final concrete lining. Placing reinforcing steel over the waterproofing system increases the potential for

damaging the water proofing. In all cases where it is practical, cast-in-place concrete linings should be designed and constructed as plain concrete, with no reinforcing steel. The presence of the waterproofing systems precludes load sharing between the final lining and the initial ground support. A basic design assumption is that the final lining carries long term earth loads with no contribution from the initial ground support.

Ground water chemistry should be investigated to ensure that chemical attack of the concrete will not occur if the lining is exposed to ground water. If this is an issue on a project, mitigation measures should be put in place to mitigate the effects of chemical attack. The waterproofing membrane can provide some protection against this problem. Admixtures, sulphate resistant cement and high density concrete may all be potential solutions. This problem should be addressed on a case to case basis and the appropriate solution be implemented based on best industry practice.

Concrete behaviour in a fire event must also be considered. When heated to a high enough temperature, concrete will spall explosively. This produces a hazardous condition for trains and for emergency response personnel responding to the incident. Spalling and loss of reinforcing strength can cause changes in the shape of the lining, redistribution of stresses in the lining and possibly structural failure. The lining should be protected against fire.

Mixes for cast in-situ concrete should have high enough slump to make placement practical. Air entrainment should be used. Compressive strength should be kept to a minimum. High strength concretes require complex mixes with multiple admixtures and special placing and curing procedures. Since concrete lining acts primarily in compression, 28 day compressive strengths in the

range 25 to 30 MPa are generally adequate.

3.4.2 Precast Segmental Lining: Precast segmental linings are used in circular tunnels excavated using Tunnel Boring Machine (TBM). It can be used in both soft and hard ground. Several curved precast elements or segments are assembled inside the tail of the TBM to form a complete circle. The segments are relatively thin, 20 to 30cm thick, and typically 1 to 1.5m wide measured along the length of the tunnel.

Precast segmental linings can be used as initial ground support followed by a cast in-situ concrete lining (the “two-pass” system) or can serve as both the initial ground support and final lining (the “one-pass” system) straight out of the tail of the TBM. Precast segmental linings, used as both initial support and final lining, are built to high tolerances and quality. They are typically heavily reinforced, fitted with gaskets on all faces for waterproofing and bolted together to compress the gaskets after the ring is completed but prior to advancing the TBM. As the completed ring leaves the tail of the shield of the TBM, contact grouting is performed to fill the annular space that was occupied by the shield. This provides continuous contact between the ring and the surrounding ground and prevents the ring from dropping into the annular space. Bolting is often performed only in the circumferential direction. The shove of the TBM is usually sufficient to compress the gaskets in the longitudinal direction. Friction between the ground and the segments hold the segment in place and maintain compression on the gasket. Segmental linings were initially fabricated in a honeycomb shape that allowed for bolting in both the longitudinal and circumferential directions. Recent lining designs have eliminated the longitudinal bolting and the complex forming and reinforcing patterns that were required to accommodate the

longitudinal bolts. Segments now have a flat inside surface. Once adequate strength is achieved, the segments are inverted to the position they must be in for erection inside the tunnel. Horizontal and vertical tunnel alignment is achieved through the use of tapered segments.

Precast segmental lining have following advantages:

- Provides complete stable ground support that is ready for follow-on work.
- Materials are easily transported and handled inside the tunnel.
- No additional work, such as forming and curing, is required prior to use.
- Provides a regular sound foundation for tunnel finishes.
- Provides a durable low maintenance structure.

Precast segmental lining have following advantages:

- Segments must be fabricated to very tight tolerances.
- Reinforcing steel must be fabricated and placed to very tight tolerances.
- Storage space for segments is required at the job site.
- Segments can be damaged, if mishandled.
- Spalls, cracked and damaged edges can result from mishandling and over jacking.
- Gasketed segments must be installed to high tolerances to assure that gaskets perform as designed.

Segments used as an initial support lining are frequently designed as structural plain concrete. Reinforcing steel is placed in the segments to assist in resisting the handling and storage stresses imposed on the segments. Reinforcement is often welded wire fabric or small reinforcing steel bars.

Ends of the segments that form the joints should be reinforced to facilitate the transfer of load from one segment to another without cracking and spalling. The primary load carried by the precast segments is axial load induced by ground forces acting on the circumference of the ring. However, loads imposed during construction must also be accounted for in the design. Loads from the jacking forces of the TBM are significant and can cause segments to damage and require replacement. These forces are unique to each tunnel and are function of the ground type and the operational characteristics of the TBM. Reinforcement along the jacking edges of the segments is usually required to resist this force. The segments should be checked for bearing, compression and buckling from TBM thrust loads. Initial lining segments are considered to be temporary support, therefore, long term durability is not considered in the design of the linings.

Segments used as final lining are designed as reinforced concrete. The reinforcement assists in resisting the loads and limits cracking in the segment. Limiting cracking helps make the segments waterproof. Final lining segments can be fabricated with straight or skewed joints. The orientation of the joint should be considered in the design of the lining to account for the mechanism of load transfer across the joint between segments. Skewed joints will induce strong axial bending in the ring and this should be accounted for in the design of the ring. Whether using straight or skewed joints, segments are rotated from ring to ring so that the joints do not line up along the longitudinal axis of the tunnel. Joints should be adequately reinforced to transfer load across the joints without damage. The primary load carried by the precast segments is axial load induced by ground, hydrostatic and other forces acting on the circumference of the ring. The presence of the waterproofing systems precludes

load sharing between the final lining and the initial ground support. A basic design assumption is that the final lining carries long term earth loads with no contribution from the initial ground support.

Lifting and erecting the segments also impose loads. The segments should be designed and reinforced to resist these loads. Grouting pressure can also impose loads on the lining. Grouting pressures should be limited to reduce the possibility of damage to the ring by these loads. The anticipated grouting pressure should be added to the load effects of the earliest ground loads applied to the lining.

Concrete mixes for precast segments for initial linings do not require special designs and can generally conform to the structural concrete mixes provided in most standard construction specifications. Strengths in the range of 25 to 35 MPa are generally adequate. Air entrainment is desirable since segments may be stored outdoors for extended periods of time and final lining segments may be exposed to freezing temperatures inside the tunnel.

3.5 Shotcrete: It is also known as torcrete concrete, sprayed concrete or gunite. Essentially it is a concrete mix which is sprayed with compressed air, it hydrates on the substrate and hardens. It has become an essential part of support in almost all tunneling projects. Admixtures can be used to improve the properties of strength and adhesion. Shotcrete provides immediate support after excavation by filling small openings, cracks and fissures. This reduces potential relative movement of rock bodies or soil particles and limits loosening of exposed ground surrounding tunnel.

Adhesion of shotcrete depends on condition of ground surface, dampness/presence of water,

composition of shotcrete and pressure of shotcrete. Generally rougher ground surface provides better adhesion. Dry rock surface have to be adequately dampened prior to application. Dusty or flaky surfaces, water inflow or water film on surface reduce adhesion. Modern admixtures can improve adhesion of shotcrete significantly to reduce rebound, however, these should be used judiciously to avoid its ill effect e.g. accumulation around reinforcement bars, voids etc.

Owing to its' flexibility, faster placement time and continuous contact with excavated surface, as it takes the shape of excavated surface even after its convergence, it is very effective as a support system in mobilizing rock support interaction. In most of the tunneling projects, a layer of shotcrete is sprayed immediately after excavation, which in addition to providing initial support prevents and seals the cracks on the surface of the excavated boundary and prevents further degradation of the newly formed rock mass surface.

Fig. 7.25: Shotcrete Machine

Fig. 7.26: Shotcreting

In dry shotcreting, the dry mix and water are mixed at the head of spraying nozzle and it requires very high degree of skill by the operator. Dry shotcrete in not used very commonly now-a-days. In wet shotcrete, which is very commonly used, ready mixed concrete is poured into the shotcrete machine (Fig. 7.25) and then it is sprayed using manual nozzle or robotic arm (Fig. 7.26). In a single layer, thickness of 100-400 mm can be deposited, depending on the type of mix used, type of equipment used and the surface on which it is sprayed.

Shotcrete can be unreinforced or reinforced by putting a welded wire mesh in the shotcrete layer. In case, toughness and ductility are desirable, shotcrete reinforced with randomly oriented steel fibers, known as Steel Fiber Reinforced Shotcrete (SFRS), is used as alternative. The steel fibers come in bundles which can be crumbled into individual pieces by hand (Fig. 7.27). The typical dimensions of these fibers is 0.75 to 1.0mm diameter and 60mm length. They are added to the mixing drum of the shotcrete machine and they get spayed with the concrete.

Occasionally, structural plastic fibers are used in lieu of steel fibers when shotcrete is expected to undergo high deformation and ductility post-cracking is of importance.

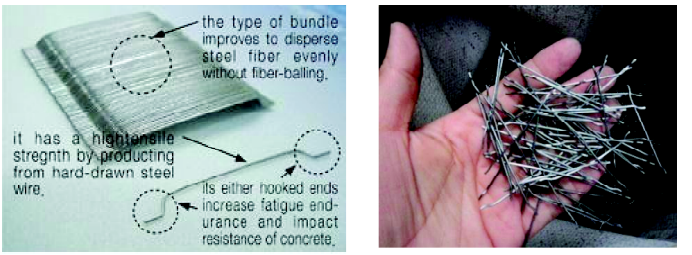


Fig. 7.27: Steel Fibers

Sometimes shotcrete is also used as permanent lining, which is normally reinforced to provide long term ductility.

3.5.1 Initial Shotcrete Lining: Initial shotcrete lining typically consists of 100 to 400mm thick layer mainly depending on the ground conditions and size of the tunnel opening, and provides support pressure to the ground. It is also referred to as shotcrete lining. A shotcrete ring can carry significant ground loads although the shotcrete lining forms a rather flexible support system. By deforming, it enables the inherent strength and self-supporting properties of the ground to be mobilized as well to share and re-distribute stresses between the lining and ground. From the ground support point of view the design of the shotcrete lining is governed by the support requirements, i.e., the amount of ground deformations allowed and ground loads expected as well as economical aspects. The earlier the sprayed concrete gains strength the more the support restrains ground deformation. However, by increasing stiffness the support system increasingly attracts loads. It depends on the ground conditions and local requirements how stiff or flexible the support system has to be and thus what early strength requirements, thickness and reinforcement should be specified.

In shallow tunnels and deformation sensitive structures (e.g. buildings) being located on ground surface, ground deformations and consequently surface settlements have to be kept within acceptable limits. The advantage of the mobilization of the self-supporting capacity of the ground can, therefore, be taken to a very limited extent. Here, early strength of the shotcrete is required to gain early stiffness of the support to limit ground deflections. Under these conditions, the shotcrete lining takes on significant ground loads at an early

stage. Early strength can be achieved with admixtures and modern types of cement.

In contrast, for tunnels under high overburden, the prevention of ground deformation and surface settlement plays a secondary role. By allowing the ground to deflect (without over-straining it) the ground's self-supporting capability, mainly shear strength, is mobilized. Consequently, the ground loads acting upon the shotcrete lining can be limited significantly because the ground assumes a part of the support function and a portion of the ground loads is dissipated before the initial support is loaded. For rock tunnels under high cover, early strength is not a necessity but final strength of the entire system (including rock) is of importance.

Shotcrete support and rock reinforcement are designed to form an integrated support system in view of the excavation and support sequence. The design engineer must define the requirements for the support system based on thorough review of the ground response anticipated.

Friction between the ground and the sprayed concrete lining (tangential subgrade reaction) is paramount for the support system. This friction reduces differential movement of ground particles at the ground surface and contributes to the ground-structure interaction.

An important aspect of shotcrete linings is the design and execution of construction joints. These joints are located at the contact between shotcrete applications in longitudinal and circumferential directions between the initial lining shells of the individual excavation rounds and drifts. An appropriate location and shape as well as connection of the reinforcement through the longitudinal joints is of utmost importance for the integrity and capacity of the support system. Longitudinal joints have to be oriented radially, whereas circumferential joints

should be kept as rough as possible. Splice bars/ clips and sufficient lapping of reinforcement welded wire fabric maintain the continuity of the reinforcement across the joints. Rebound, excess water, dust or other foreign material must be removed from shotcrete surface against which fresh concrete will be sprayed. The number of construction joints should be kept to a minimum.

In case of ground water ingress, the ground water has to be collected and drained away. Any build-up of groundwater pressure behind the shotcrete lining should be avoided because increased ground water pressure in joints and pores reduces the shear strength in the ground, undue loads may be shed onto the shotcrete lining (unless it is designed for that, which is unusual for initial shotcrete linings); softening of the ground behind the lining; increased leaching of shotcrete and shotcrete shell detachment from the ground.

3.5.2 Final Shotcrete Lining: Shotcrete as a final lining is typically utilized in combination with the initial shotcrete supports in NATM applications when the following conditions are encountered:

- The tunnels are relatively short in length and the cross section is relatively large and therefore investment in formwork is not warranted, i.e. tunnel of less than 250m in length and larger than about 12m in spring line diameter.
- The access is difficult and staging of formwork installation and concrete delivery is problematic.
- The tunnel geometry is complex and customized formwork would be required. Tunnel intersections, as well as bifurcations qualify in this area.

When shotcrete is utilized as a final lining in dual shotcrete lining applications, it will be applied against

a waterproofing membrane. The lining thickness will be generally 200 to 300mm or more and its application must be carried out in layers with a time lag between layer applications to allow for shotcrete setting and hardening. To ensure a final lining to behave close to monolithic from structural point of view, it is important to limit the time lag between layer applications and ensure that the shotcrete surface to which the next layer is applied is clean and free of any dust or dirt films that could create a de-bonding feature between the individual layers. It is typical to limit the application time lag between the layers to 24 hours. Shotcrete final linings are applied onto a carrier system that is composed of lattice girders and welded wire fabric mounted to lattice girders toward the waterproofing membrane side. This carrier system also acts fully or partially as structural reinforcement of the finished lining. The remainder of the required structural reinforcing may be accomplished by rebars or mats or by steel or plastic fibers.

Unlike the hydrostatic pressure of cast-in-place concrete during installation the shotcrete application does not develop pressures against the waterproofing membrane and the initial lining and therefore one must ensure that any gaps between water proofing system and initial shotcrete lining and final shotcrete lining be filled with contact grout. To ensure a proper grouting around the entire lining circumference it is customary to use longitudinal grout hoses arranged radially around the perimeter.

General trends in tunneling indicate that the application of shotcrete for final linings presents a viable alternative to traditional cast-in-place concrete construction. The shotcrete lining system fulfills cast-in-place concrete structural requirements. Design and engineering, as well as application procedures, can be planned such as to provide a high quality product. Excellence is needed

in the application itself and must go hand-in-hand with quality assurance during application.

4. Design of tunnel lining: IS:15026-2002 (Reaffirmed-2012) gives the formula for calculating the capacity of shotcrete lining or Steel Fiber Reinforced shotcrete (SFRS) lining. The load to be taken by the concrete lining, after taking into account the load taken by the rock bolts or other supports, depends type of rock mass as well type of support. Therefore, design of lining is done on case to case basis, by the design consultant, based on the methodology being followed for tunnel construction.

5. Need for Design Consultant: Reliable and sufficient knowledge of geological and geotechnical properties of the ground encountered during tunneling is essential to assess the loads for which tunnel supports are to be designed. Hence detailed geological and geotechnical investigation need to be carried in consultation with construction engineer and design engineers/consultant. For this, it is recommended that Detailed Design Consultant (DDC) is engaged, to provide inputs on design and construction related issues in the beginning as well during execution of work.

6. Initial and Final Support System: Tunnel support systems mainly include initial ground support as well as final/permanent support system.

Initial or Primary support is installed immediately after excavation, within the standup time, to make the tunnel opening safe until permanent support is installed. The initial support may also function as the permanent or final support or as a part of the permanent support system.

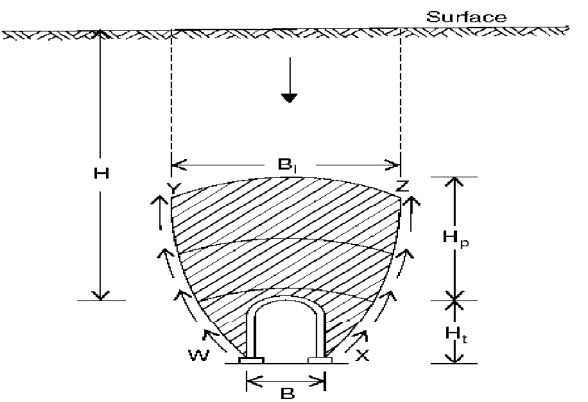
Initial support system is usually not subject to rigorous design, but it is selected on the basis of one or more of three basic approaches (viz. Empirical, Analytical and Numerical) for support design.

CHAPTER-8

EMPIRICAL METHODS OF TUNNEL SUPPORT DESIGN

The Empirical Methods of tunnel support design have evolved based on the extensive experience of various researchers from numerous tunneling projects. The main characteristic of these methods is breakdown of a wide range of rock mass quality by matching each of these categories with various support measures. Moreover, in these support measures, the stages of excavation, presence of underground water and stresses developed are also taken into account. Most commonly used empirical approaches for design of initial support system are covered as under:

1. Terzaghi's Rock Load Factors Terzaghi (1946) defined rock load factor H_p as the height of loosening zone over tunnel roof which is likely to load the steel arches (Fig. 8.01).



$$C = B + H_t$$

Fig. 8.01: Terzaghi's Rock Load Factor

Terzaghi stipulated Rock Load Factor (H_p) based on tunnel width (B) and tunnel height (H_t) for each of the nine rock mass classes defined by Terzaghi *(Ref.: Para-2 and Table-3.02 in Chapter-3)* as given in Table 8.01 below:

Table 8.01: Terzaghi's Rock Load Factors

Rock Class	Rock Load H_p in feet	Remarks
I	Zero	Light lining, required only if spalling or popping occurs.
II (*)	0 to 0.5B	Light support.
III	0 to 0.25B	Load may change erratically from point to point.
IV	0.25B to 0.35C	No side pressure.
V	(0.35 to 1.10) C	Little or no side pressures.
VI	1.10 C	Considerable side pressure. Softening effect of seepage toward bottom of tunnel requires either continuous support for lower end of ribs or circular ribs.
VII	(1.10 to 2.10) C	Heavy side pressure, invert struts required.
VIII	(2.10 to 4.50) C	Circular ribs are recommended.
IX	Up to 250 feet irrespective of value of C	Circular ribs required. In extreme cases, use yielding support.

Note: 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 IV to VI can be reduced by 50%.

(*) Some of the most common rock formations contain layers of shale. In un-weathered 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 the tunnel like squeezing or even swelling 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 rock is likely to reduce very considerably the capacity of the rock located above the roof of bridge. Hence in such rock formations the roof pressure may be as heavy as in a very blocky and seamy rock.

Terzaghi's rock load estimates were derived from an experience of tunnels excavated by blasting methods and supported by steel ribs or timbers. Ground disturbance and loosening occur due to the blasting prior to installation of initial ground support, and the timber blocking used with ribs permits some displacement of the rock mass. As such, Terzaghi's rock loads generally should not be used in conjunction with methods of excavation and support that tend to minimize rock mass disturbance and loosening, such as excavation with TBM and immediate ground support using shotcrete and rock bolts. Moreover, these estimates were not found to be reliable for tunnel wider than 6m.

2. Modified Terzaghi's Theory: Singh et al. (1995) compared support pressure measured with estimated pressure from Terzaghi's rock load theory and found that support pressure in rock tunnels does not increase directly with excavation size (as assumed by Terzaghi). They recommended vertical and horizontal support pressure (p_v and p_h) for various rock mass conditions, are given in Table 8.02 below:

Table 8.02: Clarification of Singh et al. (1995)

Category	Rock Condition	Recommended support pressure (MPa)		Remarks
		p_v	p_h	
I	Hard and Intact	0	0	-
II	Hard stratified or schistose	0.04-0.07	0	-
III	Massive, moderately jointed	0.0-0.04	0	-
IV	Moderately blocky seamy very jointed	0.04-0.1	0-0.2p_v	Inverts may be required
V	Very blocky and seamy, shattered highly jointed, thin shear zone or fault	0.1-0.2	0-0.5p_v	Inverts may be required, arched roof preferred
VI	Completely crushed but chemically unaltered, thick shear and fault zone	0.2-0.3	0.3-1.0p_v	Inverts essential, arched roof essential
VII	Squeezing Rock Condition			
VIIA	Mild Squeezing (u_a/a up to 3%)	0.3-0.4	Depends on primary stress values, p_h may exceed p_v	Inverts essential. In excavation flexible support preferred. Circular section with struts recommended
VIIB	Mild Squeezing (u_a/a = 3 to 5%)	0.4-0.6		
VIIC	High Squeezing (u_a/a > 5%)			
VIII	Swelling Rock			
VIIIA	Mild Swelling	0.3-0.8	Depends on type and content of swelling clays, p_h may exceed p_v	Inverts essential in excavation, arched roof essential.
VIIIB	Moderate Swelling	0.8-1.4		
VIIIC	High Swelling	1.4-2.0		

Notations:

p_v = Vertical support pressure, ph = Horizontal support pressure,

u_a = Radial tunnel closure, a = B/2,

Thin shear zone = Shear Zone up to 2m thick

3. Supports based on Rock Quality Designation (RQD): Deere et al. (1970) modified Terzaghi's classification system by introducing the RQD *(Ref.: Para-1 in Chapter-3).* They also distinguish between blasted and machine excavated tunnels, and proposed support system for 6 to 12m diameter rock tunnels, as given in Table 8.03 below:

Table 8.03: Support

Rock Quality	Tunneling Method	Steel sets[3]	Rockbolts[3]	Shotcrete
Excellent[1] RQD>90	TBM	None to occasional light set. Rock load (0.0–0.2)B	None to occasional	None to occasional local application
	D&B	None to occasional light set. Rock load (0.0–0.3)B	None to occasional	None to occasional local application 2 to 3 inch
Good[1] 75<RQD<90	TBM	Occasional light sets to pattern on 5 to 6 ft. center. Rock load (0.0 to 0.4)B	Occasional to pattern, 5 to 6 ft. centers	None to occasional local application 2 to 3 inch
	D&B	Light sets 5 to 6 ft. center. Rock load (0.3 to 0.6)B	Pattern, 5 to 6 ft. centers	Occasional local application 2 to 3 inch
Fair 50<RQD<75	TBM	Light to medium sets, 5 to 6 ft. center. Rock load (0.4 to 1.0)B	Pattern, 4 to 6 ft. centers	2 to 4 inch crown
	D&B	Light to medium sets, 4 to 5 ft. center. Rock load (0.6 to 1.3)B	Pattern, 3 to 5 ft. centers	4 inch or more crown and sides

Fair 50<RQD<75	TBM	Light to medium sets, 5 to 6 ft. center. Rock load (0.4 to 1.0)B	Pattern, 4 to 6 ft. centers	2 to 4 inch crown
	D&B	Light to medium sets, 4 to 5 ft. center. Rock load (0.6 to 1.3)B	Pattern, 3 to 5 ft. centers	4 inch or more crown and sides
Poor[2] 25<RQD<50	TBM	Medium circular sets on 3 to 4 ft. center. Rock load (1.0 to 1.6) B	Pattern, 3 to 5 ft. center	4 to 6 inch on crown and sides. Combine with bolts.
	D&B	Medium to heavy circular sets, 2 to 4 ft. center. Rock load (1.3 to 2.0)B	Pattern, 2 to 4 ft. center	6 inch or more on crown and sides. Combine with bolts.
Very poor[3] RQD<25 *(Excluding squeezing or swelling ground)*	TBM	Medium to heavy circular sets on 2 ft. center. Rock load (1.6 to 2.2)B	Pattern, 2 to 3 ft. center	6 inch or more on whole selection. Combine with medium sets.
	D&B	Heavy circular sets on 2 ft. center. Rock load (1.6 to 2.2)B	Pattern, 3 ft. center	6 inch or more on whole selection. Combine with medium sets.
Very poor[3] *(squeezing or swelling)*	TBM	Very heavy circular sets on 2 ft. center. Rock load up to 250 ft.	Pattern, 2 to 3 ft. center	6 inch or more on whole section. Combine with heavy sets.
	D&B	Very heavy circular sets on 2 ft. center. Rock load up to 250 ft.	Pattern, 2 to 3 ft. center	6 in. or more on whole section. Combine with heavy sets.

Notes:

1 In good and excellent rock, the support requirement will be, in general, minimal but will be dependent upon joint geometry, tunnel diameter and relative orientations of joints & tunnel.

2 Lagging requirements will usually be zero in excellent and will range from up to 25 percent in good rock to 100 percent in very poor rock.

3 Mesh requirements usually will be zero in excellent rock and will range from occasional mesh (or strip) in good rock to 100 percent mesh in very poor rock.

4 B = tunnel width.

TBM: Tunnel Boring Machine

D&B: Drilling and Blasting

4. Supports based on RMR Classification: Bieniawski (1989) provided guidelines for selection of tunnel supports based on the RMR classification, as given in Table 8.04 below:

4.1 Support pressure based on RMR value: Using measured support values from 30 instrumented Indian tunnels, Goel and Jethwa (1991) have proposed following equation for estimating the short-term support pressure for underground openings in both squeezing and non-squeezing ground conditions in the case of tunnelling by conventional blasting method using steel rib support (but not in rock burst condition):

$$p_v = (7.5\ B^{0.1}\ H^{0.5} - RMR)/\ (20\ RMR)$$

Where:

B = span of opening (in m),

H = overburden or tunnel depth (in m) 50-600m,

p_v = short-term roof pressure (in MPa), and

RMR = post-excavation rock mass rating just before supporting

Table 8.04: Supports based on RMR Classification

Rock mass class	Excavation	Rock bolts (20 mm dia, fully grouted)	Shotcrete	Steel sets
I – Very good rock	Full face 3 m advance	Generally no support required except spot bolting		
II – Good rock	Full face, 1-1.5m advance. Complete support 20 m from face	Locally, bolts in crown 3m long, spaced 2.5m with occasional wire mesh	50mm in crown where required	None
III – Fair rock	Top heading and bench 1.5-3m advance in top heading. Commence support after each blast. Complete support 10m from face.	Systematic bolts 4m long, spaced 1.5-2m in crown and walls with wire mesh in crown.	50-100mm in crown and 30mm in sides	None
IV – Poor rock	Top heading and bench 1.0-1.5m advance in top heading. Install support concurrently with excavation, 10m from face.	Systematic bolts 4-5m long, spaced 1-1.5m in crown and walls with wire mesh.	100-150mm in crown and 100mm in sides	Light to medium ribs spaced 1.5m where required
V – Very poor rock	Multiple drifts. 0.5-1.5m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6m long, spaced 1-1.5m in crown and walls with wire mesh. Bolt invert.	150-200mm in crown, 150mm in sides, and 50mm on face	Medium to heavy ribs spaced steel lagging and forepoling if required. Close invert.
Shape: Horseshoe; Width: 10m; Vertical stress < 25 MPa; Construction: Drilling & blasting.				

Note:

(i) The recommendations are applicable only to tunnels excavated with conventional drilling and blasting method.

(ii) The support measures recommended above are permanent and not temporary or primary supports.

(iii) Double accounting for a parameter should not be done in the analysis of rock structures and in estimating the rating of a rock mass. For example, if pore water pressure is being considered in the analysis of rock structures, it should not be accounted for in RMR. Similarly, if orientation of joint sets is considered in stability analysis of rock slopes, the same should not be accounted for in RMR.

5. Standup time v/s Unsupported span: For with arched roof, Bieniawski (1989) gave correlation between standup time and unsupported span, for various RMR values (Fig. 8.02).

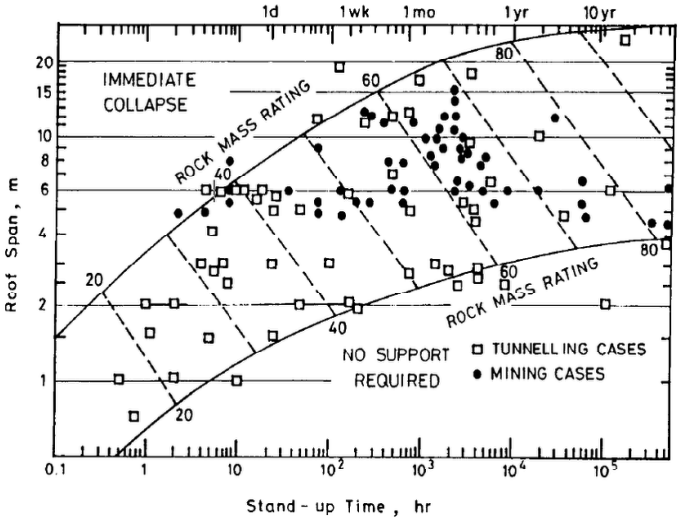


Fig. 8.02: Standup Time v/s Unsupported Span

Note:

(i) There should not be unnecessarily delay in supporting the roof in case of a rock mass with high standup time as this may lead to deterioration in the rock mass which ultimately reduces the standup time.

(ii) Standup time improves by one class of RMR value in case of excavations by TBM (Lauffer, 1988).

(iii) RMR system is found to be unreliable in very poor rock masses. Care should therefore be exercised to apply the RMR system in such rock mass. Q system is more reliable for tunnelling in the weak rock masses.

6. Supports based on Q System of rock mass classification: Estimated support categories based on the Rock mass quality Q, as given by Norwegian Geotechnical Institute (NGI), are indicated in the nomogram in Fig. 8.03. Values of ESR (Excavation Support Ratio), for various types of excavation are given in Table-7.02 in Chapter-7. The explanatory notes for the fig 8.03 are given in fig 8.04

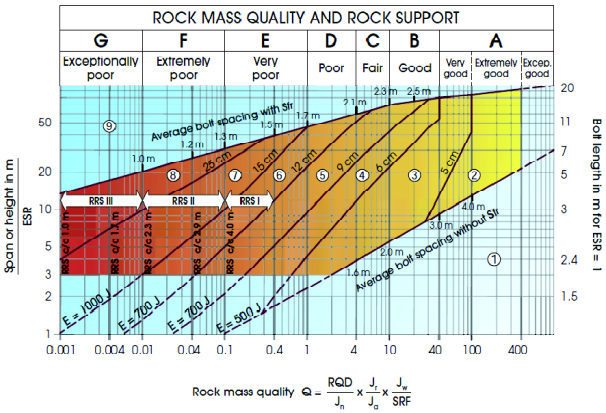


Fig. 8.03: Supports as per Q System

Support Categories

1. Unsupported or spot bolting
2. Spot bolting, **SB**
3. Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B + Sfr**
4. Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500) + B**
5. Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700) + B**
6. Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E700) + RRS I + B**
7. Fibre reinforced sprayed concrete > 15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000) + RRS II + B**
8. Cast concrete lining, **CCA or Sfr (E1000) + RRS III + B**
9. Special evaluation

Fig. 8.04: Explanatory Notes for Fig. 8.03

"E" is energy Absorbed in the Fibre reinforced Sprayed Concrete (in Unit Joules) and it indicates the thickness of concrete (E=500J, E=700J and E=1000J means concrete thickness of 6-9 cm, 9-12 cm and >15 cm respectively).

6.1 Estimating support requirement: In Fig. 8.02, draw a vertical line from the given value of Q and draw a horizontal line for the given value of "effective span D_e" (Span or Length/ESR). Find out the zone where these two lines intersect (from Zone 1 to 9). The support requirement for each zone in given in Fig. 8.04.

If systematic bolting is stipulated, in Fig. 8.03, then find out the spacing by extending the vertical line upwards (in case of bolting with fiber reinforced sprayed concrete) or downwards (in case of bolting without fiber reinforced sprayed concrete) till it cuts the upper or lower envelope line. Read the bolts spacing value at this intersection point.

If fiber reinforced sprayed concrete is stipulated, in Fig. 8.03, the range of concrete thickness at the zone boundaries can be read from Fig. 8.03 or Fig. 8.04. Exact value of the concrete thickness can be estimated by interpolating the concrete thickness values at the zone boundaries, based on location of the intersection point of vertical and horizontal lines between the zone boundaries.

In case the intersection point of vertical and horizontal lines falls in Zone-6, 7 or 8 (weak rock mass with low Q values), then support system RRS (Rib Reinforced Shotcrete) of Type-I, II or III is to be provided. When Q values are below 0.1, it can be expected that there will be possibility of large over-break, low standup time and significant early deformations. The use of steel sets should be avoided in such situations, due to the actual relatively large rock-block loosening that they allow, unless followed immediately by bolting or shotcrete, or both. It is for this category of problems that RRS has been developed (Fig. 8.05).

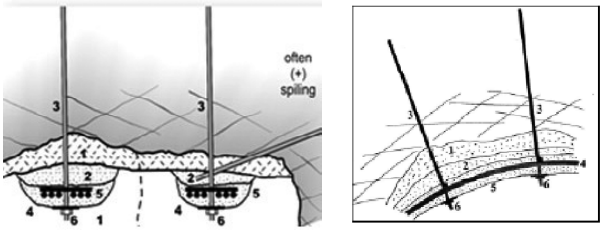


Fig. 8.05: Rib Reinforced Shotcrete

The process used for building up RRS is: (1) Spray first layer of fiber reinforced shotcrete, (2) Buildup local, smooth but not necessarily circular arch or arches of fiber reinforced shotcrete, (3) Drill bolt holes and install end anchored bolts with pre-fabricated welded cross bars, (4) Attach (wire and weld) reinforcing bar steel arches to each bolt head cross-bar (pre-fabricate in bundles for easier attachment). These bars can be bent into over-break zone, therefore requiring less shotcrete volumes than with lattice girder, (5) Spray fiber reinforced shotcrete over reinforcing bars to complete arch, (6) Bolts and washers tensioned (bolt thread protected with plastic caps), and (7) Spray fiber reinforced shotcrete over bolt head to complete the arch.

The requirements of RRS for the given zone are given in Fig. 8.04, in terms of single/double layer of rebars, numbers of rebars thickness of shotcrete, diameter of rebars etc., for any given span. For example, "D40/ 6+2 Ø 16-20 (Span 20m)" in RRS-I means "40cm sprayed concrete with Double layer of rebars, 6 and 2 nos. rebars in each layer, rebar dia. 16 to 20mm and this arrangement is suitable for span of 20m".

For road tunnel of 10m dia, the ESR value is 1.0 and, therefore, the effective span (D_e) will be 10m. For water tunnel of 10m dia, the ESR value is 1.6 and, therefore, the effective span (D_e) will be 10/1.6 = 6.3m. For these tunnels, the support requirements for some given value of Q have been worked out using Fig. 8.03 and 8.04:

Table 8.05: Estimation of support requirements

D_e	Roof Support requirement when		
	Q = 20	Q = 2	Q = 0.2
10m	Systematic bolting 2.5m x 2.5m 5.5 cm shotcrete	Systematic bolting 1.9m x 1.9m, 7 cm shotcrete	Systematic bolting 1.4m x 1.4m, 14 cm shotcrete
6.3m	No support	Systematic bolting 1.9mx1.9m, 5 cm shotcrete	Systematic bolting 1.4m x 1.4m, 11 cm shotcrete

6.2 Length of Rock bolts: The length of rock bolts can be estimated from the following formulae *(Barton et al., 1974):*

$$L = 2 + (0.15B/ESR)$$

where:

L - Length of rock bolts
B - Excavation width
ESR - Excavation Support Ratio

6.3 Maximum unsupported span: The maximum unsupported span can be estimated from following formulae:

Maximum Unsupported Span = 2 ESR $Q^{0.4}$

7. Elements of commonly used Excavation and Support Classes (ESC) in Rock and Soft Ground, as given by Hung et al. (2009) are tabulated in Appendix-8.1 and Appendix-8.2 respectively.

8. Example of estimating support requirements: Design tunnel supports for a road tunnel by various empirical approaches, for a tunnel with following details:

- Dia. of Tunnel:
  By Drill and Blast - 7.4m + 0.6m (Over break) = 8m
  By TBM = 7.4m
- Located 20m below ground surface
- Rock : Shale
- γ = 2.66 t/m^3
- RQD : 55 – 85% (Weighted Average = 72%)
- UCS = 45 MPa
- Joint Spacing : 50 – 900 mm
- 0.8-1.1 mm Separation, Slightly Weathered, Rough
- Large inflow or High Water Pressure
- Strike perpendicular to Tunnel Axis, Dip 20^0
- Two Joint Sets, Random
- Rough Planar Joints
- Unaltered Joint Walls, Surface Staining
- Medium Stress σ_c / σ_1= 45/(0.027*20) = 83

(A) <u>Based on RQD</u>
RQD 72%

<u>(Ref. Para-3 and Table 8.03 above)</u>

For TBM –

Light to Medium Steel Sets @5 to 6' (Say 1.5m c/c)

Rock bolts @ 4 to 6' (Say 1.2m c/c)

Shotcrete – 2 to 4' (Say 100mm) thick in Crown

For D&B –

Light to Medium Steel Sets @4 to 5' (Say 1.2m c/c)

Rock bolts @ 3 to 5' (Say 0.9m c/c)

Shotcrete – 4' Say 100mm) thick in Crown and Sides

(B) Based on RMR Classification

Calculation of RMR Value: *(Ref. Appendix 3.1 in Chapter-3)*

Sl. No.	Parameter	Rating
(i)	UCS = 45 MPa	4
(ii)	RQD = 72%	13
(iii)	Discontinuities spacing 50-900 mm	10
(iv)	Condition: 0.8-1.1mm Separation, Slightly Weathered, Rough	25
(v)	Ground water Inflow: Large Inflow, High Water Pressure	0
	Total = Basic RMR =	52
(vi)	Discontinuity Orientation: Strike perpendicular to tunnel Axis, Dip 20^0 (Favourable)	-2
	Total = RMR =	50

Class – III "Fair" Rock Mass

(Ref. Para-4 & Table 8.04 above):

For Class-III "Fair" Rock mass

By Top Heading and Benching.

1.5 to 3m advance in top heading.

Commence support after each blast.

Complete support 10m from face.

Rock bolts 4m Long, @1.5-2m c/c, in crown and Walls with Wire mesh in Crown

Shotcrete 50-100mm (Crown) and 30mm (Sides)

No Steel sets required.

(Ref. Para-5 & Fig. 8.02 above):

For RMR=50 and Roof Span= 8m

Standup time = 70 Hours

(C) Based on Q System

Calculation of Q Value: *(Ref. Appendix 3.2 in Chapter-3)*

Sl. No.	Parameter	Value of
(i)	RQD = 72%	-
(ii)	Two Joint Sets, Random	$J_n = 6$
(iii)	Rough, Planar Joints	$J_r = 1.5$
(iv)	Unaltered Joint Walls, Surface Staining	$J_a = 1.0$
(v)	Large inflow or High Water Pressure	$J_w = 0.5$
(vi)	Medium Stress $\sigma_c / \sigma_1 = 45/(0.027*20) = 83$	SRF = 1.0

$$Q = (RQD/J_n) * (J_r/J_a) * (J_w/SRF)$$
$$= (72/6) * (1.5/1.0) * (0.5/1.0) = 9$$

"Fair" Rock Mass – Group "2"

(Ref. Para-6 and Fig. 8.02 & 8.03 above)

For Road tunnel, ESR = 1.0 *(Ref. Table 7.02 in Chapter-7)*

Hence, De = 8.0/1.0 = 8.0

For D_e = 8.0m and Q = 9 *(From Fig. 8.03)* - Falls in Zone-3

For Zone-3 *(From Fig. 8.04)*

Systematic Rock bolts & Fiber reinforced shotcrete is required.

Bolt spacing *(From Fig. 8.03)* @ 2.3 m c/c

Shotcrete thickness *(From Fig. 8.03)* 5.5 cm.

Length of Rock bolt: *(Ref. Para 6.2 above)*

L = 2 + (0.15B/ESR) = 2 + (0.15*8/1.0) = 3.2 m

Maximum unsupported span: *(Ref. Para 6.3 above)*

$= 2\ ESR\ Q^{0.4} = 2 * 1.0 * 9^{0.4} = 4.8m$

9. Limitations of empirical approach: Following are the limitations of the empirical approach of support design:

(i) These methods provide ground support scheme based on parameters that can be determined from explorations, observations and testing. They are far from perfect and can sometimes lead to the selection of inadequate ground support. It is, therefore, necessary to examine the available rock mass information to determine if there are any applicable failure modes not addressed by the empirical systems.

(ii) These methods lead the user directly from the geologic characterization of the rock mass to a recommended ground support without the consideration of possible failure modes. Potential modes of failure are not covered by some or all of the empirical methods and must be considered independently, including the following:

- Failure due to weathering or deterioration of the rock mass.
- Failure caused by moving water (erosion, dissolution, excessive leakage, etc.).
- Failure due to corrosion of ground support components.
- Failure due to squeezing and swelling conditions.
- Failure due to overstress in massive rock.

(iii) These methods are largely based on blasted tunnels. System recommendations should be reinterpreted based on current methods of excavation. Similarly, new ground support methods and components must be considered.

Appendix- 8.1

Elements of commonly used Excavation and Support Classes (ESC) in Rock

Ground Mass Quality - Rock	Excavation Sequence	Rock Reinforcement	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation influences progress
Intact Rock	Full face or large top heading & bench	Spot bolting (fully grouted dowels, Swellex)	Patches to seal surface in localized fractured areas	Typically several rounds behind face or directly near face to secure isolated blocks, slabs/wedges	None	No
Stratified Rock	Top heading & bench	Systematic doweling or bolting in crown considering strata orientation (fully grouted dowels, swellex, rock bolts)	Fiber reinforced, typically 100mm to bridge between rock reinforcement in top heading, alternatively chain link mesh; installed with rock reinforcement.	Two to three rounds behind face	None	No or eventually
Moderately Jointed Rock	Top heading & bench	Systematic doweling or bolting in top heading considering joint spacing (fully grouted dowels, swellex, rock bolts)	Systematic shell with reinforcement (welded wire fabric or fibers) in top heading and potentially bench; dependent on tunnel size thickness of 150 to 200mm; installed with the rock reinforcement.	One to two rounds behind face	Locally to limit over break	Yes

Ground Mass Quality - Rock	Excavation Sequence	Rock Reinforcement	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation influences progress
Blocky and seamy Rock	Top heading & bench	Systematic doweling or bolting in top heading & bench considering joint spacing	Systematic shell with reinforcement (welded wire fabric or fibers) in top heading and bench; depending on tunnel size thickness 200 to 300mm.	At the face or maximum one round behind face	Systematic spiling in tunnel roof or parts of it	Yes
Crushed but chemically intact rock	Top heading, bench, invert	N/A	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 300 mm and more; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required	After each round	Systematic grouted pipe spiling or pipe arch canopy	Support installation dictates progress
Squeezing Rock	Top heading, bench, invert	Systematic doweling or bolting in top heading &bench considering joint spacing; extended length	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 300mm and more; potential use for yield elements; for initial stabilization and to	After each round	Systematic grouted pipe spiling or pipe arch canopy	Support installation dictates progress

Ground Mass Quality - Rock	Excavation Sequence	Rock Reinforcement	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation influences progress
			prevent desiccation, a layer of flashcrete may be required			
Swelling rock	Top heading, bench, invert	Systematic doweling or bolting in top heading & bench considering joint spacing; extended length	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 300mm and more; potential use for yield elements	After each round	Systematic grouted pipe spiling or pipe arch canopy may be required depending on degree of fracturing	Support installation dictates progress

Appendix-8.2

Elements of Commonly Used Excavation and Support Classes (ESC) in Soft Ground

Ground Mass Quality - Soil	Excavation Sequence	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation	Remarks
Stiff/hard cohesive soil above groundwater table	Top heading, bench & invert; dependent on tunnel size, further sub-divisions into drifts may be required	Systematic reinforced (welded wire fabric or fibers) shell with full ring closure in invert; dependent on tunnel size 6 in. (150 mm) to 16 in. (400 mm) typical, for initial stabilization and to prevent desiccation, a layer of flashcrete may be required	Installation of shotcrete support immediately after excavation in each round. Early support ring closure required. Either temporary ring closure (e.g. temporary top heading invert) or final ring closure to be installed within one tunnel diameter behind excavation face	Typically none; locally spiling to limit over-break	Support installation dictates progress	Overall sufficient standup time to install support without pre-support or ground modification
Stiff/hard cohesive soil – above groundwater table	Top heading, bench & invert; dependent	Systematic reinforced (welded wire fabric or fibers) shell with full ring	Installation of shotcrete support immediately after excavation in each round. Early support	Typically none; locally pre-spiling to limit over-break	Support installation dictates progress	Sufficient standup time to install support without pre-

Ground Mass Quality - Soil	Excavation Sequence	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation	Remarks
	on ground strength, smaller drifts required than above	closure in invert; dependent on tunnel size 6 in. (150 mm) to 16 in. (400 mm) typical, for initial stabilization and to prevent desiccation, a layer of flashcrete may be required; frequently more invert curvature than above	ring closure required. Either temporary ring closure (e.g. temporary top heading invert) or final ring closure to be installed within less than one tunnel diameter behind excavation face; typically earlier ring closure required than above			support or ground improvement; dependent on water saturation, swelling or squeezing can occur
Well consolidated non-cohesive soil – above groundwater table	Top heading, bench &invert; dependent on tunnel size, further sub-divisions into drifts	Systematic reinforced (welded wire fabric or fibers) shell with full ring closure in invert; dependent on tunnel size 6 in. (150 mm) to 16 in. (400 mm)	Installation of shotcrete support immediately after excavation in each round. Early support ring closure required. Either temporary ring closure (e.g. temporary top	Frequently systematic pre-support required by ground pipe spiling or ground pipe arch canopy, alternatively	Support installation dictates progress	Stand-up time insufficient to safely install support without pre-support or ground improvement

Ground Mass Quality - Soil	Excavation Sequence	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation	Remarks
	may be required	typical, for initial stabilization and to prevent desiccation, a layer of flashcrete is required	heading invert) or final ring closure to be installed within less than one tunnel diameter behind excavation face	ground improvement		

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CHAPTER-9

ANALYTICAL AND NUMERICAL METHODS OF TUNNEL SUPPORT DESIGN

1. Analytical Methods: The support pressures, moments, displacements etc. can be estimated by using closed form structural analysis methods, with dimensions of tunnel, engineering properties of rock mass and other boundary conditions as inputs. Carrying out such analysis manually is quite time consuming and, therefore, many custom made software for tunnel analysis/design can be used for this purpose. Described below are some such software, but this is not complete or exhaustive list of such software.

1.1 Examine (formerly Examine^{3D}) is software for two/three dimensional engineering analysis of underground excavations in rock. It generates geometry and boundary element discretization for underground openings; computes stresses and displacements using the direct boundary element method; and visualizes the analysis results. Using this program, the stresses and deformations can be calculated at any location and can be presented in various formats.

1.2 RocSupport is software tool for estimating deformation in circular or near circular excavations in weak rock and visualization of the tunnel interaction with various support systems. It can be used as a tool for the preliminary design of tunnels and support systems. It is not applicable for excavations in hard rock, where failure is controlled by structural discontinuities or brittle failure.

2. Numerical Methods: For tunnels, it is not possible to carry out engineering analysis of the underground excavation due to following reasons:

(i) Rocks may behave in complex ways (e.g. Elastic, Elasto-plastic/Visco-plastic, strain Softening etc.). Analyzing all such behaviour is not possible by normal engineering analysis.

(ii) Rock mass is non-homogeneous and anisotropic, difficult to analyze by normal engineering analysis.

(iii) Complex boundary conditions.

(iv) Difference in behaviour of intact rock vis-a-vis rock mass.

(v) It is difficult to account for excavation sequence in normal engineering analysis.

In such cases, numerical methods are used wherein the undergrounds excavations are analyzed by constructing models catering to relevant loading conditions, boundary conditions and engineering properties of the rock mass. The Numerical methods use some of the following techniques:

- Finite Difference Method (FDM)
- Finite Element Method (FEM)
- Boundary Element Method (BEM)
- Coupled Finite Element Boundary Element Method (FEBEM)
- Distinct Element Method (DEM)

Out of the above techniques, the Finite Element Method (FEM) is most commonly used technique. Listed below are some of the FEM based software for analysis and design of tunnels, but this is not complete or exhaustive list of such software:

- ABAQUS
- Plaxis 2D and Plaxis 3D

- RS2 (Formerly Phase^2) and RS3 (Formerly RS^3)
- MIDAS GTS
- SOFiSTiK
- The FEM – Tunnel (GEO5)

The numerical methods always provide approximate solutions. Hence, these results need to be validated by recording actual displacements/stresses etc. by instrumentation and constitutive model needs to be fine-tuned accordingly.

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CHAPTER-10

TUNNEL EXCAVATION METHODS

1. Selection of Tunnel Excavation Method depends on various factors, such as:

(i) Geological conditions

(ii) Cross Sectional area and Shape of tunnel

(iii) Length of tunnel

(iv) Ground water condition and expected water inflow

(v) Vibration restrictions

(vi) Allowable ground settlements

(vii) Availability of resources (machinery/equipment, funds & time)

Commonly used Tunnel excavation methods can be grouped under two categories:

(a) Excavation Methods for Rock Tunnels

(b) Excavation Methods for Soft Ground Tunnels

2. Excavation Methods for Rock Tunnels: Three commonly used excavation methods for rock tunnels are:

(i) Drill and Blast (Full Face or Partial Face Excavation)

(ii) Road Header (Full Face or Partial Face Excavation)

(iii) Mechanized Tunnelling

3. Drill and Blast Method: The Drill and Blast (D&B) method is the most common method for medium to hard rock conditions. Some of its major advantages are fast start-up and relatively low capital cost for the equipment. On the other hand, the cyclic nature of the drill and blast method requires good work site organization. This excavation method has been used

throughout the world for a long time and still remains the conventional method for rock tunnels.

This tunneling method requires use of explosives. Compared with tunneling by Tunnel Boring Machine, this generally results in higher duration of vibration levels and noise, which restricts use of this method in urban areas. The excavation rate is also less than TBM (usually 3 to 5m per day).

Drill and blast method is best suited for medium to hard rocks hard rocks and relatively short tunnels (where use of TBM/Road header is uneconomical) and can also be used when encountering too great a variety of geologies or other specific conditions such as mixed face, squeezing ground, etc. It is suited to any type of tunnel cross section. Appropriate controlled blasting technique needs to be implemented at site to reduce over breaks and minimize damage outside the minimum excavation line.

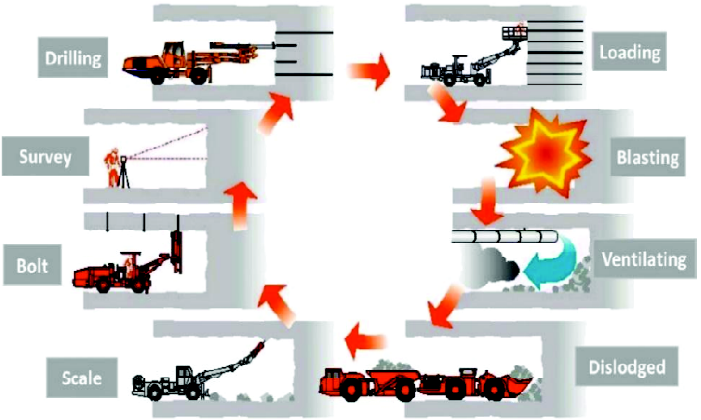


Fig. 10.01: Drill and Blast Cycle

Various activities involved in a cycle of drill and blast method are shown in Fig. 10.01 and they are discussed in following paras.

3.1 Drilling: Based on the blasting plan, determined in advance, holes are drilled in the foremost front wall of the tunnel (working face). The holes can be drilled manually (Fig. 10.02) or by using a Jumbo (Fig. 10.03). The Jumbo is also known by name Boomer.

Fig. 10.02: Manual

Fig.10.03: Jumbo drilling

Now-a-days most of the drilling work is done using jumbos having multiple drilling arms and an operator tower. It is run by electric cable, a hose brings water to the drills and drills are pneumatic. That means that the drill bits both hammer and rotate. Broken bits of rock are flushed out by water. The length of drill holes depends upon the length to be blasted in one cycle (called as Step, Advance, Pull etc.) which typically varies from 0.8 to 3.5m depending on the type of rock, the method used for tunnelling (i.e. full face or partial face blasting) and support system used.

The drilling pattern to be adopted shall ensure minimum over breaks and shall consume least quantity of explosives. The drilling pattern depends upon the texture and formation of rock which determine rock drillability & blastability, size/shape of tunnel, drilling equipment used, type of explosives & means of detonation available, expected water leaks, blast vibration restrictions and accuracy requirements of the blasted wall etc.

Following are the most commonly used drilling patterns for tunnels:

(i) Wedge Cut or "V" Cut Drilling Pattern: In this pattern, horizontal cut holes are driven around the middle of the face, in an inclined angle of 60^0 to the face towards the centre. Maximum explosives charge concentration is required at the apex end of the blast holes as they are to be blasted at the first instant for creation of the free face.

(ii) Cone/Pyramid/Diamond Cut Drilling Pattern: Four or six holes are cut at the middle of the face, which converge at the end, to form either a cone or pyramid or diamond shape. This cut is suitable for laminated rocks, sedimentary in nature. It also helps in drivage of smaller cross-sectional area tunnels so as to break the rock along the cleavage planes.

(iii) Burn (Parallel) Cut Drilling Pattern: The burn cut holes are drilled parallel to the tunnel advance and perpendicular to the face of the tunnel. Some of the drilled holes (mostly in middle section of the face) are holes of large diameter (say 100mm) or a set of closely spaced holes of conventional diameter (46 to 56mm) and are left as dummy holes without any explosive charge, so that they act as free face for the heavily charged blast holes around. Specific geometrical relationship between the diameter of dummy holes and spacing in between dummy holes and charged blast holes is required to be maintained for the given rock in order to create the free face.

With Burn or Parallel cut, possible advance per round is longer as compared to that with Angular cut holes. Success of Burn or Parallel Cut depends upon accuracy in keeping the holes parallel. This requirement and requirement of holes of large diameter or a set of closely spaced holes of conventional diameter calls for use of drilling

Jumbos.

Drilling patterns can be decided manually, but advanced computer programs are available, which make it easier to modify the patterns and predict fairly accurately the effects of changes in drilling, charging, loading and production.

3.2 Loading: During this activity, also known as charging, the drill holes are now filled with explosives, detonators are attached to the explosive devices and the individual explosive devices are connected to one another (Fig. 10.04).

Fig. 10.04: Loading

3.2.1 Choice of Explosives: The best choices are emulsion explosives, produced by intimate and homogeneous mix of oxidizer and fuel. Basically emulsion explosives consist of micro droplets of super saturated oxidizer solution in oil matrix. They are in the form of water-in-oil emulsion. Various advantages associated with the use of emulsion explosives are as below:

(i) Emulsion explosives are much better water resistant than water gel slurry or ANFO (Ammonium Nitrate and Fuel Oil). This is because the oil-phase envelopes the water phase.

(ii) They are safe to handle, store and use because of their relative insensitivity to detonation by friction, impact or fire.

(iii) Due to the oxidizer droplet size (0.2 to 10 micron) they have higher value of Velocity of Detonation (VOD) which can tackle the toughest rock conditions very effectively and that too without comprising on safety standards.

(iv) Since emulsion explosives are more oxygen balanced they generate minimum noxious fumes and very less smoke. This in-turn can reduce the ventilation time after the blasts and further can shorten the cycle time of operations. Liquid emulsion explosives are pumpable and charging time can be cut down.

3.2.2 Requirement of Explosives: Normally, the consumption of explosives in tunnel blasting is much more than that required for open cut. For road/rail tunnels, the Specific Charge or the Powder Factor is normally 1.2 kg/cum of in-situ rock on an average. However, the amount of explosives required (in kg/cum of in-situ rock) may vary from site to site depending upon the geological strata involved, cross section of the opening and advance length (Fig. 10.05).

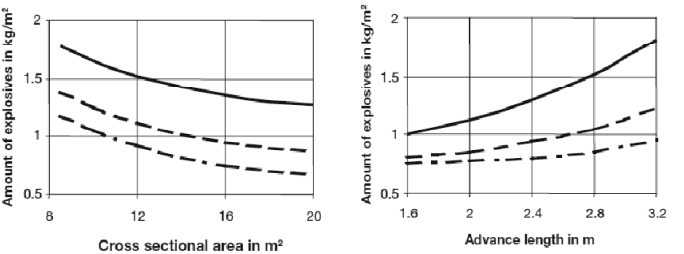


Fig. 10.05: Requirement of Explosives

3.3 Blasting: The holes, charged with explosives, are blasted in a proper sequence, from the center outward, one after the other. Although more than 100 explosions may be set off, one after the other, the blast sequence is completed in several seconds. The devices should not explode at the same time, but rather one after the other at specified intervals. Only when the blast master has ensured that nobody is left in the danger zone, the explosion can be triggered by the blasting machine.

3.3.1 Choice of Initiation System and Selection of Delay Sequence: Selection of delays in initiation system and timing shall be such that the rock volume after swell from subsequent blasts must be accommodated. For this, the fracturing and breaking time of the rock and time required for spreading out cracks in the rocks is to be studied. It, basically, all depends on condition of rock. In hard and brittle rocks, the speed of developing cracks is faster than that in softer rocks. This rock breaking speed can vary from 1 to 3 milliseconds per meter. But the ejection speed of rock after blast may vary from 20-30m per second i.e. 20-30mm per millisecond and it can be understood that for a 4m long blast holes the broken rock takes about 300-400 milliseconds for complete removal from the face. Precisely, for this reason long delays (half second, 300 to 500 milliseconds) are preferred for tunnel blasting. The advantages of delay detonators are:

- Better fragmentation,
- Reduced secondary blasting,
- More uniformity in size of fragmentation,
- Possible to fire more holes in a single blast with less vibrations and noise.

3.3.2 Controlled Blasting is adopted for minimizing over breaks and has following additional advantages:

- Less damage to peripheral rock,

- Reduced requirement of support system,
- Safer tunnel operation in general, and
- Reduced requirement of scaling.

Controlled blasting generally involves closer spacing of contour (perimeter) blast holes which are also called as trimmers. They are charged with fewer explosives than that of the production blast holes. The spacing thumb rule is about 10-12 times of blast hole diameter in hard rocks and about 5-6 times of blast hole diameter in soft rocks.

3.3.3 Air Column Method: In order to reduce the cracking or damage to the strata at the periphery it is advisable to use "Air Column Method" which can minimize the radial vector component of blast induced vibrations. This method consists of inserting into the blast holes an inert spacing device of a length about four times the diameter of the blast hole, prior to charging of first explosive cartridge. This leaves requisite air gap in between the explosive cartridge and the end of the blast hole. With this method, a plain straight tunnel face can be secured for next drilling cycle without any cracks.

3.3.4 Efficiency of Blasting: Efficiency of blasting is routinely assessed by tabulating Pull, Specific Charge, Specific Drilling, Detonator or Hole Factor, Blast-induced damage and Over break/Under breaks against the values assumed during planning.

3.4 Ventilation (De-fuming): Due to blasting, lot of rock fragments are flung around, dispersing clouds of dust which gets mixed with combustion gases of the explosion. To enable resumption of work in the tunnel, the dust and fumes are removed by using air-ducting system, long steel or plastic pipes which are attached to the roof of the tunnel and blow fresh air onto the working face (Fig. 10.06). This gives rise to localized excess pressure and the bad air is pushed towards the tunnel exit.

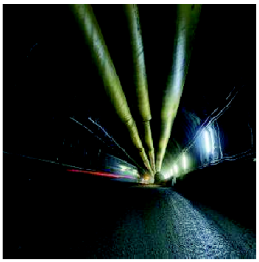


Fig. 10.06: Ventilation

Fig. 10.07: Mucking

3.5 Mucking (removing rubble): The blasted material, rubble or spoil, is carried out of the tunnel, by either loading onto dump trucks with wheel loaders or on conveyer belts (Fig. 10.07).

3.6 Scaling (Dislodging): This refers to stripping away and removal of loose pieces of rock, which were not completely released from the rock during the blasting procedure. This is done using a tunnel excavator or road header with suitable scaling tool attached to it.

3.7 Supporting (Securing): At this stage, the temporary or initial supports are provided. A layer of shotcrete is used very commonly, as it enables a cavity-free connection of support to the rock. Depending on the design of support systems, various types of supports can be implemented. After shotcreting, rock bolts are used very commonly (Fig. 10.08). A jumbo is used to carry out all the operations need for installing rock bolts. If the support system stipulates, the steel ribs or lattice girder are placed in position.

Fig. 10.08: Rock Bolting

3.8 Geological Mapping: The working face is now freely accessible and the geologist has a few minutes to map it. In the process, he determines what type of rock is present and how the rocks lie, i.e. whether they dip in a flat or steep manner, whether they are folded or even broken. At the same time, the strength of the rock, the reaction of the rock mass to the excavation process and any mountain water infiltration are also documented. The mapping report created from this – with sketches and photos – serves as the basis for the selection of appropriate supporting measures.

4. Full Face v/s Partial Face Excavation: A key decision to be taken by engineers is whether to go for "Full Face Excavation" or "Partial Face Excavation".

4.1 Full Face Excavation: Excavating the complete tunnel section in one operation is termed as Full Face Excavation. Wherever practical, this is preferred for higher rate of progress and ease in construction. The decision for excavating full face has to be taken after careful consideration of the geology, the span) of opening and the stand-up time.

4.2 Partial Face Excavation: Large openings or openings in weak rocks are less stable. Therefore, in many cases the tunnel cross section is not

excavated at once, but in parts. This type of excavation is called Partial Face Excavation.

Various types of partial face excavations are as under:

(A) Heading and Benching. This method involves excavation of top portion (called heading or calotte) first and excavation of the bottom portion (called bench) is done only after securing the top heading (Fig. 10.09). With the stand-up time problem eliminated (unless there will be a problem with wall stability), longer incre¬ments of bench can be excavated.

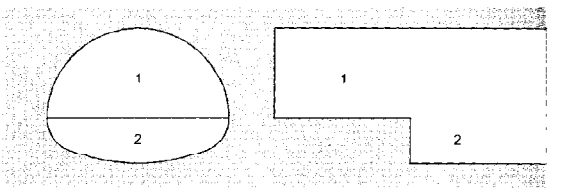


Fig. 10.09

Heading **Benching**

A ramp needs to be constructed for accessing the heading face, if the gap between faces of top heading and bench warrants so. If the heading and the bench are excavated simultaneously, then the ramp must be moved forward every now and then (Fig. 10.10).

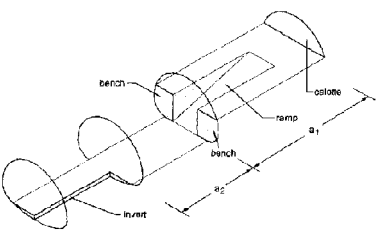


Fig. 10.10: Ramp in Heading and Benching

Installation of rock reinforcement in top heading pre¬sents no special problems. However, when steel ribs are used, the necessity for a full-strength steel rib arch creates a problem at the abutments. The arch must have a temporary foundation while the bench beneath it is excavated. Wall plates are installed longitudinally beneath the ribs at the top heading invert. A wall plate is a horizontal structural steel member placed under the arch to act as an abutment and spread the reaction while the bench is being excavated. For smaller or lightly loaded ribs, the wall plate is a single wide flange beam with its web horizontal; arch and post segments fit inside its flanges. For heavier loads or larger spans, a pair of beams joined together, with webs vertical, is located directly beneath or over the arch/post flanges.

(B) Multiple Drift Advance: If the stand-up time is insufficient for advance by heading and benching method, because of either the geology or large spans, the top heading and/or benches must be divided into two or more drifts. This is advantageous because the reduced span increases stand-up time, the reduced volume decreases mucking time and time required to install support or reinforcement is also reduced. When using steel sets, the appropriate final arch segment is used and supported temporarily on one or more steel posts. When the adjacent drift is excavated, the next arch segment is erected, connected, posted and so on. Once the wall plates are in place and the full arch erected, the temporary posts are removed.

While tunnel excavation from top down is preferred, in exceptionally poor ground it may be necessary to work from the bottom up. Driving bottom sidewall drifts first permits concreting abutments and eliminates need to establish, undercut and re-establish supports.

Some well-established schemes of multiple drift excavation are as below:

Core heading: This is also known as the **German heading method.** It consists of excavating and supporting first the side and top parts of the cross section and subsequently the central part (core) **(Fig. 10.11).** The ring closure at the invert comes at the end. The first gallery also serves for exploration. The crown arch is founded on the side galleries thereby keeping the related settlements small.

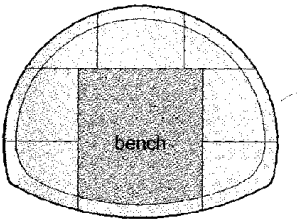


Fig. 10.11: Core Heading

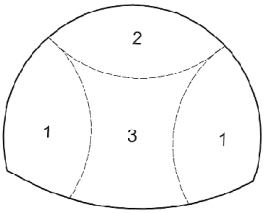


Fig. 10.12: Sidewall Drift

Sidewall drift: The side galleries are excavated and supported first (Fig. 10.12). They serve as abutment for the support of the crown, which is subsequently excavated. This method is preferred in soils/rocks of low strength. Note that a change from top heading to sidewall drift is difficult to accomplish.

In addition to above, many other variations of multiple drift sequence are also in practice.

5. Excavation by Road header: Road header is a self-propelled equipment, moving on wheels or crawler travelling track, consisting of a telescopic boom which can be rotated in any direction, boom mounted cutting head and a loading device usually involving a conveyor (Fig. 10.13).

Fig. 10.13: Road Headers

The cutting head can be a general purpose rotating drum mounted in line or perpendicular to the boom, or can be special function heads such as jack hammer like spikes, compression fracture micro-wheel heads like those on larger tunnel boring machines, a slicer head like a gigantic chain saw for dicing up rock, or simple jaw-like buckets of traditional excavators.

Road headers are used for moderate rock strengths and for laminated or joined rock. The cutter is mounted on an extension arm (boom) of the excavator and cuts the rock into small pieces. Thus, over profiling can be limited and also the loosening of the surrounding rock is widely avoided. Measures against dust (suction or water spraying) have to be provided. The required power of the motors increases with rock strength.

The width of tunnel excavated varies from slightly more than the width of the machine body plus treads to twice that width. Much less heading equipment is required and start-up costs are only a fraction of that for TBM excavation. The compactness, mobility, and relatively

small size of the road header combined with simultaneous mucking makes it practical to install rock bolts and/or shotcrete quickly and easily.

The principal constraint in use of road headers is that they curently are usable only in rock of less than about 80 MPa compressive strength. Somewhat stronger rock can be cut, or chipped away, if it is sufficiently fractured.

This method could also be called partially mechanized tunneling. Whereas TBMs are generally purpose-built, road headers are nearly "off-the-shelf" equipment requiring relatively little lead time. Excavation by road header is suited for any type of tunnel cross sections & may be done either partial face or full face.

6. Mechanized Tunnelling: Mechanized tunneling offers many advantages, some of them are:

(i) Industrialization of the tunneling process, with reductions in costs and construction times.

(ii) Possibility of crossing complex geological and hydrogeological conditions safely and economically.

(iii) Good quality of the finished product.

(iv) Enhanced health and safety conditions for the workforce

However, there are still risks associated with mechanized tunneling, for the choice of technique is often irreversible and it is often impossible to change from the technique first applied or only at the cost of immense upheaval to the design and/or the economics of the project.

The selection of tunnelling technique to use must be made on the basis of the known and suspected ground conditions, in combination with other aspects such as access, experience and skill of the officials/workers, as well as costs. Adaptability of the technique to variability of the ground could also be an important factor.

“Tunneling Shields” & “Tunnel Boring Machines” are the

two principal types of machines employed for mechanized tunneling. But quite often these two terms are used interchangeably as distinction between two is getting blurred.

7. Tunnelling Shields: In its simplest form a tunneling shield is a steel frame with a cutting edge on the forward face (Fig. 10.14). For circular tunnels this is usually a circular steel shell under the protection of which the ground is excavated and the tunnel support is erected. A shield also includes back-up infrastructure to erect the tunnel support (lining) and to remove the excavated spoil.

Fig. 10.14: Simple Tunnelling Shield

There are two main types of tunnelling shield, one with partial and other with full face excavation. In partially-mechanized shields, an excavator or a partial cutter head/ road header works on the face. Partially-mechanized shields (also called boom-in-shield tunneling machines) are used where the cost of full face tunnel boring machines cannot be justified. Manual excavation, i.e. by “hand”, is considered for very special applications only, e.g. very short advances, due to the low advance rate. This type of tunneling is called the manual shield technique. Full Face Shields are discussed under Shielded TBMs & Soft Ground TBMs.

Tunneling shields do not have an “engine” to propel themselves forwards, but push themselves forward using hydraulic jacks. In order to create the necessary force to push the tunnel shield forwards, jacks are placed around the circumference of the shield. These jacks push against the last erected tunnel segment ring and also push the shield against the tunnel face in the direction of the tunnel construction. Of course, this principle does not work at the start of the tunnel construction and therefore in the starting a reaction frame is necessary to take the jacking forces.The jacks can be operated individually or in groups, allowing the shield to be steered in order to make adjustments in line and level and to be driven in a curve if required. When the shield has advanced by the width of a tunnel segment ring, the jacks are retracted leaving enough room in the tail of the shield to erect the next tunnel segment ring.

The support usually adopted with shield tunnelling these days is circular segments. These segments form, when connected together, a closed support ring. As the tunnel segments are connected together inside the shield tail, the diameter of the completed tunnel segment ring is smaller than that of the shield. This creates a gap between the ground and the tunnel lining. When the shield is jacked further into the ground the size of this gap is approximately 50 and 250mm. In less supporting soft ground, it has to be expected that the ground settles by this value. This can result in the softening of the ground and, especially with shallow tunnels, in the settlements reaching the ground surface and having undesirable consequences on surface or near surface structures. In order to avoid these settlements, the gap is generally injected with mortar.

8. Tunnel Boring Machine (TBM) also known as a “mole“, is a machine used to excavate tunnels with a circular cross section through a variety of soil and rock strata. They can bore through hard rock, sand and almost anything in between. Tunnel diameters can range

from a meter (done with micro TBMs) to almost 17-18m to date. Tunnels of less than a meter or so in diameter are typically done using trenchless construction methods or horizontal directional drilling rather than TBMs.

TBMs have the advantages of limiting the disturbance to the surrounding ground and producing a smooth tunnel wall. This significantly reduces the cost of lining the tunnel and makes them suitable to use in heavily urbanized areas. The major disadvantage is the upfront cost and difficulty in transportation. TBM tunnels have very high start-up (pre-excavation) costs and accompanying long lead time, though the high rate of advance reduces the per m excavation cost. The decision on undertaking excavation by TBM requires careful consideration of techno-economic factors.

A Herrenknecht mega-Mixshield TBM of 17.6m diameter became the largest machine in the world when it was launched in 2015 for Tuen Mun-Chek Lap Kok undersea highway link in Hong Kong. The Mixshield is marginally larger than what is now relegated to being only the second-largest machine in the world, the 17.48m diameter Hitachi Zosen EPBM on the Alaskan Way viaduct replacement highway tunnel project in Seattle.

Although TBMs are often designed for specific projects, i.e. with a specific diameter and to cope with certain ground conditions, these days refurbished machines are becoming more common and projects are actually designed around the machines available. An example of this is when the diameter of the new project is chosen to suit the old machine, with just the cutter head being redesigned for the specific ground conditions expected.

One of the general requirements for the use of a TBM is consistent geology along the route of the tunnel as the different cutting tools are only suitable for a small variation in material characteristics. A universal machine for all types of ground and soil conditions does not exist (although TBMs with multiple modes of

operation such as Mix-shields are there). The combination of different cutting tools on the cutter head can increase the application of machines to a greater range of ground conditions.

Although TBMs can have different mechanisms for moving through the ground, most have to start outside and hence need a reaction frame to start the drive.

9. Stages of TBM Construction: Following stages are involved, before and after the main activity of tunnelling by TBM:

(i) Excavation for launching Shaft and Retrieval Shaft (Fig. 10.15)

Fig. 10.15

(ii) Assembly of TBM at the launching Shaft (Fig. 10.16)

Fig. 10.16

(iii) Tunnel excavation by TBM (Fig. 10.17)

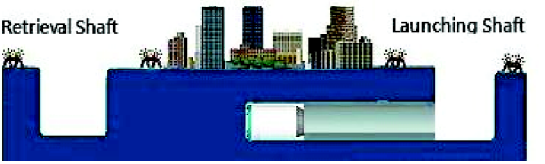


Fig. 10.17

(iv) Arrival of TBM in retrieval Shaft, to be dismantled and taken out (Fig. 10.18)

Fig. 10.18

10. Types of TBM: TBMs are often grouped under categories of "Hard Rock TBMs" & "Soft Ground TBMs".

10.1 Hard Rock TBMs: In hard rock, either shielded or open-type TBMs can be used. All types of hard rock TBMs excavate rock using disc cutters mounted in the cutter head. The disc cutters create compressive stress fractures in the rock, causing it to chip away from the rock in front of the machine, called the tunnel face. The excavated rock, known as muck, is transferred through openings in the cutter head to a belt conveyor, where it runs through the machine to a system of conveyors or muck cars for removal from the tunnel.

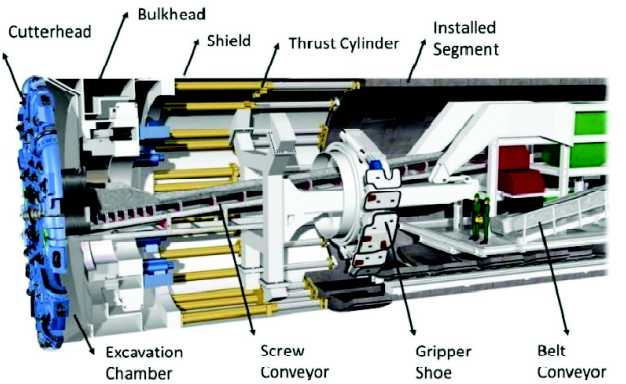


Fig. 10.19: Typical Hard Rock TBM

Hard rock TBMs comprise of following four key sections (Fig. 10.19):

(i) Boring section, consisting of the cutter head,

(ii) Thrust and clamping section, which is responsible for advancing the machine,

(iii) Muck removal section, which takes care of collecting and removing the excavated material, and

(iv) Support section, where the tunnel support is erected.

Open-type TBMs have no shield, leaving the area behind the cutter head open for rock support. To advance, the machine uses a gripper system that pushes against the side walls of the tunnel. The machine can be continuously steered while gripper shoes push on the side-walls to react the machine's forward thrust. At the end of a stroke, the rear legs of the machine are lowered, the grippers and propel cylinders are retracted. The retraction of the propel cylinders repositions the gripper assembly for the next boring cycle. The grippers are extended, the rear legs lifted and boring begins again. The open-type TBM does not install concrete segments behind it as other machines do. Instead, the rock is supported using ground support methods such as ring beams, rock bolts, shotcrete, steel straps and wire mesh *(Stack, 1995).*

In fractured rock, shielded hard rock TBMs can be used, which erect concrete segments to support unstable tunnel walls behind the machine. Double Shield TBMs are so called because they have two modes; in stable ground they can grip against the tunnel walls to advance forward and in unstable fractured ground, the thrust is shifted to thrust cylinders that push off against the tunnel segments behind the machine. This keeps the significant thrust forces from impacting fragile

tunnel walls. Single Shield TBMs operate in the same way, but are used only in fractured ground, as they can only push off against the concrete segments *(Stack, 1995).*

Hard Rock TBMs primarily fall under following categories:

- Gripper TBM: Open hard rock TBM - suited for boring in stable rock.
- Shielded TBM: Shielded hard rock TBM - suited for tunneling in varying rock formations that alternate between stable and unstable formations.

Working principles of these TBMs are briefly described below:

(I) Gripper TBM: The Gripper TBM is braced radially with grippers against the tunnel wall, with hydraulic cylinders pressing the cutter head against the tunnel face to enable a further section of tunnel to be excavated. The maximum boring stroke is governed by the length of the pistons in the thrust cylinder. The cutter head is fitted with cutter rings (disks). The rotating cutter head forces the disks against the tunnel face under high pressure. In this process, the disks roll over the tunnel face, thereby loosening the native rock.

The excavated rock, or "chips" as it is commonly known, is collected in muck bucket lips (openings in the cutter head) and discharged via hoppers onto a conveyor belt. The excavated material is brought outside the tunnel by conveyers. Typical advance of cutter head is approximately 0.7 to 1.2m. After completion of a boring stroke, the drilling process is interrupted and the machine moved forwards, with the Gripper TBM being stabilized by an additional support system. A new working cycle can begin when the gripper shoes of the machine are once more engaged.

Unlike shielded TBMs, where tunnel support, e.g. segmental lining, is fixed and does not change during tunnel construction, the tunnel support system, when using a Gripper TBM, can vary depending on the ground quality. The appropriate rock support devices can be installed immediately behind the cutter head. These devices can include anchors, steel arches, shotcrete and even segmental linings.

The tunneling performance of a Gripper TBM depends essentially on the time required to install rock supporting devices. The Gripper machine enables comprehensive rock support measures to be taken even right behind the cutter head.

(II) Shielded TBM: In contrast to Gripper TBMs, the body of the shield TBM has an extended shield over the front section of the machine. This shield has the function of supporting the ground and protecting the personnel, thus allowing safe erection of the tunnel lining. There are two basic types of shield TBMs for hard rock available; the single-shield and double-shield.

The single-shield TBM in hard rock is mainly used in unstable conditions where there is a risk of ground collapse. With these machines, the pushing forces are maintained axially against the installed lining segments. The Single Shield TBM belongs to a category of machines which are fitted with an open shield. Tunneling machines described as open shields are machines without a closed system for pressure compensation at the tunnel face. Protected by the shield, the Single Shield machine extends and drives forward the tunnel practically automatically. In order to drive the tunnel forwards, the Single Shield TBM is supported by means of hydraulic thrust cylinders on the last segment ring installed. The cutter head is fitted with hard rock disks, which roll across the tunnel face, cutting notches in it. These notches dislodge large chips of rock. Muck bucket lips, which

are positioned at some distance behind the disks, carry the extracted rock behind the cutting wheel. The excavated material is brought outside the tunnel by conveyers.

The double-shield machine (or telescopic shield) combines the ideas of the gripper and single-shield techniques and can therefore be applied to a variety of geological conditions. The double-shield machine consists of a front shield with cutter head, as well as a gripper section with gripper shoes, a tail shield and auxiliary thrust jacks. Both parts of the machine are connected by a section called the telescopic shield. The operating principle is based on the gripper shoes pressing against the tunnel wall while excavation and segment installation are performed at the same time. The system adds some flexibility to allow the machine to work either in gripper mode or as a shield TBM.

10.2 Soft Ground TBMs: The application of a TBM technique in less stable soft ground commonly requires the face to be supported. This is in contrast to the open face TBMs (often used in hard rocks) where the ground is able to support itself during excavation by virtue of its significant strength and stand-up time.

In soft ground, with little or no standup time, the ground would simply collapse into the machine and attempts to control the excavation of this material and to prevent large displacements occurring within extensive amounts of the ground around the tunnel heading would be very difficult. In addition, for tunnels constructed below the groundwater table in permeable materials, water flow into the tunnel must be controlled in order to prevent the machine and tunnel from flooding.

Soft ground TBMs are designed to simultaneously provide immediate peripheral and frontal support and as such they belong to the closed-faced group of TBMs.

Except for mechanical-support TBMs, they all have a cutter head chamber at the front, isolated from the rear part of the machine by a bulkhead, in which a confinement pressure is maintained in order to actively support the excavation and/or balance the hydrostatic pressure of the ground water.

The face is excavated by a cutter head working in the chamber. The TBM is jacked forward by rams pushing off the segmental lining erected using an erector integrated into the machine. Soft ground TBMs are classified into following types depending upon frontal support technique they employ:

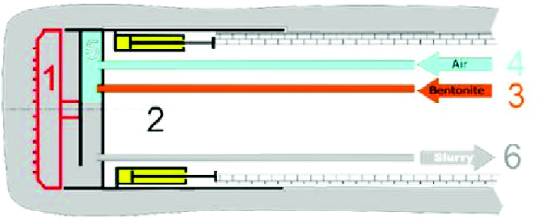
(I) Mechanical support TBM: A mechanical-support TBM has a full-face cutter head which provides face support by constantly pushing the excavated material ahead of the cutter head against the surrounding ground. Its specific field of application is, therefore, in soft rock and consolidated soft ground with little or no water pressure.

(II) Compressed air TBM: A compressed-air TBM can have either a full face cutter head or excavating arms like those of the different boom-type units. Confinement is achieved by pressurizing the air in the cutting chamber. Compressed-air TBMs are particularly suitable for ground of low permeability with no major discontinuities (i.e. no risk of sudden loss of air pressure). The ground tunneled must necessarily have an impermeable layer in the overburden.

Compressed-air TBMs tend to be used to excavate small-diameter tunnels. Their use is not recommended in circumstances where the ground at the face is heterogeneous. They should not be used in organic soil where there is a risk of fire.

(III) Slurry Shield TBM: In soft ground with very high water pressure and large amounts of ground water, Slurry Shield TBMs are needed. These

machines offer a completely enclosed working environment (Fig. 10.20). Soils are mixed with bentonite slurry, which must be removed from the tunnel through a system of slurry tubes that exit the tunnel. Large slurry separation plants are needed on the surface for this process, which separate the dirt from the slurry so it can be recycled back into the tunnel.



1. Cutter Head 2. Shield 3. Bentonite Injection

4. Air regulation 5. Air Bubble 6. Extraction of slurry with Soil

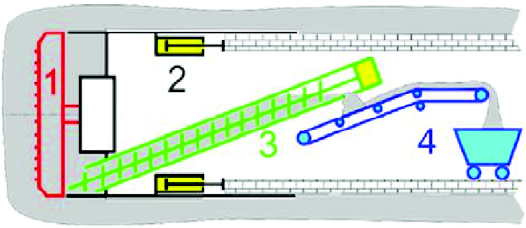
Fig. 10.20: Slurry Shield TBM

While the use of TBMs relieves the need for large number of workers at high pressures, a caisson system is sometimes formed at the cutting head for slurry shield TBMs. Workers entering this space for inspection, maintenance and repair need to be medically cleared as “fit to dive” and trained in the operation of the locks.

Slurry shield TBMs are particularly suitable for use in granular soil (sand, gravel etc.) and heterogeneous soft ground, though they can also be used in other terrain, even if it includes hard rock sections. There might be clogging and difficulty separating the spoil from the slurry if there is clay in the soil. These TBMs can be used in ground with high permeability, but if there is high water pressure special slurry has to be used to form a watertight cake on the excavation walls.

(IV) Earth Pressure Balance TBM: Earth Pressure Balance Machines are used in soft ground with less than 7 bar of pressure. An earth pressure balance machine has a full face cutter head. The cutter head does not use disc cutters only, but instead a combination of tungsten carbide cutting bits, carbide disc cutters and/or hard rock disc cutters. The EPB gets its name because it is capable of holding up soft ground by maintaining a balance between earth and pressure (Fig. 10.21). The TBM operator and automated systems keep the rate of soil removal equal to the rate of machine advance. Thus, a stable environment is maintained.

Support of the face is achieved by pressurizing the mud, formed of the excavated soil in the cutter head chamber. In most cases, water and some other additives (e.g. polymer foams) are added to render the excavated soil supple.



1. Cutter head 2. Shield 3. Screw conveyor

4. Belt Conveyor

Fig. 10.21: Earth Pressure Balance Machine

EPBMs are particularly suitable for soils which, after churning, are likely to be of a consistency capable of transmitting the pressure in the cutter head chamber. They can handle ground of quite high permeability, and are also capable of working in ground with occasional discontinuities requiring localized confinement.

Both types (EPB and SS) are capable of reducing the risk of surface subsidence and voids if operated properly and if the ground conditions are well documented. This makes them very suitable for urban tunnelling.

10.3 Special purpose TBMs: In addition to above, there are Special Purpose TBMs also. Some of these are:

(I) Reaming Boring Machines - allows a tunnel made using a TBM (pilot tunnel) to be widened (reaming).

(II) Raise Borer - used for shaft excavation which enables the top-to-bottom reaming of a small diameter pilot tunnel created using a drilling rig.

(III) Mixed Face TBMs – allows tunneling under mixed face conditions.

(IV) Multi-Mode TBMs – can operate in different modes with appropriate modifications to configuration & support techniques.

11. Back-up systems for TBM: Behind all types of tunnel boring machines, inside the finished part of the tunnel, are trailing support decks known as the back-up system. Support mechanisms located on the back-up can include conveyors or other systems for muck removal, slurry pipelines (if applicable), control rooms, electrical systems, dust removal, ventilation and mechanisms for transport of pre-cast segments.

12. Selection of TBM: Careful and comprehensive analysis should be made to select proper machine for tunneling taking into considerations its reliability, safety, cost efficiency and the working conditions. In particular, the following factors should be analyzed:

- Suitability to the anticipated geological conditions.
- Applicability of supplementary supporting methods, if necessary.

- Tunnel alignment and length.
- Availability of space necessary for auxiliary facilities behind the machine and around the access tunnels.
- Safety of tunneling and other related works.

13. Excavation Methods for Soft Ground Tunnels: Two principal Methods for tunneling in soft ground are:

13.1 Multiple Drift Method: This method has already been described under "Excavation methods for rock tunnels". Forepoling *(Ref.: Chapter-14 for details)* is normally done before doing excavation, particularly in heading portion.

13.2 Excavation by Tunnel Shields: This method has also been discussed in Para-7 above.

The control of ground water is of utmost importance in soft ground tunneling. To control groundwater, dewatering & grouting are the most common methods. Methods using "compressed air" & "freezing" are also sometimes used. Further details on this aspect may be seen in Chapter-14.

14. Stipulation about machinery in contract document: The type and number of machines required for construction of a particular tunnel will primarily depend on excavation method used for tunnelling and length of tunnel. Availability of sufficient number of machines, commensurate with desired rate of progress, of appropriate type at work site is prerequisite for successful and timely completion of work. As such, type (viz. minimum specifications/ capacity/output rate) and minimum number of machines required for construction of a particular tunnel should be stipulated in the tender document.

The list of such machineries may include excavators, road headers, drilling jumbos/boomers, loaders, dump trucks, dozers, shortcrete machines (with/without robotic arm), transit mixers, concrete pump, total stations etc.

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CHAPTER-11

TUNNELLING METHODS/ PHILOSOPHIES

Tunneling Method or Philosophy does not mean the tunnel excavation method only, but it covers all activities relevant to tunnel construction in totality i.e. data collection (geotechnical investigation, surveying etc.), design of support systems, excavation methods of tunnel, instrumentation & monitoring etc. Various Tunneling Philosophies (generally called as Tunneling Methods) are discussed in this chapter.

1. Conventional Tunnelling: The definition of "Conventional Tunneling" is rather arbitrary, and subject to variations, depending on the context. For Indian Railway context, the term conventional tunneling is used for any method other than Mechanized or Observational methods of Tunneling.

Design of support system in conventional tunneling can be done using any of the approach i.e. Empirical, Analytical or Numerical *(Ref. Chapter-8 & 9)*, using any type of support *(Ref. Chapter-7)* as per the design. The excavation can be done by Drill & Blast Method (with full face or partial face excavation) or By Road header (with full face or partial face excavation). The supports can be initial/primary supports with or without secondary/final supports. The basic principle used in this method is that the load coming on the supports is estimated and the supports are designed to resist this load.

In case exceptional ground conditions are encountered, the Conventional tunneling Method can react with a variety of auxiliary construction technologies like:

- Grouting: Consolidation grouting, fissure grouting, pressure grouting, compensation

grouting etc.

- Technologies to stabilize and improve the ground ahead of the actual tunnel face like forepoling, pipe umbrella, horizontal jet grouting, ground freezing etc.

2. Ground (Rock) Support Interaction: The term "tunnel lining" or "tunnel support" encompasses a broad range of concepts, materials, construction methods and details *(Ref. 27)*. Several common characteristics pervade all systems:

(i) Tunnel lining is not an independent structure acted upon by well-defined loads, and its deformation is not governed by its own internal elastic resistance. The loads acting on a tunnel are ill defined, and its behaviour is governed by the properties of the surrounding ground. Design of a tunnel lining is not a structural problem, but a ground-structure interaction problem, with the emphasis on ground.

(ii) Tunnel lining behaviour is a four-dimensional problem. During construction, ground conditions at the tunnel heading involve both transverse arching and longitudinal arching or cantilevering from the unexcavated face. All ground properties are time dependent, particularly in short term, which leads to the commonly observed phenomenon of stand-up time, without which most practical tunnel construction methods would be impossible. The timing of lining installation is an important variable.

(iii) The most serious structural problems encountered with actual lining behaviour are related to absence of support – inadvertent voids left behind the lining – rather than to intensity and distribution of load.

(iv) In virtually all cases, the bending strength and stiffness of structural linings are small compared with those of the surrounding ground. The

properties of the ground control the deformation of the lining, and changing the properties of the lining will not significantly change this deformation. The proper criterion for judging lining behaviour is therefore not adequate strength to resist bending stresses, but adequate ductility to conform to imposed deformations.

2.1 Ground Reaction Curve (GRC) and Support Reaction Curve (SRC): The radial deformation/ displacement, which occurs in the vicinity of an advancing tunnel face (Fig. 11.01) *(Hoek, 1999):*

- Begins a certain distance ahead of the tunnel face (about two and one-half tunnel diameters).
- Reaches about one third of its final value at the tunnel face.
- Reaches its maximum value at about four and one-half tunnel diameters behind the face.

It is important to note that even for an unsupported tunnel, the tunnel face provides an “apparent support pressure”. It is this apparent support pressure that provides the stability to give sufficient stand-up time for the actual support to be installed.

The rock mass around excavated tunnel entering into plastic state does not necessarily mean that the tunnel collapses. This material can still have considerable strength, provided that thickness of the plastic zone is small compared with the tunnel radius, and only evidence of failure may be a few fresh cracks and a minor amount of raveling. On the other hand, when a large plastic zone is formed and large convergence of tunnel occurs, the loosening of the failed rock mass can lead to severe spalling and eventual collapse of an unsupported tunnel. The primary function of

support is to control the convergence of excavated tunnel and to prevent the loosening, which can lead to collapse of the tunnel. The supports play a major role in controlling tunnel deformation *(Hoek et al. 1995).*

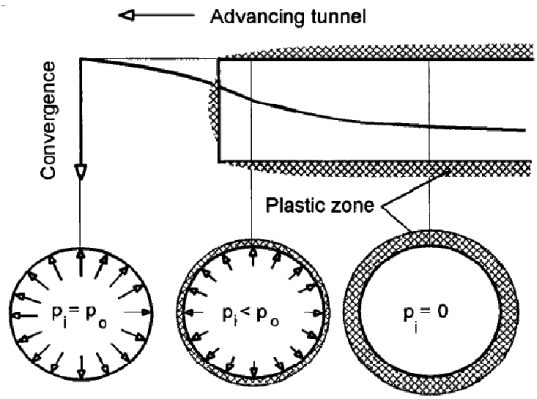


Fig. 11.01: Support Pressure (pi) at different positions

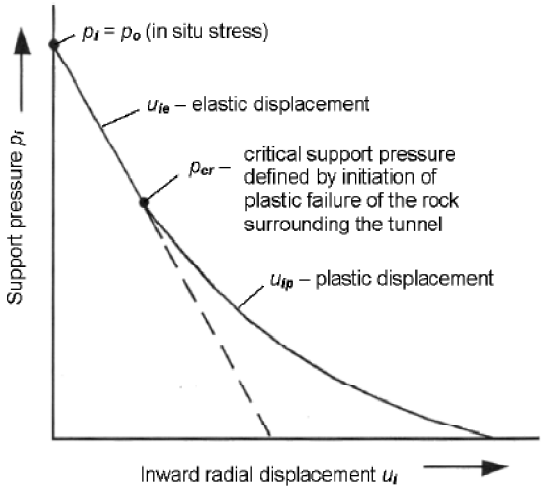


Fig. 11.02: Ground Reaction Curve

Assume a circular tunnel of radius r_o is subjected to in-situ stress p_o and a uniform internal support pressure p_i. Failure of the rock mass surrounding the tunnel occurs when the p_i is less than a critical support pressure p_{cr}. If pi is greater than p_{cr}, no failure occurs and the rock mass surrounding the tunnel is in elastic stage.

The plot between the Support Pressure (p_i) and radial convergence/ squeezing (u_i) is called Ground Reaction Curve (GRC), or Fenner-Pacher Curves (Fig. 11.02). This plot shows that:

- Zero displacement when the support pressure equals the hydrostatic stress ($p_i = p_o$)
- Elastic displacement u_{ie} for $p_o > p_i > p_{cr}$
- Plastic displacement u_{ip} for $p_i < p_{cr}$
- Maximum displacement when the support pressure equals zero

For a given tunnel radius and in-situ stress, the shape of the ground reaction curve depends on the rock mass failure criterion which is assumed and the specific rock mass characteristics.

In order to complete the rock support interaction analysis, the reaction curve for the rock support must be determined. This is a function of three components:

(i) The convergence that has occurred before support is installed.

(ii) The stiffness of the support system.

(iii) The capacity of the support system.

A certain amount of deformation takes place ahead of the advancing face of the tunnel. In addition, there is a gap between the excavation face and the closest installed support element. Therefore, further deformation occurs before the support becomes effective. This total initial displacement is uso (Fig. 11.03).

Once support is installed and it is in full/effective contact with rock mass, the support starts to deform elastically. The plot of support pressure and elastic deformation of support is called Support Reaction Curve (SRC). Slope of this curve gives stiffness of the support. The maximum elastic displacement which can be accommodated by the support system is usm and the maximum support pressure psm is defined by the yield of the support system.

Equilibrium is achieved if Support Reaction Curve intersects the Ground Reaction Curve before either of these curves has progressed too far. If the support is installed too late, the rock mass may have already deformed to the extent that loosening of the failed material is irreversible. On the other hand, if the capacity of the support is inadequate then yield of the support may occur before the rock mass deformation curve is intersected. In either of these cases, the support system will be ineffective.

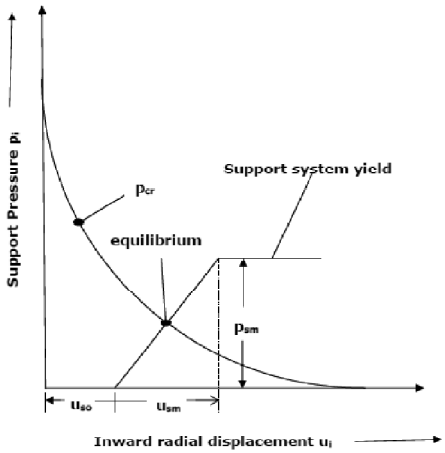


Fig. 11.03: Support Reaction Curve

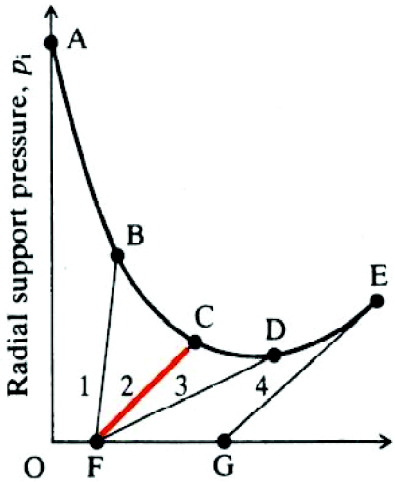
Various types of supports, depending upon their "Stiffness" and "Installation Timing", and their implications are as under (Fig. 11.04) *(Brady and Brown, 2006):*

Support-1: Too Stiff; Attracts excessive load; May fail causing catastrophic failure of rock mass.

Support-2: Lower stiffness; Good soultion; Support not loaded excessively.

Support-3: Much lower Stiffncess; Acceptable only temporarily; Situation dangerous.

Support-4: Same Stiffness as Support-2; Installed late permitting excessive convergence; Support may become overstressed before equilibrium.



Radial displacement, u_i

Fig. 11.04: Right type of Support

The Ground (Rock) Support Interaction is represented by "Ground Reaction Curves" and "Support Reaction Curves". These curves can be plotted by using closed form solutions given by Deere et al. (1970), Hoek & Brown (1980) etc., for

circular tunnels with hydrostatic in-situ stresses. They can also be plotted by using some software like “RocSupport”. For complex problems, Numerical Methods (e.g. FLAC software) are used.

3. Observational Method of Tunnelling: In the conventional tunnelling, the support is normally aimed to be provided at zero displacement/convergence, though some displacement always takes place before the support is put in position and it starts resisting the displacement. Thus, in this method, the stiffness required from the support is relatively high *(Ref. Fig. 11.03).* If some amount of displacement/convergence is permitted (within the permissible value from point of view of serviceability of the tunnel and the extent of plastic zone remaining within the critical limit), the required stiffness of the support system will reduce. This in-turn facilitates lighter (and more flexible) support system design, leading to overall reduction/ economy in the support system. But for this, the displacement/ convergence of tunnel and support (and some other parameters also) need to be monitored to ensure that support is provided at the right time and it is of right stiffness. This approach is called “Observational Method of Tunnelling”, wherein support is designed based on observation of displacement and by controlling the deformation. In this approach:

(i) Rock support is applied based on preliminary assessment (using empirical/analytical/ numerical approach or some software).

(ii) Ground displacement is monitored with time and the support is installed at the specified displacement.

(iii) If the support is insufficient, displacement continues. In such case, additional support is applied to ensure that displacement is stabilized.

(iv) The tunnel displacements and support pressures are monitored to develop a suitable support system to stabilize the tunnel.

When the tunnel behaviour is not known with confidence, this approach is very useful. This is also called “build-as-you-go approach”. In Eurocode-7, the “Observational Approach” is defined as under *(Clause 2.7, Ref. 28).*

(1) When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as “the observational method”, in which the design is reviewed during construction.

(2) The following requirements shall be met before construction is started:

- *acceptable limits of behaviour shall be established;*
- *the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;*
- *a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;*
- *the response time of the instruments and the procedure for analyzing the results shall be sufficiently rapid in relation to the possible evolution of the system;*
- *a plan of contingency actions shall be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.*

(3) During construction the monitoring shall be carried out as planned.

(4) *The results of monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put in operation if the limits of behaviour are exceeded.*

(5) *Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.*

4. New Austrian Tunnelling Method (NATM): New Austrian Tunnelling Method (NATM) was developed by the Austrians Ladislaus von Rabcewicz, Leopold Müller and Franz Pacher in the 1950s. The name was introduced in 1962 *(Rabcewicz, 1963)* to distinguish it from the "Austrian Tunnelling Method", today referred to as the "Old Austrian Tunnelling Method". It is one of the most popular observational method of tunnelling.

The method has often been mentioned as "value engineered version of tunneling", due to its use of light supports. It has long been understood that the ground, if allowed to deform slightly, is capable of contributing to its own support. The NATM, with its use of modern means of monitoring and surface stabilization, such as shotcrete and rock bolts, utilizes this effect systematically. The main idea is to use the geological stress of the surrounding rock mass to stabilize the tunnel itself.

4.1 Definition of NATM: Austrian National Committee defines NATM as *"a concept which makes the ground (rock or soil) surrounding the void a supporting construction element through the activation of a ground supporting arch".*

Another useful definition as given by H. Lauffer is *"NATM is a tunnelling method in which excavation and support procedures, as well as measures to improve the ground -which should be distorted as low as possible - depend on observations of deformation and are continuously adjusted to the encountered conditions".*

4.2 Principles of NATM: The NATM is not a set of specific excavation and support techniques and has often been referred to as a "design-as-you-go" approach to tunnelling providing an optimized support based on observed ground conditions but more correctly it is a "design-as-you-monitor" approach based on observed convergence and divergence in the lining as well as prevailing rock conditions. There are seven features on which NATM is based:

(i) Mobilization of the strength of rock mass – The method relies on the inherent strength of the surrounding rock mass being conserved as the main component of tunnel support.

(ii) Shotcrete protection – Loosening and excessive rock deformation must be minimized. This is achieved by applying a thin layer of shotcrete immediately after face advance. Shotcrete and rock bolts applied close to the excavation face help to maintain the integrity of the rock mass.

(iii) Measurements – Every deformation of the excavation must be measured. NATM requires installation of sophisticated measurement instrumentation. It is embedded in lining, ground, and boreholes. This ensures that support is not installed too early or too late.

(iv) Flexible support – The primary lining is thin and reflects recent strata conditions. Active rather than passive support is used and the tunnel is strengthened not by a thicker concrete lining but by a flexible combination of rock bolts, wire mesh and steel ribs. The lining should not be too stiff or too weak.

(v) Closing of invert – Quickly closing the invert and creating a load-bearing ring is important. It is crucial in soft ground tunnels where no section of the tunnel should be left open even temporarily.

(vi) Contractual arrangements – Since the NATM is based on monitoring measurements, changes in support and construction method are possible. This is possible only if the contractual system enables those changes.

(vii) Rock mass classification determines support measures – There are several main rock classes for tunnels and corresponding support systems for each. These serve as the guidelines for tunnel reinforcement.

4.3 Construction Sequence: The constr-uction process in NATM is normally as follows:

- Excavation: The tunnel advance can be achieved using blasting, a partial face boring machine or simply using an excavator, depending on the ground conditions. Generally, the advancement is spatially and timely staggered in the heading, benching and invert.
- Sealing the exposed ground if necessary.
- Mucking.
- Installation of lattice girders or mesh reinforcement, and application of shotcrete. Depending on the quality of the ground the support might be installed first before the spoil is removed.
- Potential installation of a second layer of reinforcement and application of shotcrete.
- Installation of anchors.
- Construction of inner/secondary lining.

4.4 Limitations of NATM: In order to use NATM, the ground has to be capable of supporting itself over the length of each advance section, which means that the ground must have a stand-up time. The limit of this construction technique is reached when the stand-up time of the ground has to be improved by artificial measures, such as freezing or grout injection.

4.5 Pre-requisites for NATM

- Balanced Contract Structure- Providing for equitable risk sharing and flexible approach as per varying technical requirements.
- Qualified Personnel-Particularly for ensuring quality of shotcrete, rock bolts.
- Better Site Management for:
- Coping with unseen events.
- Proper application of the observational method.
- Elimination of human errors.
- Continuous Monitoring of geology- By qualified & experienced geologists, for interpretation of exploration data & keeping thorough geological record.

4.6 Advantages of NATM

- Flexibility to adopt different excavation geometries and very large cross sections.
- Lower cost requirements for the tunnel equipment at the beginning of the project.
- Flexibility to install additional support measures, rock bolts, dowels, steel ribs, if required.
- Easy to install a waterproof membrane.
- Flexibility to monitor deformation and stress redistribution so that necessary precautions can be taken.

5. Norwegian Method of Tunneling (NMT): This is also an observational method, quite similar to NATM. The difference between NATM and NMT are mainly as under:

(i) NATM is most suitable for soft ground which can be machine or hand excavated, where jointing and over break are not dominant, where a smooth profile can often be formed and where a complete load bearing ring can be

(and often should be) established. NMT is most suitable for harder ground, where jointing and over break are dominant, and where drill and blasting or hard rock TBMs are the most usual methods of excavation.

(ii) In NMT, bolting is the dominant form of rock support since it mobilizes the strength of surrounding rock mass in the best possible way. Rigid steel sets or lattice girders are inappropriate in Norway's harder rocks due to the potential over break. Bolting and SFRS (Steel Fiber reinforced shotcrete) are the two most versatile tunnel support methods. A thick load bearing ring (RRS – reinforced rib of shotcrete) can be formed as needed, and matches an uneven profile better than lattice girders or steel sets.

(iii) In NMT, Q-system is used for regulating the description of rock mass conditions and support recommendations. Instrumentation and monitoring is done only in critical cases.

(iv) The shotcrete used in NMT is only SFRS. Concrete lining is used only in extreme conditions such as when tunneling through fault zones, swelling clay and very weak rock that may squeeze.

6. ADECO-RS Method of Tunneling: ADECO-RS is abbreviation of Italian word which is translated in English as *"Analysis of Controlled Deformation in Rocks and Soils".* This method was developed by Pietro Lunardi in 1980 in Italy. It has been used in many tunnels in Italy and in some tunnels outside Italy.

6.1 Important Components in Tunneling: The three important components in tunneling operation which influence the stabilization of the ground, after creation of the cavity for tunnelling, are Medium (properties of the ground through which tunneling is being done), Action (whole set of

operations to excavate the tunnel) and Reaction (deformation response of the medium to excavation) (Fig. 11.05).

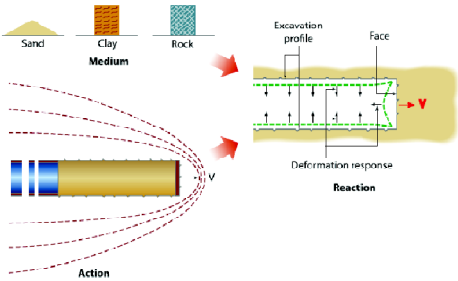


Fig. 11.05: Important Components in Tunnelling

6.2 Deformation Response of the Medium: Any point in rock mass just ahead of tunnel excavation face is in tri-axial state of stress. With the tunnel excavation passing from this point, the rock mass removal from this point modifies the stress field at this point (with minor principal stress along the tunnel axis becoming zero). Thus, the stress field at this point changes from tri-axial to plane stress state, due to tunnel excavation advance, with the confinement pressure at the excavation face progressively reducing to zero. Depending upon the medium, the stress state and the way in which face is advanced, the deformation response of the medium may be of three types (Fig. 11.06).

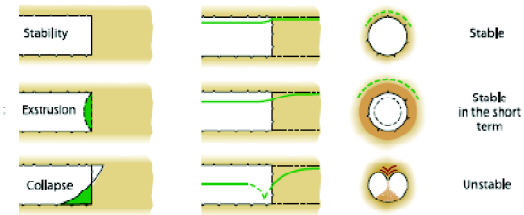


Fig. 11.06: Types of Deformation Response

If the stresses around the periphery of the cavity, near the face, are in elastic range and the deformation is limited/absolutely negligible, the face remains “Stable” and the redistribution of stresses around the cavity (the “Arch Effect”) is produced by natural means close to the profile of the excavation (Fig. 11.07).

Fig. 11.07: Stable Face

If the stresses around the periphery of the cavity, near the face, are in elasto-plastic range, then the face will deform in an elasto-plastic manner towards the interior of the cavity and it gives rise to a condition of “short term stability”. This means that in the absence of intervention, plasticization is triggered, which by propagating radially and longitudinally from the walls of excavation, produces a shift of the “arch effect” away from the tunnel periphery further into the rock mass. Development of elasto-plastic zone can only be controlled by intervention to stabilize the ground (Fig. 11.08).

If the stresses around the periphery of the cavity, near the face, are in failure (plastic) range, then the deformation response is unacceptable and a condition of instability exists in the ground ahead of the face, which makes the formation of “arch effect” impossible (Fig. 11.09).

Fig. 11.08: Stable Face in Short Term

Fig. 11.09: Unstable Face

6.3 Some terms used in this methodology:

- **Advance Core:** The volume of ground that lies ahead of the face, virtually cylindrical in shape, with the height and diameter of the cylinder the same as the diameter of the tunnel.
- **Extrusion:** The primary component of the deformation response of the medium to the action of excavation that develops largely inside the advance core. It manifests on the surface of the face along the longitudinal axis of the tunnel and its geometry is either more or less axial-symmetrical (bellying of the face) or that of gravitational churning (rotation of the face).

- **Pre-convergence of the cavity:** Convergence of the tunnel profile ahead of the face (the area yet to be excavated), strictly dependent on the relationship between the strength and deformation properties of the advance core and its original stress state.

6.4 Experimental and Theoretical Research: This research in Italy, over a period of 20-30 years, has brought out following points:

(i) It is important to keep "Excavation Rates" high and constant. This prevents "extrusion" and "pre-convergence", which are starting point for subsequent "Convergence" of Cavity. Pre-convergence can be calculated, using simple volumetric calculations, though not possible to measure it directly.

(ii) Chronologically, "deformation in the cavity (convergence)" normally follows and is dependent on "deformation in the core at face (extrusion)". There is close correlation between the magnitude of "extrusion allowed in advance core" and the "convergence manifested after passage of the face". Both of these decrease as the "rigidity of core" is increased.

(iii) The advance core extrudes with 3 types of deformations (Fig. 11.10), depending on the material involved and the stress state.

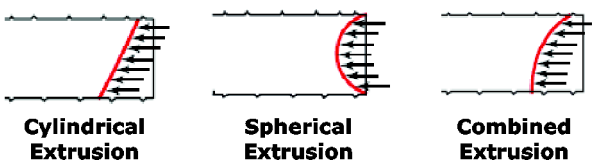


Fig. 11.10: Types of Core Extrusion

(iv) There is close connection between the "failure of the core face" and "collapse of the cavity", even if it has already been stabilized.

"Deformation of advance core" constitutes the factor most capable of conditioning the "deformation response of the cavity" and must therefore be true cause of it.

(v) It seems logical to make use of the "core" as a new instrument for controlling it, by acting on its rigidity with appropriate "interventions".

6.5 Type of Interventions: The "interventions" for increasing rigidity of the "core" can be broadly of two types:

6.5.1 Protective Intervention: These interventions channelize the stresses around the advance core to perform protective function, thereby ensuring that natural strength and deformation properties of the core are conserved. Some such interventions are:

(A) Putting drainage pipes in the advance core, to drain out the water from it and prevent reduction in its' strength due to presence of water.

(B) Full face sub-horizontal jet grouting, which creates a grout ring around the advance core, before excavation (Fig. 11.11).

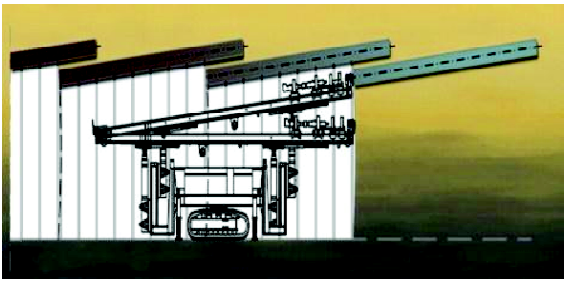
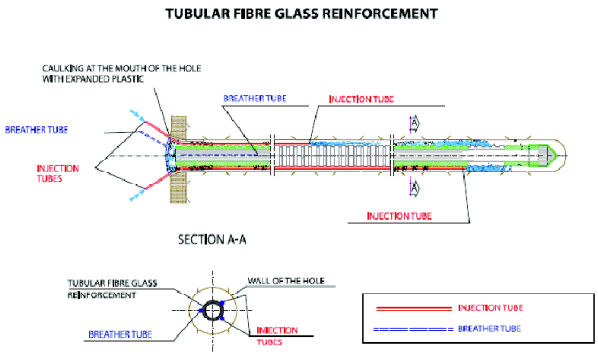


Fig. 11.11: Full face sub-horizontal Jet Grouting

Fig. 11.12: Mechanical Pre-cutting Machine

(C) Full face mechanical pre-cutting, using a machine (Fig. 11.12) to create a slit around the advance core, which is filled using concrete or grout.

6.5.2 Reinforcement Interventions: These interventions act directly on the consistency of the advance core to improve its' natural strength and deformation properties by means of appropriate ground reinforcement techniques. Most commonly used such reinforcement is fiber glass structural elements (Fig. 11.13), which are placed in holes drilled in advance core. Advantage of using these elements is that after excavation, it is very easy to break/snap the elements hanging from the face.



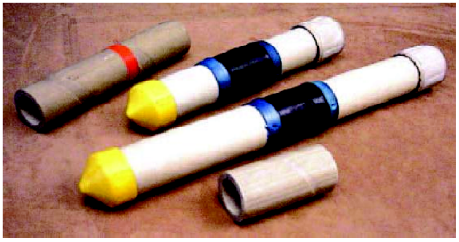


Fig. 11.13: Fiber Glass Reinforcement Elements

6.6 Behaviour Categories of Core Face: In full face excavation, the behaviour of core face after excavation can be categorised in following categories:

(A) "Category–A" Stable Core Face (Rock Type Behaviour): The face as a whole is stable and only local instability is found due to the fall of isolated blocks (Fig. 11.14). In this case, stabilization is needed for preventing deterioration of rock and to maintain profile of the excavation.

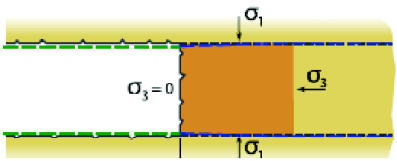


Fig. 11.14: Stable Core Face

(B) "Category–B" Stable Core Face in Short Term (Cohesive Type Behaviour): Instability manifests in the form of widespread spalling at the face and around the cavity (Fig. 11.15). Sufficient time is available to employ traditional radial confinement measures, after passage of face. In some circumstances, it may be necessary to resort to pre-confinement of cavity, to contain deformation within acceptable limits. Presence of water shall be prevented by suitable drainage arrangements.

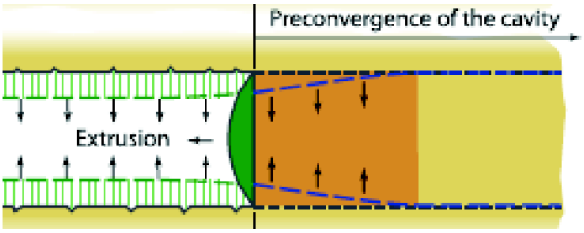


Fig. 11.15: Stable Core Face in Short Term

(C) "Category–C" Unstable Core Face (Loose Ground Type Behaviour): Deformation is unacceptable because it develops immediately in failure range (Fig. 11.16). Ground reinforcement is needed ahead of face to develop pre-confinement action. Presence of water is absolutely unacceptable and shall be prevented by drainage arrangements.

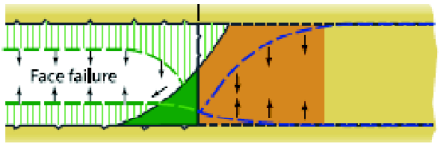


Fig. 11.16: U nstable Core Face

6.7 Stages of ADECO-RS Approach: There are two stages in this methodology:

(A) Design Stage: This stage consists of following three phases:

(I) Survey Phase, to determine characteristics of the medium. It involves collecting full information about geological / geo morp-hological / hydro-geological data of the area, location & definition of terrain through which the alignment passes, tectonics/geological structure & the stress state of rock mass, hydro-geological regime of rock mass and geo-mechanical characteristics of the materials.

(**II**) **Diagnosis Phase** to divide the tunnel sections in Category A/B/C, defining the details of deformation development and types of loads mobilized by excavation. Analytical and/or Numerical Methods are used to predict the behavior category and deformation response for various sections of the tunnel alignment.

(III) Therapy Phase to decide type of confinement and/or pre-confin-ement and test their effectiveness. It involves following steps:

(a) Decide the pre-confinement and/or confinement action (in the context of behaviour Category A/B/C).

(b) Select suitable pre-confinement and/or confinement intervention based on recent advances in the technology,

(c) Design & test the proposed intervention using mathematical modelling.

(B) Construction Stage: This stage consists of following three phases:

(I) Operational Phase in which stabilization measures are employed, followed by excavation operation. This phase proceeds hand-in-hand with "Monitoring Phase". The tunnel section, specified by design engineer, guides in this phase about type of intervention to be performed. Success of this phase depends on the accuracy of the predictions made in "diagnosis phase" and on the "design decisions" made as a consequence. Full face advance shall be employed, wherever possible, giving concave shape to the face and avoiding over-breaks. All stabilization works shall be performed rapidly.

(II) Monitoring Phase in which deformations are measured to verify accuracy of predictions made and to fine tune the design further. During service life, monitoring is continued for safety of the tunnel. Reliability of prediction made in "diagnosis" and "therapy" phases are tested here. This phase starts as soon as construction begins and sometimes even before it. Scale of instrumentation to be done depends upon the category of core face also (Category A, B or C).

(III)Final Design Adjustment Phase based on the results of the monitoring.

6.8 Tunnelling with ADECO-RS Approach: Typical stages involved in tunnelling with ADECO-RS will be as under (any particular stage may or may not be involved depending upon type of ground):

- **Phase-1:** Reinforcement of the core face using suitable reinforcement elements (e.g. Fiber glass tubular elements).
- **Phase-2:** Carry out full face excavation.
- **Phase-3:** Placement of a layer of shotcrete on newly excavated face.
- **Phase-4:** Cavity pre-confinement by a suitable method (e.g. Mechanical pre-cutting).
- **Phase-5:** Providing other support measures, behind the excavation face.
- **Phase-6:** The cycle continues repeating Phases- 2, 3, 4 & 5, until required advance length is reached.
- **Phase-7:** Excavation and casting of invert.
- **Phase-8:** Placing waterproofing system and casting of final lining.

6.9 Difference between NATM and ADECO-RS: The major differences between NATM (and derived methods) and ADECO-RS approach are as under:

(i) In ADECO-RS methodology, it is always full face excavation whereas in NATM this it is not so necessarily.

(ii) In NATM, only “excavation” behind the excavation face is reckoned for design and monitored later on. But in ADECO-RS method, “extrusion” of core face as well as “pre-convergence” of rock mass ahead of excavation face are considered.

(iii) In NATM, the “intervention” in the form of supports is provided only in the cavity behind the excavation face. But in ADECO-RS method, pre-confinement and reinforcement of advance I also done, if needed.

6.10 Case History with ADECO-RS Method: This case history pertains to "Construction of *Tartaiguille Tunnel* for TGV *Mediteranee* High Speed Rail Line from *Marseilles to Lyon* in France".

Total length of tunnel was 2330m, with cross section area of 180 m^2. Tunnel Advance began in Feb'1996 with NATM Principle. Construction Sequence followed was: Top heading with Road header, Bench excavation with Excavator Hammer, Shotcrete (25 cm thick), Steel Ribs (HEB 240), End Anchored Radial Bolts (4m long), micro piles beneath base of ribs, casting of side wall, casting of invert, water proofing layer and Concrete Lining (70 cm thick). In Sept'1996, it became practically impossible to continue, due to heavy swelling in Montmorillonite formation.

Convergence observed was 60mm in Heading and 150mm in Benching, leading to cracks in shotcrete and its spalling. SNCF constituted a Study Group and major European experts were consulted. None could offer Safety and Reliability, with required completion times; except Rocksoil S.p.A. Italy (with whom Pietro Lunardi was associated).

Work of balance 860m length was awarded to Rocksoil in March'1997. Tunnel advance resumed in July'1997, with ADECO-RS methodology. The tunneling work was completed in July'1998, with good advance rate. The progress rate chart is shown in Fig. 11.17.

The photos of this tunnel work during progress are shown in Fig. 11.18 and the view of finished tunnel is shown in Fig. 11.19.

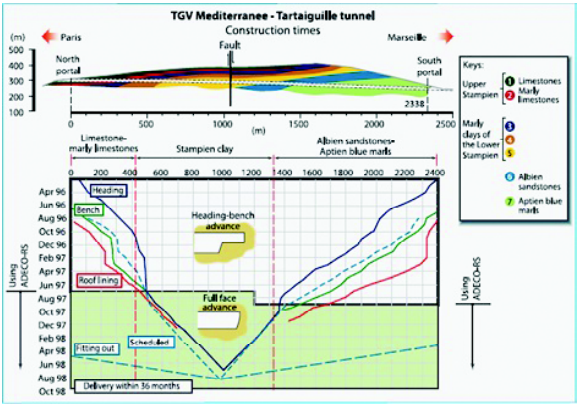


Fig. 11.17: Progress Rate Chart

Fig. 11.18: Work in Progress

Fig. 11.19: Finished Tunnel

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CHAPTER-12

INSTRUMENTATION AND MONITORING OF TUNNELS

In past, there was little emphasis on instrumentation, although ad-hoc measurements were sometimes made if the failure appeared imminent. However, because of problems faced in tunnelling like rock bursts, support failures, water inflow etc., the field instrumentation gained popularity among both designers and construction engineers. In recent times, the geo-technical instrumentation and monitoring is generally an integral part of the tunneling project.

1. Need for Geotechnical Instrumentation: There is a limit to geotechnical data acquisition and no amount of expenditure can result in 100% data acquisition, leaving at least 20% uncertainty at best (Fig. 12.01). In many large Indian projects, this is supposed to be upto 80% also.

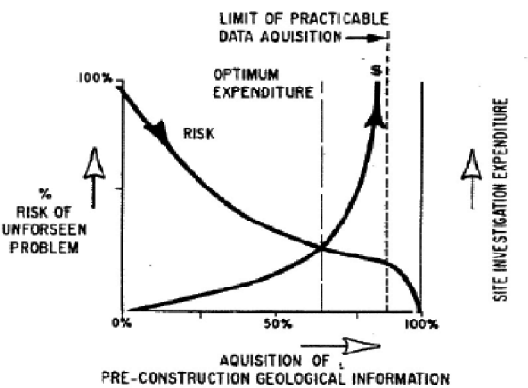


Fig. 12.01: Extent of Geotechnical Investigation

Lack of geotechnical investigation and consequent data inaccuracies have potential of leading to poor designs, additional construction costs and long term maintenance liabilities. This brings into picture the need for instrumentation and monitoring. In addition, the instrumentation and monitoring becomes essential, if "observation approach" of tunneling is followed.

2. Purpose of Instrumentation and Monitoring: In tunnelling projects, it serves following purposes:

2.1 Design and Design Verification:

- Can be used to obtain data from pilot tunnels or shafts, which can be used for design of tunnel.
- Helps in deciding design of final support system.
- In "observational approach", monitoring of displacements and loads is essential part of the construction process.
- To verify the assumptions made in the initial design and to verify that performance is "as predicted' or not.
- Data from initial phase is used to improvise the support design in later phases.

2.2 Construction Control:

- Instruments are used to monitor the effects of construction.
- Instrumented data can help in deciding how fast construction can proceed without risk of failure.
- Can be used to diagnose flaws in the contractor's construction methodology and indicate the required improvements.

2.3 Safety / Stability:

- Monitoring the stability of excavation or adequacy of ground support also serves a safety function, by warning the potential for

ground failure during construction, in some cases during service life.

- Checking adjacent structures and facilities for their safety and serviceability due to tunnel construction.

2.4 Regulatory/Environmental requirements: To ensure compliance with regulatory/ environmental requirements (e.g. groundwater lowering, ground settlements, vibrations etc.).

2.5 Performance Monitoring: To monitor in-service performance of structure (e.g. monitoring loads on rock bolts and movements within a tunnel can provide an indication of its performance)

2.6 Contractual Documentation: Monitoring data can also be used for avoiding/settling disputes with the contractor.

The geotechnical instrumentation and monitoring by a geotechnical engineer, for a tunnel, is quite similar to the way a doctor treats his patient (Fig. 12.02).

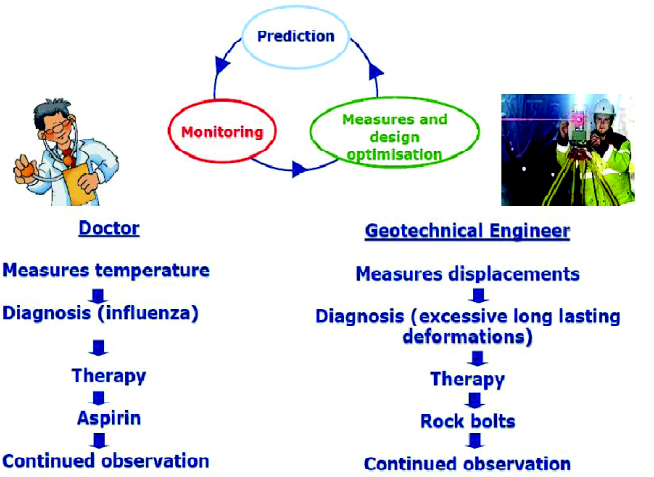


Fig. 12.02: Geotechnical Monitoring Cycle

3. Items to consider in Instrumentation and Monitoring Program:

3.1 Project conditions: The official(s) planning and designing should be familiar with project conditions including type & layout of tunnel, subsurface stratigraphy, engineering properties of subsurface materials, groundwater conditions, status of nearby structures or other facilities, environmental conditions and planned construction method.

3.2 Mechanisms that control behavior: Before defining a program of instrumentation and monitoring, one or more working hypotheses must be established for mechanisms that are likely to control behavior. Instrumentation should then be planned around these hypotheses.

3.3 Purpose of Instrumentation: Every instrument should be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation.

3.4 Parameters to be Monitored: Following are the typically monitored parameters:

- Convergence
- Crown settlement
- Floor heave
- Load in rock bolts/anchors
- Stress in shotcrete/ concrete
- Groundwater pressure
- Water pressure acting on lining
- Surface settlement
- Vertical and horizontal deformation of buildings and other structures
- Vertical and horizontal deformation of the ground at depth

3.5 Expected values of parameters recorded: An estimate of maximum possible value will determine the instrument range. The minimum value of interest determines the instrument sensitivity/accuracy.

3.6 Instrument selection: Reliability is the most desirable feature when selecting monitoring instruments. First lowest cost should not dominate the selection of an instrument. A comparison of the overall cost of procurement, calibration, installation, maintenance, reading and data processing should be made. The least expensive instrument necessarily may not result in least overall, as cost of instruments is usually a minor part of the overall cost.

3.7 Location for Installation: Selection of locations for the instruments should be based on predicted behavior of the tunnel or shaft. A practical approach to select instrument locations involves first identifying areas of particular concern (e.g. structurally weak zones or areas that are most heavily loaded) and locating requisite instruments there. Then select zones where predicted behavior is considered representative of behavior as a whole (primary instrumented sections). Lastly, install simple instrumentation at a number of secondary instrumented sections to serve as indices of comparative behavior. If the behavior at one or more of the secondary sections appears to be significantly different from the primary sections, additional instruments can be installed at the secondary section as construction progresses.

3.8 Threshold Values: A predetermination should be made of instrumentation readings that will indicate need for remedial action. The concept of green (all is well), yellow (need for cautionary measures including an increase in monitoring frequency) and red (need for timely remedial

action) response values should be adopted.

3.9 Remedial action and Implementation: Assign duties and responsibilities for all phases, including planning, instrument procurement, calibration, installation, maintenance, reading, data processing/presentation/ interpretation, data reporting and deciding on implementation of the results. When duties are assigned for monitoring, the party with the greatest interest in the data should be given direct responsibility for producing it accurately.

3.10 Factors affecting measurements: For proper interpretation of site instrumentation data, it is essential to monitor and record site activities and climatic conditions that can affect the measurements obtained.

3.11 Ensuring data correctness: In critical situations, more than one of the same type of instrument may be used to provide a backup system even when its accuracy is significantly less than that of the primary system. Repeatability can also give a clue to data correctness. It is often worthwhile to take many readings over a short time span to determine whether a lack of normal repeatability indicates suspect data. Also plan for regular calibration and maintenance of instruments to ensure correctness of date recorded by them.

4. Instruments used: The instruments typically used for monitoring various parameters in tunnels are as under:

4.1 Surface Settlement: Surface settlement/ movement is measured to forewarn about surface settlement and/or to monitor stability of open cut excavations. The instruments used for this are as under:

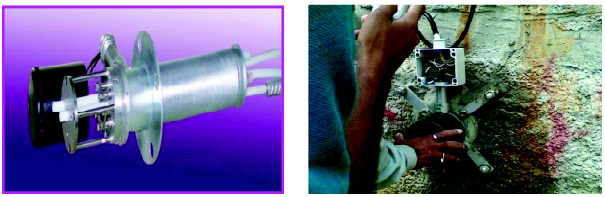


Fig. 12.03: Borehole Extensometer

(A) Borehole Extensometer: It is used to monitor displacement or deformation in soil and rock mass at various depths (Fig. 12.03). It normally consists of anchors, rods, protective tubes and a VW (vibrating wire) displacement sensor. The anchors coupled to the rod are installed in the borehole. The anchors and anchor rods referenced to stable ground, move up or down as movement in borehole occurs. This changes tension of the vibrating wire inside the VW transducer, which is transmitted through a cable to the readout unit.

(B) Multiple Point Borehole Extensometer: They are installed in borehole to monitor displacements at various depths. Relative movements between the anchors (which are fixed at different depths) and the reference head (which is common for all anchors) are measured (Fig. 12.04).

Fig. 12.04: Multiple Point Borehole Extensometer

4.2 Sub-surface Horizontal Movement: This is measured to forewarn about tunnel instability by monitoring ground movement towards excavation or heading and/or to verify adequacy of rock bolting/

other supports. The instruments used for this are as under:

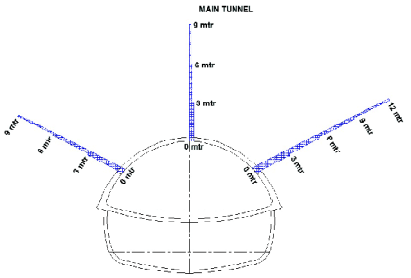


Fig. 12.05

(A) Borehole Extensometer: The Borehole Extensometer, details given in Para 4.1(A) above, can be used for measuring horizontal movements of rock mass around tunnel cavity, by fixing it in horizontal direction or the direction in which movement is to be measured (Fig. 12.05).

(B) Inclinometer: The inclinometer system consists of casing and measurement system (Fig. 12.06).

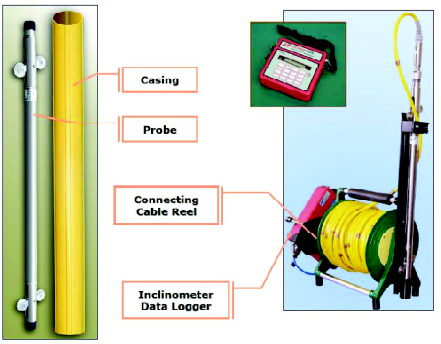


Fig. 12.06: Components of Inclinometer

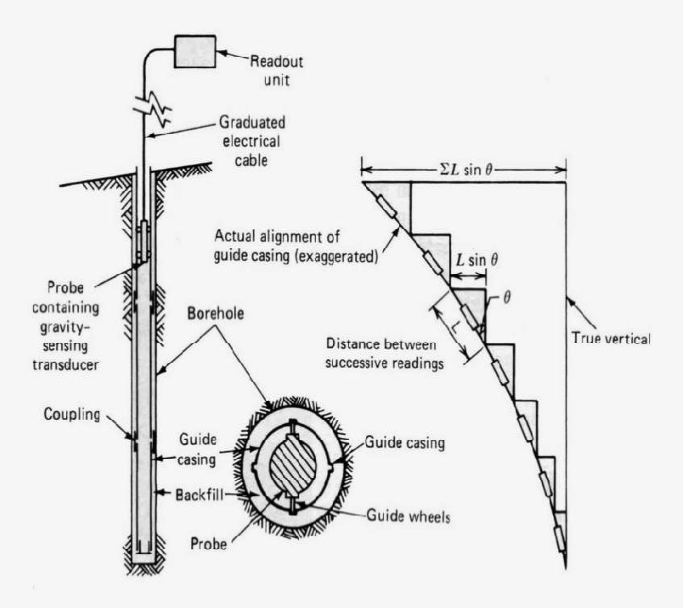


Fig. 12.07: Working Principle of Inclinometer

The casing provides contact for sub-surface measurements, while its grooves control the orientation of the inclinometer sensor and provide a uniform surface for measurements. The casing is usually installed in a borehole; however, it can also be buried in a trench, cast into concrete or attached to a structure. It is used for measuring angles of slope/tilt and deviation from true vertical (Fig. 12.07).

Plot of inclinometer readings over a period of time can show trend of sub-surface movements along the depth of borehole (Fig. 12.08).

There are many types of inclinometers like Electronic Inclinometer, Mercury Inclinometer, Manual Inclinometer and Gravity Inclinometer. The Electronic Inclinometer enables precise readings and is used very commonly.

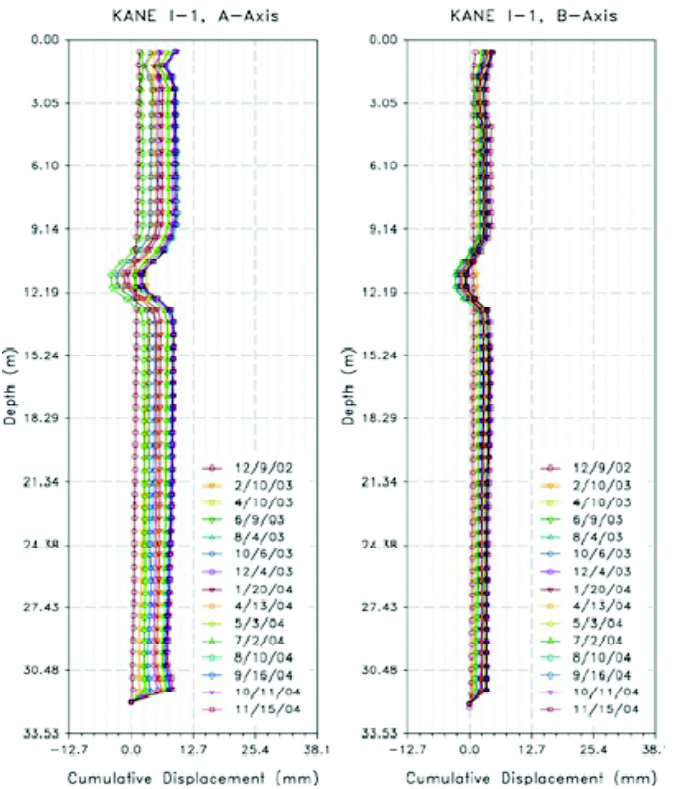


Fig. 12.08: Plot of Inclinometer Readings

4.3 Diameter or Width Change: This is done to check the convergence of the cavity after excavation. The instruments used for this are as under:

(A) Tape Extensometer: It is a portable device designed to measure any change in distance between these two points (Fig. 12.09 and Fig. 12.10). The instrument consists of a precision punched steel tape incorporating a repeatable tensioning system and

dial gauge readout. Both ends of the equipment are fixed to the anchors provided at the two points, between whom the change in distance is to be measured. A precision dial gauge measures the change in distance between two anchor points. They are normally used for maximum length of 25m.

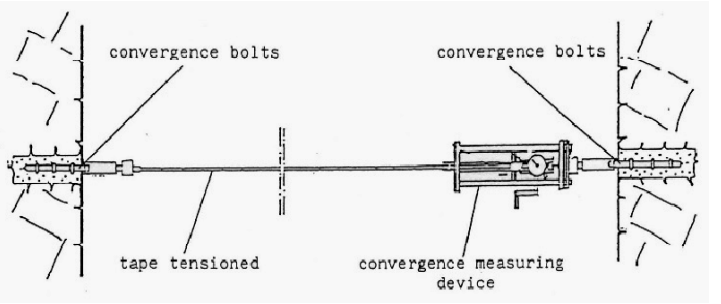


Fig. 12.09: Schematic diagram of Tape Extensometer

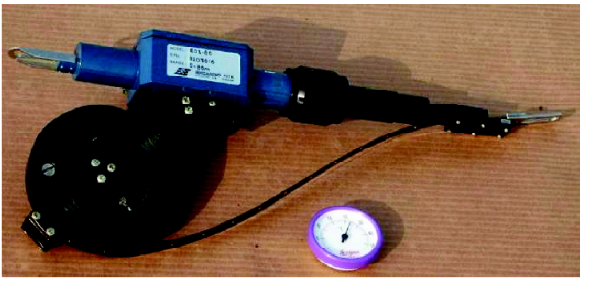


Fig. 12.10: Tape Extensometer

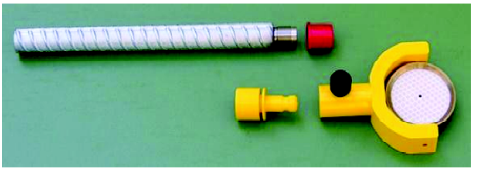


Fig. 12.11: Bi-reflex Targets

Fig. 12.12: Mini Prism Targets

(B) 3-D Optical Targets: Use of 3-D Optical Reflectors is a cost effective and it is precise product for recording deformations in tunnel, over a long period of time. These targets can be of Bi-Reflex or Mini Prism type (Fig. 12.11 and Fig. 12.12).

They consist of two parts, the bottom part (anchor bolt) is fixed permanently at the location where deformation is to be measured. When not in use, the anchor bolt is covered by a cap.

When the deformations are to be measured, the top part (the reflective target) is fixed to the anchor bolt and three-dimensional co-ordinates of the target is determined in an absolute reference system by optic-trigonometric surveying of targets in repeated measurement cycles, using a TotalStation or similar surveying equipment. This is achieved by also including a number of reference points which are considered stable in the survey. The measuring instrument is positioned to provide the best possible lines of sight to the targets and reference points and can otherwise be freely positioned (Fig. 12.13).

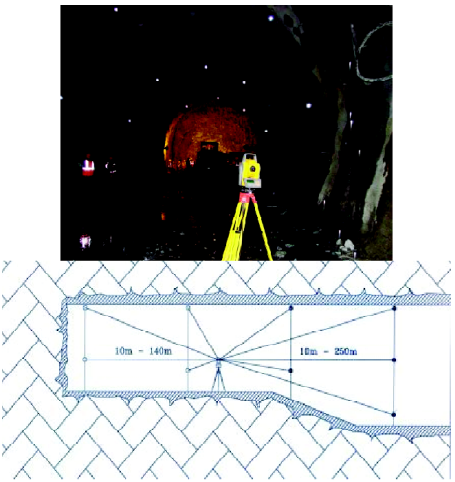


Fig. 12.13: Measurement of Deformations

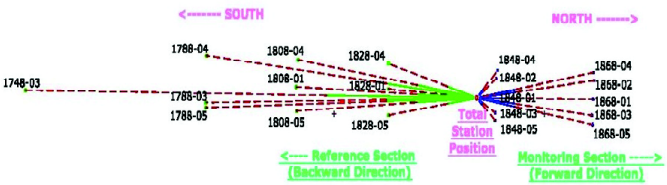


Fig. 12.14: Sample Deformation Measuring Scheme

These targets are used for measuring deformations during construction phase of the tunnel but some of the target are continued to be monitored during service life of the tunnel also as part of inspection and maintenance. A sample of deformation measuring scheme used in 10.96 km long Pir Panjal tunnel in Udhampur Srinagar Baramulla Rail Link (USBRL) Project of Indian Railway is shown in Fig. 12.14.

4.4 Tilt: Tilt is measured for the buildings affected by the tunnelling work. It is measured using Tilt meter. It is a sensitive equipment designed to measure very small changes from the vertical level, either on the ground or in structures. Tilt meters may be purely mechanical or incorporate vibrating-wire or electrolytic sensors for electronic measurement. A sensitive instrument can detect changes of as little as one second.

4.5 Load or Stress in Structural Supports: This is done to verify adequacy of structural support (rock bolts, ribs, etc.) and increasing knowledge of support behavior as input to improved design procedures. Various equipment used for this are as under:

(A) Load Cell: Centre hole type load cells are fixed in the rock bolt itself to measure the load taken by the rock bolts (Fig. 12.15).

(B) Pressure Cell: Pressure cells (Fig. 12.16) are used to measure the pressure between the two surfaces where they are fixed.

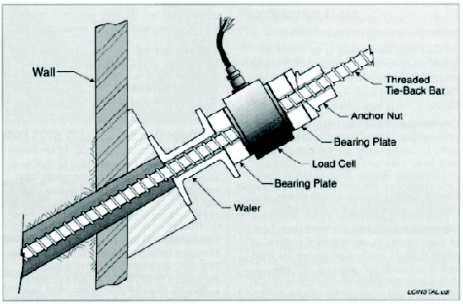


Fig. 12.15: Centre Hole Load Cells

Fig. 12.16: Pressure Cell

By fixing pressure cells in the appropriate direction, the tangential or radial stresses in the shotcrete can be measured (Fig. 12.17).

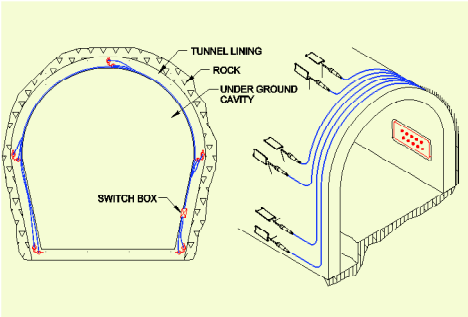


Fig. 12.17: typical use of Pressure Cell

Fig. 12.18: Measuring Anchor

(C) Measuring Anchor: They are sort of instrumented rock bolts and can be called as combination of rock bolt and extensometer (Fig. 12.18). They are used to record the extension of rock bolt.

(D) Shotcrete Strain Meter: This is used to measure stain in shotcrete (Fig. 12.19). Two parallel steel bars, embedded in the shotcrete at a defined distance to each other, distort a central tube when they move relative to each other. This distortion corresponds to the mean compression or elongation of the concrete in between. The distortion is measured by strain gauges. The full bridge strain gauge signal is transmitted to a data acquisition system via a 6-conductor cable.

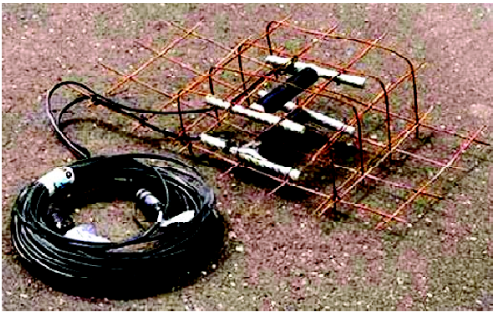


Fig. 12.19: Shotcrete Strain Meter

4.6 Pore water Pressure: This measurement is used for forewarning of distress to buildings due to movement of soil or water. It is done using Piezometer. It is a device which measures the pressure (more precisely, the piezometric head) of groundwater at a specific point. It is designed to measure static pressures. There are many types of piezometers available in the market like Electric, Hydraulic or Vibrating Wire type (Fig. 12.20).

Fig. 12.20: Vibrating Wire Piezometer

4.7 Groundwater Level: This is recorded to monitor draw down of groundwater table due to tunnelling work. This can be done by measuring groundwater level in observation wells, using water level sounder.

4.8 Vibrations: These are recorded to verify that ground and building vibrations due to tunneling activities do not exceed an acceptable limit. They are measured using Engineering Seismograph (Fig.

12.21), which is a device used for recording earth tremors. Basically, it is a heavily weighted horizontal rod (pendulum) suspended from a pole. This rod is free to swing from side to side if the earth shakes. One end of the rod rests against the pole, while the other holds a pen or stylus. This stylus marks a slowly moving roll of paper. If there is no shaking, the passing paper is marked with a straight line. If there is a tremor, the paper is marked with a squiggly line.

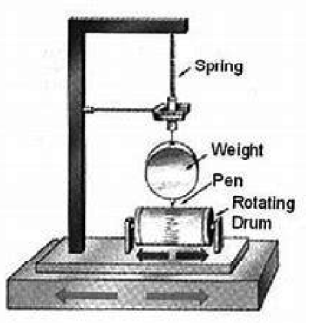


Fig. 12.21: Seismograph

A typical instrumented tunnel section is shown in Fig. 12.22.

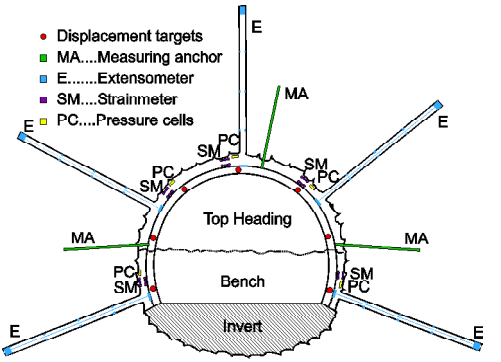


Fig. 12.22: Typical Instrumented Section

5. Monitoring Sections at Construction Stage: During the construction stage, measurements can be undertaken at Standard Monitoring Sections, Principal Monitoring Sections and on the Surface.

5.1 Standard Monitoring Section:

(a) The purpose of standard monitoring section is to evaluate the structural stability of the temporary support.

(b) At each monitoring section, the progressive displacement is determined by recording three dimensional coordinates on different days. These measurements can be used to derive settlements, convergences, divergences and displacements along the tunnel.

(c) The targets are fixed to the tunnel wall through Convergence Monitoring bolts. The layout of monitoring stations and their spacing depends primarily on the geological conditions. Typically, seven number targets are fixed in a cross-section – five in heading & two in benches.

(d) In zones of rock susceptible to swelling, levelling points are also provided in the invert.

(e) Following spacing can be adopted for the standard monitoring sections:

- Stable Rock: maximum 30 m
- Unstable Rock: maximum 20 m
- Squeezing Rock: maximum 10 m

(f) Survey points are installed at a distance of less than 1m from the face before the next round and surveyed (zero position). Subsequently, the points are normally surveyed at least once a day for the first few days. Near fault zones or in the event of heavy deformation, the interval is shortened. When a number of successive measurements show decreasing rates of deformation, the

measurement interval can be lengthened.

(g) The vertical movements of the points at the feet of the top heading cannot be directly surveyed since the points have to be about 0.8 to 1.0m above the invert in order to be recorded by instruments. If large displacements occur at the feet of the top heading, it is often better to measure the force at the foot of the top heading with load cells.

5.2 Principal Monitoring Sections: The main purpose of principal monitoring sections is to check the input data for the design and the suitability of the calculation model.

(A) Measurements: Following individual measurements are normally carried out:

- Horizontal/diagonal convergences and settlements of crown/foot points): Through optical surveying instruments (Total Station & Targets) or Tape extensometers.
- Displacement of the surrounding rock mass: Through Multipoint Borehole extensometers
- Strain & pressures on the outer support layer/steel arches: Shotcrete strain meters & Pressure cells
- Loading of the Rock Bolt: Axial Force Meter (to determine the load development along the rock bolt) & Rock Bolt Load Cell (to determine maximum anchor load and the degree of utilization of the anchor).
- In areas with clay minerals susceptible to swelling, invert heave is also measured.

(B) Number of instruments in each cross-section should be decided in consultation with designer & geologist.

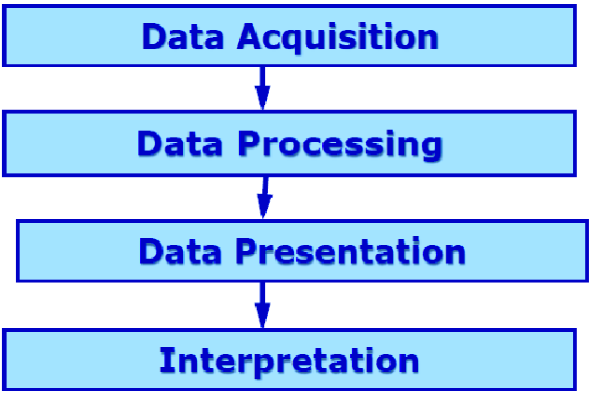
(C) Number of Principal Monitoring Sections: In addition to the standard monitoring sections, at

least two principal monitoring sections should be provided in every tunnel. It may also be appropriate to provide surface monitoring sections at the same location if it can be expected that deformation could extend to the surface. As soon as the working sequence has become established after driving the first 50 to 100m of tunnel, the first principal monitoring section should be setup. Subsequent spacing may vary from 150 to 500m depending on geological conditions.

5.3 Surface Monitoring: Surface measurements are taken to monitor open Excavations near portal and for monitoring surface movements in case of shallow tunnels, particularly when there are buildings in the area affected by the tunnel.

This normally entails precision levelling to observe the behaviour of settlement with time, supplemented when necessary with extensometer. All settlement points should be installed as to allow for reliable zero readings without any influence of construction activities.

6. Stages in Geotechnical Monitoring: Following are the stages in Geotechnical monitoring:



7. Graphical Presentation of Monitoring Data: The data recorded by instrumentation is processed/analyzed and then presented in required for, including graphical form, for better appreciation. This is done by the customized software, which are part of the instrumentation and monitoring scheme. Some of the commonly used graphical representations are listed here.

7.1 Time Displacement Diagram: Time Displacement diagrams are used to present vertical, horizontal and longitudinal displacement components versus time. Typically, the results of displacement measurements of all or one targets in one monitoring cross section are plotted in a single diagram. Construction phases are also presented on the same plot showing correlation between construction activities and displacements. The displacement history is used to assess stabilization process. A sample of time-displacement diagram for crown settlement, in a tunnel excavated with heading and benching, is shown in Fig. 12.23.

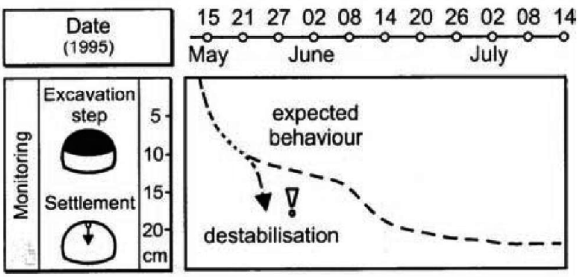


Fig. 12.23: Time Displacement Diagram

7.2 Vector Diagrams: The vector diagrams show plot of displacement development with time, at selected points, in sections perpendicular and parallel to the tunnel axis (Fig. 12.24 & Fig. 12.25). The vector plots can show the influence of

geological structure(s) and quality of surrounding ground, which has a significant influence on displacement characteristics.

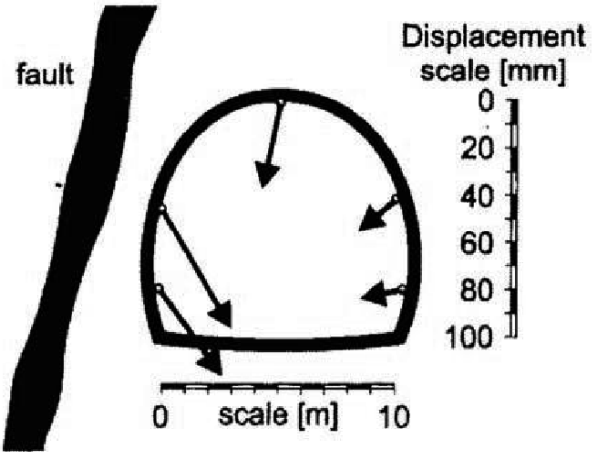


Fig. 12.24: Vector Diagram

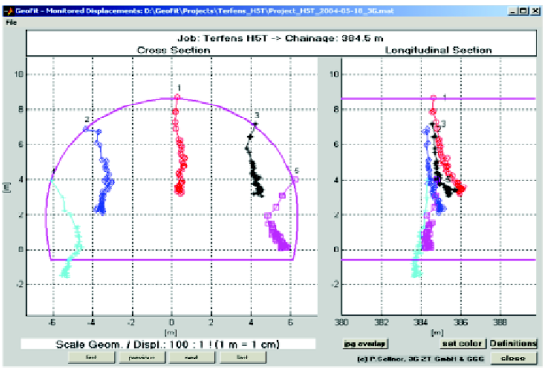


Fig. 12.25: Screen shot of Vector Diagram

7.3 Lines of Influence: Lines of Influence are produced by connecting displacement values of a number of monitoring points along the tunnel axis

at the same time. An example of lines of influence for "displacement of crown" due to top heading excavation is shown in Fig. 12.26. As the excavation approaches the Fault (9), in excavation Step-8, a significant deviation of the previously uniform behaviour can be observed. During tunnelling through the fault, a further increase in settlements is measured.

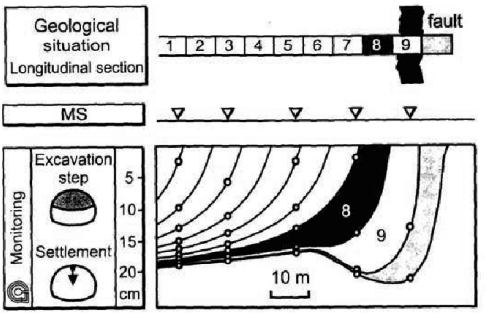


Fig. 12.26: Lines of Influence

7.4 Trend Lines: Trend lines are generated by connecting settlement values of individual lines of influence at a predefined distance behind the face (Fig. 12.27 & Fig. 12.28).

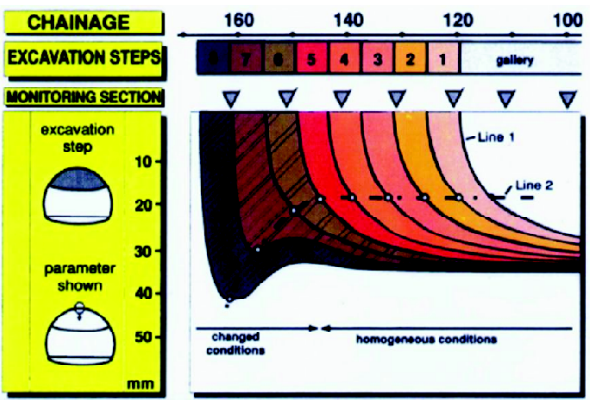


Fig. 12.27: Trend Lines

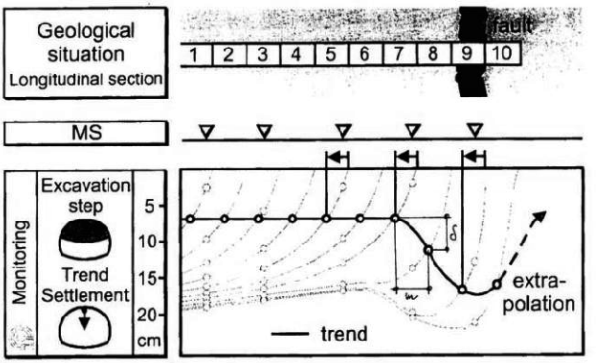


Fig. 12.28: Trend Lines

They give a good overview of the displacement development along the tunnel and are quite useful for extrapolation of the displacement behaviour ahead of the excavation face. Trend lines which show increasing displacement can indicate critical situations and must be considered as a serious warning signal.

8. Control limits in Observational Approach: In observational approach, various parameters (e.g. displacement, stress, force etc.) are monitored and based on the values recorded, various decisions are taken. For deciding action and type of action, control limits for various parameters are required to be set. Concept of various control limits and their application in observational approach is shown in Fig. 12.29.

Comparison of monitoring data with control limits will give first indication for identification of potential areas which are close to or exceeding design limits. For judgment of rock mass behaviour and performance of the primary support, control limits are normally established for:

- Primary lining displacements,
- Displacement velocities,
- Shotcrete strains,
- Settlements etc.

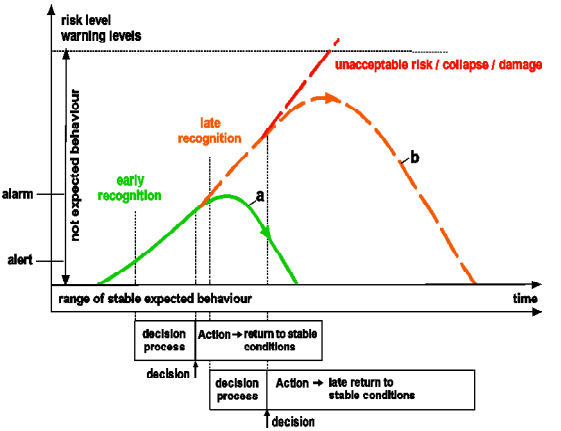


Fig. 12.29: Control Limits

The definition of control limits shall be considered as flexible and adjustable, which means control limits shall be updated regularly, if necessary, on basis of experience gained during construction.

8.1 Type of Control Limits: As shown in Fig. 12.29, normally three types of control limits are established by following three trigger levels:

(A) Alert Level: This relates to to threshold values, on exceedance of which, certain routines will be started to impose an increased attention and surveillance to these specific areas. It indicates that specific area is approaching a level where additional actions and/or contingency measures may be necessary.

(B) Alarm Level: This relates to threshold values, on exceedance of which the element of work may be approaching a critical state. The Geotechnical Engineer shall decide for the specific case and the overall support and rock mass performance.

(C) Action Level: This relates to threshold values on exceedance of which the element of work is considered to be outside the expected range of

assessed behaviour and may be close to its ultimate limit capacity. The overall performance shall be re-checked with a related risk assessment. Design review shall be done together with assessment of need for additional support. Additional support and/ or contingency measures, to guarantee safety of the works, shall be implemented. For ease of identification of an unacceptable safety risk, the works shall be stopped and remedial measures shall be implemented immediately.

8.2 Threshold Values for Control Limits: The parameters in tunnelling, on which mostly control limits are set, and their typical threshold values are discussed in following paras.

(A) Control Limits–Displacement Velocity: It shows rate of displacement with time, at monitoring points (Fig. 12.30).

Usually, time interval between successive observations is one day, with progress in top heading being 2-3m per day. It is an important indicator of stability development. Immediately after excavation, increase of displacement velocities is expected due to stress redistribution. After installation of the primary support, this must decrease and stabilize after excavation face has advanced further and stress redistribution is completed. Continuing/increasing displacement velocity indicates that rock mass is not stable and may indicate progressive destabilization.

As a guideline, the control limits related to measured displacement velocities are defined as follows:

Alert Level $\delta_n = 0.8\ \Delta_{(n-1)}$

Alarm Level $\Delta_n = 1.0\ \Delta_{(n-1)}$

Action Level $\Delta_n = 1.1\ \Delta_{(n-1)}$

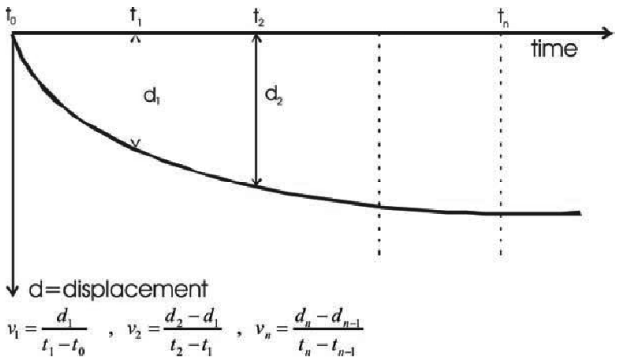


Fig. 12.30: Displacement Velocity

(B) Control Limits–Differential Settlement: The Differential Settlement between the top heading crown and the top heading footing ($\Delta_S = S_{crown} - S_{footing}$) shall be monitored to identify potential instabilities at the shotcrete lining footing.

As a guideline, the control limits related to differential settlement are defined as follows:

Alert Level	+ 5mm
Alarm Level	+ 1mm
Action Level	- 3mm

(C) Control Limits-Trend Lines: Control Limits for trend lines are defined in terms of ratio "δ/ Advance".

Where, "δ" is increase in displacement, and

"Advance" is corresponding face advance.

As a guideline, the control limits related to the ratio above are defined as follows:

Alert Level	10^{-3}
Alarm Level	5×10^{-3}
Action Level	10^{-2}

(D) Control Limits–Shotcrete Strain: As a guideline, the control limits for shotcrete strains (in %) are defined as follows:

Alert Level	0.2%
Alarm Level	0.4%
Action Level	0.6%

9. Ignoring Instrumented Data: If the data obtained from instrumentation is not properly interpreted in a timely fashion, or if no action is taken based on this data, the instrumentation program will serve no purpose. Some of the examples of collapse/unsafe conditions as a consequence of ignoring instrumented data, are shown below:

Fig. 12.31 - The deformation started increasing significantly from 30th December onwards. Ignorance of this led to collapse of cavern on 6th January.

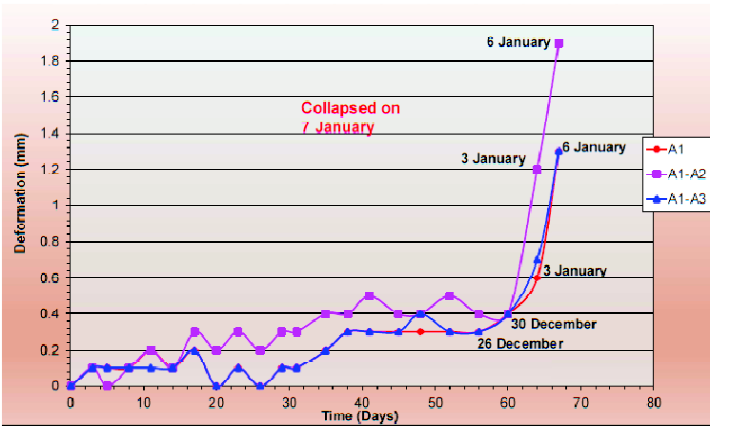


Fig. 12.31: Collapse of a Cavern

Fig. 12.32 - The deformation, which had stabilized, started increasing significantly from 16th March onwards. Ignorance of this led to collapse of cavern on 1st April.

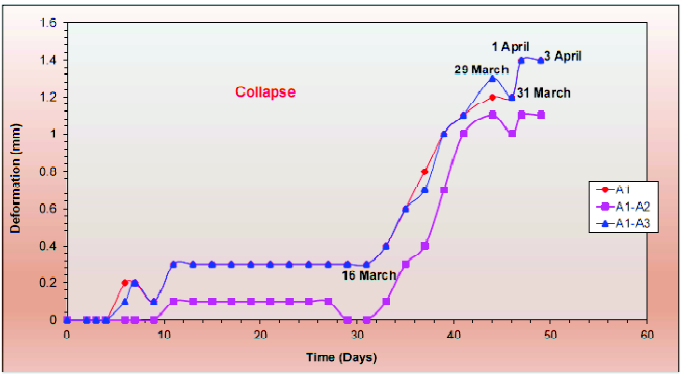


Fig. 12.32: Collapse of underground excavation

Fig. 12.33 – Sudden increase in deformation, after about 150 days, is an impending sign of collapse.

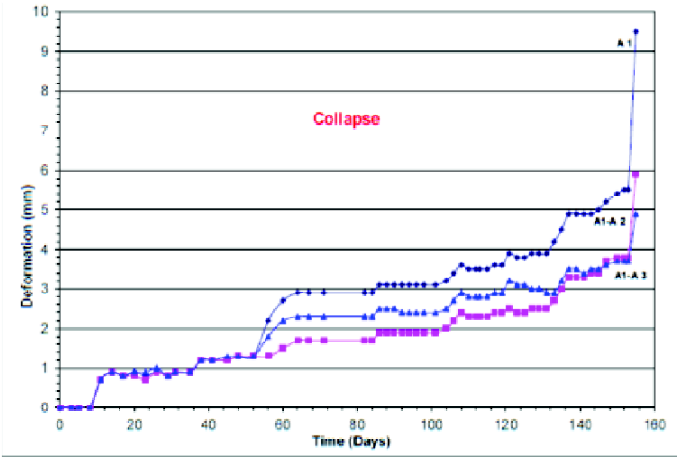


Fig. 12.33: Signs of Collapse

Fig. 12.34 – Sudden increase in load taken by anchors, after about 150 days, is an impending sign of collapse. This is for same site shown in Fig. 12.33.

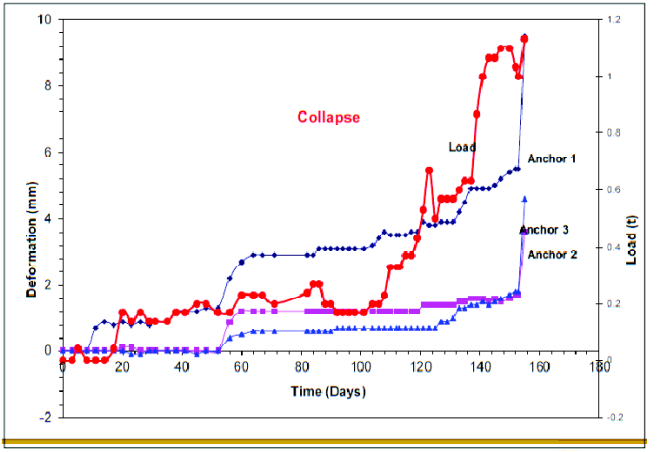


Fig. 12.34: Signs of Collapse

10. Contract Document: Responsibilities for installation & commissioning, calibration, monitoring, information flow, data interpretation and reporting etc. must be clearly defined in contract documents.

CHAPTER-13

DRAINAGE AND WATER PROOFING IN TUNNELS

Presence of water inside the tunnel affects the progress of construction work as well as the operation of the completed tunnel. A dry tunnel provides a safer and friendlier environment and significantly reduces construction, operation and maintenance costs. Adequate geological and hydrological investigations should be done in advance so as to identify the stretches where water ingress is expected to be encountered along with a reasonable assessment of the quantity of water. The water entering the tunnel, during the construction stage, has to be collected and drained away until the final measures planned for the completed tunnel are implemented.

1. Stages of providing Drainage: The design of permanent drainage system and control systems begins during the geotechnical exploration phase with an assessment of stretches where water ingress is expected to be encountered along with a reasonable assessment of the quantity of water. The stages of providing drainage and water proofing arrangements in tunnels are as following:

1.1 Pre-drainage: Water should be prevented from entering the tunnel before starting the tunnel construction work. Following are the methods/ systems employed for this purpose:

(A) Diverting water channels: All existing water channels, water from which can find entry into the tunnel, should be diverted away from the tunnel (to the extent possible) or such channels may be sealed/ lined to prevent entry of water from these channels into the tunnel. This requires detailed survey of the area around the tunnel.

(B) Ground Water Pumping: This may be done by means of collecting water in ponding/collection areas near the tunnel and then pumping it away from the tunnel area.

(C) Grouting: Appropriate type of grouting (cement/ chemical/resin based) besides reducing the permeability of strata also increases the stability of the ground. A grouted body in form of close ring is created ahead of tunnel advance (Fig. 13.01). This ring helps in resisting hydrostatic pressure also.

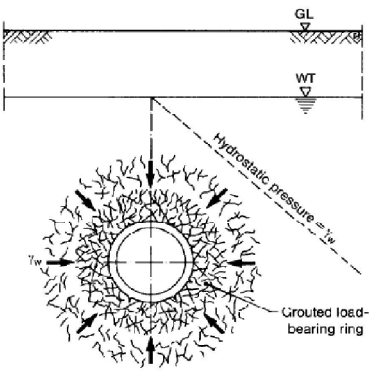


Fig. 13.01: Grouting

The suitability of the ground and the grout material for grouting depend on the geological, hydrological and chemical factors including mineralogical composition of the soil, grading, permeability of the ground, hydrostatic pressure of the groundwater, speed of groundwater flow and chemical properties of the groundwater. Use of grouting is particularly suitable when water pressure in joints is very high and their volume is small.

(D) Ground Freezing: In this method, the pore water is converted to ice by circulation of a chilled liquid via a system of small diameter pipes placed in drilled holes. Ice creates a frozen mass of soil and/or rock particles, with improved compressive

strength and impermeability. Brine is typical cooling agent, although fast acting liquid nitrogen can be used for projects where the freeze only needs to be quickly established and maintained for a short period of time.

Ground freezing can temporarily seal and consolidate the ground under conditions that are water-bearing but not suitable for grouting. The availability of technology and experience for ground freezing is rather limited in India.

1.2 Drainage during Construction: During construction, water may come in tunnel from:

(i) Wash water, which is used for washing drill holes and water from other construction related activities; and

(ii) Ground or sub-soil water (if the base of tunnel is at lower level than the ground water level).

Proper planning should be done to prevent the water in tunnel creating obstruction in the construction work. Precaution should be taken while dewatering the area outside the excavation limits, because lowering of water table could cause settlement of existing structures, impact the vegetation and drying of existing wells. Various ground water collection and dewatering measures can be used either individually or in combination depending on various factors including the ground conditions and the construction process. These include:

(A) Construction of Cutoff Walls: Impervious retaining walls, such as steel interlocking sheeting or concrete slurry wall, could be placed into deeper less pervious layers, to reduce ground water inflow during construction and limit drawdown of existing ground water table.

(B) Use of Plastic Gutters, Channels and Pipes: Plastic Gutters and Channels of various profiles can be used to collect localized water ingress and drain it out, during tunnel construction (Fig. 13.02).

Fig. 13.02: Use of Pipes

Fig. 13.03: Drainage or Dimpled Mats

(C) Drainage or Dimpled Mats These mats, made out of plastic (Fig. 13.03), are used when large quantity of groundwater is expected to appear from large areas. The dimpled side should ideally be installed towards the ground/rock. They are often laid on the shotcrete support layer followed by in-situ/sprayed concrete inner lining (with or without waterproofing).

(D) Dewatering Pumps: They are particularly required when tunnel is progressing downhill or when the available longitudinal slope is inadequate for draining out the water. Sumps at appropriate locations need to be constructed for this purpose. This can be in the form of "Deep Wells", around perimeter or along alignment of underground excavation (Fig. 13.04) or can be in the form of "Well Points", perforated tubes (covered with screen), sunk in ground (Fig. 13.05).

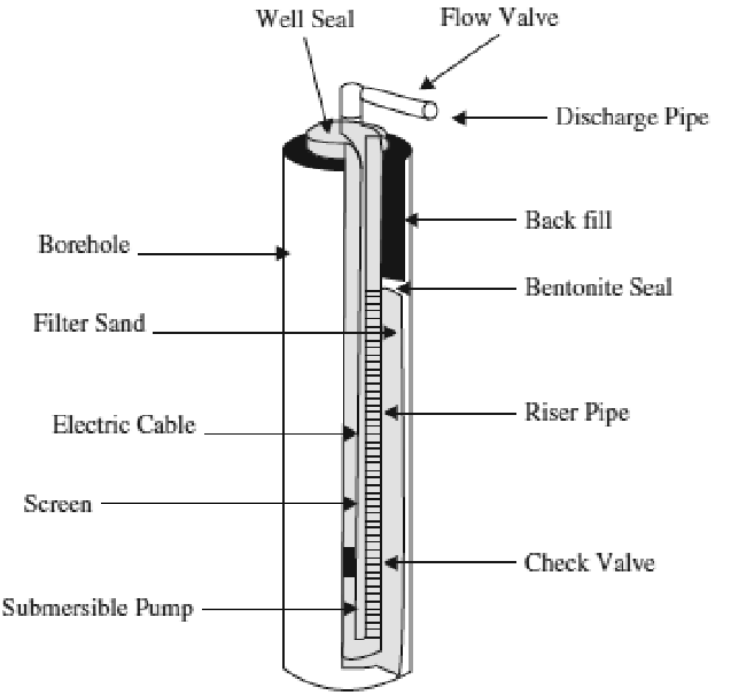


Fig. 13.04 Deep Wells

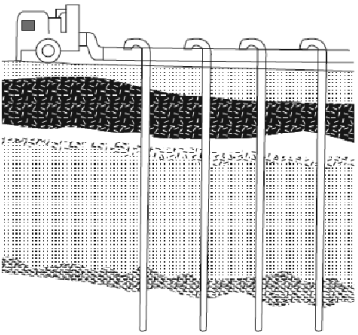


Fig. 13.05: Well Points

(E) Drainage Boreholes before Tunnel Advance: Such boreholes (Horizontal Drains) should be done when heavy water inflow and/or sudden (but not predictable) water inflows can be expected. Advance drainage is expected to considerably simplify the subsequent work because flow pressures can have severe adverse effect on structural stability.

Slotted hard PVC pipes or perforated plastic drainage pipes with glass wool filters are often inserted into the drainage holes (typically 35 to 100mm diameter) if flow of water can lead to erosion of surrounding strata. The pipes are normally extended with a flexible plastic hose at the exposed end of the hole to drain the water into the side drainage channels or into a temporary drainage pipe hung from the wall.

(F) Drainage boreholes after Tunnel Advance: Drainage boreholes drilled after the advance can prevent build-up of pressure behind the shotcrete layer and also help in reducing static pressure on the support. Drainage boreholes are particularly useful for draining out localized water. The extent to which water pressure is relieved by drilling depends on location of the boreholes, their direction, spacing, length and diameter. The most favourable

combination of spacing, length and diameter of the holes should be determined experimentally.

In Tunnel T-3 in Udhampur – Katra Section of Udhampur Srinagar Baramulla Rail Link (USBRL) Project of Indian Railway, there was heavy flow of water through the drainage holes constructed (Fig. 13.06) and stopping this heavy flow required construction of a series of grout curtain walls on the side of tunnel walls, along the tunnel axis (Fig. 13.07).

Fig. 13.06: Heavy Water Inflow

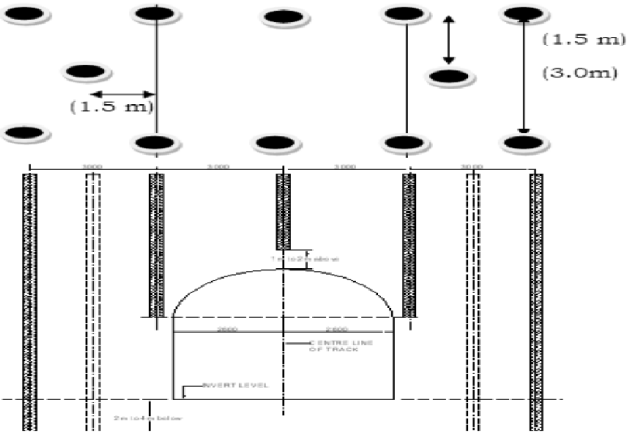


Fig. 13.07: Grout Curtain Walls

1.3 Permanent Drainage System: In operational phase of tunnel, a permanent drainage system is required in tunnels to drain out the water that could accumulate in tunnel from Rainfall, Tunnel washing operations, Tunnel seepage and Fire-fighting operations.

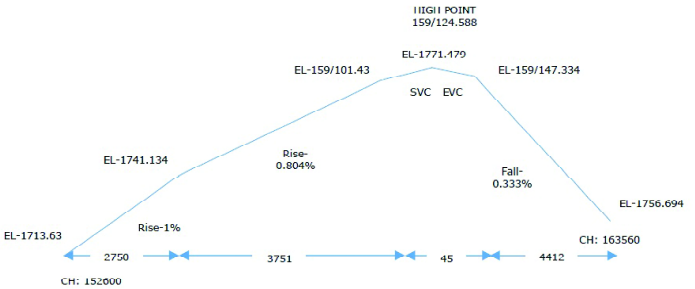


Fig. 13.08: Gravity Flow System

Water accumulated in tunnel can be drained out either by a gravity flow system or a pumped system. Gravity flow system will suffice for tunnels with continuous grades. Example of drainage under gravity flow system is Pir Panjal tunnel (10.96 km long) in Udhampur Srinagar Baramulla Rail Link (USBRL) Project of Indian Railway (Fig. 13.08) which is having peak point in the middle of the tunnel and 6534m and 4426m lengths on both sides having falling gradient towards tunnel exits, with a vertical curve of radius 2500m joining the two grades. If a low point occurs within the tunnel, it may be necessary to have a pumped system to drain out the collected water.

2. Extent of Waterproofing: The degree of extent of waterproofing in tunnels can be of following types:

2.1 Watertight/Waterproof Tunnel: In such tunnels no water is allowed to enter the tunnel and, therefore, no drainage system is constructed inside the tunnels (Fig. 13.09). “Tunnel De Viret”

in Lausanne Metro, in Switzerland, is an example of such tunnel.

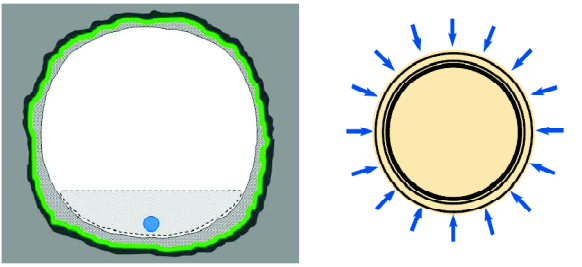


Fig. 13.09: Watertight / Water Proof Tunnel

A watertight tunnel does not adversely affect natural groundwater regime. Operation and maintenance of such tunnels is relatively much simpler due to absence of water inside tunnel. Such tunnels are environment friendly and ground settlements due to water drainage are reduced. The disadvantages of these tunnels include higher construction cost and difficulty in ensuring a reliable long term water proof system.

2.2 Partially Drained Tunnel: Such a system involves drainage of water until selected limit water level or selected portion of the tunnel cross section (Fig. 13.10) or up to permissible water drainage quantity.

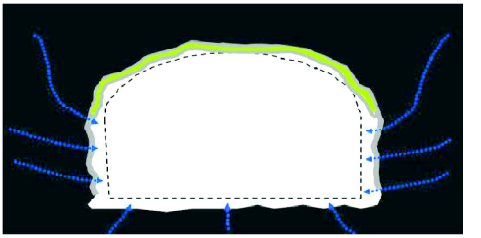
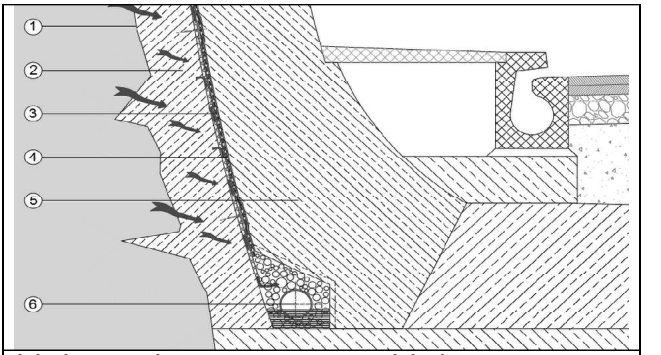


Fig. 13.10: Partially Drained Tunnel

The advantage of such system is that impact on the groundwater regime remains within specified

limits. The cost of construction of such a tunnel is high due to the required elaborate drainage system. The availability of technology and experience for such system is presently not available in India.

2.3 Fully Drained Tunnel: In such tunnel, the whole quantity of water coming through the cross section of the tunnel is collected inside the tunnel and then drained out of the tunnel using Permanent Drainage System (Fig. 13.11).



(1) Theoretical Excavation (2) Shotcrete
(3) Drainage Sheet (4) Waterproofing Layer
(5) Concrete Lining (6) Line Drainage

Fig. 13.11: Drainage System

The construction cost of a fully drained tunnel is relatively low, since elaborate pressure-tight waterproofing and an inner lining designed to resist pressure are not required. In addition, such tunnels function relatively reliably. However, environmental issues arising out of depletion of ground water need to be considered. Such tunnels entail higher maintenance cost.

The decision on selecting appropriate drainage system should be taken keeping in view ground conditions, expected water inflow, technical feasibility and cost effectiveness. Completely drained tunnel should generally be the preferred option for all new tunnels.

3. Requirements of Water-tightness in Tunnels

(i) Following classification should be adopted for specifying degree of water tightness in the contracts:

Table 13.01: Classification of Water-tightness

Water-proofing Class	Damp Characteristics	Water proofing requirements
1	Completely dry	The sides of the lining must be so waterproof that no damp patches are detectable on the inner face.
2	Largely dry	The sides of the lining must be so waterproof that only a slight dampness (e.g. noticeable through discolouration) is detectable on the inner face in isolated locations. When the slightly damp patches are touched with the hand, no trace of water should be left on the hand. Blotting paper or absorbent newspaper laid against the patch may not discolour due to moisture absorption.
3	Capillary moisture	The sides of the lining must be so waterproof that only isolated and localized patches, which are wet to touch, occur. Patches, which are wet to touch, are defined in that moisture penetration of the tunnel sides is noticeable and blotting paper or absorbent newspaper laid against the patch discolors due to moisture absorption, but no water drips occur.

(ii) Waterproofing scheme should take into account type, quantity and aggressiveness of the water acting on the tunnel structure:

(iii) The water proofing scheme/material should provide:

- Permanent resistance against the prevailing mix of soil and water, including all chemical contents.
- Resistance against all adjacent construction materials like shotcrete admixtures, grouting chemical etc.
- Resistance against the expected static and dynamic loading.
- Adequate mechanical strength.
- Resistance in case of fire. Attention should be paid not only to the flammability but also the release of poisonous fumes.
- Durability for the design life.
- Ease of installation, maintenance and repair.
- Adequate Construction detailing.
- Environmental compatibility- materials used should not contaminate percolating water or groundwater.
- Bedding between protective layers.
- Division of the waterproofing into compartments in order to be able to localize and repair any leaks.
- Multi-layer construction of the waterproofing, or if there is one layer, reliable feasibility of checking its function.

- Feasibility of testing with appropriate testing procedure.
- It must be possible to clearly describe the waterproofing system and its technical and material specification in the contract.
- It must be possible for the contractor to comply with the guarantee requirements of the specification.

(iv) Waterproofing system should always be an integrated system that takes into account intermediate construction stages, final conditions of structures and their ultimate usage, including maintenance and operations.

4. Waterproofing Systems: Available waterproofing systems fall under following two categories:

4.1 Rigid Systems: These include water-resistant plaster, sealing mortar and resin concrete. One major disadvantage of this system is its dependency on quality of application. However, experience in use of synthetically modified concretes or mortars as sealing materials for tunnels is rather limited.

4.2 Flexible Systems: Flexible systems include following:

(A) Bitumen Waterproofing Layer: Bituminous waterproofing materials are now hardly used in underground tunnelling, because of following reasons:

- Bituminous waterproofing materials need a mostly dry and flat support, requiring profiled levelling of the tunnel sides.
- Installation of reinforcement for the inner lining can easily damage the waterproofing layer.

- Groundwater that cannot be completely collected leads to waterproofing not adhering to the support with a risk of defects.
- Enhanced fire risk.

(B) Use of Geo-synthetics: Geo-synthetics, manufactured by polymers, are used very commonly for waterproofing because of their good mechanical properties (high tension strength, high failure strain, good flexible behaviour etc.).

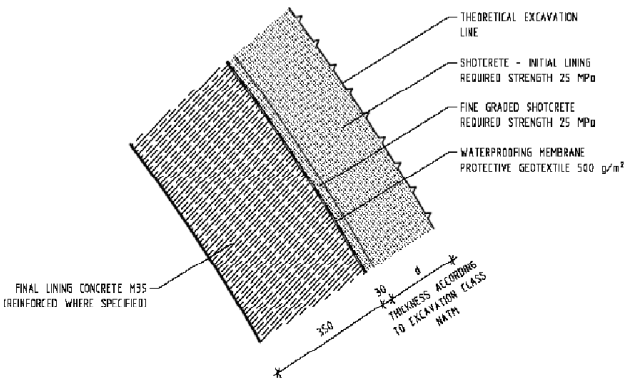


Fig. 13.12: Geo-synthetic Waterproofing System

Tunnels in rocks are waterproofed by a geo-composite sandwich (Fig. 13.12) consisting of:

- A non-woven geotextile layer on the initial shotcrete layer.
- A waterproof HDPE/PVC geomembrane.
- Cast-in-situ/sprayed final (inner) lining of concrete.

Typical construction sequence involved in placing this waterproofing layer is as following:

(i) Laying a non-woven geotextile layer directly on the initial shotcrete layer, held in place by pins/nails driven or shot into the rock (Fig. 13.13).

(ii) Over this, a continuous waterproof HDPE/ PVC geomembrane is installed (Fig. 13.14).

(iii) The membrane has to be cut and fit to all shapes and corners, welding together (by heat) to make a continuous waterproofing membrane (Fig. 13.15).

(iv) Membrane joints are tested for water tightness (Fig. 13.16).

(v) Finally, cast-in-situ/sprayed lining of concrete is placed to provide inner (final) lining, without damaging the membrane.

Fig. 13.13: Fixing Non-woven Geotextile Layer

Fig. 13.14: Laying Geomembrane

Fig. 13.15: Membrane Welding

Fig. 13.16: Testing of Membrane Joints

(C) Sprayed Waterproofing Layer: It involves spraying of the polymer based waterproofing material in liquid form, of minimum 2mm thickness, directly onto the support. Termed as composite shell lining system, it consists of a double-bonded spray applied membrane embedded between two concrete linings (Fig. 13.17). This bonding property makes the interface between membrane and concrete

impermeable. The waterproofing material is sprayed in liquid form directly onto the concrete. Common materials include Reaction resins, Cement-plastic combinations, Polymers, Bitumen-plastic combinations. The addition of about 20% by weight of glass fiber 3 to 5cm long can also improve the properties.

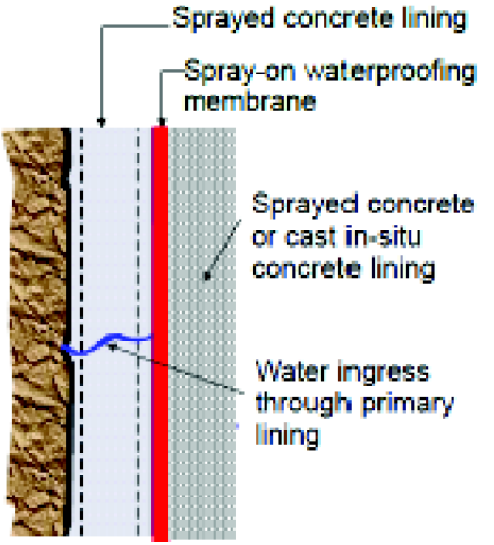


Fig. 13.17: Sprayed Waterproofing Membrane

The main advantages of sprayed waterproofing system are absence of seams, easy installation and easy to repair any defective patch. One major limitation is that the application area should not have any local bumps larger than the layer thickness. The conditions in a tunnel (dampness etc.) may also cause problems for the application. High humidity can delay the setting process of sprayed waterproofing materials.

CHAPTER-14

TUNNELLING IN WEAK/SOFT GROUNDS

Tunnelling in weak/soft ground presents different challenges because of the fact that such grounds have very less (or almost zero) standup time and before creation of cavity as well as after creation of cavity, they require extensive stabilization measures to stabilize the ground again. In soft/weak strata, ground loosening breaches the integrity of natural arch, with a consequence that without supporting the excavation, soon after it is completed, the walls may squeeze together and the roof may fall in. Some of the "tunnel excavation methods in soft ground" have been elaborated in Chapter-10. In this chapter, the whole issue of "Tunnelling in Soft/Weak Grounds" is being elaborated in detail.

1. Conventional Methods: Before the advancements in tunneling technology, from second part of 20th century onwards, the soft ground tunnelling was mostly done by "multiple drift" methods. There were many versions of it, mostly originated from European countries.

1.1 Belgian Method: In this method, the tunnel section was excavated in stages; starting from the part of the top heading, then widening it sideways and finally opening the benching portion in stages (Fig. 14.01). The openings created in stages were supported on timber struts.

This method was first employed in Chaleroy tunnel (in Belgium) in 1828. But experience of using this method was catastrophic during construction of Gotthard Tunnel (1872-1882). The key problem was that the sequencing following top heading required the arch to be underpinned. However, this

proved difficult in the yielding ground conditions encountered, leading to the timbers giving way, followed by the cracking or total collapse of the masonry arch.

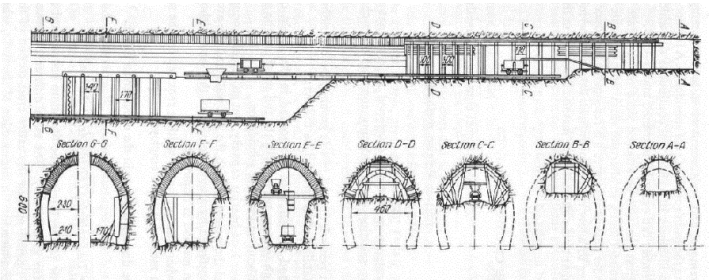


Fig. 14.01: Belgian Method

1.2 German Method: The underlying principle of this method was to leave a central bench of ground to be excavated last and to use it to support roof and wall timbering (Fig. 14.02). This allowed the arching to be built in one operation (unlike the Belgium method).

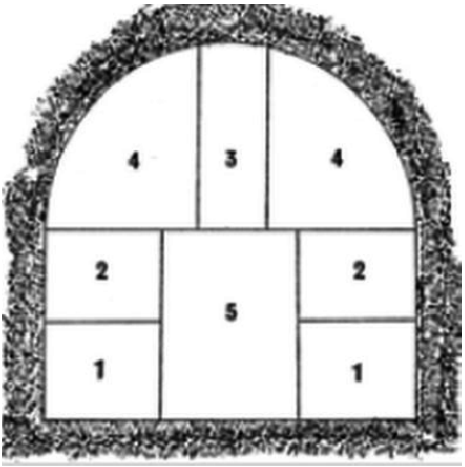


Fig. 14.02: German Method

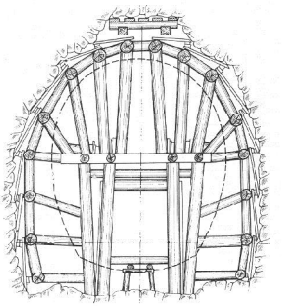


Fig. 14.03: English Method

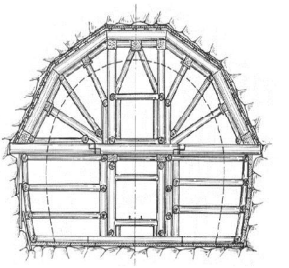


Fig. 14.04: Austrian Method

1.3 English Method: In this method, the excavation started from a central top heading (Fig. 14.03), which allowed two timber crown bars to be hoisted into place. Development of the heading then allowed additional bars to be erected around the perimeter of the face with boards between each pair to exclude the ground. The system is economical, permits construction of the arch of the tunnel in full-face excavation, and is tolerant of a wide variety of ground conditions, but depends on relatively low ground pressures.

1.4 Austrian Method: This method requires a strongly constructed central bottom heading upon which a crown heading was constructed. The timbering for full-face excavation was then heavily braced against the central headings, with longitudinal poling boards built on timber bars carried on each frame of timbering (Fig. 14.04). As the lining advanced, so was the timbering propped against each length to maintain stability. The method was capable of withstanding high ground pressures but had high demand for timber.

2. Ground Improvement: Ground improvement in tunneling signifies improvement in mechanical and hydrological properties of the ground, before or during

or after tunnel excavation, to facilitate tunnel excavation and ground stabilization. Ground improvement may be required:

- To create an improved zone for the excavation, prior to excavation.
- To prevent instability during construction, related to poor geotechnical properties (loose and un-cemented ground, flowing ground).
- To control water inflow when excavating.
- To control settlements of the structures above/around excavation.

Ground improvement is a general term for all kinds of improvement techniques. Technically it can be of following categories:

(i) Ground Consolidation: This term is used when the ground is consolidated to improve the properties (e.g. draining the water, reducing the voids in the soil matrix which increases the density etc.).

(ii) Ground Compaction: Increasing the density of soil by compressing soil or applying external load on it, to reduce the void space in the soil matrix.

(iii) Ground Treatment: This term is used when the ground is treated with some compounds (chemicals, resins, lime, and etc.) which results in increase in strength and decrease in permeability of the ground.

(iv) Ground Reinforcement: This term is used when the ground is reinforced with steel or fiber glass elements or geo-synthetics increase its load bearing capacity and shear strength.

Some of the commonly used ground improvement techniques, used for tunnelling, are elaborated in Para-3 to 7 below.

3. Drainage: Control of groundwater is important in soft ground tunnelling. The presence of a small amount of water in granular soils above the water table may be beneficial in providing an increase in stand-up time because of apparent cohesion brought about by negative capillary forces (until they dissipate), but below water table the presence of water reduces effective strength drastically and seepage pressures cause rapid and complete failure in the non-cohesive soil. Presence of water in clays is of primary importance in determining the strength, sensitivity and swelling properties of the material.

In some cases the tunnel construction is only possible with the application of special dewatering measures. For various stages for providing drainage (or dewatering) and various methodologies the Chapter-13 on *"Drainage and Waterproofing in Tunnels"* may be referred.

In case of low overburden, dewatering measures can be carried out from the ground surface. In other cases, dewatering has to be done from the tunnel cross section or from pilot tunnels.

4. Grouting: Several types of grouting are used to modify and/or stabilize soils in-situ. Recent improvements in grouting have made it a valuable tool in both groundwater control and soil stabilization for tunnelling projects. It can be very effective in following situations:

- To strengthen loose or weak soil and prevent cave-ins due to disturbance of loose, sensitive or weak soils by the tunnelling operations.
- To decrease permeability and in-turn grou-ndwater flow.
- To reduce the subsidence effects of dewatering or to prevent the loss of cines from the soil.
- To stabilize sandy soils those have a tendency to run in a dry state or to flow when below the water table.

4.1 Location for Grouting: Grouting can be carried out in the tunnel excavation as face grouting or as radial grouting from the excavated tunnel or from a pilot tunnel (Fig. 14.05).

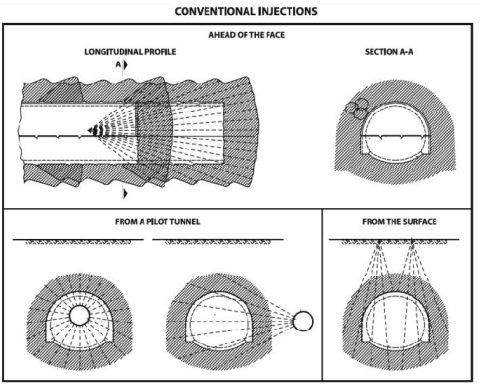


Fig. 14.05: Grouting Locations

4.2 Groutability of Ground is primarily determined by the permeability of the ground or the percentage fines (passing 75 micron sieve). The thumb rule is that soil having less than 10% fines could be successfully grouted and those with more than 20% fines could not. However, advancements in grouting technology have raised this limit approximately by 5%. Groutability is also assessed by using Groutability Ratio (N), as given in Table 14.01

Table 14.01: Groutability of different Grounds

Ground Type	N =	Groutability
Soil	$(D_{15})_{Soil}$ / $(D_{85})_{Grout}$	Yes if > 24 No if < 11
Cohesive Soil	$(D_{10})_{Soil}$ / $(D_{95})_{Grout}$	Yes if > 1 No if < 6
Rock	Width of Fissure)/ $(D_{95})_{Grout}$	Yes if > 5 No if < 2

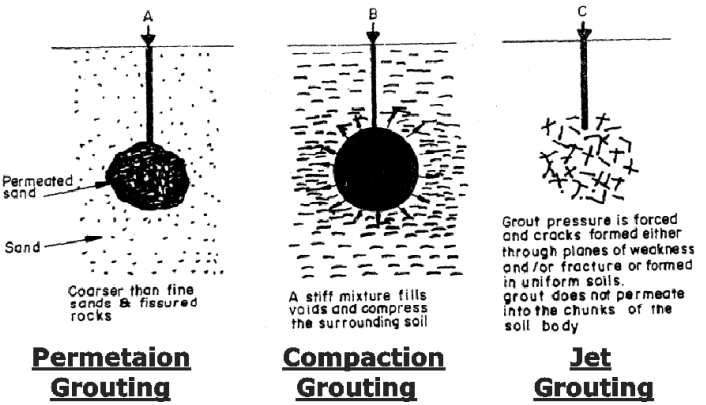


Fig. 14.06: Types of Grouting

4.3 Types of Grouting: Three ways of introducing grout material into the soil are possible (Fig. 14.06):

(A) Permeation Grouting: In this method, the grout fills the voids in the soil and there is no change in volume or structure of the original ground. Permeation grouting may be done with either cement based or chemical based grout, with latter being necessary for satisfactory penetration of fine soils. This type of grouting can be used for creation of a support ring around the tunnel excavation boundary or to create support for foundation of any structure in the vicinity of tunnel.

(B) Compaction or Displacement Grouting: In this method, soil is densified during tunnelling by injection of a stiff grout. The thick mortar mix acts as a radial hydraulic jack, creating bulbs or lenses and thus displacing and compressing the surrounding soil. This type of grouting is useful in controlling settlement of foundation of structures located above the tunnel or underpinning of foundations of structures located in vicinity of the tunnel.

(C) Jet Grouting : In this method, the ground is fragmented by deliberate hydrofracturing, in order to increase total stresses by wedging action of sucessive thin grout lenses, to fill unconnected voids and possibly consolidate the soil under injection pressure. Jet grouting can be used for:

- Forming an umbrella (canopy) ahead of the face.
- Reinforcing and stabilizing the tunnel face.
- Reinforcing the walls of tunnel.
- Underpinning the steel ribs.
- Creating impermeable diaphragms (e.g. before starting the excavation with TBM).

Jet grouting is applied mainly horizontally or at a slightly upward or downward angle from within the face of the tunnel. An improvement of the roof arching behaviour is achieved by applying one or more layers of jet grouting columns in stages corresponding to the excavation operations.

An improvement of the stability of the face is achieved by placing individual jet columns parallel to the direction of advance in the working face.

Less common in tunneling is vertical or steeply inclined jet grouting, except in shallow tunnels where it is applied from the surface. From within the tunnel vertical or steeply inclined jet grouting is mainly applied to underpin the bottom of the roof arch.

4.4 Grouting Material: The most commonly used grout material is cement. In special cases chemical products such as resins or foams are also applied. In these cases, the environmental and safety restrictions have to be considered specially. Fig. 14.07 may be referred as a rough guide for assessing material of the grout to be used.

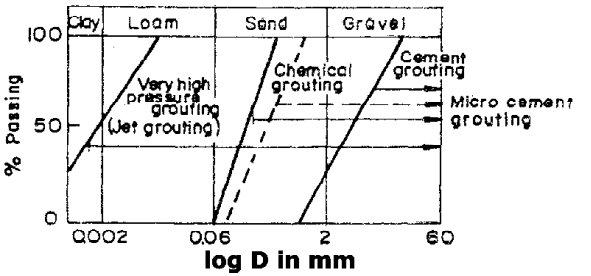


Fig. 14.07: Grout Material Suitability

5. Ground Freezing: In this method, pore water present in the soil is converted into ice by extraction of the latent heat. The ice then acts like a cement to bind the soils grains together, thereby raising the strength and lowering the permeability of soil mass. This method is successful only when sufficient water pore water is present in the ground. It may be noted that presence of organic material or salt water will result in greater difficulty in freezing. Another major deterrent is moving ground water, which makes the freezing difficult.

Following advantages can be derived by the freezing:

- Makes water-bearing strata temporarily impermeable.
- Increases compressive and shear strength of ground.
- Provides structural underpinning (temp-orary supports).

5.1 Refrigeration Process: The typical freezing installation consists of a refrigeration plant that cools a brine solution, which is then pumped down the center of an annular freeze pipe to the bottom of the hole, returning via the outer annulus in contact with the soil. The warmed brine is returned to the refrigeration plant and the cycle continues. In practice, a number of freeze pipes are connected to a pair of headers for the flow and

return lines. For tunnel construction, it is not necessary to maintain continuous freezing and keep on lowering the temperature of the soil, the only requirement is that pore water should be kept at a temperature below the freezing point. For special purposes, especially for projects of limited extent and duration, boiling of liquid nitrogen in the freezing elements may be appropriate.

5.2 Ground Freezing Techniques: Following ground freezing techniques are known:

(A) Continuous frozen bodies which provide long-term load-bearing. The freezing is achieved by a drilled tube system, through which coolant is pumped.

(B) Short-term local freezing of damp zones close to the face or in the immediate vicinity outside the excavated cross section.

6. Face Consolidation: Once the analysis of ground behaviour indicates possibility of caving phenomenon, the need for face reinforcement (consolidation) may be of great importance, in addition to other support measures.

Face bolts are used for face reinforcement. Depending on the anticipated ground condition/behaviour, the relevant bolt type and length have to be determined in the design. As a protection against rock fall, spot bolts may be sufficient whereas in difficult ground conditions (e.g. squeezing ground condition) systematic anchoring with a high number of long and overlapping bolts may be necessary. Face bolts are placed during the excavation sequence, if necessary in each round or in predefined steps.

Though steel bolts are also used for face reinforcement but most commonly used bolts for this purpose are “fiber glass reinforcement elements” (Fig. 14.08). These elements can be grouted after inserting in the bore holes.

Fig. 14.08: Fiber Glass Reinforcement Elements

Advantage of using these elements is that after excavation, it is very easy to break/snap the elements hanging from the face.

7. Advance Supporting or Pre-Supporting: The main problem with conventional methods, when tunnelling through difficult geotechnical conditions, is control of deformation. Without support or treatment, the ground weakens and tends to sink into the opening (fall of ground from the upper part of the tunnel face, tunnel face extrusion and tunnel face failure), a phenomenon called "decompression". Elimination of this decompression may require a "cavity pre-confinement action" (any active action that favours the formation of an arch effect in the ground ahead the tunnel face) that can be achieved through reinforcement and/or protective intervention ahead of the tunnel excavation. Such interventions, done in advance of tunnel face excavation, are called as Advance Supports or Pre-supports. Some of the commonly used Advance Support or Pre-support are as under:

7.1 Mechanical Pre-cutting: In this method, a peripheral cut (like slit) is made slightly outside the periphery of the area to be excavated. This is done using a slit cutting machine (Fig. 14.09).

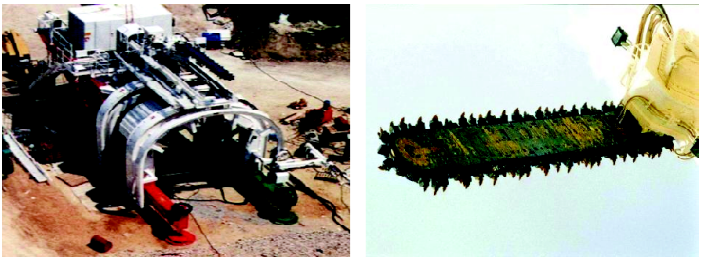


Fig. 14.09: Mechanical Pre-cutting Machine

The slit is cut at a small outwards angle from the direction of tunnel axis and then filled with shotcrete. This forms an annular ring around the periphery of the area to be excavated (Fig. 14.10), which provides a suppression or reduction in vibrations and over-excavation, and keeps the rock mass intact.

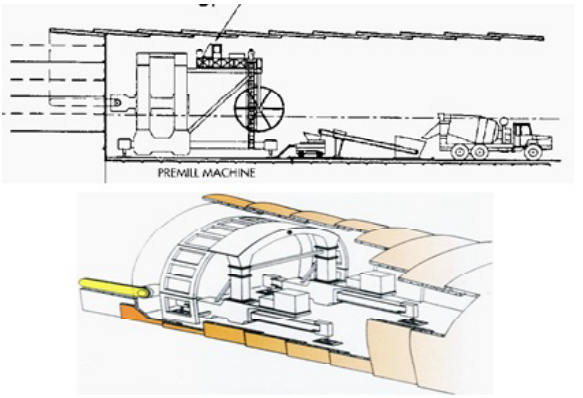


Fig. 14.10: Mechanically Pre-cut Ring

7.2 Spiles: Spiles are steel bars inserted in the ground, on the boundary of the excavation in roof area, for local short-term stabilization of the roof section and at the working face. The spiles rest on the first steel rib or lattice girder support in front (Fig. 14.11). The spiles act as advance

support at the tunnel face and limit over break. Their length is about 3 times the round/step length to ensure sufficient overlap and they are spaced around 30cm. Depending on the type of soil, the spiles can be jacked, rammed or inserted in drill holes.

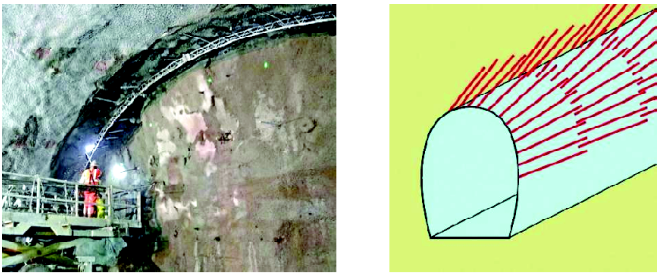


Fig. 14.11: Spiles

7.3 Umbrella Arch: Umbrella Arch Method (UAM) or Forepole is an economical method to increase the excavation front sustainability, minimize the land subsidence, preventing rock debris falling and landslides. In this method, first bore holes are drilled in a semi-circular pattern around the crown of proposed tunnel profile. Then steel pipes are placed into each of these holes and they are filled by grout, forming a strong umbrella arch tube above the tunnel crown. Pipes are installed in two consecutive steps, with overlapping length, and are provided at a small outwards angle from longitudinal axis of the tunnel (Fig. 14.12). Length of pipes is from 15 to 30m and they are spaced at about 30 to 50cm. The main advantages of this method are:

- Stress reduction in drilling front.
- Increased safety during drilling operations.
- Simplicity of drilling operations and in-turn increase in drilling progress.

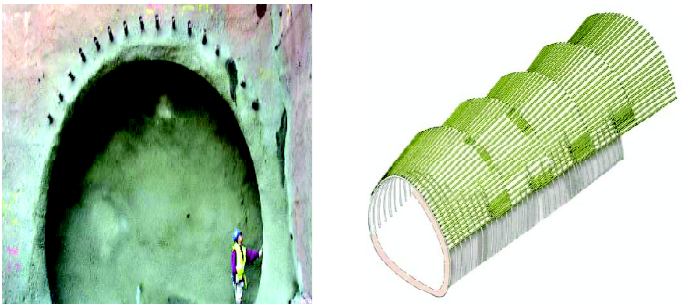


Fig. 14.12: Umbrella Arch Roofing

Umbrella arch roofing is useful at following locations:

- For Portal development.
- For constructing shallow tunnels in soft grounds.
- For tunneling in ground with mix of boulders and soil.
- For tunnelling in sandy gravelly terrain.

8. DRESS Method: In SJVNL (Sutlej Jal Vidyut Nigam Limited) Hydro-power Project of India, a special method named as "DRESS" was employed for construction of tunnel in a wide shear zone, near Rattanpur adit. This length of about 360m was very difficult due to encountering of sheared rock, high ingress of water and high stress condition. DRESS is acronym for "Drainage Reinforcement Excavation Support Solution". Though this was a hydropower tunnel, but the methodology can be applied in railway tunnel also, with similar ground conditions.

Three alternatives (A, B & C) of tunneling advance with DRESS methodology were proposed, depending on the rock conditions. Alternatives A, B and C were proposed for the regions having GSI value < 15, 15 to 35 and 35 to 45 respectively. The steps involved in each alternative were as under:

Step	Description	Iternative		
		A	B	C
1	Drainage/Exploratory Drill Hole	E	E	E
2	Face Improvement by Grouting	E	E	W
3	Steel Pipe Umbrella	E	E	-
4	Side Drift	E	-	-
5	Rock Reinforcement by Forepoling ahead of face	E1	-	-
6	Radial Rock Reinforcement at the Face	W	W	W
7	Temporary Invert (Top Heading)	E	E	E
8	Enlargement of Heading	E	-	-
9	Integration of Rock Reinforcement behind the face	E	W	W
10	Improvement of Side Wall Footing	E	E	E
11	Benching & Steel Arch Concrete Invert	E	E	E
12	Rock Reinforcement of Bench Profile	E	E	E
13	Final Lining	E	E	E

E–Essential, W–When & If Necessary, E1-Only in Left Wall

The sequence of activities was as under:

(i) Draining of Rock ahead of the face: Before opening of the face, advance drainage was done all around and ahead of the face, to eliminate the detrimental influence of water pressure on the face stability.

Six to eight drainage holes of 77mm diameter up o 24m length, depending on the site strata, in an upwards inclination of 15^0, were drilled with a hydraulic drill using DTH hammer. M. S. Pipe of 50mm diameter 12m grouted and 12m perforated, protected with geo-textile were provided in the drilled drainage holes to avoid the blockage of drainage system. These drainage holes were provided in alternate forepoling blocks.

(ii) Face Improvement: After providing the drainage system, stability of the face and ahead was improved by cement grouting with W/C ratio of 1:1. Sometimes when grouting was not possible due to encountering of gouge sheared material mixed with clay, the face was stabilized by Shotcreting and grouted anchor bars of 25mm diameter 8m long.

(iii) Steel Pipe Forepole Umbrella Arch: Forepoling (casing) of steel pipes was provided ahead of the face before excavation of the face using the hydraulic drilling rig. In this the crown of the tunnel above springing level was supported with 12m long steel pipe forepoles (casing) of 114.3mm outer diameter with 6mm thick wall and in an upward direction of 6^0 over rib R1 of the block. The forepole were spaced @400mm c/c spacing. After drilling and installing of the forepoles, cement grout in W/C ratio of 0.75 to 0.45 was placed at a maximum pressure of 5 kg/ cm^2.

(iv) Face Advance in Heading Excavation: After stabilizing the crown and the zone ahead of the tunnel face by forepoles, drainage holes, shotcreting and grouting, the tunnel advance in one forepoling block of 12m length was carried out up to 8.75m length of a variable diameter of excavation from 11.65m to 13.45m before the next block of forepoling. In this 8.75m length of tunnel advance, a total number of 12 sets of ribs of ISMB 300x140 @ 750mm c/c spacing were provided in a sequential advance of 0.75m to 1.50m depending upon the stand-up time of rock strata.

Excavation was done by mechanical means in top heading up to 1.0m below springing level, in rounds of 0.75m to 1.50m in the form of half ring, leaving the central portion to brace the face and the walls against bulging/collapse.

After protecting the crown with shotcrete & wire mesh, the excavated section was supported with ribs and the space between the rock surface and the rib intrados were filled with shotcrete.

In the second round of excavation, the central portion was first excavated up to the previous round advance and again the excavation for this round was carried out in an advance of 1.5m in the same way as explained above.

After excavating and supporting the forepoling block up to rib R12 (last rib of the block), the excavation of rib R1 of next block was done and the rib was installed and supported with wire mesh and shotcrete.

The ribs were anchored at springing level with 25mm diameter 6m long cement grouted anchor bolts with ISMC 150x75 runners joining three to four sets of ribs. The face was then sealed off with shotcrete for improvement of face and ahead before start of excavation of next forepoling block.

The excavated reach was further supported with radial rock reinforcement in the form of 32mm diameter 6m long hollow core self-drilling cement grouted rock bolts. Grout was then pumped through the bolts itself forcing out water, debris etc. and filling of all fissures voids and complete grouting of the bolt was ensured.

A temporary invert arch of 350mm thick shotcrete was also provided to prevent heave of an unsupported invert and punching of steel ribs from the arch support into soft rock.

(v) Benching Excavation: The benching excavation was taken up about 50m behind the face in order to have proper drainage system and stabilization of the heading strata. The benching was also done with hydraulic hammering technique. After finishing the excavation of

benching, the side walls were protected with initial layer of 50mm thick shotcrete, followed by wire mesh fixing, extension of heading ribs in the benching and providing steel ribs arch invert. The spaces between the rock surface and intrados of the ribs on wall sides were filled with shotcrete and invert steel arch encased in 400mm thick M20 concrete. The benching profile was supported with 25m diameter 6.0m long cement grouted anchor bolt.

(vi) Monitoring during Construction: The behaviour of the ground around the opening was closely monitored during the progress of construction with tape extensometer.

In these squeezing rock conditions, it has been observed that the behaviour of the convergence was initially faster with respect to face distance and it subsided subsequently with the elapsed period of opening and face distance.

Deformation in the ribs and cracks in shotcrete were observed in some of the reaches in the blocks but these were within permissible and safe limits. The maximum movement observed in 205m length of the tunnel excavated at that stage was to the tune of 14.6cm in an elapsed period of 460 days.

Progress of up to 25m in a month was achieved in extremely poor rock conditions. Despite the initial investment for hydraulic rig, in view of such progress of work coupled with other factors like safety and stability of operations, DRESS was found most appropriate in extremely poor rock mass conditions.

9. Tunnelling in Swelling Grounds: Swelling phenomena is generally associated with argillaceous soils or rocks derived from such soils. In the field, it is difficult to distinguish between squeezing and swelling ground, especially since both conditions are often

present at the same time. However, except in extreme conditions, squeezing is almost always self-limiting and will not recur vigorously, or at all, once the intruding material has been removed. But swelling may continue as long as free water and swelling minerals are present, especially when the intruding material has been removed, thereby exposing fresh, unhydrated rock.

Most swelling is due to the simultaneous presence of unhydrated swelling clay minerals and free water. Tunnel construction commonly creates these conditions. Minerals such as montmorillonite form layered platy crystals; water may be taken up in the crystal lattice with a resultant increase in volume of up to 10 times the volume of the unhydrated crystal. The displacements resulting from this increase in volume give rise to the observed swelling pressures, whether in soil or in rock.

Swelling grounds cause major problems of supporting both during construction and during operation life of a tunnel due to wall displacements. In certain cases, invert heave of over 25cm per year has been observed. In a tunnel in Udhampur-Katra section of Northern Railway, India, floor heaving of 40-60 cm was observed in year 2004 because of swelling claystone having montmorillonite, kaolinite and illite. The swell rates in several old (75-100 years) Swiss tunnels decreased to 0.5 to 1.0 cm per year with the passage of time. However, the total invert heave was of the order of several meters requiring repairs of the inverts.

Sometimes, it is felt that swelling could be prevented if the ground is hermetically (completely) sealed by shotcrete against air moisture. But many case histories show that this approach has not succeeded. This is because of the fact that while shotcreting prevents ingress of atmospheric water, the swelling still takes place due to presence of pore water within the clays.

The swelling pressure on tunnel supports may be very high. Swelling grounds have normally low modulus of deformation and are capable of exerting high pressure

even at moderate depths. Baldovin and Santovito (1973) measured contact pressures up to 3.5 MPa on provisional concrete lining (placed about 20 days after excavation) in the Foretore Tunnel in South Italy.

Einstein and Bischef (1975) have proposed an analysis-cum-design method for tunnels in swelling rocks. The procedure consists of following seven steps:

(i) Determination of the primary state of stress.

(ii) Determination of the swell zones around the opening, based on the primary state of stress and the stress changes caused by the opening.

(iii) Laboratory swell tests in the Oedometer, on samples taken from swell zones.

(iv) Determination of time-swell properties from Oedometer tests, measuring the time displacement relations for several stress increments.

(v) Derivation of the swell-displacements for the stress difference between the primary state of stress and the state of stress after excavation.

(vi) Performing swell-time computations.

(vii) In-situ measurements of swell-displacements and swelling pressures.

Further, they recommended following design features based on the above procedure:

(i) Use an invert arch instead of horizontal struts.

(ii) Bolt the swelling zone with the ground below it.

(iii) Use compressible backfill between the support and the ground.

(iv) Trim the floor.

(v) Employ grouting to seal off preferential paths supplying water to the swell prone zone.

(vi) Provide constraint to the swelling ground by cutting slots and injecting grout under pressure through these slots.

(vii) Avoid exposure of the swelling ground to atmospheric moisture by applying sprayed concrete (SFRS).

(viii) Provide good drainage inside the tunnel.

10. Tunnelling in Squeezing Grounds: The squeezing or elasto-plastic pressure is mobilized due to failure of a weak rock mass around a tunnel under influence of high overburden pressure or tectonic stresses. The overstressed zone of rock fails where tangential stress (σ_θ) exceeds the mobilized UCS of the rock mass. The failure process will then travel gradually from the tunnel boundary to deeper regions inside the unsupported rock mass. The zone of the failed rock mass is called the "broken zone". This failed rock mass dilates on account of the new fractures. A support system after installation restrains the tunnel closure and gets loaded by the support pressure.

"Commission on Squeezing Rocks in Tunnels" of ISRM has defined squeezing as the time dependent large deformation, which occurs around a tunnel/other underground openings, and is essentially associated with creep caused by stress exceeding shear strength. Deformation may terminate during construction or continue over a long time period.

High deformability, low shear strength and the high in-situ stress state are the major factors that govern the tunnel wall stability and extent of closure. Prediction of squeezing conditions is of great importance to a designer for designing a stable support system of the tunnel.

It is the time-dependent displacement which dominates in fragile rock masses under high overburden, particularly when a broken zone is formed around an opening. Therefore, the support attempts to curb these time-dependent tunnel closure and in-turn attracts higher loads *(Jethwa, 1981: Dube, Singh & Singh, 1986).*

Terzaghi (1948) advocated that support pressure for

squeezing rocks is higher for greater overburden and is directly proportional to the tunnel width. But the research later on proved that the support pressure for squeezing rocks in all cases is not directly proportional to overburden. The researchers on the subject have been able to predict the degree of squeezing, short term & long term support pressures, allowable tunnel cloures and strategy of supporting squeezing ground.

Steel fiber reinforced shotcrete with embedded ribs has proved to be successful in supporting tunnels in the mild to severe squeezing ground conditions. Following detailed strategy has been adopted in squeezing grounds:

(i) Circular or horseshoe shaped tunnel should be planned in the suqeezing ground condition. The tunnel width should preferably be less than 6m in severe or very severe squeezing grounds. The excavated diamater may be 10% more than the design diamater.

(ii) The excavation should be by heading and benching method in minor squeezing ground and by multiple drift method in severe or very severe squeezing grounds. Drill 10m advance probe hole ahead of the tunnel face to know the rock mass quality and drain out ground water, if any.

(iii) The horizontal drill holes of 3m length are drilled ahead of the tunnel face and the forepoles of mild steel rods are inserted and welded to the nearest steel ribs. Then smooth blasting is adopted with short length of blast holes (1m) to cope up with the low stand-up time.

(iv) A steel fiber reinforced shotcrete (SFRS) layer of 2.5cm thickness is sprayed immediately to prevent rock loosening. Full-column grouted bolts are installed all around the tunnel including the bottom of tunnel.

(v) Steel ribs with struts at the bottom are erected and designed to support the forepole umbrella and rock support pressure. The struts should be strong enough to resist high wall support pressures in the squeezing grounds.

(vi) Additional layers of SFRS are sprayed after some delay to embed the steel ribs. It will provide lateral stability to ribs and also create a structurally robust lining.

(vii) The SFRS should also be sprayed on the floor to cover steel sets and counter heaving tendency of the squeezing ground by withstanding high bottom support pressures.

(viii) The convergence of the tunnel roof and walls should be monitored and plotted with time. In case rate of convergence/closure is not dropping with time, additional SFRS layers need to be sprayed. It is a good tunnelling practice if multple borehole extensometers are installed to know what is happening within the broken zone particularly in severe or very severe squeezing ground conditions.

Barla Giovanni (Ref.: 38) has reported some case histories about full face tunneling usiing fibre glass reinforcement elemets, inlcluding the Saint Martin *La Porte access adit along Lyon-Turin base tunnel (at France-Italy border)* located in severe squeezing ground conditions. The overburden was about 300-600m. In this tunnel, a yield-control support system combied with full-face excavation was adopted successfully in order to cope with the large deformations experienced duirng face advance through a highly heterogeneous, disrupted & fractured rock mass of Carboniferous formation, often effected by faulting. A near circular cross section (of radius 6.10m) was excavated with support system as under (Fig. 14.13 & Fig. 14.14):

(i) Stage 0: Face reinforcement, including a ring of fiber glass elements around the tunnel

perimeter, over a 2–3m length in direction of tunnel advance.

(ii) Stage 1: Mechanical excavation carried out in steps of 1m in length, installation of 8m long rock dowels along the perimeter, yielding sets with sliding joints and a 10cm thick shotcrete layer. The tunnel is excavated in the upper cross section to allow for a maximum convergence of 600mm.

(iii) Stage 2: The tunnel is opened to the full section at a 30m distance from the face, with the application of 20cm shotcrete lining, yielding steel sets with sliding joints fitted with hiDCon elements. The tunnel is allowed to deform in a controlled manner with maximum convergence not to exceed 400mm.

(iv) Stage 3: Installation of the final concrete lining at a distance of 80m from the face.

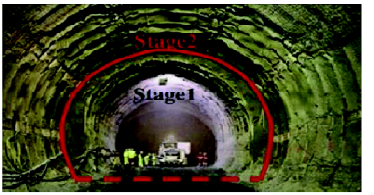


Fig. 14.13: Stages of Excavation

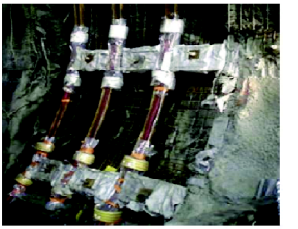


Fig. 14.14: Yielding Supports

Nine hiDCon elements (one in the invert) are installed in slots in the shotcrete lining between the yielding type steel sets. These elements (height 40cm, length 80cm and thickness 20cm) yield at approximately 40-50% strain with a yield strees of 8.5 MPa. With 9 elements installed, if each element attains 50% strain the maximum allowed radial displacement is equal to 20cm approximately, resulting into total convergence of 40cm. Also, with yield stress of 8.5 MPa, the radial confinemet stress on the surrounding rock results to be 0.3 MPa approximately.

11. Comparison between Squeezing and Swelling is shown in Table 14.01 below:

Table 14.01: Comparison between Squeezing and Swelling

[Jethwa (1981) and Jethwa & Dhar (1996)]

Parameter	Squeezing	Swelling
Cause	Small volumetric expansion of weak and soft ground upon stress-induced shear failure. Compaction zone can vary from within broken zone	Volumetric expansion due to ingress of moisture in ground containing highly swelling minerals.
Closure Rate of Closure	Very high initial rate, several cm per day for the first 1-2 weeks of excavation. Reduces with time.	High initial rate for first 1-2 weeks till moisture penetrates deep into ground. Decreases with time as moisture penetrates into the ground deeply with difficulty
Period of Closure	May continue for years in exceptional case.	May continue for years if the moist ground is scooped out to expose fresh ground.
Extent	The affected zone can be several tunnel diameters thick.	The affected zone is several m thick. Post-construction saturation may increase swelling zone significantly.

12. Tunnelling in Seismic Areas: A study of the published literature indicates that the tunnels and caverns in rock medium do not suffer as much damage as the surface structures during major earthquakes (M = 8.5), particularly if they are located at a depth of more than 20m and there is no fault zone in the neighbourhood.

The explanation of drastic damage to surface during shallow major earthquakes is that surface waves (called Rayleigh waves) have major energy than primary and shear waves. The amplitude of Rayleigh waves decays exponentially with depth and it becomes negligible at a depth of about 15-20m below the ground level in rock masses (just like surface waves in ocean).

A dynamic analysis of an underground structure is essential when it is meant to accommodate human activities. In addition, transport tunnels may require a dynamic analysis when they are located in the area of high seismic activities and the active fault may be crossing it or may be very near to it.

When a dynamic disturbance strikes an underground structure, some deformations are caused. These deformations may be decomposed in three components, namely, radial, axial and tangential. The axial component may be further decomposed into the longitudinal and transverse (wave) components. The radial deformation of the underground structures is important when the source of the dynamic disturbance is located within the structure, which is normally not the case in transport tunnels.

The longitudinal (axial) deformations are represented by alternating regions of compressive and tensile strains that travel as a wave train along the tunnel axis. The transverse (axial) component creates alternate regions of negative and positive curvatures propagating along the tunnel. A tunnel lining that is stiff compared with the surrounding soil responds as an elastic beam. For a positive bending associated with the transverse (axial) deformations, the top of the lining is in compression while its bottom is in tension. The same is not true, however, for rock tunnels with flexible or no lining at all. In such cases, the tunnel in positive curvature experiences tensile strain on top and compressive strain at bottom. This dynamic effect consisting of alternating cycles of compressive and tensile strain superimpose on the existing static state of strain in the rock and lining.

The tangential deformations result when waves propagate normal or nearly normal to the tunnel axis. These may result into distortion of the tunnel cross section and may lead to additional stress concentration. This effect is not severe as the tunnel diameter is much less than half the wavelength. Another aspect associated

with the tangential deformations characteristic of the dynamic disturbance is that of ringing i.e., entrapment and circulation of dynamic wave energy around the tunnel *(Owen et al., 1979).* This is not possible as the wavelength of the dynamic disturbance is much more than the tunnel radius. In general, the seismic wavelengths are very large (25-500m) compared to the normal tunnel sizes.

Analysis of the maximum longitudinal strains in the concrete linings during earthquakes, by researchers and by use of software packages, suggest that there is no cause for worry for tunnel stability because of earthquakes in rock masses below 20m from ground surface, except in the active fault zones. The Himalayan experience, about large number of shrines located in deep caves remaining unaffected in spite of being located in seismically active region and experiencing several big earthquakes, confirm this observation.

Use of some empirical equations *(Ref.: 39)* or Computer Software can be made to design the support systems of tunnels in seismic regions.

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CHAPTER-15

PROBLEMS AND HAZARDS IN TUNNELLING

Knowledge of potential tunnelling problems and hazards plays an important role in the selection of excavation method and designing a support system for underground openings. Different conditions of the ground need different approach to tunnel excavation and support. Common problems and hazards encountered during tunneling are elaborated in this chapter.

1. Rock Burst: Rock burst is defined as any sudden and violent expulsion of rock pieces from an apparently (temporarily) stable opening. A schematic representation of the phenomenon of rock burst is shown in Fig. 15.01 below.

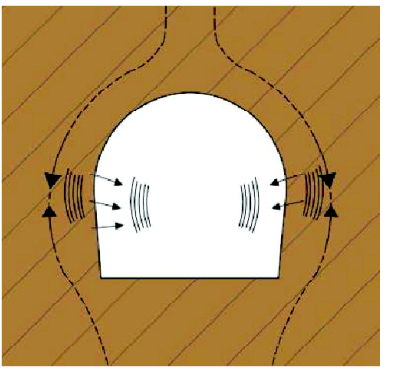

Fig. 15.01: Rock burst

Manifestation of slabbing and release of micro-seismic energy may be the first sign but suddenly several thousands of tons of rocks may breakout like an explosion releasing seismic energy of a mild earthquake. Experience shows that deeper an opening is made in hard rocks, more vulnerable it becomes to rock burst.

It is obvious that failure of rock mass will occur where tangential stress exceeds its biaxial (plane strain) compressive strength. In-situ stresses should be measured in drifts, in areas of high tectonic stresses, to know in-situ stress and tangential stress on opening realistically. It will help in predicting rock burst conditions in massive rock masses. Rock bursts predominantly occur within the first few hours after a blast or an excavation step at tunnel sidewalls, where the tangential stresses reach their maximum and at the tunnel face. For this reason, rock bursts are mostly relevant for the primary support design.

Following approach to tunnelling is recommended in rock burst prone areas:

- A way of reducing chances of rock burst is to make openings of small size. This is because amount of strain energy released per unit area of excavation will be reduced considerably.
- Since stress concentration is responsible for initiation of cracking, it may help to select a shape of excavation which gives minimum stress concentration.
- Slow down the rate of excavation in the zone of stress concentration to avoid sudden release of high strain energy.
- The modern trend is to convert the brittle rock mass into a ductile rock mass by using full-column grouted resin bolts. The plastic behavior of mild steel bars will increase the overall fracture toughness of a rock mass. So the overstressed rock mass will tend to fail slowly, as the propagation of fractures will be arrested by the reinforcing bars.
- A sequence of excavation must be so designed that rock fails in a controlled manner. At least no rock burst should occur near working face during working hours for protection of workers.

2. Chimney Formation or Daylighting: Collapse of tunnel roof or a part of tunnel, extending up to the top of the tunnel and exposing the tunnel to daylight is called as "Chimney Formation" or "Daylighting" (Fig. 15.02).

There may be local thick shear zones dipping towards a tunnel face. The soil/gouge may fall down rapidly, unless it is supported carefully and immediately. Thus, a high cavity/chimney may be formed along the thick shear zone. The chimney may be very high in water-charged rock mass. This cavity should be backfilled by lean concrete completely (Fig. 15.03).

Fig. 15.02: Chimney Formation

Fig. 15.03: Chimney Backfilling

Chimney formation is normally caused due to reasons like weakness in the crown of a tunnel, insufficient cover to overlaying permeable water bearing strata and insufficient cover to surface or insufficient cover to overlaying deposit materials; as shown in Fig. 15.04, Fig. 15.05, Fig. 15.06 and Fig. 15.07 below.

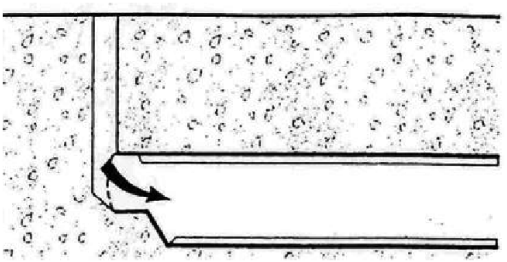

Fig. 15.04: Due to weakness in crown of Tunnel

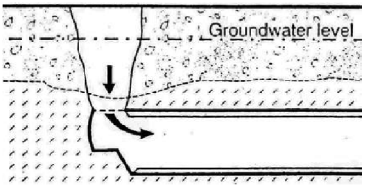

Fig. 15.05: Due to insufficient cover to overlaying permeable water bearing strata

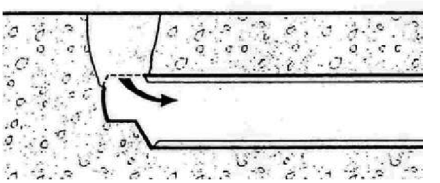

Fig. 15.06: Due to Insufficient cover to surface

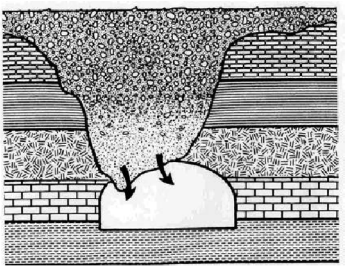

Fig. 15.07: Due to insufficient cover to overlaying deposit materials

3. Face Collapse: Collapses can not only occur in the tunnel itself, i.e. in the crown, but also at the excavation face. In poor rock conditions, with a possibly instable tunnel face, the appropriate counter measures have to be taken.

With the exception of tunneling through cohesion-less granular soil, the stability of the tunnel face is in general time-dependent, i.e. a face that is stable in the short-term may collapse in the long-term. Schematic representation of face collapse is shown in Fig. 15.08 below.

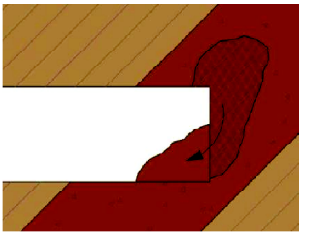

Fig. 15.08: Face Collapse

Face collapse can significantly affect the tunnel advance rate as well as the support requirement. Therefore, stability of the face shall be ensured to avoid uncontrolled collapse. Temporarily or as an emergency measure, the face collapse can be controlled by Face Plugging (Fig. 15.09) but as a permanent measure ground improvement techniques like grouting, pipe roof umbrella, face dowels etc. can be used to strengthen the face and avoid its collapse. These methods have been elaborated in Chapter-14.

Fig. 15.09: Face Plugging

4. Water Ingress: The inclined beds of impervious rocks (shale, phyllite, schist etc.) and pervious rocks (crushed quartzite, sandstone, limestone, fault etc.) may be found along the tunnel alignment. The heavy rain/snow charge the beds of pervious rocks with water like an aquifer. While tunnelling through the impervious bed into a pervious bed, seepage water may gush out suddenly (Fig. 15.10 and Fig. 15.11).

Fig. 15.10: Water Ingress

Fig. 15.11: Water Pooled in Tunnel

Water inflow influences the construction procedure, tunnel stability, and the environment. Groundwater flow in hard rock depends on discontinuities such as joints and their permeability. Several factors can influence on the hydraulic conductivity of the rock mass:

- Joint characteristics like orientation, continuity/ length, roughness and frequency.
- Stress situation.
- Faults and adjacent fractures.
- Dykes.
- Composition and thickness of the overburden.

Sudden flood accompanied by huge washout of sand and boulders may also occur ahead of tunnel face where several shear zones exist. The flooding problem becomes dangerous where the pervious rock mass is squeezing ground also due to the excessive overburden. The machines, including TBMs, get buried partly or fully. In Parbati Hydroelectric Project Stage-II, near Kullu in Himachal Pradesh, India, where part length of Head Race Tunnel was planned to be excavated by TBM, in May'2007 routine probe drilling ahead of TBM punctured a water bearing horizon which resulted in inflow of water of over 120 liter/second containing 40% sand & silt debris. The inflow was sudden and occurred at a high pressure which could not be contained. Eventually over 7500 m^3 of sand and silt debris buried the TBM. This delayed the commissioning of project by about 10 years.

Much depends on preparedness, and on whether or not discontinuity and fault infillings are washed out in the process. This may cause exaggerated over-break and chimney formation, an unsafe working environment.

The seepage should be monitored near the portal regularly. The discharge of water should be plotted along chainage of the face of tunnel. If the peak discharge is found to increase with tunnelling, it is very likely that sudden flooding of the tunnel may take place on further tunnelling. It is suggested that experts be consulted for tackling such situations.

Following approach for tunnelling is recommended to guard against water ingress:

- Groundwater control by rock mass impermeabilization using pre-grouting: Groundwater control is achieved by probe drilling ahead of the face followed by pre-excavation grouting (i.e. pre-grouting) of the rock mass. The primary purpose of a pre-grouting scheme is to establish an impervious zone around the tunnel periphery, by reducing the permeability of the most conductive features in the rock mass. The impervious zone ensures that the full hydrostatic pressure is removed from the tunnel periphery to outside of the pre-grouted zone. The water pressure is gradually reduced through the grouted zone and the water pressure acting on the tunnel contour and the tunnel lining can be close to nil. In addition, pre-grouting will have the effect of improving the stability in the grouted zone within the rock mass.
- Constructing a drained structure of the rock mass in combination with rock support: This means that the support measure installed is not constructed to take external water pressure. Excessive water must therefore not be allowed to build up behind the rock support measure. Excess water must either be piped to the water collection system in the tunnel or taken care of by a water protection system.

5. Portal Collapse: The portal areas frequently represent some of the most problematic points during the excavation of a tunnel. Thus, Portal Collapse (Fig. 15.12) is another problem to be tackled in tunnel constructions.

Several factors, (e.g. the direction of excavation, the morphology of the site, the geo-mechanical characteristics of the terrain etc.) influence the portal problems. While it is highly desirable that the location selected for the portal be in fresh rock with cover of the same order as tunnel width and height, environmental constraints or other relevant considerations will sometimes dictate that the portal be located where there is low cover, weathered rock, or even soil. Where rock is exposed, the preconstruction of a reinforced concrete portal structure will still be of substantial assistance.

Fig. 15.12: Portal Collapse

The philosophy of design used for portals is similar as used for any slope stability analysis. Active support and effective drainage are key elements to ensure stability of the portal slopes. Chapter-18 may be referred for further details on Portals.

6. Toxic Gases and Geothermal Gradient: There are serious environmental hazards due to toxic or explosive gases while tunnelling in the argillaceous rocks. Sometimes methane gas is emitted by blasted shales.

Improper ventilation also increases concentration of toxic gases like carbon monoxide, carbon dioxide, hydrogen sulphide and sulphur dioxide. Attention should be given to the physical properties of the gases, as some gases tend to collect either in high or low pockets in a tunnel complex. Monitoring of gases and oxygen should be carried out near the face of a tunnel especially where blast fumes and gas emission are maximum. Oxygen must be maintained at a level of 20% or greater. Dust inside the tunnel should be controlled for reducing health hazards. Therefore, wet drilling method is recommended for both blast holes and bolt holes.

As rock engineers are going deeper and deeper, workers face a high temperature. The temperature may increase at a rate of about 30^0 C per km. The temperature inside a 1400m deep NJPC (Nathpa Jhakri Power Corporation) tunnel in Himalaya, India, was as high as 45^0C or more. The efficiency of workers in such a high temperature was reduced drastically. They worked for two to three hours only after taking bath frequently with ice-filled water. If possible, cool fresh air should be used for ventilation to maintain a working temperature of around 30^0C at the tunnel face.

7. Timely Decision: Tunnels require deployment of considerable skill and care in their investigation, planning, design, construction and monitoring, if they are to be safely constructed. Several of the tunnel problems described above often arise due to failure to properly plan and design for uncertainties, in particular for an unfavourable change in ground conditions. Procedures should be developed to overcome these uncertainties and permit safe tunnel construction. But successful application of these procedures requires timely action to either prevent the failure or prevent consequential damages due to the failure. Any failure normally gives some warning signs in advance. There must be procedures and protocols in place to notice these signs and activate the remedial action immediately

without wasting time. If the warning signs are ignored or timely action is not taken on them, it leads to catastrophic failure.

8. Failure Analysis: Every collapse or failure shall be analyzed in detail and documented properly. This is required not only for settling cost questions from contractual and insurance point of view but it also helps in deciding about continuation of work and need for any additional measures before starting/continuing the work. Documentation and sharing the failure analysis, helps as a repository of knowledge not only for the project concerned but also for all such/similar projects in future.

CHAPTER-16

SAFETY ASPECTS IN TUNNEL CONSTRUCTION

Working in underground structures is an inherently risk-prone activity. In view of this, there is a responsibility on all stakeholder in tunnel projects– owner, consultant and contractor – to ensure absolute safety, as well as broader Safety, Health and Environment (SHE) aspects of construction.

1. Applicable Regulations: Various activities involved in tunnel construction are covered by number of acts, regulations and codes. Some of them are:

- Indian Explosives Act – 1884
- Mines Act – 1952
- Mines Rules – 1983
- IS:4081 (1986): Safety Code for Blasting & Related Drilling Operations
- IS:4756 (1978): Safety Code for Tunnelling Works

2. Risk Assessment: Before commencement of construction work, risk assessment should be undertaken after identification of likely hazards and for each risk occurrence an estimate of the possible consequences should be determined. Based on this assessment, appropriate risk mitigation and control measures should be put in place to control or minimise the risks. On-going risk assessment will be needed during the construction phase, particularly if design changes are made or unforeseen ground conditions are encountered.

3. Project Safety Plan (PSP): Since each underground project has its own peculiarities and special features, it is essential for each tunnel project to carry out comprehensive Risk Analysis of the particular project and evolve a Project Safety Plan (PSP) or Health and

Safety Plan (HSP), before commencement of work. While the agencies concerned may adopt standard provisions of their respective organizations, it is essential to have a project-specific safety plan. The PSP shall be prepared by the concerned construction agency and got approved from the competent authority. The engineer-in-charge for the work should join with designers and contractors for developing the PSP & monitoring its implementation. The plan should also contain details of emergency procedures, workers welfare measures and medical facilities. A copy of the plan should be made available to all supervisors/engineers at site. Workers should regularly be briefed/educated about provisions of the plan relevant to their work area.

4. General Safety Measures

4.1 Basic Philosophy: For underground, a fundamental safety measure would be to assess the type and category of rock and its' stand-up characteristics. It is common practice to divide the rock in different classes, depending on the rock mass classification system being used. It is essential to provide adequate rock supporting measures, as per the design of the support system being adopted, before the expiry of the permissible stand-up time for each class of rock. In case of tunnelling in weak/soft grounds, suitable measures for ground improvement/reinforcement shall be taken, as per the design and as per the provisions of PSP.

4.2 Personal Protective Equipment: All personnel entering the tunnel during construction shall wear all applicable Personal Protective equipment (PPE). The PPE shall comprise, at minimum, Safety Helmet, Safety (Hard) Shoes, tight clothing with no loose ends and Jackets/ clothing with reflective strips. Additional PPE such as goggles, gloves, dust masks, helmet lamps, ear plugs/ muffs, safety harnesses etc. shall also be adopted wherever conditions so warrant.

4.3 Access Control Systems: A proper access control system should be in place to have a clear idea at all times on the identity of all personnel who are inside the underground installations in case any accident takes place and rescue operations are to be launched. It is also essential to keep track of all equipment inside the tunnel. It is common to issue tokens to all concerned personnel while entering the tunnel and retrieving the same on exiting.

4.4 Signage: Well-illuminated boards shall be placed at required locations to inform people of safety hazards inside the tunnel and the precautions to be taken. Appropriate Safety Signage should be provided as per applicable standards like *IS:9457 (2005)- Code of practice for Safety Colours and Safety Signs and IS:12349 (1988)-Fire Protection Safety Signs etc.*

Names, contact numbers and addresses of officials/ organisations to be contacted in case of emergency should be displayed at prominent locations of the site.

All safety signs shall comply with the internationally recognized Safety Colours as indicated below:

- Yellow – Danger (Fig. 16.01)
- Blue – Mandatory (Fig. 16.02)
- Red – Prohibition (Fig. 16.03)
- Green - Safe Condition (Fig. 16.04)

Fig. 16.01: Danger (Yellow) Signage

Fig. 16.02: Mandatory (Blue) Signage

Fig. 16.03: Red (Prohibition) Signage

Fig. 16.04: Green (Safe Condition) Signage

4.5 Safety Management and Training: Safety policies should be established for all the agencies involved in the project, with a clear chain of command/communication and responsibilities for safety.

All operations inside the tunnel or shaft shall be carried out under supervision of a competent engineer, who shall be responsible for ensuring safety stipulations and he should brief all workmen on the plan of work before it is started with special emphasis on all potential hazards and on the ways to eliminate or guard against them. In larger jobs, these responsibilities of safety management may be delegated to an independent safety officer working under the overall control of the engineer in-charge. Periodical meetings, preferably once every month, shall be conducted to review effectiveness of safety measures.

Appropriate training courses should be designed and put in place for people unfamiliar with tunnelling, before they are allowed to work underground. Where 25 or more employees have to work underground at any one time, at least one rescue crew of 5 employees per shift must be trained in rescue procedures and resuscitation, use of oxygen breathing apparatus and use of firefighting equipment. Where less than 25 employees work underground, not less than 5 employees must have such training in rescue work.

All workers should also be trained in use of safety devices and appliances provided to them. In case any worker feels that he cannot perform a work safely he should immediately inform the site in-charge of his inability to carry on with the work.

The training of workers would include at minimum, safety induction (initial training in basics of safety) and training (routine training) exercises, medical screening of personnel for working inside tunnels,

system of permits for simultaneous operations in various locations, pep talks (regular talks to workmen before they commence work, on importance of safety and how necessary it is for them to observe safety regulations for their own welfare) and tool box talks (specific safety instructions at site in the specific area of work for the workmen), talks on specific operations to be carried out on the day, safety walkabouts (general safety observance checks carried out by safety stewards by going around the site and checking observance of the various safety regulations etc.), safety audits, safety reviews and mock drills etc.

The occurrence of any accident shall be reported to the supervisory staff/officer and adequate precautionary measures shall be taken by the engineer in-charge to prevent recurrence. An accurate record of such accidents shall be properly maintained in format approved by Engineer-in-charge. Probable reasons of accidents shall be investigated and precautionary measures taken to avoid further recurrence. Accidents occurring during the fortnight shall be discussed in the safety meetings and adequate publicity shall be given to the causes of these accidents and their preventive measures.

Mock drills should be conducted periodically (preferably once every six months) to assess the level of safety preparedness/awareness. Concerned State/District administration authorities should also be briefed about and involved in conducting of mock drills. Mock drill can include scenarios for evacuation and rescue/relief in case of different types of accidents/hazards.

4.6 Medical Facilities: Arrangements for rendering prompt and adequate first-aid to the injured persons shall be maintained at work site. At least one experienced first-aid attendant with his distinguishing badge shall be available on each

shift to take care of injured persons. Engineer or foreman, who is normally present at each working face in each shift, can be given adequate training on first-aid methods to avoid employment of a separate attendant. Arrangements shall be made for calling the medical officer, when such a need arises. Depending upon the magnitude of the work, availability of an ambulance at a very short notice (at telephone call) shall be ensured. Stretchers and other equipment necessary to remove injured persons shall be provided at portal.

Where there are more than 50 persons working in a shift, effective artificial respiration arrangements shall be provided, with trained men capable of providing artificial respiration.

4.7 Ventilation: Ventilation shall be carried out in tunnels to make the working space safe for workers by keeping the air fresh and by eliminating harmful and obnoxious dust, explosive fumes, exhaust from operating equipment, particularly diesel operated equipment, and other gases. Mechanical ventilation shall be adopted where necessary to force the “air” in or exhaust the air out from the working face of the portal through ducts. Externally located fans operate in forced ventilation and induced ventilation modes to supply air through rigid or more commonly, flexible ducts. Immediate booster fans ensure better supply for longer ducts. Ventilation shall be properly designed considering the tunnel topography and emission levels inside. A minimum of 200 cubic feet of fresh air per minute is to be supplied for each employee underground. Mechanical ventilation, with reversible airflow, is to be provided in all of these work areas, except where natural ventilation is demonstrably sufficient. Where the temperature is high or heavy blasting is resorted to suitably augmented volume of air shall be provided. Where blasting or drilling

is performed or other types of work operations that may cause harmful amounts of dust, fumes, vapours, etc., the velocity of airflow must be at least 30 feet per minute.

It is important to be alert all the time for the presence of toxic gases in underground works and appropriate instrumentation should be provided to keep track of the ambient air quality at all times. Proper records shall be maintained of specific measurements of air quality at regular intervals throughout the day after blasts or major rock falls. Particularly after each blasting for underground rock excavation, the ventilation measures should be set in place quickly and effectively for de-fuming and personnel should be allowed to enter only after establishing that the air quality is sufficiently acceptable. In certain regions geothermal conditions prevail and cooled air should be supplied to enable safe and comfortable working conditions. In any case appropriate and well-designed ventilation systems should be put in place to ensure proper ambient conditions.

Air quality test shall be carried out once every 24 hours but in any case after every blast or a major rock-fall. Portable instruments should be provided to test the atmosphere quantitatively. A record of all tests should be maintained and be kept available for inspection. In case any of the gases are detected to have crossed the threshold value indicated therein, the workmen shall be withdrawn immediately till the percentage is brought down well below the threshold value by improving the ventilation or by other effective measures. In case presence of gases like methane is detected, further tunnelling work shall be stopped and the advice of Director General Mines Safety (DGMS) shall be sought.

4.8 Noise Protection: Sufficient steps should be taken to reduce noise levels to acceptable limits and workmen and visitors should be asked to wear

ear plugs/muffs etc. where required. Exposure to a noise level of 85 dB(A) can cause damage to hearing. Steps must therefore be taken to reduce the noise. As proper protection is only possible when the protective devices are properly fitted and worn, an effective assessment, fitting and training program should be put in place.

4.9 Lighting: Adequate lighting should be provided at the face and at any other point where work is in progress and at equipment installations such as pumps, fans and transformers. A minimum of 50 lux shall be provided at tunnel and shaft headings during drilling, mucking and scaling. When mucking is done by tipping wagons running on trolley trucks a minimum of 30 lux shall be provided for efficient and safe working. The lighting in general in any area inside the tunnel or outside an approach etc. shall not be less than 10 lux.

Emergency lights (battery operated) shall be installed at the working faces and at intervals along the tunnel to help escape of workmen in case of accidents. All supervisors and gang-mates shall be provided with cap lamps or hand torches. It shall be ensured that at least one cap lamp or hand torch is provided for every batch of 10 people. Any obstruction, such as drill carriages, other jumbos and drilling and mucking zones in the tunnel shall be well lighted.

4.10 Communication System

(A) Warning Signs and Notice Boards: Irrespective of length and bends in the tunnel, arrangements shall be made for transmitting of warning signals by any one of the following means:
(a) By electrically operated bells, operated by battery/dry cells with the bell placed outside the tunnel and the position of the switch shifting with the progress of the tunnelling work. The position of

the operating switch, shall be so chosen as to ensure proper accessibility and easy identification. (b) By the use of field (magnet type) telephone. (c) Any other suitable arran-gement like walkie-talkie.

For tunnels up to 100m, only one of these systems may be adequate whereas in tunnels of length more than 100m at least two systems shall be installed with the wires running along opposite sides of the tunnel, if practicable. These system(s) shall be subject to daily checks regarding proper serviceability, under the supervision of a responsible person.

Red and green lights of adequate size and brightness shall be provided at suitable intervals on straight lengths and curves etc. to regulate the construction traffic.

(B) Telephone System: A telephone system shall be provided to ensure positive and quick method of communication between all control locations inside tunnel and portal of the tunnels when longer than 500m and for shafts when longer than 50m.

(C) CCTV System: Closed Circuit TVs are often deployed to keep a continuous watch on underground installations from the Control room.

4.11 Fire Protection

(A) General: All combustible materials like rubbish shall be continuously removed from such areas where flammable liquids are stored, handled and processed. All spills of flammable liquids shall be cleared up immediately. Containers of flam-mable liquid shall be tightly capped. All waste and combustible rubbish shall be removed at least daily from the tunnel.

(B) Fire System: Fire Incidence Detection Sys-tems should be able to detect the fire very early in its development and also accurately locate the position of the fire. The degree of accuracy depends on the

type of active fire safety systems installed in the tunnel. Clearly visible Fire Points shall be established inside the tunnel (near any room, recess or compartment etc.) for use in an emergency. Each fire point should have available as a minimum Dry Powder Extinguisher, Water Type Extinguisher and Bucket of Sand. Recharging of fire extinguishers and their proper maintenance should be ensured. Supervisors and workmen at the site should be trained in the use of fire-fighting equipment provided at the site.

Water supply for firefighting purposes should be provided at the construction site. This may be in the form of static water tank of adequate capacity or a hydrant line with adequate water pressure at outlet points. Sufficient number of fire hoses with branch pipes should be provided at site so that the fire can be controlled until the arrival of the Fire Brigade. Telephone Number of the local fire brigade should be prominently displayed near each telephone on site. Fire Alarms should be provided at appropriate locations inside the tunnel.

(C) Electrical Installations: The entire electrical installation shall be carried out according to the existing Indian Electricity Rules as modified from time to time. Voltage for lighting in a tunnel should be 125V between phases as specified for underground lighting in terms of Rule 118 (c) of Indian Electricity Rules, 1956. The electrical installations should be designed and executed and regular tests should be carried out to ensure safe conditions and emergency cut-off procedures. Electrical leakage monitoring system should be in place. All parts of electrical installation shall have all conductors and contact areas of adequate current carrying capacity and characteristics for the work they may be called upon to do and all joints in conductors shall be properly soldered or otherwise efficiently made. They shall be so constructed,

installed and maintained as to prevent danger of fire, external exposition and electric shock, be of adequate mechanical strength to withstand working conditions underground, be not liable to be damaged by water, dust or electrical, thermal or chemical action, to which they may be subjected, be efficiently insulated or have all bare live parts enclosed or otherwise protected, and be installed at such location that dumpers or wagons do not come in contact with the same.

On the occurrence of a fire caused by any electrical apparatus or a fire liable to effect any electrical installation; the supply of electricity should be cut off from such apparatus or installation as soon as practicable, and the fire shall be attacked and reported to the nearest available supervisor. As far as practicable, combustible material shall not be used in the construction of any room or recess containing electrical apparatus. No combustible material should be stored in rooms, recesses or compartments containing electrical apparatus.

A passageway not less than 60 cm wide shall be maintained in front of switchboards. In no case, space in front or back of a switchboard shall be allowed to be used as a change room, locker or storage room. Rubber mats shall be provided in front and in back of the switch boards. No one shall be permitted at the back of switchboards when the current is on.

All electric wires carrying voltage 440 and above, installed underground, shall be in the form of insulated, lead covered cables, armoured effectively against abrasion and effectively grounded.

Most tunnels are wet or damp providing a perfect ground for short circuits. Steel forms and drill carriages shall, therefore, be properly grounded. The switches shall be located on a high ground and these shall be properly grounded. All electrical apparatus

including portable tools shall be connected only to an electrical supply system, which shall have proper earthing and grounding.

Following notice shall be kept exhibited at suitable places:

• A notice on the board of 45x30cm prohibiting unauthorized persons from entering electrical equipment rooms.

• A notice on the board of 45x30 cm prohibiting unauthorized persons from handling or interfering with electrical apparatus.

• A notice on the board of 60x90 cm containing directions as to the rescue of persons in contact with live conductors and the restoration of persons suffering from electrical shocks.

• A notice specifying the person to be notified in case of electrical accident or dangerous occurrence, and indicating how to communicate with him.

Adequate fire extinguishing equipment suitable for use on live parts shall be kept ready for immediate use in or near any room, recess or compartment containing such parts as will be readily accessible safely for use in case of emergency. This equipment shall be inspected at least once in a month.

4.12 Housekeeping

(A) General: Only the material required for work in progress shall be kept inside the tunnel. All other material shall be removed from inside the tunnel. Sufficient width of formation as even as possible and without any obstacles shall be created to enable the workers to get out of the tunnel quickly in case there is any collapse or any other mishap inside the tunnel.

(B) Traffic Control: Vehicles carrying pipe, rail and timber shall be properly loaded for safe passage through the tunnel. The load shall be kept within

the side limits for the vehicle as loads projecting over the sides are dangerous to men working in the tunnel. For transportation of wide loads, special care shall be taken in operation of the vehicles, with prior warning to the workmen along the tunnel to ensure a safe journey.

A safe and smooth walkway system shall be provided for employees, suitably separated from the vehicular roads by guard railing. For transportation of employees by vehicles proper safety precautions shall be taken.

(C) Pipes and Cables: All water and air pipes as well as electrical cable shall be arranged along the sides of the tunnel, duly supported at regular intervals and in a systematic and neat fashion.

(D) Water Control: Sudden water ingress can be a catastrophic situation in certain underground areas and emergency dewatering systems should be in place to tackle such situations.

Many times water seepage is encountered in underground excavations. Prima-facie this is not a dangerous indication by itself. It is an indication of fissures in the rock and presence of water streams nearby, which have to be kept in watch. Excessive ingress of water can give rise to unsuitable conditions and has to be carefully monitored. Also, for good working conditions inside underground enclosures, continuous dewatering to remove the excessive inflow is essential.

A study of boring data and geological formations shall be made to have an indication of locations, where water can be expected. Water inflow may be reduced or even entirely stopped by grouting of the wet seams. A wet area covering more than a single seam shall be sealed off by installing a suitable section of concrete lining. In case of a steady flow of water from the roof or side of the tunnel the flow shall be deflected down the sides to sumps by metal

shields. The number of pumps provided at site shall be 50% more than the requirements calculated on the basis of the estimated pumping needs, or at least one number, whichever is more. In case of steeply inclined tunnels steps shall be provided for quick exit in case of failure of haulage. Gutters and sumps shall be kept clean. Suitable arrangements shall be made to indicate the position of sumps in case tunnel invert is flooded.

4.13 Working with Machinery & Equipment: Precautions for safety while working with construction machinery & equipment shall be followed as per Manufacturer's guidelines & applicable codes. Special care should be taken while working with compressed air.

4.14 Insects, Leeches, Vermins and Snakes: Protection against insects, vermins, leeches or snakes shall include the following:

- Use of boots, hoods, netting, gloves, masks or other necessary.
- Drainage or spraying of breeding areas.
- Elimination of unsanitary conditions which propagate insects or vermins.
- Approved first-aid remedies for the affected.

4.15 Emergency Management System: An Emergency Management Plan shall be part of the approved Project Safety Plan and shall be well communicated to all working personnel and well displayed at the site. Emergency Rescue Measures should be drawn up to take care of various possible contingencies. It would also be advisable to provide safe rooms in deep installations where people can take shelter for a few hours in case of emergency. Buried large diameter pipe lines leading to outside can be provided to offer a medium for communication and feeding in air supply in case of any collapse and blockage of entrance to underground structures.

5. Safety Requirements for Various Activities

5.1 Drilling and Blasting

(A) Drilling Operations: Only wet drilling shall be permitted. Drilling shall not be resumed after blasts have been fired until a thorough examination has been made by the blasting foreman to make sure that there are no misfired charges. A drill, pick or bore shall not be inserted in the butts of old holes even if examination fails to disclose explosives. Separate holes shall be so drilled as to nowhere less than 30cm clear distance away from the previous hole. Charging of drilled holes and drilling shall not be carried out simultaneously in the same area, unless Nonel type of detonators are used and adequate precautions have been taken.

(B) Blasting Operations: Where blasting operations are to be conducted, sufficient warning shall be given to all staff and workmen prior to blasting. Cell phones are usually prohibited in areas where blasting operations are conducted. Sufficient protective bulkheads etc. shall be provided to enable personnel to take shelter behind during blasting.

All explosives shall be handled and used with care either by or under the direction of competent persons and following the Indian Explosives Act-1884, Explosive Rules-1983 and IS:4081(1967)- Safety code for blasting and related drilling operations. Explosives and detonators shall be placed in separate insulated carriers whether carried by persons or conveyed mechanically and an attendant shall ride with the explosives being conveyed mechanically on slopes in shafts or in underground work areas. For carrying explosives mechanically, prior permission of Chief Inspector of Explosives shall be obtained. Insulated containers, used for carrying explosives or detonator, shall be of approved manufacturer and shall be provided with suitable non-conductive carrying device, such as rubber,

leather or canvas handle or a strap. Explosives and detonators shall be brought to the working places in separate, tight, well insulated containers, and kept in the containers until removed for placement in drill holes. If drill holes are not ready, they shall be stored in locked box type magazines located at a safe distance of at least 170m from the working space. No person other than a shot firer shall carry any priming cartridges into a shaft, in which the sinking is in progress. No such cartridge shall be so carried except in a thick felt bag or other container sufficient to protect it from shock.

Electric firing shall be done by an approved method. All drilling equipment and personnel not engaged on loading shall be removed from the site before loading of holes starts. Loading of a round shall be completed by the crew starting the work of loading. Firing of round shall be the responsibility of the blasting foreman. Only clay sticks or pneumatic air locks shall be used for separation of charge/ stemming of the holes.

Before use, each and every electric detonator shall be tested with the help of an ohmmeter. Before shot firing, the circuit shall be tested for insulation and for continuity. Before a shot is fired in an underground working place due warning shall be given to persons within 330m in all directions and every entrance to the place where a shot is about to be fired shall be guarded so as to prevent any person, not having received warning from placing himself in dangerous proximity to the shot.

In case an exploder is used the revolving handle of the exploder shall be in the custody of the blasting foreman to prevent anybody else firing the shot when the blasting foreman and other persons are inside. Stray currents may cause accidents while loading and utmost care shall be taken in removing all faults from electrical circuits. Electric power, light and other circuits in the vicinity within 70m of the loading

points shall be switched off after charging the explosives and before the blasting operation starts. Power supply is to be switched on only after the blasted area has been properly inspected by the blasting foreman for misfires. All tracks, airlines and vent pipes shall be kept properly grounded. The heading shall be properly lighted with the electric floodlights before and after blasting.

(C) Inspection after Blasting: Immediately after a blast has been fired, the firing line shall be disconnected from the blasting machine or other source of power. When at least 5 minutes have passed after the blast was fired, a careful inspection of the face shall be made by the blaster to determine if all charges have been exploded. Electric blasting misfires shall not be examined for at least 15 minutes after failure to explode. Other persons shall not be allowed to return to the area of blast until an “ALL CLEAR” signal is given by the blasting foreman.

All wires shall be carefully traced and search made for any exploded cartridge by the man-in-charge of the blasting operation. Sufficient time shall be given for the fumes to clear before permitting the labour to work for mucking operation.

(D) Misfires: Misfired holes shall be dealt by the blaster preferably by the same person who has done the charging operations. If broken wires, faulty connections or short circuits are determined as the cause of a misfire, the proper repairs shall be made, the firing line reconnected and the charge fired. This shall be done, however, only after a careful inspection has been made of burdens remaining in such holes and no hole shall be fired when the burden has been dangerously weakened by other shots. The charge of explosives from a misfired hole shall not be drilled, bored or picked out. Misfired charges, tamped with solid material, shall be detonated by a safe approved method.

The stemming shall be floated out by the use of water or air jet from hose until the hole has been opened to within 60 cm off the charge, and the water shall be pumped out or siphoned off and the new charge placed and detonated. Whenever this method is not practicable, a new hole shall be drilled 30 cm deep and spaced not nearer than 60 cm, shall be loaded and detonated. A careful search shall be made of the unexploded material in the debris of the second charge.

(E) Scaling and Mucking: It is essential to carry out proper scaling operations after each blast to remove all the loose rock pieces and guard against rock falls. Careful and frequent inspection of walls and roofs as well as of tunnel supports shall be carried out. Thorough scaling of loose rocks at all weak spots is the best prevention against rock falls. Periodic inspection of unsupported sections of the tunnel from a travelling scaling platform shall be carried out for locating weak spots. Supported sections shall also be inspected regularly to make sure that the weakness of the formation has not spread beyond the supports. Loosened rock shall be supported/removed forthwith. All supports shall be checked occasionally to make sure that there is no member under distress. All scaling platforms shall be equipped with safe ladders.

(F) Explosives Disposal: No explosive shall be abandoned. They shall be disposed or destroyed strictly in accordance with the approved methods and in doing so the manufacturers or the appropriate authority shall be consulted. The expired deliveries shall be sent back to the manufacturer. Explosives, caps, boxes, or material used in packing of explosives shall not be left lying around in places to which children or unauthorised persons or livestock can have access. Paper or fibrous material employed in packing explosives shall not be put to any

subsequent use. Such material shall be destroyed by burning in the presence of a responsible person.

(G) Explosive Accountal: A day-to-day account of the explosives shall be maintained in a register in an approved manner, which shall be open to inspection at all times by the concerned authorities. Explosives shall be issued only to competent persons upon written requisition signed by the blaster or by an official authorized for the purpose and only against the signature or thumb impression. Such requisitions shall be preserved by the person-in-charge of the magazine.

5.2 Installation of Supports: Design and installation of appropriate supports within the stand-up time for the particular type of rock is the most important step to ensure proper safety for all personnel inside the tunnels. Special watch shall be maintained for uncontrolled collapse of the face or adjacent areas, sliding in of muck and water etc. The stability of temporary supports should be checked regularly. The bolts should be tested at regular intervals determined by rock conditions and the distance from vibration sources.

5.3 Structural Steel Erection: All employees working in places where they are exposed to falling hazards should use safety belts. Workmen should stand clear from lifted loads, when derrick is in operation. When guiding a beam, it should be so held that the hands do not get jammed against other objects. When lifting an object in a group, one person should be designated to give the signal for all to lift or set the object down in unison. There shall be no riding on steel that is being hoisted, no riding on the overhauling weights, hooks, cables or slings, nor sliding down on ropes or cables.

5.4 Scaffolds: Safe means of access shall be provided to every place in scaffolds at which

workers are required to work. Construction and dismantling of every scaffold shall be under the supervision of a competent person. Boards and planks used for the floors shall be of uniform thickness, well jointed, closely laid, and securely fastened in place. While dismantling, the entire scaffold shall be maintained stable and rigid so as to avoid the danger of collapse. Nails from the planking and various members of the scaffold shall be carefully removed and all material carefully piled.

5.5 Working Platforms: All working platforms, from which workers are liable to fall more than 2m, shall be of adequate width depending upon the type of work to be done and the width shall not be less than 60 cm. Suitable guardrails of height 1m above the working surface shall be provided. Platforms shall be kept free from unnecessary obstruction and when they become slippery, appropriate steps shall be taken to remedy this.

5.6 Concreting

(A) Mixing Plant: Precautions shall be taken to protect workmen from falling objects. The operations of the plant shall be coordinated by signals etc. to ensure safety of all workmen. An air exhaust system shall be installed to remove cement and other dusts from the inside of the plant. Duct masks should be worn when necessary.

(B) Pumped Concrete: The pipeline shall be anchored at all curves and near the end. Air release valves shall be installed at high points to release entrapped air. If and when necessary to open a pipe to clear it of an obstruction, the work must be carefully done in order that workmen are not injured by concrete blown out by air pressure in the pipe.

(C) Grouting and Shotcreting: Only experienced man should be employed for grouting and

shotcreting, which are special type of concrete work. All hoses and mixers shall be inspected daily and maintained in a safe working condition.

5.7 Welding and Cutting

(A) General: All welding and cutting shall be done by workmen who are thoroughly trained in the work. Shields shall be placed around the work to protect person from glare. Welding and cutting shall not be done in the immediate proximity of flammable materials. A helper shall always be at hand to shut off the gas in case of an accident when the welder is working in a space from where escape is difficult. All welding operations should be carried out in a well-ventilated space. Eye exposed to welding or flashes should be washed with rose water for better relief.

(B) Oxy-acetylene Cutting and Welding: Keep hose and cylinder valves free from grease, oil, dust and dirt. Keep cylinders away from stoves, furnaces and other sources of heat. Only “Gas Lighter” shall be used to light the torch. Valve protection caps shall be in place when cylinders are not in use.

(C) Gas Cylinders: Gas cylinders shall be kept upright in safe places where they cannot be knocked over and well separated from combustible materials. Loaded and empty cylinders should be kept in separate places. Tampering with or attempting to repair safety devices or valves of gas cylinders shall be prohibited. Cylinders found to have leaky valves or fittings, shall be taken into the open away from any source of ignition, and slowly drained of gas.

(D) Hoses and Torches: Special care shall be taken to avoid interchange of oxygen and acetylene hoses, as the mixture of these gases is highly explosive. Compressed air shall never be used to clean hoses as it may contain oil from the compressor. Oxygen shall be used to clean oxygen hoses and acetylene shall be used to clean acetylene hoses. Copper or

brass wire shall be used to clean the tips. Hardwood sticks may also be used.

(E) Electric Arc Welding and Cutting: Only heavy duty electric cable with unbroken insulation shall be used. When it is necessary to couple several lengths of cables for use as a welding circuit and occasional coupling or uncoupling is necessary, insulated cable connectors shall be used. Frames of all electric welding machines operated from power circuits shall be effectively grounded.

5.8 Paints: Most paint materials are highly combustible, and every precaution should be taken to eliminate danger from fire. Fire extinguishers of appropriate capacity shall always be at hand where flammable paint materials are being mixed, used or stored. Sand buckets or extinguishers of the carbon dioxide and carbon tetrachloride type are generally affective.

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CHAPTER-17

SAFETY ASPECTS IN OPERATION IN TUNNELS

Three types of accidents can occur in tunnels; derailments, collision and fires. An integrated comprehensive approach towards tunnel safety has to be implemented at all the stages i.e. planning, design, construction and operations. The philosophy followed in design and implementation of safety aspects for train operation in tunnels vary from country to country. The European countries (who have probably longest and oldest network of railway tunnels) adopt UIC provisions, whereas Japan have their own standards which are quite different from UIC standards. In USA, the provisions of their statutes (e.g. for Fire protection provision of *NFPA-National Fire Protection Association - Code 130 "Standard for Fixed Guideway transit and Passenger Rail System - 2017")* are applicable. There are no laid down standards for Indian Railway so far and, therefore, the standards being presented in this chapter are based on UIC approach, as per UIC Code 779-9 R (1st Edition, Aug'2003).

1. General: Tunnels of length up to 1 km are termed as "Short Tunnels", tunnels of length 1 to 15 km are termed as "Long Tunnels" and tunnels of length more than 15 km are termed as "Very Long Tunnels".

The recommendations are for tunnels in mixed passenger/ freight traffic and normal operating conditions (up to 200 trains/day). It does not cover underground platforms and surfaces in urban areas.

Safety in tunnels is the result of an optimum combination of infrastructure, operations and rolling stock measures. A general principle shared by all railways can be divided in following categories:

(i) Prevent accidents

(ii) Mitigate the impact of accidents

(iii) Facilitate self-rescue

(iv) Facilitate rescue

For each of the above categories, a set of safety measures can be defined for a combination of Infrastructure Measures, Rolling Stock Measures and Operations Measures.

2. Important Safety Aspects: Some important aspects related to safety are discussed in this para.

2.1 Tunnel Ventilation: Ventilation of tunnel is one of the important aspects related to passenger and crew comfort during passage of train inside the tunnel. It is also important for workmen working inside the tunnel from their health point of view. Movement of trains inside tunnel transforms its environmental features. Some of the pollutant gases, emitted from locomotives, may be potential hazards to the health, physiological and psychological comfort of human being. For safe operation, it is necessary that these hazardous features, especially gases emitted from locomotives, should not cause discomfort to crew, passenger and workman inside the tunnel. Concentration of pollutant gases and rise in temperature of air inside tunnel depends upon effectiveness of ventilation in tunnel. Thus, it is necessary that tunnels are provided with adequate ventilation, so that concentration of hazardous gases and rise in temperature of air inside tunnel remain within permissible limits.

2.2 Effect of Movement of Train inside Tunnel: Passage of a train in a tunnel creates following environmental hazards:

(A) Air Quality Deterioration: Emission from diesel locomotive contains potentially hazardous gases such as oxides of nitrogen (NO, NO_2), oxides

of carbon (CO, CO_2) Sulphur-dioxide (SO_2) and hydrocarbons. These gases get mixed up with the air inside tunnel and pollute it. High concentration of carbon monoxide gas causes headache and discomfort and may be fatal if stay is prolonged. Nitrogen Oxides (NO, NO_2) have toxic effects. Sulphur-dioxide (SO_2) is bronchial and nasal irritant.

(B) Thermal Environment Hazards: As a locomotive/ generator cars traverses through a tunnel, heat from exhaust gases is emitted. The air inside the tunnel gets heated up due to heat emitted from exhaust gasses/ locomotive surface. For safe operation of trains in the tunnel, the thermal environment is to be controlled within a safe range for efficient functioning of locomotive and comfort of passengers, crew and workman.

(C) Pressure Transient Hazards: When a train passes through a tunnel, aerodynamic effects come into play. Due to this, the drag and propulsion power increases and the pressure environment around the train gets changed. The change of pressure environment around the moving vehicle may cause severe discomfort to passengers.

2.3 Permissible Values of Pollutants: The permissible values for the concentration of pollutants in tunnels depend upon the time of exposure. Threshold levels for various pollutants inside tunnels are tabulated in Table 17.01.

* These values are from the consideration of passengers comfort and shall depend upon the length of the tunnel and speed of the train.

As workers are required to remain in tunnel for 8 hours, values for 8 hours exposure need to be considered for the design of ventilation system.

Maximum temperature of air inside tunnel needs to be limited to 40^0C considering passengers and workmen comfort.

Table 17.01: Threshold Levels for Pollutants

Pollutant Gas	8 Hours Exposure Values	15 Minutes Exposure Values*
CO	50 ppm	400 ppm
NO	25 ppm	5 ppm
NO_2	5 ppm	5 ppm
CO_2	5000 ppm	18000 ppm
SO_2	5 ppm	5 ppm

2.4 Types of Ventilation Systems: The ventilation in a tunnel can be achieved by:

(A) Natural Ventilation: When a train travels inside tunnel at a relatively high speed and ratio of train frontal area to tunnel cross section is of the order 0.5 to 0.6, it induces considerable air flow inside tunnel. This type of ventilation is called as natural ventilation. The amount of induced air flow will depend upon orientation of tunnel and atmospheric pressure difference between inside and outside the tunnel. If length of tunnel is small, the induced air flow may be sufficient to keep the pollutants concentration and rise in temperature inside tunnel, within permissible limits. In such case there may not be any necessity for provision of artificial ventilation.

(B) Artificial Ventilation: In long tunnels, natural ventilation is not sufficient to keep concentration of pollutant gases under permissible limit. In such cases, artificial ventilation may have to be provided by means of ventilation shafts with or without of ventilation fans, with suction and delivery arrangement. Where provision of shaft is not feasible, longitudinal ventilation with the help of an axial blower fan at the portal supplemented by auxiliary fan of smaller capacity, spaced at suitable intervals along the length of tunnel may be considered (Fig. 17.01).

Fig. 17.01: Artificial Ventilation

2.5 Tunnel Illumination System: Tunnel lighting (Fig. 17.02) is required for the following purposes:

(i) Passengers and staff to detrain and make their way safely out of the tunnel.

(ii) To assist train crews in their orientation and improve their visibility of the track. Adequate tunnel lighting allows drivers to quickly adjust to the light within, identify possible obstacles and negotiate their passage without reducing speed.

(iii) To let the inspectors or workers clearly see the track elements/condition or go through their routine inspections without using flashlights.

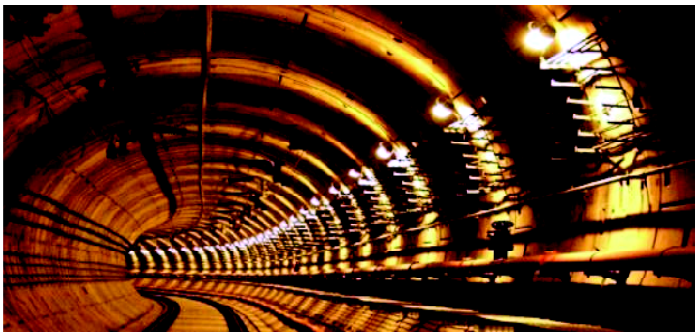

Fig. 17.02: Tunnel illumination System

2.6 Design of Tunnel Lighting Systems: The design of the lighting system should also:

(i) Take full account of the possible conditions of the tunnel following an emergency (for example, fire and smoke).

(ii) Consider when and how the lighting should function. Options include:

- Permanently switched on
- Switched on automatically following an incident or condition
- Manually controlled

(iii) Consider how maintenance illuminance will be provided in the event of failure of the normal power supply.

2.7 Normal Tunnel Lighting: Illumination system for General Lighting would be provided to achieve minimum 10 lux illumination level. Lighting fixtures for general lighting should be energy efficient type such as LED lighting fixtures. HPSV (High Pressure Sodium Vapour) and MH (Metal Halide) lighting fixtures which are power consumption intensive should be avoided. When there is no train operation for more than half an hour in tunnel or no maintenance activity is going on, the lighting should be reduced (about 30% or as considered appropriate) of full lighting to conserve energy in main and escape tunnels.

2.8 Emergency Tunnel Lighting: Emergency lighting having luminance of at least 5 lux at walkway level shall be provided to guide passengers and staff to a safe area in the event of emergency. Emergency tunnel lighting shall be installed on one or both sides of the tunnel. Emergency lighting shall be reliable and operative under emergency conditions (like presence of smoke). Lighting fixtures shall be energy efficient type such as LED type. Emergency lighting circuit

shall be alternative supply system (such as on UPS and DG set supply).

2.9 Power Supply System: The Power Supply arrangement shall be designed keeping in view the reliability, redundancy, voltage drop at the farthest end of feeding system. Generally, the Power Supply should be availed at 33 kV on both the tunnel portals from two independent sources which should be stepped down to 11kV first and then to 415 V at the sub-station near tunnel for feeding E&M system at 415V. The standby DG set of full load capacity should be provided at both tunnel portals to provide the standby Power Supply with redundancy in the system. Electrical equipment should be protected against damage, mechanical impact, heat or fire. The design of distribution system should be able to tolerate unavoidable damage by energizing alternate links. For Power Supply of control system and emergency lighting, 120 minute UPS backup with adequate SMF battery bank should be provided at all the Sub-stations at Tunnel Portals and inside the tunnel.

Alternatively, on electrified routes, Power Supply for Tunnel Ventilation and illumination systems can be tapped from OHE by providing 25kV/415V three phase Auxiliary Transformers in order to maintain reliability of the Power Supply. The step down transformers of 25kV single phase to 415V three phase will need to be specifically designed.

2.10 Cables: The 33kV double circuit, 11kV double circuits and LT circuits with redundancy shall be provided for which suitable cable ducts/ conduits/trays should be planned keeping in view the maintainability. Fire resistant, Halogen Free Low Smoke (HFLS) cables on protected cable ducts should be provided. Physical protection of cables against impact from derailments or construction work should be provided.

2.11 Leaky Feeder: In the main tunnel and in the accesses, a leaky feeder cable (or other alternative system) shall be used as antenna. It consists of a coaxial cable with gaps or slots in it, with a regular span in its outer conductor for radiating or receiving signals along its entire length.

2.12 Sensors inside Main and Escape Tunnels: For monitoring of environmental conditions/ pollutants, CO Sensors, Visibility Sensors, Train Location Sensors, Temperature Sensors, Wind Velocity Sensors and Heat detection cables should be provided inside tunnels with preset threshold values. The data of these sensors should be transmitted to the SCADA system provided in Tunnel Control Room through Optical fiber cable/ dupe line communication. The Tunnel Control Rooms would be manned round the clock where the tunnel operator can take the action as required as per the condition monitoring of the tunnel environment.

2.13 Safety walkway near tunnel portals: Safety pathways of adequate width (on both sides of track) should be provided near portals for full tunnel length for quick evacuation of passengers in case of emergency.

3. Infrastructure Measures

3.1 Speed Monitoring/Signaling System: Monitoring of speed can be affected on the locomotives, on speed-checking sections through automatic train control at fixed points (ATCS), by means of radar or ahead of signals, using signal based controls. It is specified based on Operating characteristics; train density and speed (e.g. >160 km/h).

(A) Specifications: The system should be able to prevent train from overrunning a stop signal and exceeding maximum speed, with a high reliability level.

(**B) Assessment:** New Tunnels: Speed monitoring recommended, if equipment on the specific route is planned.

Existing Tunnels: If upgrade of an existing system is possible, then tunnels should have high priority for upgrade.

3.2 Train Radio: A train radio system permits communication between the train crew and operations centre and with passengers in coaches. It includes fixed installations in tunnels and equipment on board trains (including coaches).

(A) Specifications: System is installed depending on the standard of the line on which tunnels are located. Reliability is highly important.

(B) Assessment: New Tunnels: Recommended as a standard measure (including the possibility to communicating messages to all coaches in a train).

Existing Tunnels: Recommended (a) if tunnel is on a line equipped with train radio system, then tunnel shall be also equipped. (b) if line is not yet equipped, tunnels shall be a relevant argument when setting priorities (c). A tunnel may not be equipped if it is on a line with a low volume of traffic or of secondary standard.

3.3 Train Detection (axle counter, track circuit): Checking that a track section has been completely cleared and trains are complete.

(A) Specifications: installation depending on operations conditions also e.g. very low-density traffic. It is combined with an adequate train protection/signaling system. This measure is not specific to tunnels only: if decided to equip, it means equipping the whole line.

(B) Assessment: New Tunnels: Recommended as standard measures. Both measures are equally effective.

Existing Tunnels: Recommended as standard (exceptions: for lines with very low-density traffic and simple operating conditions).

3.4 Train Control: Lineside fixed temperature sensors for the detection of hot axles and wheels, so that trains can be stopped in a safe place before entering the tunnel.

(A) Specifications: Appropriate distance between two installations: depending on the network-wide concept adopted for installations (typical range of between 25 and 100 km). Depending on the operation mode for double-track lines, one or both tracks are equipped. Rules and procedures to check a train.

(B) Assessment: New and Existing Tunnels: Recommended at the approach to sections with many tunnels. Isolated tunnels will be covered by the ordinary network of installations.

3.5 Arrangement of Switches: In tunnels and at the approach to tunnel entrances, the installation of switches or other track discontinuities should be avoided (completely remove or shift the location). Accidents caused or influenced adversely by switches will then not occur in tunnels.

(A) Specifications: Minimal distance between switches and tunnel entrance optimized to take into account line speed.

(B) Assessment: New and Existing Tunnels: Switches or other track discontinuities should be reduced to the operating minimum in tunnels. If not possible, movable-point-frog switches should be considered (depending on speed, axle load and operating requirements).

3.6 Access Control: Measures to prevent unauthorized access to the tunnel portals or exits: signs, fencing, secure locks, remote or local surveillance.

(A) Specifications: Signs: warning and entry prohibited at tunnel entrances. Emergency exits: locked doors, possibility of opening doors from inside by anyone and from outside by railway/rescue services (remote or on the spot). Fences for the portal area and emergency exits depending on the exposure and possible security hazard scenarios. Large doors for emergency access. Closed-circuit TV-monitoring of sensitive areas such as tunnel entrances depending on the exposure and possible security hazard scenarios. Remote monitoring by a control/operations centre for the tunnel.

(B) Assessment: New and Existing Tunnels: Security measures must be taken on the basis of a risk assessment including the location/exposure, accessibility of tunnel objects, their attraction as targets for vandalism or sabotage and local experience/tendency to be subjected to vandalism and sabotage. For new tunnels, it is recommended as a standard measure to put signs and fences at tunnel entrances and to lock all exits (see specifications). Further measures are recommended only if an assessment of security risks shows high risks. It is recommended that existing tunnels be upgraded (optimization) if reasonable because of the local situation (in general in urban areas).

3.7 Double-bore Single-track Tunnels: Double-bore single-track tunnels instead of single-bore double-track tunnels to avoid accidents caused by a derailed train obstructing the adjacent track and allow for better rescue conditions in the event of an accident or fire.

Assessment: New Tunnels: The optimal system should be result of evaluation of all relevant parameters. The more cost-effective system should be chosen provided that required escape distances and operating restrictions (e.g. mixed traffic) can be observed.

Existing Tunnels: Not applicable if the tunnel is a double-track tube.

3.8 Fire, Smoke and Gas detection in Tunnels: Installation of fire, smoke and gas detectors in tunnels, enabling rapid location of a fire in ignition phase in main tunnel and in technical rooms.

(A) Specifications: A distinction should be made between the main tunnel and technical rooms.

(B) Assessment: New Tunnels: Main tunnel - Not recommended as standard. Gas detectors recommended for tunnels with a low point in the tunnel (u-shaped) and if gas could enter the tunnel from the surroundings.

Technical rooms - Fire and/or smoke detectors are recommended for technical installations concentrated in separate rooms in a tunnel.

Existing Tunnels: In the course of a renewal/general upgrade, the recommendations for new tunnel should be followed as far as reasonable.

3.9 Fire Extinguishing System: Automatic or manually-triggered fire extinguishing systems in order to fight the fire in an early stage, in the main tunnel and in technical rooms.

(A) Specifications: To define the efficiency is part of a specific project.

(B) Assessment: New and Existing Tunnels: Main tunnel- no extinguishing systems are recommended. Automatic extinguishing systems are recommended only for rooms with highly sensitive technical installations.

3.10 Ventilation System: A distinction must be made between three main situations:

(a) Main tunnel: mechanical smoke extraction system in the main tunnel to draw out smoke or to create a defined air stream in order to obtain a smoke-free site for rescue.

(b) Smoke extraction if a tunnel on a double-track line consists of double-bore single-track tubes or at passages between double-bore single-track tubes (to keep the parallel tubes free of smoke, to prevent air streams)

(c) Safe places: ventilation systems to keep emergency exits, cross passages or a parallel safety tunnel free of smoke (produce overp-ressure).

(A) Specifications:

(a) No specification for the main tunnel.

(b) Combination of double-track/single-track tunnel or at passages between double-bore single-track tubes: the ventilation/smoke extraction system has to be designed so that smoke transfer from one tube into the other through the passage between the two tubes is reduced to a minimum. A detailed concept and sufficient dimensioning of the system are necessary.

(c) Safe places: The ventilation system has to be designed so that smoke transfer into the safe place is reduced to the minimum when opening doors to the main tunnel. If there are alternatives to an active ventilation system meeting that requirement, they are acceptable as well.

(B) Assessment: New Tunnels:

(a) Smoke extraction in the main tunnel: not recommended as standard measure to control smoke spread.

(b) & (c) Recommended for specific situations, where safe areas should be kept free of smoke (e.g. parallel tubes, emergency exits). In order to achieve this goal, alternative measures such as doors or locks may also be adequate.

Existing Tunnels: Not reasonably feasible for existing tunnels.

3.11 Escape Routes (Routes, Handrail, Marking): Provision of walkways in tunnels to facilitate escape (normally beside tunnel wall, also in or between tracks if there is not enough space). Handrail along the tunnel wall and especially around obstacles. The escape route and directions are marked by pictograms.

(A) Specifications: Minimum width of walkway for new tunnels: >70cm, optimally 1.20m. In double-track tunnels on both sides of tunnel. Existing tunnels: optimisation of surface (e.g. compressed gravel, cable duct with larger slab). Hard and smooth surface, free of obstacles as far as possible. Handrail leads around obstacles. Signs are to be located at lighting points: indication of escape direction and distance to nearest exit.

(B) Assessment: New Tunnels: Recommended as a standard measure as specified.

Existing Tunnels: Improvements to enable adequate movement are recommended for existing tunnels as basic equipment, solutions should be optimised and consider the specific risk situation (tunnel length, traffic, rescue concept).

3.12 Emergency Tunnel Lighting: Lights along one or both tunnel walls for lighting the escape routes in the event of a train evacuation. The lighting shall ensure uniform illumination of the escape route in order to enable evacuees to walk safely.

(A) Specifications: Following specifications are based on the assumption of electric lighting. Alternative technical solutions are possible as well if they fulfil the intended functions.

- On one or both sides

  Single-track tube: one side (same as walkway)

  Double-track tube: both sides

- Luminosity

  Enable a safe walking as far as possible also under smoke conditions and poor visibility

- Height of lights

  Above walkway, as low as possible, depending on free space

- Autonomy and reliability:

(a) Guaranteed power supply for emergency or alternative concepts to ensure high reliability.

(b) Supply cables protected against mechanical impact and fire

(c) It is recommended to build sections for power supply/lighting

- Other specifications:

  Possibility to switch on from operating centre, portals and inside the tunnel.

  Minimum distance between portal and first switch is 250 m if security aspects are relevant.

  Under normal operation, lighting is switched off.

(B) Assessment: New Tunnels: Reco-mmended for new tunnels as specified in the specifications.

Existing Tunnels: Tunnel lighting is generally recommended for tunnels of about >1 km. Decisions should be based on a risk assessment considering at least operations data and tunnel length. In order to keep a good cost-effectiveness ratio, specifications may be less stringent: lighting on one side only, lower requirements for luminosity and reliability.

3.13 Emergency Telephones/Communication Means: Emergency telephones or similar means of communication so that passengers, too, can use them in emergencies, connected with operations centre (independent of train radio or mobile phone). Emergency telephones shall permit

adequate and reliable communication during any emergency.

(A) Specifications: Clearly visible and easy to use (indications necessary). Reliability. Direct and easy-to-use connection to the operating centre. Distance between phones: 500–1000 m as guideline (depending on distance between exits or cross passages). Additional/alternative locations: Portals and exits.

(B) Assessment: New Tunnels: Recommended as standard.

Existing Tunnels: Recommended as standard, optimisation of existing telephones as far as reasonably possible.

3.14 Escape Distances: A maximum distance between two safe places (portal, emergency exit, cross passage) in the tunnel is defined in order to enable self-rescue.

(A) Specifications: Distance between safe places: 1000m (mean escape distance for self-rescue of 500m) as general guideline. For double-bore single-track tubes and parallel safety tunnel: reduced distance of 500m (cost-effective). This distance can vary depending on the local situation, operating parameters and the total safety concept.

(B) Assessment: New Tunnels: The cost effectiveness ratio depends on the local situation (costs). Under favourable conditions, a good cost-effectiveness ratio can be assumed.

Existing Tunnels: For existing tunnels additional construction work is very expensive and if for safety reasons only, the cost-effectiveness mostly will be very unfavourable.

3.15 Vertical Exits/Access: Construction of vertical exits from the tunnel which are used for escape as well as for access by rescue services.

(A) Specifications: Maximum height should be less than 30 m, width of stairs of about 1.2m as a guideline. Design or installation necessary that prevents smoke spreading into the safe place (possible solution: locks or ventilation system). Equipped with lighting and communication means (e.g. telephone). Design or installation that prevents unauthorised access from outside.

(B) Assessment: New Tunnels: If vertical exits are planned, a distance of about 1000m between the exits.

Existing Tunnels: For high risk tunnel: possibility to improve tunnels in the course of a total renewal, if opportunities (nearness to surface) exist. Decision should be made based on a sound evaluation.

3.16 Lateral Exits/Access: Construction of lateral exits from the tunnel which are used for escape as well as for access of rescue services.

(A) Specifications: Cross section: 2.25m x 2.25m as a guideline. Maximum length of about 150m as a guideline, but if longer, it should be accessible with road vehicles. Design or installation that prevents smoke from spreading into the safe place (possible solution: locks). Equipped with lighting and communication means (e.g. telephone). Design or installation that prevents an unauthorised access from outside.

(B) Assessment: New Tunnels: If lateral exits are planned, a distance of about 1000m between the exits.

Existing Tunnels: For high-risk tunnels: possibility to improve tunnels in the course of a total renewal. Decision should be made based on a sound evaluation.

3.17 Cross Passages: Cross passages between double-bore single-track tunnels or between a double-track tunnel and a safety tunnel.

(A) Specifications: Cross section: 2.25m x 2.25m as a guideline. Design or installation that prevents smoke from spreading into the safe place. Equipped with lighting and communication means (e.g. telephone). Design or installation that prevents unauthorised access to the neighbouring tube if train operation has not yet been stopped.

(B) Assessment: New Tunnels: If cross passages are planned, a distance of about 500m between the passages is recommended.

Existing Tunnels: For high risk tunnels: possibility to improve tunnels in the course of a total renewal. Decision should be made based on a sound evaluation

3.18 Parallel Service and Safety Tunnel: Provision of a service and safety tunnel parallel to the main tunnel (double-track). The tunnel is kept free of smoke and provides a safe place in the event of fire and other accidents. The safety tunnel can also be used by emergency services.

(A) Specifications: Cross passages to the main tunnel. Cross section: 3.5m x 3.5m as a guideline, accessible by road vehicles, possibilities to reverse and pass. Independent ventilation system (or similar installation) in order to keep the safety tunnel free of smoke (produce overpressure in relation to the cross passages and the main tunnel).

(B) Assessment: New Tunnels: Should be the result of an evaluation of the optimal system. Not recommended as general solution.

Existing Tunnels: For high-risk tunnels: possibility to improve tunnels in the course of a total renewal. Decision should be made based on a sound evaluation.

3.19 Access to Tunnel Entrance and Exits: Access road to portals and emergency exits for rescue services.

(A) Specifications: Access roads shall be accessible for normal fire brigades vehicles. Solid surface (damage after a large intervention is acceptable). Minimal width: 3m. The road ends at the rescue area or at a solid turning place. As close as reasonable to the entrance, depending on local topography.

(B) Assessment: New Tunnels: Recommended in combination with rescue area.

Existing Tunnels: Recommended to improve situations as far as reasonably practicable. If not possible, helicopter landing areas in the vicinity should be defined and prepared as far as reasonably practicable.

3.20 Rescue Areas at Tunnel Entrance or Exits: Rescue areas are situated in the vicinity of tunnel entrances and emergency exits as the base for rescue operations.

(A) Specifications: The area at the entrances of new tunnels should include:

- Access road to the area, accessible for fire-fighting lorries, solid road surface, possibility for two vehicles to cross on the way.
- Power supply, lighting, fixed provisions/ installations for communication.
- Possibility for water supply (on site or in the vicinity).
- Defined helicopter landing area (20mx20m) with road connection to the rescue area.
- Access to the portal.

The area at exits should include:

- Access road to the area, accessible for fire-fighting lorries, solid road surface, turning area and if this is not possible, helicopter landing area.

- Power supply, lighting, possibility for water supply (on site or in the vicinity).

  The area for existing tunnels should include

- Access road to the area, accessible with heavy fire-fighting lorries, turning area and/or defined helicopter landing area (20mx20m) as far as reasonably practicable.
- Power supply, lighting, possibility for water supply (as far as reasonable practicable).

Local possibilities have to be taken into account optimisation.

Existing roads, places and land area should be included in the considerations.

(B) Assessment: New Tunnels: Recommended as standard safety measure, in the light of local possibilities.

Existing Tunnels: Generally recommended with the restriction that local topography and possibilities should be taken into account for optimising.

3.21 Water Supply (as access, in tunnel): Continuous water main through the tunnel: permanently filled or dry pipe. Branch lines to tunnel entrances: portals, emergency exits: permanently filled or dry pipe.

(A) Specifications: Supply of water pipe: pool, hydrant in the vicinity, connected to water supply system, other sources (e.g. river). Reserve of 100 m^3 at tunnel entrances if water supply is based on pools. Hydrants in the tunnel: every 250m if there is a continuous pipe; at emergency exits, if supply is only through these exits. Installed on one side of the track. Design should especially consider maintenance aspects.

(B) Assessment: New Tunnels: Water supply as continuous pipe through the tunnel or branch lines to portals and exits are recommended as standard. Alternative solutions with mobile railway means are adequate too, if they are based on "professional" rescue organisation (e.g. fire-fighting and rescue train).

Existing Tunnels: If the intervention concept is based on railway resources: mobile water supply is recommended (e.g. rescue train). If the concept is based on fire brigades: water supply to the tunnel portals is recommended, e.g. mobile means by road, water reserves in the vicinity. Additional equipment of an existing tunnel only in the course of a renewal.

3.22 Electrical Supply for Rescue Services: Power supply suitable for the equipment needed by emergency services in tunnels.

(A) Specifications: Distance between outlets: 125–250m. Ensure compatibility for rescue service and maintenance. Location in niches, concentrated with other electrical installations and communication means. On one or both sides of the track. For short tunnels and/or existing tunnels: mobile means as alternative

(B) Assessment: New Tunnels: Recommended to integrate in a comprehensive concept for power supply and installations.

Existing Tunnels: Recommended to upgrade existing tunnels in the course of a renewal of a tunnel or else to provide mobile means.

3.23 Radio Installation for Rescue Services: Ensure radio communication for emergency services in a tunnel between emergency services, operations centre, railway personnel (in general: own frequencies for rescue services).

(A) Specifications: Channel with common frequency necessary.

(B) Assessment: New Tunnels: Recommended as a standard measure.

Existing Tunnels: Generally recommended, but depends on the specific situation, alternatives possible.

3.24 Control Systems: Tunnels with large electromechanical installations shall be equipped with a centralised control system (tunnel control centre).

(A) Specifications: Control of: ventilation/smoke extraction system, lighting, communication means, power supply and all other safety systems, etc. Security measures such as closed circuit TV monitoring system. Eventually also operating tasks. Professional staff/24h-operation.

(B) Assessment: New Tunnels: Not recommended for new tunnels less than 15 km long. It is reasonable to integrate these functions into ordinary operations centres which are also responsible for the stretches on the approach to a tunnel.

Existing Tunnels: Only for new tunnels.

3.25 Tunnel Rescue Train: Railway vehicles for rescue purposes can be defined on different levels:

(a) Provision of carrier wagons to carry rescue vehicles and of tank wagons for water supply. Fire brigades load their vehicles onto the carrier wagon hauled by a locomotive or tractor.

(b) Special rescue unit/train: rescue train for rapid transport of staff and equipment. The train is specially built for intervention and serves as a means of transport, a base for fire-fighting, first aid, transport of injured people and for communication. The staff is composed of railway staff and local fire brigades.

Assessment: New and Existing Tunnels:

(a) Recommended if it is part of a comprehensive rescue concept which includes alternative ways to reach the site of an accident in a tunnel (e.g. through exits) or for an individual tunnel in a lower risk class.

(b) Recommended if it is part of a comprehensive rescue concept based primarily on railway resources for rescue.

3.26 Road/Rail Vehicles for Rescue: The relevant fire brigades are provided with road/rail vehicles which are able to run on track to convey staff and equipment rapidly to the accident site. The main goals are interception, support of self-rescue, first aid and initial fire-fighting action.

(A) Assessment: New and Existing Tunnels: Recommended only if road/rail vehicles are part of a comprehensive rescue concept based on fire brigades.

4. Prevention of Fire on Rolling Stock

4.1 Fire Load and Prevent Fire Spreading:

(a) Constructive measures/vehicle design to prevent outbreak and spread of fire.

(b) Avoiding the use of materials producing toxic substances/a large amount of smoke in the event of fire.

(A) Specifications: Reduction of the fire load; separation (compartment-type construction with interconnecting doors constructed as fire doors); use of fire-resistant materials; replacing flammable by hardly-flammable material; introducing fire resistant layers inside seats although these increase the fire load.

(B) Assessment: New and Existing Tunnels: It is recommended that fire safety aspects be emphasised and integrated in the specifications for new rolling

stock and also that it be ensured that they are taken into account for coach renewals.

4.2 Onboard Fire Detection:

(a) Automatic fire detection on traction units to detect fire at an early stage (with notification to the driver).

(b) Automatic fire detection on coaches to detect fire at an early stage (with notification to the driver).

Assessment: New and Existing Tunnels:

(a) Recommended for traction units

(b) Not recommended for passenger coaches in general. To be considered for technical installations in separate compartments.

4.3 Onboard Fire Extinguishing Equipment:

(a) Portable fire extinguishers on traction units and in coaches. The use of more effective extinguishing agents would improve extinguishing performance, reliability and ease of use.

(b) Automatic or manually-operated extinguishing systems on traction units (e.g. sprinklers for defined compartments).

(c) Automatic fire-extinguishing systems in coaches (technical compartments, passenger compartments).

Assessment: New and Existing Tunnels:

(a) Portable fire extinguishers on traction units and coaches: Recommended as a standard measure, ensure proper functioning and improve the effectiveness.

(b) Automatic or manually-operated extinguishing systems on traction units: Recommended for new traction units and for specified mechanical or electrical components on networks with large number of tunnels (especially very long tunnels).

(c) Automatic fire-extinguishing systems in coaches: Not recommended for all coaches. May be a reasonable solution under defined conditions such as a closed network, operation with fixed consists (typically commuter trains).

4.4 First-aid Equipment Onboard: Each train is equipped with at least one first aid box.

Assessment: New and Existing Tunnels: Recommended as a general safety measure (not only tunnel safety).

4.5 Escape Equipment and Design of Coaches:

(a) Escape equipment: the train crew is equipped with megaphones for communication and lamps to be able to inform passengers in the event of evacuation, also for use outside the train.

(b) Escape design: coaches (doors, windows, body shell) are designed with defined emergency exits/accesses. The respective places are visible/indicated for passengers and rescue services.

Assessment: New and Existing Tunnels:

(a) Recommended as suitable

(b) It is recommended to integrate the aspect of emergency exits/accesses in further specifications for coaches (but this is not something specific solely to tunnels)

5. Operations Measures

5.1 Regulations for Carriage of Dangerous Goods: Restrictions on transit through tunnels of passenger trains and freight trains carrying dangerous goods:

(a) Dangerous goods in general (including single loads or wagons in a freight train).

(b) Block trains carrying dangerous goods only.

Assessment: New and Existing Tunnels: Recommended for high risk tunnels, if operating conditions permit (optimise operations from safety standpoint).

5.2 Emergency Information for Passengers: Passengers are informed about what to do in the event of an emergency with special emphasis on incidents in tunnels.

(A) Specifications: Means are: Posters, leaflets, spot publicity, on-board TV.

(B) Assessment: New and Existing Tunnels: Implement.

5.3 Emergency and Rescue Plans: Preparation of emergency plans consisting of:

- Strategy for dealing with critical events
- Emergency call-out plans
- Tunnel-specific plans for rescue services

Assessment: New and Existing Tunnels: Recommended as a standard measure.

5.4 Information on carriage of Dangerous Goods:

(a) Notification of movements of exceptionally dangerous goods (to be defined, e.g. chlorine, propane, vinyl chloride) to inform rescue services concerned along the route to be prepared in the event of emergency and to be able to take the right action in time (e.g. evacuation).

(b) Information system to identify rapidly the loads involved in the event of an accident in order to take the right precautions and action for intervention (precise and rapidly accessible database).

Assessment: New and Existing Tunnels: Reasonable as a general safety measure if information concerning carriage of dangerous

goods is improved, but not recommend as safety measure specific to tunnels.

5.5 Provision of Rescue Equipment: Provision of rescue equipment for fire-fighting in a tunnel.

(A) Specifications: All rescue services persons are equipped with breathing apparatus for use when fighting a fire in tunnel. Rolling pallets are located at tunnel entrances and exits. Depending on the rescue concept, further minimum equipment is located at entrances.

(B) Assessment: New and Existing Tunnels: Provision of adequate breathing apparatus is a standard measure (prerequisite for an intervention).

6. Additional Measures for Very Long Tunnels

6.1 Segmentation of Overhead Lines: Disconnection of the overhead line into segments in very long tunnel. Earthing devices including voltage measuring instruments should be positioned at the entrances, portals and emergency exits for the nearby segments of the overhead line.

(A) Specifications: The different segmented parts of the OHL must take into account the escape routes possible for trains in the tunnel in the event of an emergency so that rescue services can enter the tunnel safely and “captured” electric trains can exit from the tunnel. Communication means (e.g. telephone) and lighting of the place are ensured.

Procedures and responsibilities are defined (including communication between rescue services and the relevant centre).

(B) Assessment: New and Existing Tunnels: Recommended as a safety measure for very long tunnels as specified.

7. Safety measures for existing tunnels: For existing tunnel up to 2 km long, suitable safety measures may be adopted, on case to case basis and depending on the site conditions, keeping in view the principles elaborated above for the long tunnels.

8. Mock Drills: Mock drills should be conducted periodically (recommended at least once every two years) on at least one tunnel in section of every ADEN to assess the level of safety preparedness/awareness. Concerned State Government/District administration authorities should also be briefed about and involved in conducting mock drills. Mock drill can include scenarios for evacuation and rescue/relief in case of “fire accidents” serious injuries to passengers etc.

CHAPTER-18

TUNNEL PORTALS

There are generally two modes of access to tunnel construction face: through a portal providing direct access at the surface or through a shaft providing vertical access to the level of tunnel operations. Whether to use portal, shaft or a combination of the two is determined by the elevation of the tunnel, utility of this access during construction and whether the ingress is temporary or permanent. Generally speaking, transportation tunnels rely on portals because of the relative elevation. Usually, the tunnelling through mountain will use portals and sometimes congestion may require that the portal to the tunnel be below the surface, as is often the case with urban rail tunnels. When this is the situation, a ramp is constructed by cut-and-cover method.

1. Need for Portal: The portals are constructed to provide well defined access to tunnel and protect the entrance of tunnel from rock or other objects falling on it. Structurally, the portal protects and supports the tunnel entrance and approach from the earth and rock above it. It prevents surface water from entering the tunnel and provides a means to drain the water running down from above the portal.

Often acting as a retaining wall, the portal is more critical than often realized and it should be designed and constructed based on that criticality. The loads that the portal has to support will sometimes require the utilization of geometric shapes for strength (e.g. the portal being curved in shape to use the arch support).

2. Design of Portal: Design of tunnel portal is quite simple and following factors are taken into account:

(i) Load on beam is taken as self-load plus a uniformly distributed live load calculated by

assuming 45^0 dispersion above the beam.

(ii) Design the portal frame by calculating Bending moments, Shear forces and Displacements.

(iii) Provide section of beam and columns and reinforcement accordingly.

(iv) Stabilize the slope above the portal.

3. Construction of Portal: The steps normally followed in construction of tunnel portals are as following (Fig. 18.01):

(i) Construct approach to reach working face.

(ii) Remove the loose overburden consisting of weathered rock in first few meters.

(iii) Make a working platform, at the invert level, and mark outline of the tunnel face on the exposed rock face.

(iv) Construct a RCC or Steel frame portal around the periphery of the proposed tunnel. Steel portal is embedded in concrete (Fig. 18.02).

Fig. 18.01: Portal Development

If the tunnel begins at a soil slope, the slope should be cut back so that the ground above is minimum one to two diameters distance above the portal. The ground may consist of competent rock or talus on a mountain side and everything in between.

Fig. 18.02: Tunnel Face Opening

If the ground is stable rock, a portal structure may not be necessary. Using rock bolts or rock bolts with wire mesh or adding shotcrete to the bolts and mesh may suffice. If bolt holes can be drilled vertically from the surface, it is recommended to blast the slope so that there is vertical wall of rock and where appropriate to clean the surface using controlled blasting, to provide a wall with limited blasting damage. If required, the slope can be benched and supported with wire mesh, shotcrete and tie backs. On benches, ditches (catch water drains) are provided to collect water and drain it away from the portal.

Portal construction involves excavating and supporting the tunnel entrance. If support is required to stabilize or hold the crown, Spiling can be installed. Spiling can be pipes (known as Pipe Roofing) that are driven and/ or drilled above the tunnel crown (Fig. 18.03). These pipes support the roof of the excavation. The pipes in the soft ground have angular cuts at end, making them self-drilling. The pipes are then grouted to provide a solid shell above the portal.

The goal is to support the crown. Steel beams can also be used for stabilization of the roof, for the permanent portals, or they can be used in combination with pipes (Fig. 18.04). The criticality of the portal requires that it be totally supported. Rock should be completely bolted in the portal area, with spacing of the rock bolts varying from 1.20m to 3m depending upon the quality of the

rock. In soft ground or soil, it may be necessary to use forepoling with pipe or one of the method discussed in Chapter-14.

Fig. 18.03: Spiling above Portal

Fig. 18.04: Using Steel Beams for Portals

The portal can be constructed and supported using ribs and lagging, shotcreted ribs and lagging, and liner plate, to name a few. The portal must be constructed to facilitate water handling on a slope and requisite berms and/or catch water drains may have to be built for this. Irrespective of the portal's surroundings, there must be a means to handle water, whether it is from the tunnel, groundwater, or runoff. The portal should be graded to create a low spot with a sump and a pump to handle water that enters the portal area. When planning this, one should check local regulations regarding disposal of the water.

The portal should extend from the tunnel entrance far enough to prevent falling or rolling material from landing on the approach invert. To facilitate this, a facade (also known as False Portal) may be constructed extending to the requisite distance from face of the tunnel (Fig. 18.05). Normally, a suitable thickness of earth pad (a layer of earth) is provided on the roof of false portals, with parapets on both sides. Any boulder or rolling material is retained on this earth pad with very less impact load on the roof and these materials are cleared from there as part of maintenance operations.

Fig. 18.05: False Portal

Fig. 18.06: Portal Construction using Ribs and Liner Plate

Portal, including false portal, can be constructed using liner plate and steel beams (Fig. 18.06). At the base of liner plates, footings are constructed for long term stability. If portal is on a slope, the slope may be cleaned and shotcreted, once the liner plate is tied into the slope, to protect from the rocks rolling down the slope. The shotcrete prevents erosion of the slope into the portal roof and strengthens the portal.

In addition to considering ground and water control requirements, it must be kept in mind that the portal area serves as the focal center for the tunnelling operations. Since everything goes through it, it must be designed and equipped to facilitate movement through it. Generally, electrical panels are either in the portal area or above the portal elevation on the top of the bank. If that is the case, there should be easy access from the portal to the panel, for example, using stairs.

There should be a ventilation fan in the portal area, at least during tunnel construction phase. It should be mounted on a solid frame, elevated out of the way, and should be as quiet as possible. It is dangerous for workers in the portal area to not be able to hear. Storage for general-use small tools and materials should be readily available in the portal area.

The workers, materials and muck will be travelling into and out of the portal area. If trains are used for muck haulage during construction of tunnel, the train will have to pass through the portal area to dump the muck cars. All supplies, utilities, equipment, workers and possibly liner segments will be loaded in the portal area. It will be a very active area, which means adequate safety considerations must be taken. If trains are moving in the area, walkways should be easily be seen by the locomotive operator. The portal location, design and construction should take into account all these factors.

CHAPTER-19

SHAFTS

Shafts are vertical or nearly vertical openings connecting the surface and the underground structure and when used during construction serve the same purpose as portals. Many times, the words “Shaft” and “Adits” are used interchangeably but technically speaking “adit” is an entrance to an underground structure which is horizontal or nearly horizontal in contrast to “shaft” which is vertical or near vertical entry to the underground excavation.

1. Need for Shafts: During construction phase, the shafts and adits provide extra working faces for tunnel excavation, thereby increasing the progress of tunnel construction work. Tunnel shafts can be temporary or permanent. Temporary shafts are for use during construction only. Permanent shafts may be used during construction, but will become an integral part of the tunnel structure. Permanent shafts can be used for access to tunnels, elevators/stairways or both, ventilation, pumping, utility lines or manholes, or they may be enlarged to house stations. Temporary shafts are normally backfilled at the end of construction.

2. Location of Shafts: The location of shafts is critical in planning efficient construction. Locating a shaft at the midpoint of a tunnel will permit tunnel driving in two directions; also a single set of excavation machinery and office accommodation can serve both excavation faces. Locating a shaft near vacant land will facilitate the erection of temporary buildings. The proximity of muck disposal locations and routes should also be considered.

While selecting the location of shaft, the purpose of the shaft during construction, end use, proximity to utilities and the underground needs have to be considered. If

the shaft has to serve as ventilation shaft in the transport tunnel, then the shaft location alignment is fixed.

Another major consideration is the effects on the environment. If the shaft is located on a street, traffic and local businesses will be affected: resulting in both traffic congestion and loss of business income. In addition, it may be prohibited to locate a shaft near an environmentally sensitive area such as a stream or it may create environmental problems and require facilities to treat the water, for example.

Once the shaft has been located in a grade, a pump chamber and a sump may be excavated if required. The pump should provide sufficient capacity to handle the maximum anticipated flow. Information regarding the amount of water that will enter the shaft is obtained during the shaft excavation process, and an estimate of groundwater seepage can be based on previous experience in the same soil or rock medium. Unexpectedly large inflows may occur if water-bearing strata or seams are encountered during excavation.

3. Shapes of the Shaft: Theoretically, shafts can be in any geometric shape. However, the most common shapes are the circle, rectangle, square and ellipse. The shape of the shaft is dependent on the use and ground conditions. The circular shaft is generally preferred, as it is structurally much stronger and can be more efficiently supported. Therefore, in ground with high lateral stresses or heavy loading on the shaft ground support, a circular shaft is preferred.

4. Design of Shaft: The shaft construction method is determined by:

(i) Depth of groundwater table,

(ii) Type of ground to be excavated,

(iii) Extent of working space needed, and

(iv) Depth of shaft.

While determining shaft construction method, minimum size of shaft has to be decided. During design, the minimum dimensions are typically determined by the physical layout of the final structure to be constructed or space needed for launching a Tunnel Boring Machine (TBM). In transport tunnels, shafts can be used for access, elevators, ventilation, transit stations and utility drops. It is difficult to determine exactly what size shaft the contractor will need because proposed means and methods of excavation and exact type of equipment that will be used, are not known at this stage.

5. Collar: As with any construction, if the activity is not started correctly, there will be problems for the entire activity that could have been avoided. The same is true with sinking a shaft. The starting point in the construction of a shaft is the collar. The collar is uppermost section of the shaft. A concrete collar, a ring of concrete, should be placed around the top of the shaft. The collar prevents distortion of the shaft's primary lining and prevents surface water and debris from falling into the shaft. The depth of the collar depends on the depth to the rock, the sinking method and the groundwater. It is generally considered that the collar extends to bedrock and in some shallow shafts used for the civil tunneling applications that can be the entire depth. The collar structure is constructed prior to shaft sinking. The shaft excavation area is excavated, generally as a trench. For small shafts, the collar can be made simple but should be no smaller than 1m wide and 1.5m deep and should be constructed of reinforced concrete.

The collar beam should be at least 0.3m above the ground surface to prevent things being inadvertently kicked into the shaft and to prevent the inflow of water.

The collar is where the survey is established and transferred to the tunnel. It should be remembered that a shaft collar is load-bearing structure. Therefore, a complete analysis of the loads must be conducted. The size of the collar area may be increased or modified

to suit operations. For example, if a gantry crane is to be used for shaft excavation, a concrete slab will need to be appended to the collar for the crane's rail and other equipment.

The shaft collar should be constructed based on an engineered solution. That is, it should be designed based on the engineering properties of the soils and loading of the collar area.

The material used for the collar should be the same as for the shaft lining. The lining immediately follows below the collar. The transition from collar to shaft should not be apparent.

6. Area around Shaft: The shaft area should be graded to drain all surface water away from the shaft. It is a good idea to place gravel or stone (concrete is even better) on the surface area to maintain a neat and organized working area. If, to save money, the shaft area is not organized well, in the long run this decision may cost much more than proper preparation would have.

The top of the shaft should be reviewed for prevention of falls and dropping things down the shaft. Handrails or another type of safety protection around the top of the shaft must also be provided. The federal, state or local safety regulations must be complied. The safety regulations may require only a standard railing of 1.1m height but that may be inadequate for great depths and wherever possible 1.2m high chain link fencing should be provided on the perimeter of the shaft, which eliminates considerable risk.

7. Shaft excavation in Soft Ground: Shafts in soft ground are normally excavated with a crane using a clamshell bucket to host the muck from the shaft and drop it into a hopper or a stockpile or directly into a truck on the surface.

Generally, the most difficult shaft sinking is through the overburden, because the overburden is generally

not as stable as rock. It can be practically difficult at the interface of the overburden and the rock. This is often an area of water ingress.

Primary shaft linings are normally installed at every 4 or 5 feet of advance. However, shafts up to 30 feet have been sunk without supports. The rate of installation depends on the type of lining and the nature of the soil medium. A permanent shaft usually will have a final lining of concrete and the concrete for lining may be cast either with forms on both sides or forms on the inside only with the ground support system on the outside.

Soft ground shaft sinking may disturb or damage neighbouring buildings, utilities, pipelines or stresses. The problem is especially acute with soft plastic soils; when plastic soil is excavated, the load over the excavated area is reduced, and plastic yielding may result, causing ground yielding at the surface. A properly designed shaft support system can prevent plastic yielding. Soil characteristics, shaft depth, diameter, and economic factors will dictate the choice among the many available sheeting and bracing system.

8. Support Systems used for Shaft Excavation: There are many methods for supporting the ground to excavate through the overburden. Sinking a shaft has an advantage over tunnelling when it comes to handling bad ground. With a shaft, one is on top of the ground being excavated rather than having to go under it. That means, when approaching bad ground, one is in a better position to implement ground support measures.

A shaft can be sunk through soil using various types of support depending on depth and material to be excavated. Some of these support types are as under:

8.1 Timber Sheet Piling: This is normally used only in shallow shafts, since driving the thick timbers is difficult. The method can be economical to start excavating in soft material, not deeper than about 20 feet of soil overlying rock. Although

easy to install, wood sheet piles are poor for ground water control. Moreover, availability of timber is an issue in present times.

Timber or wood sheet piles (3 to 4 inch thick) are driven vertically on the shaft perimeter prior to excavation (Fig. 19.01). Horizontal steel ribs are installed against the interior of the sheeting, to counter the lateral earth pressure acting on the sheeting, by compression or bending action of the ring depending upon whether the shaft is circular or rectangular. It is important that a template is used to maintain the line and plumb of sheet pile wall.

Fig. 19.01: Timber Sheet Piling

8.2 Steel Sheet Piling: Steel sheet piles consist of steel that is forged to different shapes providing additional flexural strength and an interlocking capability. They are constructed by driving prefabricated sections into the ground. Interlocking steel sheet piles are commonly used to brace soft, water bearing ground if the excavation depth exceeds about 20 feet. Steel sheet piles are generally limited to depths of 25m (80 feet). For greater depths, the shaft can be stepped in and another level of sheet pile can be driven. However, the stepped-in area must be wide enough to allow the driving equipment; thus it has limited uses. Steel sheet piles can be driven into harder ground. Because of its lock ends, it is very good for water control as compared to wood sheet piles.

Steel sheeting is desired when depths are so great that wood sheet pile cannot work and where high loads are present. Also, if it is required to leave the sheeting in the ground for a long period of time, steel sheeting can resist corrosion. More force is required to penetrate the ground, and thus heavier equipment is necessary. They are usually driven by a pile hammer or vibratory hammer, depending upon the type and condition of the ground being penetrated.

The steel sheet pile wall is formed by sequentially connecting the joints of adjacent sheet pile section. Steel piles must be driven carefully, to ensure proper interlocking of the joints to cut-off water seepage. Each pile is driven to the planned depth, then the next pile is driven and the two are locked together. This process is continued till the shaft wall is complete. Excavation usually begins after the pile diving operation is completed, unless the shaft is unusually deep. Horizontal steel rib sets (wales) can then be installed progressively at appropriate vertical depths as the excavation progresses downwards (Fig. 19.02). The lateral earth pressure is transferred to the wales.

Fig. 19.02: Steel Sheet Piling

Steel sheet piling is most commonly used because it provides high resistance to driving stresses, is

light weight and the pile can be easily changed using welding or bolting. Because of its reuse feature, its cost can be amortized over several projects. During driving, the joints are less likely to deform and, with some protection, can have a long service life below water.

Problems can develop when trying to drive sheet piles through difficult soils or boulders, because then driving them to desired depth may not be possible. Sheet piles can rarely be used as a permanent structure. Because of noise and vibrations, there may be complaints from habitants staying near site of work.

8.3 Soldier Piles and Lagging: Soldier piles and lagging used to retain earth are among oldest forms of retaining systems in deep excavations. The system consists of H-piles called soldier piles driven or placed in drilled holes, usually spaced from 6 to 10 feet apart. Once the soldier beams have been installed, the soil is excavated along one side of the beams to partially expose the front faces of the beams. Then, the wood lagging is installed to temporarily hold back the soil (Fig. 19.03). In some case, reinforced concrete panels can also be utilized for permanent conditions instead of timber. In moist to wet soils, the water usually drains between the lagging boards.

In deeper excavations, where large earth pressures are encountered, horizontal steel ribs sets (wales) are installed. When the shaft is greater than 6m or the deflection near existing structures (such as buildings and utilities) is very strict, wales are often used. The steel rib sets must be designed for either ring compression or bending, depending on whether the shaft is circular or rectangular. In large rectangular shafts, steel struts can be installed to span between wales. In this system, the moment is resisted by the soldier pile, and the lagging does not provide resistance to the

moment. Embedding the soldier piles below the shaft bottom provides passive soil resistance, whereas the lagging between the piles, retaining the soil, transfers the lateral load to the soldier piles.

Fig. 19.03: Soldier Piles and Lagging

With the soldier pile being the only part of the wall embedded beneath the subgrade, it is difficult to control basal movement and the system is not as stiff as other support systems. Hence soldier piles and lagging walls are generally used in temporary constructions, and if not properly backfilled behind the lagging, there can be a problem with ground losses and surface settlement.

8.4 Liner Plates: Liner plates are corrugated pressed steel pans with bolt holes in the flanges on the sides and ends to permit bolted erection of the ring. They are curved longitudinally according to the curvature of the tunnel (Fig. 19.04). Liner plates come in different thicknesses (gages) to suit the given loading conditions.

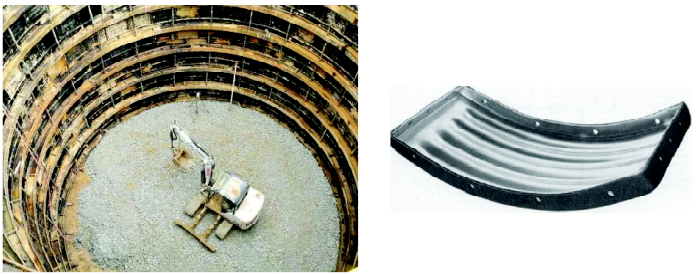

Fig. 19.04: Liner Plates

The main advantage of liner plates is that their small size permits ease of operation in the limited working spaces, and it is not necessary to have special equipment to lift or place the liner plates.

Shaft excavation begins by precisely erecting the first ring of liner plates on the ground surface and placing a concrete or earth collar around it. The soil is then excavated within the ring, and as space becomes available a liner plate is bolted to the bottom flange of the first liner plate ring; assembly of the succeeding rings proceed in the same manner. The joints of individual liner plates are staggered from the joint immediately above to increase strength. If voids are observed while sinking, material should be stuffed into them to reduce the chances of localized caving because if left unfilled such voids can continue to expand and eventually cause major ground failure. The fill behind the liner plate does not have to be structural. Hay is often used to fill such voids but being inflammable, the risk of fire must be kept in mind.

The soil pressure is carried by the liner plates in ring compression. In larger diameter shafts or in shafts where the lateral earth pressure is large, circular horizontal steel rib sets (ring wales) can be set at a predetermined vertical spacing inside the liner plate ring to increase strength.

Grout holes are fabricated in the liner plate to permit the grouting voids behind the liner plate or to stop/reduce the inflow of water.

8.5 Horizontal Ribs and Vertical Lagging: The horizontal ribs and vertical lagging method, also called as ribs and lagging method, is somewhat similar to liner plate construction. Rings are made of structural steel members, cold-formed to required curvature, the sections butted at each end. Butt plates welded to the ends of the segments are provided with bolt holes. Six to eight feet length of timbers are usually used for lagging (Fig. 19.05).

This method requires excavation of the soil to a distance equal to the length of the lagging. Curved ring segments are bolted together and held in place by tie rods and spacers that are placed between the webs of the rings. Placement of the vertical lagging follows. The steel rings can be placed at varying vertical spacing to resist the varying lateral earth pressure.

Fig. 19.05: Ribs and Lagging

Since the soil must be initially somewhat self-supporting for the height of lagging, this method is

usually employed in cohesive soils, although it can be used when the ground has moderate stand-up time. During construction, the interval between excavation and lagging placement should be minimal to prevent ground loss.

8.6 Slurry Walls: Bentonite, naturally occurring clay, has a large capacity for absorbing water. In suspension, it is a liquid when agitated, and a gel if left to stand.

A trench is dug in to build a wall using slurry, consisting of water and bentonite. Prior to starting the excavation, guide walls are constructed to maintain the slurry wall alignment. The guide walls are constructed on the surface and are approximately 50cm wide by 1m deep. The trench is excavated and kept full of slurry at all the times. The slurry prevents the trench from collapsing by providing outward pressure which balances the inward hydraulic forces and prevents water flow into the trench. Excavation is generally done upto the design depth using a special clamshell shaped digger. The excavator is then lifted and moved along the trench guide walls to continue the trench with successive cuts as needed. While maintaining the slurry level to prevent the cave-in, the excavator is moved to complete the entire length of wall once it reaches rock or designed depth. Then pre-fabricated steel reinforcement cage is lowered into the slurry filled trench, using spacers to ensure correct positioning of cage. During excavation and installation of rebar, the viscosity and density of slurry are monitored. Tremie pipes are used to pour concrete into the slurry trench. The concrete displaces the slurry and the slurry is captured and saved for other excavations (Fig. 19.06). A cofferdam can then be created by bracing the concrete with steel or reinforced concrete wales and struts.

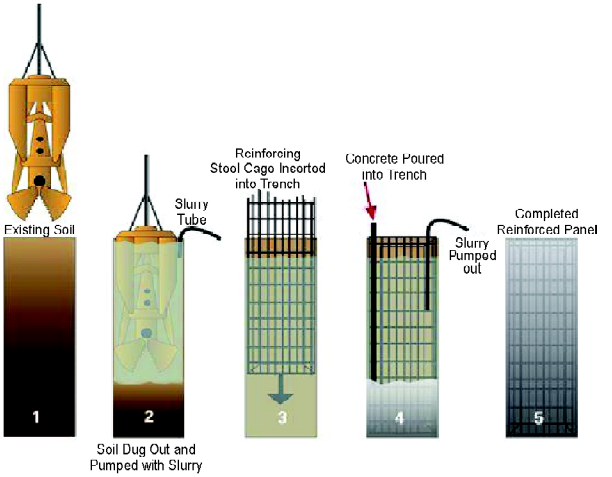

Fig. 19.06: Slurry Wall Cycle

Slurry walls are useful in soil that is unstable or has a high water table. Slurry walls are used as both temporary and permanent earth retention systems to support the sides of deep excavation.

Another method, known as “interlocked element type”, is used when the soil is very hard and bouldery, or when the excavation must reach considerable depths. Primary holes upto desired depth are first drilled using percussion rigs with the assistance of bentonite slurry. Generally the spacing of primary holes is kept as twice their diameter. This also ensures that penetration of the soil between primary holes by the bentonite slurry is almost complete, and therefore the excavation of the secondary elements involves a zone of soil stabilized through gelation in its pores. Concrete is placed in primary holes with the aid of Tremie pipes. A hydraulically expandable chisel is then used to excavate the panels between the primary holes. The secondary elements are then put into these spaces between the primary elements, interlocking with the primary elements.

The structure built can be reinforced by placing the steel in the slurry filled primary holes before placing the concrete.

8.7 NATM System: A recent development in the design of support for shafts in soft ground has been the adaption of the New Austrian Tunnelling Methodology (NATM), in the same way as it is done for tunnels.

In general, the NATM support system follows somewhat the same sequence as the installation of liner plate. A lattice-type rib ring is a key element of this system. After excavation of a section of the shaft, lattice ring is installed. After this, shotcrete with wire mesh is applied to the excavated exposed surface. Usually about 6" of shotcrete thickness is adequate, but if the instrumentation indicates movement of the earth or other undesirable characteristics, more shotcrete can be applied to obtain a thickness of 12" or more.

9. Shaft excavation in Soft and Wet Ground: Excavation in soft, wet ground can be accomplished in a number of ways. The most common method is to lower, by any of several means, the groundwater table in the working area. Other methods include freezing of the soil, the use of slurry, grouting, sinking a pneumatic caisson and sinking a dredged drop caisson with a Tremie concrete seal.

9.1 Lowering of Groundwater: Although this method is time consuming, it ensures dry, safe and firm working conditions. To determine the proper type of groundwater control system, geological and soil information should be evaluated a pumping test should be performed on the soil. The pumping test should yield results such as water volume pumped, well yield and time required to reach equilibrium.

Dewatering can result in lowering of the water table under adjacent areas. Therefore, extreme

caution must be observed if large structures are located in the vicinity of the dewatering operations. Dewatering increases effective stress in the soil, which in turn causes settlements. Also, in cases where adjacent structures are supported on piles, enough drawdrag can be developed on the pile foundation to cause settlement.

9.2 Open Pumping: This method consists of driving steel sheet piling, excavating and pumping water from the bottom of the excavation.

However, the pumping operation may cause seepage of water (and loss of fines) around the toe of the sheeting. Furthermore, if the pressure due to the upward seepage of water becomes greater than the soil pressure at the bottom of the excavation, it can result into a quick or “boiling” condition in the soil. Also, if the seepage of water around the toe of the sheeting significantly dislodges soil particles, the sheeting can be undermined.

9.3 Wellpoint System: Wellpoints are well screens, which require suitable filter material around the screen to prevent the collection of soil particles with the water. This system is generally used for dewatering to a depth of about 15 feet and this technique is best suited for medium to fine sand, for work of short duration.

Wellpoints are placed at 3 to 12 feet spacing, around the area to be excavated, and connected to a common header pipe, which is connected to a pump. The wellpoint system has to be kept in operation during shaft excavation also otherwise water table can return to its original level.

9.4 Deep Wells: Deep wells can be used to dewater pervious materials to whatever depth the excavation requires, and they can be installed outside the zone of excavation.

The deep well system consists of spacing 6 to 18 inches diameter wells at 20 to 200 feet spacing, depending on perviousness of ground and depth of dewatering required. These wells have a commercial type of water well screens surrounded with a properly graded sand-gravel filter. Each well is equipped with its own submersible pump. The excavation for the shaft can begin after water drawdown to the required elevation has been accomplished.

9.5 Freezing: In water-bearing ground where even minimal surface subsidence cannot be tolerated, such as adjacent to large buildings, the most reliable method of handling the excavation is to freeze the soil and then excavate it. There is no limitation on the depth upto which freezing may be used.

The procedure consists of sinking pipes around the area to be excavated and circulating a cold brine solution through the pipes, thereby freezing a wall of soil. As an alternative to brine refrigerant, liquid nitrogen is sometimes used to accelerate the freezing process. Excavation can then begin. If concrete of shaft lining is to be installed, the concrete can be placed against the frozen soil.

The main disadvantage of this technique is the time required to freeze the soil and the cost of the equipment.

10. Shaft excavation in Rock: Shaft excavation in rock is usually performed by the Drill and Blast method, like in tunnels. Shaft excavations for tunnels are normally less than 120 feet deep and, therefore, use of more sophisticated excavating equipment is not often economical. However, for shaft excavation for more than 120 feet depth, other methods may have to be explored. Prior to the start of rock excavation, it is advisable to grout and seal the overburden if ground water infiltration from the overburden can become a problem.

10.1 Raises: A raise is a vertical excavation proceeding from a lower elevation to a higher elevation, perhaps from one tunnel to another, or from a tunnel to the ground surface. The raise can be used to intersect another tunnel above an existing one. Shaft raising is sometimes used in urban tunnel construction to minimize surface disruption.

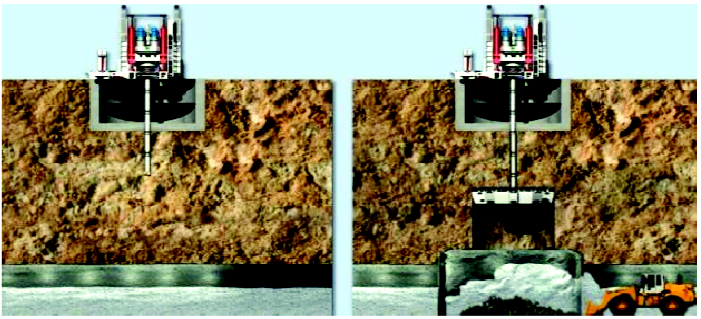

Fig. 19.07: Raise Boring

Raises are usually excavated by drilling pilot holes, usually of diameter about 9 to 12 inches, then reaming the hole to the proper diameter. The most common and successful system is to drill the pilot hole down and reaming up the required raise (Fig. 19.07). Only a few types of raise drills are widely used, and the nature of rock should be studied carefully to assure the use of proper cutters for efficiency and economy.

Raise excavation by drilling and blasting is often quite hazardous, especially the scaling down of loose rock after the blast.

10.2 Temporary Supports: In sedimentary, fractured, or blocky rock, the walls can be quite treacherous. The support can consist of steel ribs and liner plates, steel ribs with lagging, rock bolts with or without wire mesh, or shotcrete. The NATM support system is particularly adaptable for rock shafts that require temporary supports.

If rock is of reasonable good quality, it may be advantageous to merely install rock bolts into the wall and fasten wire mesh to the bolts to keep rock spalls from dropping on the workers. It is sometimes good practice to apply shotcrete to the walls. Shotcrete linings are also becoming popular as permanent linings.

11. Other Mechanized Methods: Methods are being developed to sink shafts by mechanical methods. Herrenknecht and Robbins have developed a shaft sinking machine that can sink all type of shafts. They are designed to be used in both stable and unstable grounds. They mainly consist of a sinking unit and a shaft boring machine (Fig. 19.08). Functioning like a gripper TBM, the machine is braced with gripper arms against the solid rock. The machine receives all supplies and is controlled from the surface and, with information being monitored, the operator can be constantly aware of the machine's technical parameters.

12. Lining of Shafts: Permanent shafts are usually lined. The planned permanent usage of the shaft determines the type of final lining. Shafts sunk through soft ground require an initial or "primary" lining for construction support. The secondary lining, if required, can either be poured against the primary lining, or it can be formed from both the outside and inside. If the lining is formed on the outside, the annular space between the primary and secondary lining should be tightly backfilled or packed with pea gravel, well graded sand or other suitable material.

Precast concrete segments can be used for secondary lining, where it can be placed from the bottom up, but the annular space created should be filled up.

If the shaft has been excavated through water-bearing ground, placing an impervious sheeting material on the face of primary lining, prior to the placement of the final concrete lining, has become a common practice. Additionally, grouting may be required to prevent water seepage into the finished shaft.

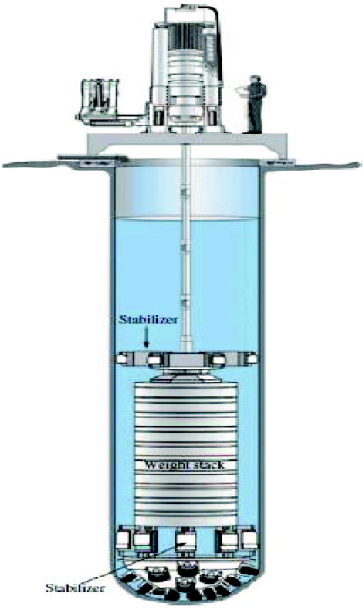

Fig. 19.08: Vertical Shaft Sinking Machine

Calculation of lining thickness is done according to structural loads and stresses to which the collar will be subjected.

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CHAPTER-20

INSPECTION AND MAINTENANCE OF TUNNELS

The philosophy of Inspections and Maintenance of tunnels remains more or less same in the tunnels, irrespective of the owning organization of the tunnel. But the frequency of the inspection, roles/responsibilities of officials involved and reporting system etc. vary from one organization to another. Therefore, it is difficult to lay down a common inspection and maintenance methodology in any text book. In this chapter, the methodology to be followed in Indian Railway, as per *"Guidelines for Civil Engineering Inspection, Maintenance and Safety in Existing Tunnels" (No. RDSO/GE/G-0015, August'2012),* have been covered.

1. Pre-requisites for Tunnel Inspection

1.1 Qualifications of Inspecting Officials: All officials performing tunnel inspection work should possess basic knowledge of tunnel components and understand how they function. The inspecting official(s) should have the ability to identify and evaluate defects that pose a threat to the integrity of a structural member and should be able to assess the degree of deterioration of structural components of tunnel. Training courses for building competency in tunnel inspections should be organized periodically.

1.2 Equipment/Tools for Inspection: Recommended list of equipment and tools to be used for tunnel inspections is as under:

- Tunnel Inspection Unit/Requisite staging mounted on the mobile units, such as open wagon, dip lorry or rail motor (Fig. 20.01): To carry out thorough inspection of the sides

and roof of the tunnels. These should be kept at suitable points for urgent use as required.

Fig. 20.01: Tunnel Inspection Unit

- Calipers: To measure steel plate thicknesses.
- Feeler gauge/Crack width ruler/Optical Crack width Microscope: For measuring crack width.
- Digital Camera (with Flash): To take photographs for documentation of the inspection.
- Markers: For making reference marks on tunnel surfaces.
- Chipping Hammer: To sound concrete
- Extension Cord: To get electricity to inspection area.
- Flashlights: Used in dark areas to improve visibility during inspection.
- Plumb Bob: To check plumbness of columns and wall faces.
- Pocket Knife: To examine loose material and other items.
- Scraper: To determine extent of corrosion and concrete deterioration.
- Screw Driver: To probe weep holes to check for clogs.
- Wire Brush or Brooms: To clean debris from surfaces to be inspected.

- Pocket Tapes: To measure dimensions of defects.
- 30m Tape (Non Metallic): To measure anything beyond the reach of pocket tapes and folding rules.
- Appropriate Safety gear.

Various non–destructive inspection methods such as ultrasonic, geo-radar and seismic testing may be used to supplement the regular inspection methods, if required.

1.3 Advance Preparation

(A) Study of Tunnel Records: To perform the inspection efficiently, it is important to plan and organize for inspection in advance. This involves study of available tunnel drawings, previous inspection reports, construction phase geological records and tunnel instrumentation data records (if available).

(B) Marking of Reference System: It is necessary to establish a system by which the location of a defect can be recorded and understood. This will allow the inspections to be referenced historically for future monitoring of condition of any defect and will increase the efficiency of the overall inspection process. In addition to locating a defect by its longitudinal position, it is necessary to note the defect's position within the tunnel cross-section. Horseshoe, rectangular and other circular tunnels can be broken down into consistently named cross-sectional elements. Nomenclature used in inspection of tunnel should be illustrated in initial page(s) of Tunnel Inspection Register.

(C) Tunnel Inspection Record: To properly gather and record tunnel inspection data, it is recommended to have following two inspection/ information registers for each tunnel:

(1) Tunnel Inspection Register: For recording of Tunnel Condition Codes & comments. Recommended Proforma is placed at Appendix-20.01 and Appendix-20.02

(2) Supplementary Tunnel Information Register: For detailed sketches and/or photographs of defects found in areas of the tunnel, it would be helpful to make sketches/take photographs of the same conditions or defects as previous inspections, so that the rate of deterioration can be ascertained. As far as possible, supplementary information should be recorded during the inspection itself. In cases (like photographs) where this is not possible, the date of recording Supplementary information should not be two weeks later than date of inspection). Recommended Proforma is placed at Appendix-20.03

(D) Inspection Methodology: Identification of structural defects during the inspection can be accomplished through visual inspection or through a combination of visual inspection and non-destructive techniques. In case any special non-destructive testing is required to be used for inspection, advance planning & preparation should be made for the same.

(E) Ensuring Safety: Inspection team should ensure that safety practices are followed at all times. Along with the safety of inspection personnel, the inspection team should take appropriate measures during inspection to prevent danger to the traffic, to staff and to members of the inspection team.

2. Common Structural Defects: Identification of structural defects during the inspection can be accomplished through visual inspection or through a combination of visual inspection and non-destructive techniques.

The visual inspection must be made on all exposed surfaces of the structural elements. All noted defects should be measured and documented for location. Severe spalls in the concrete surface should be measured in length, width and depth. Severe cracks should be measured in length and width. Corrosion on steel members should be measured for the length, width, and depth of the corrosion. The inspectors should clear away debris, efflorescence, corrosion or other foreign substances from the surfaces of the structural element prior to performing the inspection. Once the defect is noted, it should be classified as minor, moderate, or severe as explained in the following sections.

Particular attention should be paid to determining if differential settlement has occurred in transition areas of the tunnel. Transitions are those areas in which the tunnel support conditions change, such as between sections of rock and soil tunneling.

In addition to visual inspection, structural elements should be periodically sounded with hammers to identify defects hidden from the naked eye. As a result of a hammer strike on the surface, the structural element will produce a sound that indicates if a hidden defect exists. A high-pitched sound or a ringing sound from the blow indicates good material below the surface. Conversely, a dull thud or hollow sound indicates a defect exists below the surface. Such a defect in concrete may signify a delamination is present or that the concrete is loose and could spall off. Once the defect is found, the surface in the vicinity of the defect should be tapped until the extent of the affected area is determined.

For concrete or masonry surfaces that are accessible, non-destructive, ultrasonic test method such as “Impact-Echo” may be utilized. Impact-Echo is an acoustic method that can determine locations and extent of flaws/deteriorations, voids, debonding of reinforcement bars and thickness of concrete. The use of this method helps to mitigate the need for major

rehabilitation since the deterioration can be detected at an early stage and repairs performed. Common structural defects are briefly described below:

2.1 Concrete Structures

(1) Scaling: Scaling is the gradual and continuing loss of surface mortar and aggregate over an area. This is classified as follows:

- Minor Scale: Loss of surface mortar up to 6mm deep, with surface exposure of coarse aggregates.
- Moderate Scale: Loss of surface mortar from 6mm to 25 mm deep, with some added mortar loss between the coarse aggregates.
- Severe Scale: Loss of coarse aggregate particles as well as surface mortar and the mortar surrounding the aggregates. Depth of loss exceeds 25mm.

(2) Cracking: A crack is a linear fracture in the concrete caused by tensile forces exceeding the tensile strength of the concrete. Cracks can occur during curing (non-structural shrinkage cracks) or thereafter from external load (structural cracks). They may extend partially or completely through the concrete member. Cracks are categorized as Transverse Cracks, Longitudinal Cracks, Horizontal Cracks, Vertical cracks, Diagonal Cracks, Map Cracks, and Random Cracks etc. All cracks may be classified as follows:

- Minor: Up to 0.80mm.
- Moderate: Between 0.80mm and 3.20mm.
- Severe: Over 3.20 mm.

(3) Spalling: Spalling is a roughly circular or oval depression in the concrete. It is caused by the Separation and removal of a portion of the surface concrete revealing a fracture roughly parallel or

slightly inclined, to the surface. Spalling may be classified as follows:

- Minor: Less than 12mm deep or 75mm to 150mm in diameter.
- Moderate: 12mm to 25mm deep or approximately 150mm in diameter.
- Severe: More than 25mm deep and greater than 150mm in diameter and any spall in which reinforcing steel is exposed.

(4) Pop-outs: These are conical fragments that break out of the surface of the concrete leaving small holes. Generally, a shattered aggregate particle will be found at the bottom of the hole, with a part of the fragment still adhering to the small end of the pop-out cone. These are classified as under:

- Minor: Leaving holes upto 10mm in diameter or equivalent.
- Moderate: Leaving holes between 10mm and 50mm in diameter or equivalent.
- Severe: Leaving holes 50mm to 75mm in diameter or equivalent. Pop-outs larger than 75mm in diameter are spalls.

(5) Efflorescence: This is a combination of calcium carbonate leached out of the cement paste and other recrystallized carbonate and chloride compounds, which form on the concrete surface.

(6) Staining: Staining is a discoloration of the concrete surface caused by the passing of dissolved materials through cracks and deposited on the surface when the water emerges and evaporates. Staining can be of any colour although brown staining may signify the corrosion of underlying reinforcement steel.

(7) Hollow Area: This is an area of a concrete surface that produces a hollow sound when struck by a hammer. It is often referred to as delaminated concrete.

(8) Honeycomb: This is an area of a concrete surface that was not completely filled with concrete during the initial construction. The shape of the aggregate is visible giving the defect a honeycomb appearance.

(9) Leakage: This occurs on a region on the concrete surface where water is penetrating through the concrete.

- Minor: The concrete surface is wet although there are no drips.
- Moderate: Active flows at a volume less than 30 drips/minute.
- Severe: Active flows at a volume greater than 30 drips/minute.

2.2 Steel Structures

(1) Corrosion: Corroded steel varies in color from dark red to dark brown. Initially, corrosion is fine grained, but as it progresses, it becomes flaky or scaly in character. Eventually, corrosion causes pitting in the member. All locations, characteristics and extent of the corroded areas should be noted. The depth of severe pitting should be measured and the size of any perforation caused by corrosion should be recorded. Corrosion may be classified as follows:

- Minor: A light, loose corrosion formation pitting the paint surface.
- Moderate: A looser corrosion formation with scales or flakes forming. Definite areas of corrosion are discernible.
- Severe: A heavy, stratified corrosion or corrosion scale with pitting of the metal surface. This corrosion condition

eventually culminates in loss of steel section and generally occurs where there is water infiltration.

(2) Cracks: Cracks in the steel may vary from hair line thickness to sufficient width to transmit light through the member. Any type of crack is serious and should be reported at once. Look for cracks radiating from cuts, notches, and welds. All cracks in the steel will be classified as severe.

(3) Buckles and Kinks: Buckles and kinks develop mostly because of damage arising from thermal strain, overload, or added load conditions. Erection damage may also cause buckles and kinks.

(4) Leakage: This occurs on a region of the steel surface where water is penetrating through a joint or crack.

- Minor: The steel surface is wet although there are no drips.
- Moderate: Active flows at a volume less than 30 drips/minute.
- Severe: Active flows at a volume greater than 30 drips/ minute.

(5) Protection System: Steel is generally protected by a paint system or by galvanizing. Most existing structures use either paint or galvanized steel. Paint systems fail through peeling, cracking, corrosion pimples and excessive chalking. The classification of the degree of paint system deterioration is tied to both the physical condition of the paint and the amount of corrosion of the member as follows:

- Minor: General signs of deterioration of the paint system but no corrosion yet present.
- Moderate: Paint generally in poor condition and corrosion is present but not serious. No section loss.

- Severe: Paint system has failed and there is extensive corrosion and/or section loss.

2.3 Masonry Structures

(1) Masonry Units: The individual stones, bricks or blocks should be checked for displaced, cracked, broken, crushed or missing units. For some types of masonry, surface deterioration or weathering can also be a problem.

- Minor: Surface deterioration at isolated locations. Minor cracking.
- Moderate: Slight dislocation of masonry units; large areas of surface scaling.
- Severe: Individual masonry units significantly displaced or missing.

(2) Mortar: The condition of the mortar should be checked to ensure that it is still holding strongly. It is particularly important to note cracked, deteriorated or missing mortar if other deterioration is present such as missing or displaced masonry units.

- Minor: Shallow mortar deterioration at isolated locations.
- Moderate: Mortar generally deteriorated, loose or missing mortar at isolated locations; in?ltration staining apparent.
- Severe: Extensive areas of missing mortar; infiltration causing misalignment of tunnel.

(3) Shape: Masonry arches act primarily in compression. Flattened curvature, bulges in walls or other shape deformations may indicate unstable soil conditions.

(4) Leakage: A region on the masonry surface where water is penetrating through a joint or crack.

- Minor: The masonry surface is wet although there are no drips.
- Moderate: Active flows at a volume less than 30 drips/minute.
- Severe: Active flows at a volume greater than 30 drips/minute.

2.4 Connection Bolts: The connection bolts on fabricated components may be discolored due to moisture and humidity conditions in the tunnel. This condition does not down grade the structural capacity of the bolt. Particular attention should be given to bolts in regions of leakage to ensure that no detrimental loss of section has occurred. If losses in section are observed, such bolts should be noted for replacement. Also, the location of all missing or loose bolts should be noted.

- Minor: Bolts are discolored, but have no section loss.
- Moderate: Bolts are deteriorated with up to 15% loss.
- Severe: Bolts are deteriorated with greater than 15 % section loss. However, bolts with deterioration approaching 50% or more should be replaced.

2.5 Shotcrete/SFRS: Inspector should particularly look for any cracking, spalling, scaling and water seepage and rate it as mentioned earlier for concrete structures.

2.6 Rockbolts: Generally shotcreting is done after the work of rock bolting is complete and therefore the rock bolt ends are generally embedded in shotcrete. The protrusion of bolt end may only be visible. The surroundings of bolts should be closely watched for any signs of cracks, loosening of shotcrete, dislocation etc.

If any sign of looseness/cracks is observed, sounding of the location to be done for about 10m

on both side and suitable action to be taken. Any dampness observed around the rock bolt to be recorded.

A sample rock bolt selected in random in a tunnel at the rate of one rock bolt per 500 bolts or part thereof should be subjected to pull out test as a confirmatory test once in 10 years after provision of rock bolt.

3. Major Items to be inspected

3.1 Portal (including cuttings at approaches)

(a) Signs of instability of slopes.

(b) Possibility of boulder/loose mass fall.

(c) Possibility of tree fall.

(d) Drainage arrangements: Catch water drains, side drains and sumps. Catch water drains above the portals should drain away and not be allowed to percolate into the tunnel or behind the portal masonry.

(e) Weep holes: In retaining walls/breast walls and portal structure.

(f) Adequacy & condition of measures (like shotcreting, rock bolting etc.) taken for strengthening/ stabilization/erosion control of slopes/tunnel face.

(g) Structural condition of portals.

(h) Emergency access & communication.

3.2 Section of tunnel in relation to moving dimensions

(a) Visible signs of any convergence/deformation in tunnel supports.

(b) In case the conventional measurements indicate some movement/convergence, systematic convergence measurement using tape extensometer/optical methods should be done.

(c) Scrutiny of past convergence/deformation history and instrumentation data.

3.3 Tunnel Roof, Walls and Invert

(a) Signs of any structural distress in tunnel supports (concrete/RCC/Steel/Shotcrete/ SFRS/Rock bolts etc.).

(b) Signs of problem in formation (like disturbance in track alignment, levels).

(c) Seepage/dampness.

3.4 Tunnel Refuges (Man/Trolley) and Walkway

(a) No unwanted material, free of vegetation.

(b) Firm and level.

3.5 Drainage

(a) Side drains upto outfall: Cleanliness, No unwanted material& whether functioning satisfactorily.

(b) Weep holes: Check for clogging/choking.

(c) Functioning of dewatering pumps.

(d) Desilting of sumps.

3.6 Track: As per provisions of IRPWM. Detailed inspection needs to be carried out in locations where tunnel convergence/ deformation has been observed.

3.7 Ventilation, Lighting, Telephone Communication, Firefighting preparedness and Electrical/ Mechanical Systems

(a) Functioning of designed systems.

(b) Comments on adequacy of available systems.

General comments on functioning of mechanical, electrical and signaling systems should be recorded during inspection. Any issue requiring urgent attention should immediately be brought to notice of concerned department.

It should be ascertained whether Ventilation shafts/Adits are adequate and maintained free of vegetation and other growth. For tunnels more than 200m long, level of pollution and temperature condition should be enquired from the gang and Keyman working in that location of tunnel keeping in view passenger comfort and working conditions for working inside the tunnel.

4. Inspection Documentation

4.1 Recording of Defects: The inspection should be thoroughly and accurately documented. For the tunnel structure, the documentation of severe defects should include a sketch showing the location and size of the defect and a verbal description of the defect. All severe defects should be photographed. However, a representative photo of minor or moderate defects will be sufficient. All defects should be described but sketches need only to be made for severe defects.

The sketches should show the necessary plan and elevation views of the defects in structural element to which they pertain. All defects should be located on sketches by dimensioning their location in reference to the beginning or end of the element. Each defect should be dimensioned showing its length, width, and depth (if applicable).

In documenting the inspection, consistent abbreviation system (such as that given below) should be used in description, sketches and photographs to describe the defect and to classify them as minor, moderate or severe:

Description of Defect	Classification
Crack- CR	1 - Minor
Scaling - SC	2 - Moderate
Spall - SP	3 - Severe
Staining - ST	

Exposed Reinforcement - E

Corrosion - C

Honeycomb- H

Hollow Area - HA

Debris - D

Buckle- B

Efflorescence - EF

Leakage - LK

For example, a moderate crack should be labeled as CR2, a severe leakage as LK3 etc. This designation should be placed on the sketch/ photograph.

Before placing any information on Supplementary Tunnel Information Register, always ensure that the structural element (that the defects pertain to) is correctly identified.

4.2 Condition Rating System: The tunnel components mentioned in Proforma of Tunnel Inspection Register should be rated as below:

Excellent condition: No defects found.

Good condition: No repairs necessary. Isolated defects found

Fair condition: Minor repairs required but element is functioning as originally designed. Minor, moderate, and isolated severe defects are present but with no signi?cant section loss.

Poor condition: Major repairs are required and element is not functioning as originally designed. Severe defects are present.

Serious condition: Major repairs required immediately to keep structure open to traffic.

The rating is dependent upon the amount, type, size and location of defects as well as the extent to which

the element retains its original structural capacity. To judge the extent to which the structural element retains its original structural capacity, the inspector must be able to appreciate how the element is designed and how the defect affects this design.

Defects should be described as “Minor”, “Moderate” or “Severe”. “Moderate” or “Severe” defects should be sketched and/or their photographs should be placed in Supplementary Tunnel Information Register.

4.3 Repair Priority Classifications: When summarizing inspection data and making recommendations for future repairs (in “Tunnel Inspection Register” and “Tunnel Inspection Report”) it is recommended to mention priority of these repairs to be performed under following classification:

(A) Critical: The inspection may reveal severe defects that could pose danger to the traffic or to tunnel personnel. When this occurs, this particular severe defect should be categorized for a “critical repair”. A defect requires this designation if it requires “immediate” action including one of the following critical actions be taken:

- Close the tunnel/keep the defect under continuous watch (with appropriate speed restrictions, if necessary) until the severe defect is removed or repaired.
- Shore up the structural member if this is appropriate.

(B) Priority: Refers to conditions for which further investigations, design and implementation of interim or long-term repairs should be undertaken on a priority basis, i.e. taking precedence over all other scheduled work.

(C) Routine: Refers to conditions requiring further investigation or remedial work that can be undertaken as part of a scheduled maintenance program, other scheduled project or routine facility maintenance depending on the action required.

4.4 Specialized Testing Reports: If the inspection utilizes any specialized testing agencies and equipment, all such reports derived from these special testing shall become a part of the documentation of the particular inspection period.

4.5 Tunnel Inspection Report: Upon completion of the inspection, Tunnel Inspection Report should be developed that summarizes the ?ndings of inspection. This report should be submitted along with Tunnel Inspection Registers & Supplementary Tunnel Information Registers to higher official(s). The report will facilitate appreciation of items in the tunnel requiring urgent attention and will help the higher authorities in their decision making.

Suggested outline for the report along with a description of the contents to be included in each section is as under:

- **Report Number/Letter number: Identi?** cation number of report
- **Table of Contents:** Self-explanatory.
- **Executive Summary:** Provide a concise summary of the inspection, findings, and recommended repairs.
- **Major Inspection Findings:** Summarize Minor, Moderate & serious defects in the tunnels
- **Recommendations:** This section will include recommendations for repair/rehabilitation of the tunnel components that were found to have defects. The recommendations should classify the repair/ rehabilitation in to following categories:

- Critical
- Priority
- Routine.

5. Role / Duties of Inspecting Officials

5.1 Inspection by SSE / P.Way or SSE / Works

(i) As per Indian Railway Bridge Manual provisions *"Senior Section Engineer in-charge of tunnel shall inspect every tunnel on his section once a year during the prescribed month after the monsoon season but where specified by the Chief Engineer, the structural part shall be inspected by the SSE/Works".* But it is advisable that the inspection by SSE in-charge of tunnel or SSE/Works is carried out before monsoon, as is done in case of bridges.

(ii) Senior Section Engineer shall record the results of their inspection in prescribed tunnel inspection register.

(iii) Senior Section Engineer shall submit to the Assistant Divisional Engineer (ADEN) by the prescribed date a list of important defects with a certificate in duplicate to the effect.

"I certify that I have personally carried out tunnel inspection of my section in accordance with standing orders for the year ending..... and append herewith a list of important defects."

(iv) The ADEN shall issue such orders as deemed necessary to the Section Engineer and counter sign and forward one copy of the certificate of inspection to the Senior Divisional Engineer with remarks if any.

(v) The Section Engineer shall accompany the assistant Engineer on the latter's annual inspection of tunnels.

5.2 Inspection by ADEN

(i) As per Indian Railway Bridge Manual provisions *"the ADEN shall inspect every tunnel on the sub division once a year before the monsoon during the prescribed months and record the results in ink in the tunnel inspection registers".* But it is advisable that the inspection by ADEN is carried out after monsoon, as is done in case of bridges.

(ii) The tunnel, condition of which warrant special attention, should be inspected more frequently.

(iii) The instructions and index as required should be prefixed to each tunnel inspection register.

(iv) The inspection shall be detailed and cover all aspects, entries being made under each of the heads given in the register.

(v) The ADEN should make an extract of all remarks concerning repairs required, send these to SSE with explicit instructions and ensure expeditious compliance

(vi) On completion of his annual tunnel inspection the Assistant Engineer shall certify at the end of the register as follow:

"I certify that I have inspected all the tunnels shown in register during the year ending…..and have issued detailed orders in writing to the Inspectors concerned except the following on which the Divisional/Sr. Divisional Engineer's orders are solicited".

(vii) These registers should be in the Divisional/Sr. Divisional Engineer's office by specified date.

5.3 Inspection by Divisional/Sr. Divisional Engineer

(i) The Divisional/Sr. Divisional Engineers shall carefully scrutinize the ADEN's tunnel inspection register and inspect such tunnels

as called for his inspection. He shall record his orders regarding the points which require a decision by him and initial against every entry of tunnel in the registers in token of scrutiny. He should endorse on each register, below the assistant Engineer certificate, as follows:

"I have personally scrutinized this register and have issued orders regarding all essential points requiring a decision by me. The following points are submitted to the Territorial Head of the Department at Head Quarters for orders."

(ii) The Divisional/Sr. Divisional Engineer should extract the items of inspection register requiring attention and send it to the ADEN who should intimate the same to the Inspector concerned for expeditious compliance.

(iii) The register should be forwarded to the mentor Head of the Department at Headquarters who will examine each register, issue orders regarding matters referred to him, endorsing the registers to the effect and return them to the Divisional/Sr. Divisional Engineer. Subsequent action taken on the notes should be entered in the registers by the ADEN.

5.4 Special Inspection: Understanding and prediction of tunnel behavior is a complex subject. In case regular inspections and/or instrumentation records indicate a problem that may affect safety/ operation of traffic, special inspections are recommended to be undertaken with assistance of experts on relevant field(s).

It is recommended to get a detailed inspection of tunnel conducted by a tunnel expert at a frequency varying from 1 to 6 years (depending on geology/tunnel condition).

Also in case of special unexpected events like railway accident, earthquake etc. railway officials should undertake special inspection of tunnel(s).

6. Maintenance and Repair of Tunnels: Maintenance and repairs activities to be carried out for various components of tunnel are as under:

6.1 Portal and Approach Cuttings

(A) Surface Drainage: Maintenance operations carried out on surface drains usually fall into one or a combination of the following:

(i) Weed control within surface drains.

(ii) Removal of debris from other track maintenance activities.

(iii) Removal of sediment.

(iv) Re-grading (if required).

(B) Catch water Drain

(i) Catch water drain should be pucca with adequate slope to ensure development of self-cleaning velocity.

(ii) Catch water drain should not have any weep hole.

(iii) The expansion joints should be sealed.

(iv) Catch water drains should have well designed out fall with protection against tail-end erosion

(C) Removal of loose boulders etc.

(i) Loose boulders/loose mass in the portal area should be removed by loose scaling or by controlled blasting.

(ii) Where necessary, rock fall and debris diversion or containment features should be constructed to positively ensure that no rocks, rock-slides or other debris such as soil from mountain slopes above

the portal area can reach the tracks or damage equipment or structures.

(iii) Option of constructing false tunnel /cut and cover in tunnel approach should be explored, if required.

(iv) Weak/leaning trees/branches that may fall on/near the track should be cut.

(D) Structural Repairs

(i) Structural repairs including strengthening (as per requirements) in portal structure.

(ii) Structural repairs of supports (like shotcreting, rock bolting etc.) used for strengthening/ stabilization/erosion control of slopes/tunnel face. Further strengthening of support measures to be taken up as per requirements

(E) Access Road: Road access to portals (if available) should be fit for desired use particularly for evacuating passengers and for staff/material access.

(F) Weep Holes: Cleaning of clogged/choked weep holes.

6.2 Inside Tunnel

- Scaling/Sounding of tunnel
- Removal of unwanted material
- Cleaning of side drains and sumps
- Clearing walkways and refuges
- Cleaning of clogged/choked weep holes
- Structural maintenance and repairs (including strengthening)
- Track Maintenance (with special attention to problematic areas)

- Scheduled maintenance of mechanical and electrical systems (including ventilation & lighting - by respective departments)
- Scheduled maintenance of instruments
- Scheduled maintenance of safety, fire and `communication equipment and signage

6.3 Systematic Approach for repairs: Factors affecting the repair methods are the deterioration severity and the structural impact of the defect. The cause of the defect should be determined before remedial works are undertaken, otherwise the same problem may recur. It is recommended to adopt a systematic approach to tunnel repairs involving following steps:

- Determine the cause(s) of damage
- Evaluate the extent of damage
- Evaluate the need to repair
- Select the repair method
- Development and approval of detailed repair methodology
- Execution of repairs

Appendix – 20.01

Tunnel Inspection Register

General Information (last updated on .....)

1. Tunnel No.: ..... 2. Year of Construction: .....

3. Gauge/Type: ..... 4. Section: .....

5. Chainage: ..... 6. Between Stations: .....

7. Length: ..... 8. Line/Road: .....

9. Curve/Straight: ..... 10. Electrified: Yes/No

11. CWR/LWR/SWR/FPJ 12. Tunnel Profile: .....

13. Previous History of Tunnel:
Year of Construction: ..... Drawing No. .....
Date of repair: ..... Drawing No. .....

14. Year wise summery at .....

Year	Details

	UP		DN	
	On straight	On curve	On straight	On curve
15. Minimum height of roof above Rail Level				
16. Minimum distance from center line of track				

17. Tunnel Support Information:

	Chainage		Length	Support Description
	From	to		
i				
ii				
iii				
iv				
	Total			
Note: Reference drawing Number---------- is available at ---------- office				

18. Gradient

	Chainage		Length	Gradient(V:H, R/F)
	From	To		
i				
ii				
iii				
iv				
	Total			

19. Curvature

	Chainage		Length	Curve (Degree, LH/RH)
	From	To		
i				
ii				
iii				
iv				
	Total			

20. Track Structure

	Chainage		Length	Track Structure
	From	To		
i				
ii				
iii				
iv				
	Total			

21. Geotechnical Information

22. Special arrangement

(a) Lighting:	(b) Ventilation:
(c) Drainage:	(d) Shaft/Adits:
(e) Instrumentation:	(f) Other:

Appendix – 20.02

Tunnel Inspection Register

Inspection Form

Date of Inspection: ..... Name of Inspecting Official: .....

S. No.	Description	Remarks of Inspecting Official	Condition Rating	Reference to supplementary Information, if any
(A)	Portal			
	1. Portal 1 (including Tunnel approach)			
	2. Portal 2 (including Tunnel approach)			
(B)	Section of tunnel in relation to moving dimensions			
(C)	Tunnel Walls & Invert			
(D)	Tunnel Roof			
(E)	Tunnel Refuges (Man/Trolley)			
(F)	Drainage			
(G)	Track			

(H) Ventilation	NR
(I) Lighting	NR
(J) Walkway/Escapeway	NR
(K) Telephonic Communication	NR
(L) Firefighting Preparedness	NR
(M) Electrical/Mechanical Systems	NR
N) Electrical/Mechanical Systems	NR

Details of any major repairs/attention since previous inspection, if any

Details of any untoward occurrence since previous inspection, if any

Details of any untoward occurrence since previous inspection, if any

Signature of Inspecting Official

Details of Action taken/Compliance (with names, signature & date of official making the entry)

Appendix - 20.03

Supplementary Tunnel Information Register

1. Tunnel No.: ..... 2. Section: ..... 3. Gauge/Type: .....

4. Line/Road: ..... 5. Chainage: ..... 6. Length: .....

7. Date of Inspection: ..... 8. Name of Inspecting Official: .....

9. Nature of Supplementary Information (Tick the applicable type of information) Sketch/ Photograph/ Detailed Description

10. Reference to supplementary information (Provide Report no., File no. etc. in case the information is not included in/attached with Supplementary Tunnel Information Register)

Description of Supplementary Information

Signature of Inspecting Official

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CHAPTER-21

METRO TUNNELS

Rapid Transit or Mass Rapid Transit (also known by names Heavy Rail, Metro, MRT, Subway, Tube, U-Bahn or Underground) is a type of high-capacity public transport system generally constructed in urban areas. Unlike buses or trams, rapid transit systems are electric railways that operate on an exclusive right-of-way, which cannot be accessed by pedestrians or other vehicles of any sort and it can be on grade separated alignment (elevated alignment), underground alignment or subways (in tunnels) or on surface (like conventional Railway system).

1. Metro Tunnels Excavation: Tunnel Boring Machines (TBMs) with features purpose-built to the specific ground conditions are now the preferred mode for bored tunnelling in urban areas. The high capital cost is justified by the length of tunnel more than 2 km *(Sharma, 1998, Ref.: 32)*. The TBMs offer following advantages over the drill & Blast method in the metro tunnels:

- Explosives are not used and, therefore, produces much lower vibrations in built-up areas.
- Little or no over break.
- Fast excavation, thus saving money.
- Less labour cost.
- Reduces surface settlement to very low levels resulting in assured safety to the existing super structures.
- Reduces risk of life of workers by (i) Rock falls at face or behind the TBM, (ii) Explosives, (iii) Hit by vehicles; and (iv) Electrocution.

The type of TBM to be used is decided based on the nature of ground to be excavated (i.e. hard rock or soft soil), as discussed in Chapter-10. Now-a-days, dual

mode shield TBMs have been developed to bore through all types of soils, boulders and weak rocks (in non-squeezing grounds) under high groundwater table. During tunnelling, the groundwater table is lowered to the bottom of the tunnel by drilling drainage holes to keep the ground dry. In dual mode shield TBMs, both scrapper picks and as well as disc cutters are mounted on the cutter head. Otherwise, these machines are quite similar to EPBMs in dimensions and working principles, though progress per day is bit less than the EPBMs (about 6m per day vis-a-vis 10m per day as reported in Delhi Metro).

While working with dual mode shield TBM in Delhi Metro, initially it was found that a large number of scrappers and buckets were getting detached from the cutter head. This was probably because of presence of too many boulders in the soil strata. As a result, the big boulders were entangled in the large space between the arms and thereby knocking off the scrapper and buckets. The protective plates and deflector strips were added around the buckets to avoid direct impact of boulders on the buckets, in addition to other modifications. Thereafter, the dual TBMs succeeded *(Singh M., Ref.:33).*

The advantage of fully shielded TBM with segment erector is that there is no unsupported ground behind the shield. That is why TBMs have failed in poor grounds yet dual TBM has succeeded in soils, boulders and weak rock mass in non-squeezing ground conditions. There are not much of successful experiences worldwide about TBM tunnelling through squeezing grounds. Therefore, TBMs are not recommended in the squeezing ground and flowing conditions.

It is necessary to inject foam along with water at the cutter head which has the following advantages:

- Reduced permeability and enhanced sealing at the tunnel face.
- Suppresses dust in rock tunnelling.

- Excavation of wet soil or weathered rock is easier.
- Soil does not stick to the cutters.

2. Pre-cast Lining: Pre-fabricated lining is mostly suitable in underground metros, for various soils, boulders and rock conditions except squeezing grounds (due to the high overburden pressure) and flowing grounds within water charged wide shear zones (due to seepage erosion or piping failure). But generally these conditions do not occur in suburban tunnels, being relatively shallow depth tunnels. In some projects, fiber reinforced precast concrete linings have been adopted.

TBM is capable of placing lining segments in position all around the circular tunnel with the help of segment erector. Segment bolts are then tightened by impact wrenches twice. The curved alignment is achieved with the help of tapering of the lining rings. All the rings are tapered and curvature is obtained by suitably adjusting the orientation of rings. Before taking inside the tunnel, the segments are checked on ground for any cracks/ damage. As water tightness is extremely important for the durability of the tunnel lining, a double gasket system comprising a durable elastomeric gasket and a water sealing made from the hydrophobic material is used. These gaskets are located in grooves cast into the edges of the precast concrete segments. Together with the high precision casting of the segments achieved by precision steel molds, gaskets will ensure the durable and watertight tunnels. Hydrophobic seals expand 250 percent once it comes into contact with water.

Thought should be given to fire-resistant design of concrete lining. Extra thickness of concrete cover (about 75mm) should be provided over the steel reinforcement. Under-reinforced concrete segments may be used to ensure the failure in ductile phase, if it occurs.

Grouting is carried out simultaneously with the tunnelling. There are in-built ports in the tail skin of the TBM. These are used in primary grouting of annulus

(void between excavation profile and outer face of the precast ring). Grouting is continued upto 3 bars (0.3 MPa) pressure. Excavation is not commenced until the previous lining is completed. Secondary grouting is also done within 14 days of ring erection. Every third ring is grouted to pressure of 3 bar (0.3 MPa). Secondary grouting will fill up any void left during the primary grout due to its shrinkage.

3. Building Condition Survey and Vibration Limit: Open trenches and shafts are excavated by drilling and blasting method for connection to the underground metro system. The controlled bench blasting method is used in open excavation, under busy and congested roads which are flanked by old or heavy buildings and monuments. Before designing the controlled blasting, the entire rock mass is explored thoroughly. The trial blasts are detonated to determine the safe-scaled-distance, according to the nature of structures.

The next step is to assess the condition of buildings standing near the blast site to determine how much vibration can be sustained by these structures, especially old buildings and ancient monuments if any. Table 21.02 shows the permitted Peak Particle Velocities (PPV) as per German standards. It may be noted that ISRM recommends almost twice PPV values.

Archaeologists suggest that no surface metro station should be built within protected 100m periphery of a protected (heritage) monument. In such cases, an underground metro station may be a better choice.

4. Impact on the Structures: The blasting works may affect the surrounding structures slightly in spite of the controlled blasting. In worst case, small cracks may develop in RCC and masonry. The air over-pressure may also create cracks in glass works of doors and windows in nearby areas. Appendix-21.1 summarizes various types of damages to the structures. Substantial compensation may have to be paid to the owners of the buildings or structures damaged, according to the specified class of damage.

Table 21.02: Permitted PPV on Structures

S. No.	Condition of Structure	Max. PPV (mm/s)
1	Most structures in “good condition”	25
2	Most structures in “fair condition”	12
3	Most structures in “poor condition”	5
4	Water supply structures	5
5	Heritage structures/Bridge structures	5

The traffic is stopped during blasting time for a few minutes and all the roads, other exits/entries to the blasting site are closed for safety reasons. Flying of rock pieces during an urban blasting may have severe consequences.

5. Subsidence: Subsidence of ground and differential settlement of nearby structures takes place due to underground tunnelling. The dewatering due to excavation causes more widespread subsidence, primarily due to the settlement of overlying loose deposit of soil, silt or clay. In totally rocky areas, the subsidence is very small and does not cause any worry. Following instruments are recommended for precision monitoring of structures:

- Precise leveling points,
- Tilt meters,
- Crack gauges embedded in the nearby structures, and
- Vibration monitoring of old/ancient structures.

In case the actual settlement is expected to go beyond the predicted subsidence, the whole construction methodology must be reviewed. Appendix-21.1 may be used which specifies the maximum tensile strain caused by subsidence (= increment in spacing of

columns divided by the distance between columns, expressed in percentage).

Table 21.03: Preliminary Design of cut Slopes (for Height of cut less than 10m)

S. No.	Type of soil/rock protection work	Stable cut slope without any breast wall or minor protection work	Stable cut slope with breast wall
1	Soil or mixed with boulders (a) Disturbed vegetation (b) Disturbed vegetation overlaid on firm rock	1V:1H Vertical for rock portion and 1V:1H for soil portion	N:1 * Vertical for rock portion and 1V:1H for soil portion
2	Same as above but with dense vegetation, medium rock and shales	1V:0.5H	5V:1H
3	Hard rock, shale, or harder rocks with inward dip	1V:0.25H to 1V:0.10H and vertical or overhanging	Breast wall is not needed
4	Same as above but with outward dip or badly fractured rock/shale	At dip angle or 1V:0.5H or dip of intersection of joint planes	5V:1H
5	Conglomerates/very soft shale/sand rock which erode easily	Vertical cut to reduce erosion	5V:1H

* N is 5 for H< 3m; 4 for H= 3 - 4m and 3 for H= 4 - 6m

6. Portal and Cut Slopes: It is better to locate the portals deeper into the ground or mountain where rock cover of at least equal to width of tunnel is available. The slope of portal should be stable, otherwise the same should be reinforced properly with the rock anchors.

Alternatively a thick breast wall (1m) of concrete should be constructed to ensure stability of portals *(Singh & Goel, 2002 – Ref.:34).*

The side slopes of open trenches should be stable. Deoja et al. *(Ref.: 35)* have suggested the dip of safe cut slopes with and without protective measures for both rocks and soils (Table-21.03) for all types of tunnels, including metro tunnels.

Building Damage Classification

(Burland et al., 1977 and Boscardin & Cording, 1989)

Risk Category	Description of degree of damage	Description of typical damage and likely form of repair for typical masonry buildings	Approx. Crack Width (mm)	Max. Tensile Strain (%) due to subsidence
0	Negligible	Hairline cracks	-	Less than 0.5
1	Very Slight	Fine cracks easily treated during normal redecorations. Perhaps isolated slight fracture in building. Cracks in brickwork visible upon close inspection.	0.1 to 1	0.05 to 0.075
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible, some repointing may be required for weather tightness. Doors and windows may stick slightly.	1 to 5	0.075 to 0.15
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tack pointing and possibly replacement of a small amount of brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Water tightness often impaired.	5 to 15 or number of cracks greater than three	0.15 to 0.3
4	Severe	Extensive repair involving removal of sections of walls, especially over doors and windows. Windows and door frames distorted. Floor slopes noticeable. Walls lean or bulge noticeably, some loss of bearing in beams. Utility services disrupted.	15 to 25 but also depends on number of cracks	Greater than 0.3
5	Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly	Usually greater than 25 but	-

Risk Category	Description of degree of damage	Description of typical damage and likely form of repair for typical masonry buildings	Approx. Crack Width (mm)	Max. Tensile Strain (%) due to subsidence
		and require shoring. Windows broken by distortion. Danger of instability.	depends on number of cracks	

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CHAPTER-22

MISCELLANEOUS

Some topics, which are relevant to tunnels, but have not been covered so far, are elaborated in this chapter.

1. Ballastless Track in Tunnels: Most of the advance railways in the world specify laying of Long Welded Rails (LWR) in tunnels. Holding the track structure in place, with LWR, is very competitive in terms of construction and maintenance practices are well understood. But this technique requires:

- Considerable overall track height.
- Monitoring of track geometry.
- Maintenance in line with actual traffic density.

The alternative to ballasted track is ballastless track (BLT) in which ballast is replaced by foundation layers made of cement concrete (and asphalt in some cases). This type of track must provide same functions as traditional tracks by superimposing several layers of decreasing stiffness. Technological development and research in Railway track construction has led to introduction of BLT usually referred as slab track (Fig. 22.01). This system was first introduced in Japan in late 1960s. Both conventional ballasted track as well as ballastless track have their advantages and disadvantages as discussed below:

1.1 Ballasted Track Structure:

Advantages

- Known and proven method; suitable for high speed also.
- Relatively low construction cost.
- High elasticity.
- High maintainability (particularly with mechanized maintenance) at relatively low cost.

- Good noise & vibration absorption.
- Less sensitive to construction defects.
- Availability of mechanized construction technology.

Fig. 22.01: Ballastless Track in Tunnels

Disadvantages

- Over the time, the track tends to “float”, in both longitudinal and lateral directions, as a result of non-linear, irreversible behaviour of the materials. This translates into increased requirement for track maintenance.
- Limited non-compensated lateral acceleration occurs in curves, which is due to the limited lateral resistance offered by the ballast.
- At high speeds ballast can be churned up, causing damage to rails and wheels.
- Over the time, ballast bed becomes less permeable and less elastic due to contamination, breakage of the ballast and transfer of fine particles from the subgrade.
- Depth of construction of ballasted track is

relatively high, which has implications for tunnel size

1.2 Ballastless Track Structure:

Advantages

- Less frequent maintenance efforts to maintain track geometry.
- Relatively higher construction cost, but lower life cycle cost.
- Excellent riding comfort even at speed greater than 250 kmph.
- Unlike ballasted tracks where the track tends to "float" over the time, in both longitudinal and lateral directions as a result of non-linear irreversible behavior of the materials, this is not the case in ballastless tracks.
- High lateral resistance of track structure which allows future increase in speeds in combination with tilting coach technology.
- Relatively low noise and vibration nuisance.
- High impermeability.
- Lesser dead weight.
- Depth of construction is relatively less as compared to ballasted track, which reduces tunnel construction cost.
- The track can be accessible to road vehicles.
- Less environment pollution.
- Easy and economical maintainability - ease of replacing parts with minimum dislocation to traffic.
- Electrical insulation for facilitating track circuiting control.

Disadvantages

- Relatively higher initial construction cost.
- Less sensitive to construction defects.
- Difficult to repair & restore traffic in case of

accident resulting in damage to ballastless track.

- Need for providing transitions.

Considering the reduced maintenance requirement, reduced life cycle cost and enhanced safety, Ballastless track should, by and large, be provided for all new tunnels.

1.3 Ballastless Track Systems: Many types of ballastless track systems are currently in use around the world. This includes slab track system, embedded rail system, RHEDA system, Twin-block system etc. Selection of appropriate system should be made after consideration of various techno-economic factors including contingency plan for repairs & restoration. For details about these systems, literature on the subject *"Railway Track" may be referred e.g. Modern Railway Track (Second Edition) – By: Coenraad Esveld – Publisher: MRT Productions, The Netherlands, Chapter-9 & Track Compendium – By: Dr. Bernhard Lichtberger – Publisher: Eurail Press, Chapter-10*

2. Cut and Cover Tunnelling: This is a common and well-proven technique for constructing tunnels/ underground structures at shallow depths. The method can accommodate changes in tunnel width and non-uniform shapes and is very commonly used in construction of underground stations.

In cut-and-cover method, the structure is built inside an excavation and covered over with backfill material when construction of the structure is complete. Cut-and-cover construction is used when the tunnel profile is shallow and the excavation from the surface is possible, economical, and acceptable. Cut and cover construction is used for underpasses, the approach sections to mined tunnels and for tunnels in flat terrain or where it is advantageous to construct the tunnel at a shallow depth.

Several overlapping works are required to be carried out in using this method. Trench excavation, underground structure construction and soil covering of excavated area are three major integral parts of the tunnelling method. Most of these works are similar to other road construction except that the excavation levels involved are deeper. Bulk excavation is often undertaken under a road deck to minimize traffic disruption as well as environmental impacts in terms of dust and noise emissions and visual impact.

For depths of about 10m to 12m, cut-and-cover is usually more economical and more practical than mined or bored tunneling. The cut-and-cover tunnel is usually designed as a rigid frame box structure. In urban areas, due to the limited available space, the tunnel is usually constructed within a neat excavation line using braced or tied back excavation supporting walls. Wherever construction space permits, in open areas beyond urban development, it may be more economical to employ open cut construction.

Where the tunnel alignment is beneath a city street, the cut-and-cover construction will cause interference with traffic and other urban activities. This disruption can be lessened through the use of decking over the excavation to restore traffic. While most cut-and-cover tunnels have a relatively shallow depth to the invert, depths to 18m are not uncommon; depths rarely exceed 30m.

Disadvantages of this method are:

- More dust and noise impact may arise, though these can be mitigated through implementation of sufficient control measures.
- Temporary decks are often installed before bulk excavation to minimize the associated environment impacts.
- Larger quantity of cut and disposal materials would be generated from the excavation works, requiring proper handling and disposal.

Two types of construction methods are employed to build cut and cover tunnels; bottom-up and top-down. Part-A in Fig. 22.02 illustrates “Bottom-Up Construction” where the final structure is independent of the support of excavation walls. Part-B in Fig. 22.02 illustrates “Top-Down Construction” where the tunnel roof and ceiling are structural parts of the support of excavation walls. These construction types are described in more detail below.

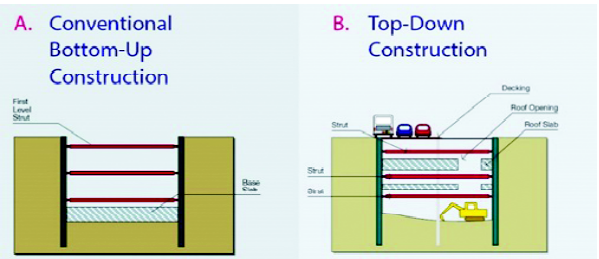


Fig. 22.02: Cut and Cover Tunnelling Methods

2.1 Bottom-Up Construction: In the conventional “bottom-up” construction, a trench is excavated from the surface within which the tunnel is constructed and then the trench is backfilled and the surface restored afterward. The trench can be formed using open cut (sides sloped back and unsupported), or with vertical faces using an excavation support system. In this method, the tunnel is completed before it is covered up and the surface reinstated.

Various stages involved are as under:

(i) The underground retaining wall is installed before excavation commences. The retaining wall can be a concrete diaphragm wall, a concrete bored pile wall or a steel sheet pile wall; depending on the site condition, soil type and the excavation depth (Fig. 22.03).

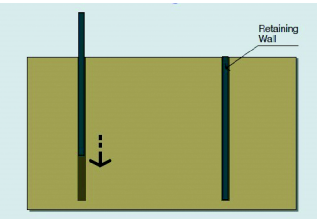


Fig. 22.03: Installation of retaining wall

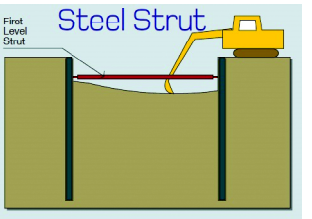


Fig. 22.04: Excavation & Installation of Steel Strut

(ii) The soil is excavated to the first strut level. The first level strut is installed before the excavation proceeds further (Fig. 22.04).

(iii) The soil is excavated to the next strut level and the second level strut is installed. This continues till the excavation reaches the final depth or formation level (Fig. 22.05). The numbers of strut levels depend on the excavation depth.

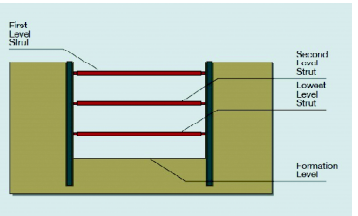


Fig. 22.05: Excavation & Installation of Steel Struts

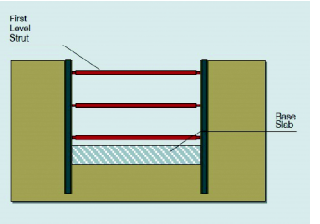


Fig. 22.06: Construction of Underground structure

(iv) At the formation level, the reinforced concrete slab or base slab is constructed, followed by the removal of lowest level strut and the side walls are constructed (Fig. 22.06).

(v) The next level of slab is constructed, followed by the removal of the strut near to that slab level. This process progresses upwards till the roof slab is constructed (Fig. 22.07).

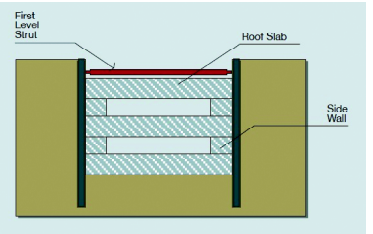


Fig. 22.07: Construction of Underground structure

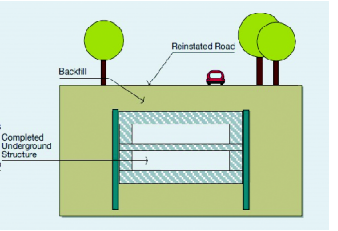


Fig. 22.08: Back filling and Reinstatement

(vi) After the roof slab is completed, the soil is backfilled to the first strut level before the first level strut is removed. This is followed by completely backfilling the top of the underground structure (Fig. 22.08). If the retaining wall is a diaphragm wall or a bored pile wall, the top 2m of the wall will be removed. If it is a sheet pile wall, the sheet pile will be extracted.

Bottom-up construction offers several advantages:

- It is a conventional construction method, well understood by contractors.
- Waterproofing can be applied to the outside surface of the structure.
- The inside of the excavation is easily accessible for the construction equipment and the delivery, storage and placement of materials.
- Drainage systems can be installed outside the structure to divert water away from the structure.

Disadvantages of bottom-up construction include:

- Somewhat larger footprint required for construction than for top-down construction.
- The ground surface cannot be restored to its final condition until construction is complete.

- Requires temporary support or relocation of utilities.
- May require dewatering that could have adverse effects on surrounding infrastructure.

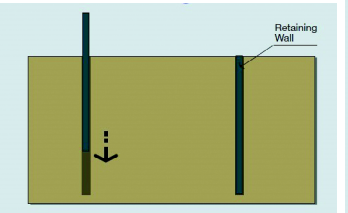
2.2 Top-Down Construction: In this system, the vertical walls are constructed first, usually using slurry walls, although secant pile walls are also used. In this method the support of excavation is often the final structural tunnel walls. Secondary finishing walls are provided upon completion of the construction. Next the roof is constructed and tied into the support of excavation walls. The surface is then reinstated before the completion of the construction. The remainder of the excavation is completed under the protection of the top slab. Upon the completion of the excavation, the floor is completed and tied into the walls. The tunnel finishes are installed within the completed structure. For wider tunnels, temporary or permanent piles or wall elements are sometimes installed along the center of the proposed tunnel to reduce the span of the roof and floors of the tunnel.

Various stages involved are as under:

(i) The underground retaining wall is usually a concrete diaphragm wall, which is installed before excavation commences (Fig. 22.09).

(ii) The soil is excavated to just below the roof slab level of the underground structure. Struts are installed to support the retaining wall, which in turn supports the soil at sides (Fig. 22.10).

(iii) The roof slab is constructed, with access openings provided on the slab for works to proceed downwards (Fig. 22.11). The roof slab not only provides a massive support across the excavation, it also acts as a noise barrier.



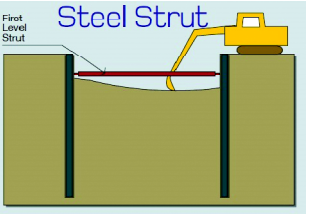
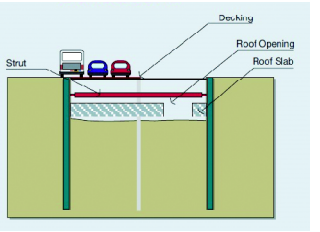


Fig. 22.09: Installation of Retaining Wall

Fig. 22.10: Excavation & Installation of Steel Strut

(iv) The next level of slab is constructed, and this process progresses downwards till the base slab is completed (Fig. 22.12).



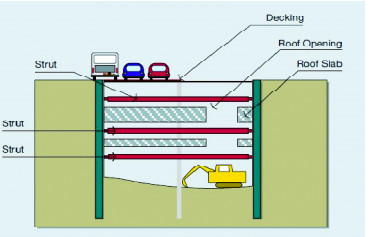


Fig. 22.11: Construction of Underground Structure

Fig.22.12: Construction of Underground Structure

(v) The side walls are constructed upwards, followed by removal of the intermediate struts. The access opening on the roof slab are then sealed (Fig. 22.13).

(vi) After the underground structure is completed, the soil is backfilled to the top strut level before the strut is removed. This is followed by completely backfilling the top of the underground structure and finally reinstating the surface areas (Fig. 22.14).

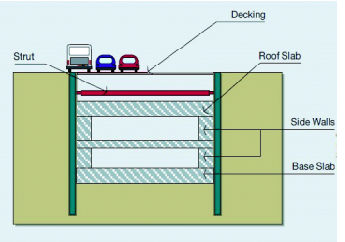


Fig. 22.13: Construction of Underground Structure

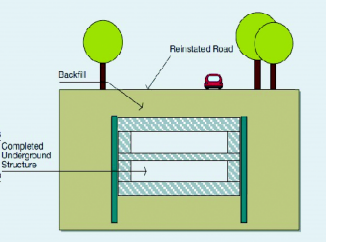


Fig. 22.14: Backfilling & Reinstatement

Top-down construction offers several advantages in comparison to bottom-up construction:

- It allows early restoration of the ground surface above the tunnel. The temporary support of excavation walls are used as the permanent structural walls.
- The structural slabs will act as internal bracing for the support of excavation thus reducing the amount of tie backs required. It requires somewhat less width for the construction area.
- Easier construction of roof since it can be cast on prepared grade rather than using bottom forms.
- It may result in lower cost for the tunnel by the elimination of the separate, cast-in-place concrete walls within the excavation and reducing the need for tie backs and internal bracing.
- It may result in shorter construction duration by overlapping construction activities.

Disadvantages of top-down construction include:

- Inability to install external waterproofing outside the tunnel walls.
- More complicated connections for the roof, floor and base slabs.

- Potential water leakage at the joints between the slabs and the walls.
- Risks that the exterior walls (or center columns) will exceed specified installation tolerances and extend within the neat line of the interior space.
- Access to the excavation is limited to the portals or through shafts through the roof.
- Limited spaces for excavation and construction of the bottom slab.

It is difficult to generalize the use of a particular construction method since each project is unique and has any number of constraints and variables that should be evaluated when selecting a construction method. The following summary presents conditions that may make one construction method more attractive than the other. This summary should be used in conjunction with a careful evaluation of all factors associated with a project to make a final determination of the construction method to be used.

Conditions favourable to Bottom-Up Construction:

- No right of way restrictions.
- No requirement to limit sidewall deflections.
- No requirement for permanent restoration of surface.

Conditions favourable to Top-Down Construction:

- Limited width of right-of-way.
- Sidewall deflections must be limited to protect adjacent features.
- Surface must be restored to permanent usable condition as soon as possible.

3. Micro-tunnelling: Micro-tunnelling is a name for remotely-operated, small-diameter tunnelling. It is typically achieved by jacking pipes of concrete or other

suitable materials, behind a tunnelling machine, from a launch shaft into a reception shaft. MTBMs are available in the 30-114 inches Outer Diameter range. Micro-tunneling operations are managed by an operator in an above-ground control container. Cutter heads can be customized for specific ground conditions (Fig. 22.15). In slurry shield machines, soil excavation takes place by way of infusing the soil with slurry at the face of the bore and cuttings are forced into slurry inlet holes in the MTBM's crushing cone for circulation to and from a separation plant through a closed system. The launch shaft is outfitted with a pit seal to prevent shaft flooding and a project specific thrust block to distribute jacking force.

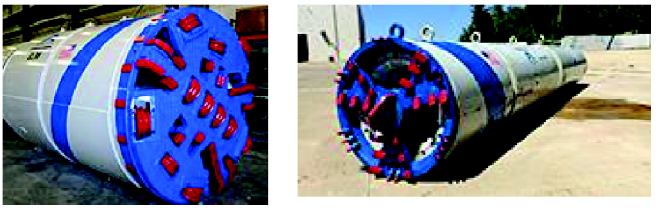


Fig. 22.15: MTBM Cutter Heads

MTBMs feature high pressure jetting nozzles, an articulated steering joint with three-point steering control and hydraulically activated dirt wings to minimize MTBM roll. Additional MTBM attributes include a live, one-way audio system & camera allowing for system monitoring during operation, gas detector and submersible pump.

A full installation system may typically include following components:

- Shield machine
- Control cabin
- Hydraulic jacking rig and the power pack to operate it
- Guidance System, including laser

- Keyhole jacking frames and series of pumps
- Slurry system with pumps and separation plant
- There may also be pipe lubrication equipment

3.1 Guidance System: The Micro-tunneling Guidance System includes an active target and three inclinometers that read and transmit data to the control console in the control container. Two inclinometers in the target and one at the rear of the MTBM track roll and incline to ensure accuracy. Another inclinometer, mounted in the front of the MTBM, assesses the incline. The proprietary software programs display the current and projected cutter head location, MTBM incline and roll to the operator's control console.

The Total Guidance System (TGS) provided in recent MTBMs is a monitoring system for extended lengths and alignments with curves (Fig. 22.16). It comprises of individual, self-leveling, station units that maintain a surveyed connection throughout the alignment without the need for continuous manual surveying. The target is axially mounted behind the machine's articulation cylinders at line and grade, and registers the position and angles of incidence of the red laser emitted from the guidance system. The combined stations communicate a continuous electronic distance measurement for the operator to monitor exact machine X and Y positioning, real-time cutter head location and horizontal and vertical deviation with the proprietary software program. This state-of-the-art guidance system can be utilized as a standalone guidance system for any tunneling, pipe jacking or micro-tunneling application, regardless of equipment manufacturer.

Fig. 22.16: Total Guidance System

Fig. 22.17: Control Container

3.2 Control Container: Control Container houses the operator's control console, motor control centers for the slurry pumps, MTBM drive motor and bulkhead panel for electrical and communication connections (Fig. 22.17). The operator monitors and controls all facets of the micro-tunneling operation from a dual-monitor display console and control station including: MTBM's pitch and yaw, rotation, torque, jetting, jacking thrust, steering, slurry flow and pressure, as well as the MTBMs anticipated position at the cutter face using proprietary software programs.

3.3 Remote Hydraulic Power Pack: The Remote Hydraulic Power Pack is a power distribution center for the jacking frame and micro-tunneling auxiliary functions (Fig. 22.18).

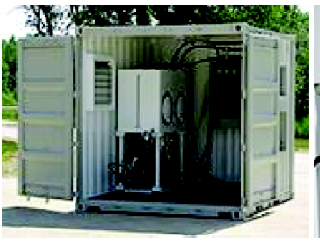


Fig. 22.18: Remote Hydraulic Power Pack

Fig. 22.19: Main Drive Power container

3.4 Main Drive Power Container: The Main Drive Power Container offers increased voltage to run the periphery drive and face access MTBM's main drive (Fig. 22.19).

3.5 Procedure of Micro-tunneling: The procedure used in micro-tunnelling, with slurry shield machine, will typically comprise of following steps:

(i) The MTBM and jacking frame are set up in a shaft at the required depth.

(ii) The MTBM is pushed into the earth by hydraulic jacks mounted and aligned in the jacking shaft.

(iii) The jacks are then retracted and the slurry lines and control cables are disconnected.

(iv) A product pipe or casing is lowered into the shaft and inserted between the jacking frame and the MTBM or previously jacked pipe.

(v) Slurry lines and power and control cable connections are made, and the pipe and MTBM are advanced for another drive stroke.

(vi) This process is repeated until the MTBM reaches the reception shaft.

(vii) Upon drive completion, the MTBM and trailing equipment are retrieved and all equipment removed from the pipeline.

3.6 Utility in Railway: When any pipe line crossing has to be made under the track (in major yards or under very high embankments), executing the work by conventional methods of using relieving girders or using conventional box pushing technique is either not possible due to site conditions or it may be very time consuming. In such situations, using the MTBM (as trenchless technology) is highly advantageous. In Indian Railways, MTBM has been used successfully recently in Bhubaneshwar Yard for constructing a pipe culvert crossing of more than 100m length

across the whole yard to address the problem of drainage during rainy season, in Igatpuri Yard for constructing a pipe culvert crossing of more than 90m length across the whole yard, to address the problem of drainage during rainy season and construction of pipe culverts below very high banks in lieu of damaged existing minor bridges in Igatpuri – Kasara section of Central Railway.

4. Provision of OHE: Planned location of Over Head Equipment (OHE) fixtures to be fitted in tunnel walls/ roof, in electrified territory, in plan and cross-section should be obtained from electrical department and incorporated in construction drawings. Self-anchored bolts of High Tensile steel/Stainless steel should preferably be used for fixing arrangements. It should be ensured that bolts do not puncture the waterproofing layer (if provided). Planning for fixation of OHE fixtures shall be kept in consideration while designing tunnel lining.

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