Guidelines for 
Design & Construction of Tunnels

रिपोर्ट सं. RDSO/2012/GE:G-0017
June 2012

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Preface

Tunneling projects have always remained a challenge for engineers owing to the unpredictable behavior of the ground strata. In spite of carrying out elaborate geotechnical investigations to evolve the design of a tunnel and to decide the appropriate tunneling methodology, unforeseen situations are often encountered during execution.

Following factors in particular differentiate Tunnels from above ground structures:
1. Unpredictability/variability of Ground- the principal construction material.
2. The magnitude of the loads is basically unknown and as such, has to be assumed considering various geological/geophysical investigations etc.
3. Impossibility of calculating a reliable factor of safety of tunnel construction due to unpredictability/variability of both the construction material and the loads.

As such Tunneling is a highly specialized field requiring a blend of technical knowledge, field experience and engineering judgment.

Indian Railways (IR) is presently undertaking some highly challenging construction projects in hilly terrains involving construction of tunnels under difficult conditions. Presently there are no Indian Railways guidelines available on Design and Construction of tunnels. Railway Board vide its letter dated 21.12.2011 advised RDSO to prepare Manual/Guidelines on design, construction & maintenance of Tunnels. Considering the importance of the subject, Geotechnical Engineering Directorate took up formulation of these guidelines on top priority. The provisions of this guideline are based on experience gained during field visits & extensive literature survey. For further details a list of various code/manual/books/web links are given in reference section.

Draft Guidelines on “Design and Construction of Tunnels” were circulated to Zonal Railways for their comments / suggestions and final guidelines are now being issued after obtaining Railway Board (Railway Board letter no: 2011/CE-I/Tunnel, dated 26.07.2012).
The guidelines cover the key components of a tunneling project including Geotechnical investigation, Design Philosophy, Construction methodology. It is expected that these guidelines will enable engineers on Indian Railways to develop a basic appreciation for complexity of Tunneling project and help them in supervising tunneling projects being executed by Professional agencies.

Following Officials of Geotechnical Engineering Directorate of RDSO were associated with the preparation of the Guidelines.

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Inputs from Zonal Railways in form of feedback/comments, field checklists, sample proformas, approved cross sections and designs would help in further improving the guidelines.

(Shirish Kesarwani)
Executive Director/Geotechnical Engineering
Chapter 1: Geo-Technical Investigations

1.0 General:

Successful planning, design and construction of a tunneling project requires various types of investigative techniques to obtain a broad spectrum of pertinent topographic, geologic, subsurface, geo-hydrological, and structure information and data.

A geotechnical investigation program for a tunnel project must use appropriate means and methods to obtain necessary characteristics and properties as basis for planning, design and construction of the tunnel and related underground facilities, to identify the potential construction risks, and to establish realistic cost estimate and schedule.

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

2.0 Phases of Geo-technical Investigation Program

It is more efficient to perform geotechnical investigations in phases to focus the efforts in the areas and depths that matter. Due to factors like high cost, lengthy duration and limited access, it is recommended that investigations be carried out in several phases to obtain the information necessary at each stage of the project in a more cost-efficient manner.

The Preliminary investigations for planning and feasibility studies can be confined to information studies (involving Collection, organization & study of available data) and preliminary reconnaissance. Carrying out of Geological mapping and minimum subsurface investigations at this stage would help in comparing alternative alignments and for arriving at a conceptual preliminary design.

More detailed geotechnical investigation need to be carried out during Preconstruction Planning and Engineering Phase to refine the tunnel alignment and profile once the general corridor is selected, and to provide the detailed information needed for design. 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.

Geo-technical Investigations are also required to be conducted during Construction phase for various purposes including confirmation of design assumptions, obtaining forewarning of unfavorable tunneling conditions etc.
Different phases of Geo-technical investigations are briefly described below:

2.1 Preliminary Geo-technical investigations for Feasibility Studies:

In this phase, the emphasis is on defining the regional geology and the basic issues of design and construction.

This phase of Geo-technical investigations is conducted in two stages:

a) Collection, organization & study of available data
b) Preliminary survey

These stages are briefly described below:-

a) Collection, organization & study of available data:

Preliminary Geo-technical investigations for a tunnel project starts with collection and review of available information to develop an overall understanding of the site conditions and constraints at little cost.

Some possible sources of available information are:

(1) Topographical maps of Survey of India
(2) Geological maps & reports of GSI
(3) Geological maps and reports from agencies other than GSI
(4) Geo-technical investigation reports from other agencies
(5) Case histories of other underground works in the region
(6) Details of land ownership (Government, Private, Forest), Access routes, environmental sensitivity from State Govt./Local bodies
(7) Satellite images and aerial photographs from public or private sources

Existing data can help 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.

Published topographical, hydrological, geological, geotechnical, environmental, and other information should be collected, organized and evaluated.

In areas where seismic condition may govern or influence the project, historical seismic records are used to assess earthquake hazards.

Records of landslides caused by earthquakes, 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 as above should be followed by a preliminary survey for concept development and preliminary design. 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 & 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 & 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.

Carrying out of following investigations at this stage is recommended to help in comparing alternative alignments and for arriving at a conceptual preliminary design:
- **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.

- **Selected exploratory borings in critical locations**

- **Geophysical explorations if appropriate**: 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.

- **Aerial photography**: to supplement existing data.

- **Hydrological survey**: to define the groundwater 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 and 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.

2.2 Detailed Geo-technical investigations for Preconstruction Planning and Engineering

This phase involves:

a) Conducting of investigations to finalize / refine the tunnel alignment and profile once the alternative alignments within the general corridor have been selected during Feasibility Study stage.

b) To provide the detailed information needed for design of tunnel & selection of appropriate tunneling methodology.

**Investigations carried out during this phase are:-**

i) Topographical Surveys

ii) Subsurface Investigations

iii) Detailed Geological Mapping

iv) Detailed Hydrological survey & Groundwater investigation

iv) Structure & utility preconstruction survey

These are briefly described below:-

i) **Topographical Surveys:**

Detailed topographic maps, plans and profiles are developed to establish primary control for final design and construction based on a high order horizontal and vertical control field survey.

The tunnel centerline should be finalized & incorporated into the contract drawings of the tunnel contract, and all tunnel control should be based on this centerline. During construction, survey work is necessary for transfer of line and grade from surface to tunnel control points.

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)
ii) **Subsurface Investigations**: It is important to remember that unlike other Railway structures, the ground surrounding a tunnel can act as a supporting mechanism, or loading mechanism, or both, depending on the nature of the ground, the tunnel size, and the method and sequence of constructing the tunnel.

Thus, for tunnel designers and contractors, the rock or soil surrounding a tunnel is a construction material, just as important as the concrete and steel used on the job.

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 and rock types)
- Determining soil and rock material properties and mass characteristics
- Identify geological anomalies, fault zones and other hazards (squeezing soils, methane gas, etc.)
- Defining hydro geological conditions (groundwater levels, aquifers, hydrostatic pressures, etc.) and
- Identifying potential construction risks (boulders, etc.).

Subsurface investigations typically consist of:

- Borings,  
- In situ testing,  
- Geophysical investigations, and  
- Laboratory material testing.

Each of these investigation techniques is briefly discussed below:

- **Borings**: to identify the subsurface stratigraphy, and to obtain disturbed and undisturbed samples for visual classification and laboratory testing.

  Vertical and slightly inclined test borings and soil/rock sampling are key elements of any subsurface investigations for tunneling projects. The 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.

  Table below presents general guidelines from AASHTO (1988) for determining the spacing of boreholes for tunnel projects:
Guidelines for Vertical/Inclined Borehole Spacing

<table>
<thead>
<tr>
<th>Ground Conditions</th>
<th>Typical Borehole Spacing (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cut-Cover Tunnels</td>
<td>100 to 300</td>
</tr>
<tr>
<td><strong>Rock Tunneling</strong></td>
<td></td>
</tr>
<tr>
<td>Adverse Conditions</td>
<td>50 to 200</td>
</tr>
<tr>
<td>Favorable Conditions</td>
<td>500 to 1000</td>
</tr>
<tr>
<td><strong>Soft Ground Tunneling</strong></td>
<td></td>
</tr>
<tr>
<td>Adverse Conditions</td>
<td>50 to 100</td>
</tr>
<tr>
<td>Favorable Conditions</td>
<td>300 to 500</td>
</tr>
<tr>
<td><strong>Mixed Face Tunneling</strong></td>
<td></td>
</tr>
<tr>
<td>Adverse Conditions</td>
<td>25 to 50</td>
</tr>
<tr>
<td>Favorable Conditions</td>
<td>50 to 75</td>
</tr>
</tbody>
</table>

The above guideline can be used as a starting point for determining the number and locations of borings. However, especially 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 will need to be applied by experienced geotechnical professionals to adapt the investigation program.

In general, borings should be extended to at least 1.5 tunnel diameters 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.

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 boring 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. Once the bore hole reached a horizontal alignment, coring can be obtained using HQ triple tube core barrels.

All borings should be properly sealed at the completion of the field exploration, if not intended to be used as monitoring wells.

- **In situ tests**: to obtain useful engineering and index properties by testing the material in place to avoid the disturbance inevitably caused by sampling, transportation and handling of samples retrieved from boreholes; in situ tests can also aid in defining stratigraphy.
In situ tests are used to directly obtain field measurements of useful soil and rock engineering properties. In soil, in situ testing include both index type tests, such as the Standard Penetration Test (SPT) and tests that determine the physical properties of the ground, such as shear strength from cone penetration Tests (CPT) or Vane Shear Tests (VST) and ground deformation properties from pressure meter tests (PMT). The parameters to be tested would depend on the nature of underground strata viz. Rock or Soil.

It is recommended that parameters to be tested 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.

- **Geophysical tests:** to quickly and economically obtain subsurface information (stratigraphy and general engineering characteristics) over a large area to help define stratigraphy and to identify appropriate locations for performing borings.

Geophysical tests are indirect methods of exploration in which changes in certain physical characteristics such as magnetism, density, electrical resistivity, elasticity, or a combination of these are used as an aid in developing subsurface information.

Geophysical methods provide an expeditious and economical means of supplementing information obtained by direct exploratory methods, such as borings, test pits and in situ testing; identifying local anomalies that might not be identified by other methods of exploration; and defining strata boundaries between widely spaced borings for more realistic prediction of subsurface profiles.

Typical uses of geophysical tests include determination of the top of bedrock, the depth to groundwater, the presence of voids, the location and depth of utilities, the location and depth of existing foundations, and the location and depth of other obstruction etc.

In addition, geophysical testing can also obtain stiffness and dynamic properties which are required for numerical analysis. Geophysical testing can be performed on the surface, in boreholes (down or cross hole), or in front of the TBM during construction.

It is recommended that Geophysical method to be used should be finalized in advance in consultation with design engineer.

Typical applications for geophysical tests are presented in Table below:
Applications for Geophysical Testing Methods (after AASHTO, 1988)

<table>
<thead>
<tr>
<th>Geological Conditions to be Investigated</th>
<th>Useful Geophysical Techniques</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SURFACE</td>
</tr>
<tr>
<td>Stratified rock and soil units (depth and thickness of layers)</td>
<td>Seismic Refraction</td>
</tr>
<tr>
<td>Depth to Bedrock</td>
<td>Seismic Refraction Electrical Resistivity Ground Penetrating Radar</td>
</tr>
<tr>
<td>Depth to Groundwater Table</td>
<td>Seismic Refraction Electrical Resistivity Ground Penetrating Radar</td>
</tr>
<tr>
<td>Location of Highly Fractured Rock and/or Fault Zone</td>
<td>Electrical Resistivity</td>
</tr>
<tr>
<td>Solution Cavities</td>
<td>Electrical Resistivity Ground Penetrating Radar Gravity</td>
</tr>
</tbody>
</table>

It is important to note that 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.

- **Laboratory testing:** provides a wide variety of engineering properties and index properties from representative soil samples and rock core retrieved from the borings.

Detailed laboratory testing is required to obtain accurate information for design and modeling purposes.

**Soil Testing:** Detailed soil laboratory testing is required to obtain accurate information including classification, characteristics, stiffness, strength, etc. for design and modeling purposes. Testings are performed on selected representative samples (disturbed and undisturbed) in accordance with relevant standards.

Table below shows common soil laboratory testing for tunnel design purposes:
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil identification</td>
<td>• Particle size distribution</td>
</tr>
<tr>
<td></td>
<td>• Atterberg limits $w_i$, $w_p$</td>
</tr>
<tr>
<td></td>
<td>• Moisture Content</td>
</tr>
<tr>
<td></td>
<td>• Unit weights $\gamma_d$, $\gamma_z$</td>
</tr>
<tr>
<td></td>
<td>• Permeability $k$.</td>
</tr>
<tr>
<td></td>
<td>• Core recovery</td>
</tr>
<tr>
<td>Mechanical properties</td>
<td>• Unconfined compressive strength</td>
</tr>
<tr>
<td></td>
<td>• Triaxial compressive strength for determination of Friction angle $\Phi_u$, $\Phi$ and Cohesion $c_u$, $c$.</td>
</tr>
<tr>
<td></td>
<td>• Consolidation Test for determination of Compressibility $m_v$, $c_v$</td>
</tr>
<tr>
<td>Mechanical properties determined by</td>
<td>• Shear strength $\zeta_u$ (Vane-test).</td>
</tr>
<tr>
<td>field testing</td>
<td>• Penetration $N$ (Standard Penetration Test).</td>
</tr>
<tr>
<td></td>
<td>• Deformability $E$ (Plate bearing, Dilatometer)</td>
</tr>
</tbody>
</table>

**Rock Testing:** Standard rock testing evaluates physical properties of the rock including density and mineralogy. The mechanical properties of the intact rock core include uniaxial compressive strength, tensile strength, static and dynamic elastic constants, hardness, and abrasive indices.

Table below summarizes common rock laboratory testing for tunnel design purposes:

**Common Laboratory Tests for Rock (after USACE 1997)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Index Properties</td>
<td>• Density</td>
</tr>
<tr>
<td></td>
<td>• Porosity</td>
</tr>
<tr>
<td></td>
<td>• Moisture Content</td>
</tr>
<tr>
<td></td>
<td>• Slake Durability</td>
</tr>
<tr>
<td></td>
<td>• Swelling Index</td>
</tr>
<tr>
<td></td>
<td>• Point Load Index</td>
</tr>
<tr>
<td></td>
<td>• Hardness</td>
</tr>
<tr>
<td></td>
<td>• Abrasivity</td>
</tr>
<tr>
<td>Strength</td>
<td>• Uni-axial compressive strength</td>
</tr>
<tr>
<td></td>
<td>• Tri-axial compressive strength</td>
</tr>
<tr>
<td></td>
<td>• Tensile strength (Brazilian)</td>
</tr>
<tr>
<td></td>
<td>• Shear strength of joints</td>
</tr>
<tr>
<td>Deformability</td>
<td>• Young’s modulus</td>
</tr>
<tr>
<td></td>
<td>• Poisson’s ratio</td>
</tr>
</tbody>
</table>
It is desirable to preserve the rock cores retrieved from the field properly for years until the construction is completed and disputes/claims are settled. Common practice is to photograph the rock cores in core boxes and possibly scan the core samples for review by designers and contractors.

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 below shows special considerations for various tunneling methods.

**Special investigation needs related to construction methods**

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

<table>
<thead>
<tr>
<th>Construction Method</th>
<th>Special Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard or Abrasive Rock</td>
<td>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.</td>
</tr>
<tr>
<td>Mixed Face</td>
<td>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.</td>
</tr>
<tr>
<td>Karst</td>
<td>Potentially large cavities along joints.</td>
</tr>
<tr>
<td>High stress</td>
<td>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.</td>
</tr>
</tbody>
</table>
| 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. |

These Geo-technical investigations provide factual information about the distribution and engineering characteristics of soil, rock and groundwater at a site, allowing an understanding of the existing conditions sufficient for developing an economical design, determining a reliable construction cost estimate, and reducing the risks of construction.

### iii) Detailed Geological Mapping

A detailed geologic mapping effort should be carried out which includes mapping & plotting of Joints, faults, and bedding planes. The geologist must then project the geologic conditions to the elevation of the proposed tunnel so that tunneling conditions can be assessed.
Geologic mapping collects detailed geologic data systematically, and is used to characterize and document the condition of rock mass such as:

- Discontinuity type
- Discontinuity orientation
- Discontinuity infilling
- Discontinuity spacing
- Discontinuity persistence
- Weathering

By interpreting and extrapolating all these data, the geologist should have a better understanding of the rock conditions likely to be present along the proposed tunnel and at the proposed portal and shaft excavations. In addition, the 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

Based on detailed geological mapping, it should be possible to divide the tunnel alignment into zones of consistent rock mass condition. 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.

The mapping data will also help in targeting subsurface investigation borings and in situ testing in areas of observed variability and anomalies.
iv) Detailed Hydrological survey & Groundwater investigation

Hydrogeological work to be carried out at this stage includes measurement of groundwater levels or pressures in boreholes, permeability tests using packers in boreholes, and sometimes pumping tests.

Groundwater is a critical factor for tunnels since it may not only represent a large percentage of the loading on the final tunnel lining, but also it largely determines ground behavior and stability for soft ground tunnels; the inflow into rock tunnels; the method and equipment selected for tunnel construction; and the long-term performance of the completed structure.

Accordingly, for tunnel projects, 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 (rising, falling and constant head tests; packer tests, etc.)
- Geophysical testing
- Pumping tests

During subsurface investigation drilling and coring, it is particularly important for the inspector 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.
v) Structure & utility preconstruction survey

Structures located within the zone of potential influence may experience a certain amount of vertical and lateral movement as a result of 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 may 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, rerouting, strengthening, reconstruction and/or temporary support, and may also require monitoring during construction.

2.3 Geo-technical Investigations During Construction phase

For tunneling projects it is generally essential to perform additional subsurface investigations and ground characterization during construction. Such construction phase investigations serve a number of important functions like:

- To verify initial ground support selection & for design/redesign.
- 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 likely 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: typically consists of drilling horizontally from the tunnel heading by percussion drilling or rotary drilling methods.

Pilot Tunnels (and shorter exploratory adits): are small size tunnels (typically at least 2m by 2 m in size) that are occasionally used for large size tunnels in complex geological conditions.

Pilot tunnels, when used, are typically performed in a separate contract in advance of the main tunnel contract to provide prospective bidders a clearer understanding of the ground conditions that will be encountered. Consideration can also be given to locating the pilot tunnel adjacent to the proposed tunnel, using the pilot tunnel for emergency exit, tunnel drainage, tunnel ventilation, or other purposes for the completed project.

3.0 Assessing Exploration requirements of a Tunneling Project:

3.1 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.

3.2 The complexity and size of an underground structure has a bearing on the required intensity of explorations. A long tunnel of small diameter does not warrant the expense of detailed explorations, and a tunneling method able to cope with a variety of conditions is required. On the other hand, a large underground structure such as an underground railway station is more difficult to construct and warrants detailed analyses that include closely spaced borings, reliable design data, and occasionally a pilot tunnel.

3.3 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.

3.4 In mountainous terrain, it is often difficult or very expensive to gain access to the ground surface above the tunnel alignment for exploratory drilling. Many tunnels have been driven with borehole data available only at the portals. In such instances, maximum use must be made of remote sensing and surface geologic mapping, with geologic extrapolations to tunnel depth. The strategy may also include long horizontal borings drilled from the portals or probe holes drilled from the face of the advancing tunnel.

3.5 The intensity of explorations can be measured in several meaningful ways:

- Cost of full geotechnical exploration program (borings, testing, geophysics) as percentage of construction cost.
- Typical spacing of boreholes.
- Number of meters of borehole drilled for each 100 m of tunnel.

3.6 The required intensity of explorations will vary with factors like complexity of geology, project environment, depth of tunnel, accessibility for explorations, and cost of individual borings etc.

3.7 A practical guide for assessing the suitability of an exploration program is shown in Table below. The guide starts with a relatively simple base case and employs factors up or down from there. The base case considered is a 6 m drainage tunnel through moderately complex geology in a suburban area at a moderate depth of about 30 m.

<p>| Table: Guidelines for Assessing Exploration Needs for Tunnels and in Rock (US Army Manual EM-1110-2-2901) |
|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|
| Cost of Boring and Testing as % of Construction cost | Borehole Spacing | Borehole Length per 100 m of Tunnel |
| Base case | 0.4-0.8 | 150-300 m | 15-25 m |
| Extreme range | 0.3-10 | 15-1,000 m | 5-1,000 m |
| For conditions noted, multiply base case by the following factors: |
| Simple geology | 0.5 | 2-2.5 | 0.5 |
| Complex geology | 2-3 | 0.3-0.5 | 2-3 |
| Rural | 0.5 | 2-2.5 | 0.5 |
| Dense urban | 2-4 | 0.3-0.4 | 2-5 |
| Deep tunnel | 0.8-1 | Increase borehole spacing in proportion to depth of tunnel |</p>
<table>
<thead>
<tr>
<th>Poor surface access</th>
<th>0.5-1.5</th>
<th>5-10+</th>
<th>Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shafts and portals</td>
<td>NA</td>
<td>At least one each</td>
<td>NA</td>
</tr>
<tr>
<td>Special problems</td>
<td>1.5-2</td>
<td>0.2-0.5 locally</td>
<td></td>
</tr>
</tbody>
</table>

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.

3.8 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.

According to experience from USA (R.A. Robinson, M.A. Kucker, J.P. Gildner), over 55% of claims relate to unforeseen ground conditions and they decrease with increasing exploration, as shown in Table below:

<table>
<thead>
<tr>
<th>Meter of exploration boring per meter of tunnel alignment</th>
<th>Claims relative to bid price</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>30-40%</td>
</tr>
<tr>
<td>1</td>
<td>&lt;20%</td>
</tr>
<tr>
<td>1.5</td>
<td>&lt;10%</td>
</tr>
</tbody>
</table>

3.9 U.S National committee on Tunneling Technology (USNC/TT) recommended in year 1995 the following level of effort for geotechnical site exploration:

1. Site exploration budgets should average 3.0% of the estimated project cost.
2. Boring footage should average 1.5 linear ft of borehole per route ft of tunnel.

3.10 While above tables can be referred to for guidance purpose, the extent of an exploration program should be based on specific project requirements and complexity, rather than strict budget limits.

However, for most tunnels, especially tunnels in mountainous areas, the cost for a comprehensive subsurface investigation may be prohibitive. The challenge to geotechnical professionals is to develop an adequate and diligent subsurface investigation program that can improve the predictability of ground conditions within a reasonable budget and acceptable level of risk.

4.0 **An effective & economical Geotechnical investigation Program should include:**

a) Planning of a comprehensive Geotechnical investigation Program in
active consultation with experienced geotechnical engineers, geologists & designers.

b) “What”, “Why”, “Where”, “How” & “How much” for each Geotechnical parameter to be tested/investigated should be included in the Geotechnical investigation Program.

c) Phasing the investigation, as discussed earlier to better match and limit the scope of the investigation to the specific needs for each phase of the project, and Utilizing existing information and the results of geologic mapping and geophysical testing to more effectively select locations for investigation. Emphasis can be placed first on defining the local geology, and then on increasingly greater detailed characterization of the subsurface conditions and predicted ground behavior.

d) Keeping the investigation program flexible enough so as to enable taking up of additional investigations as per unexpected requirements that emerge during course of work.
Chapter 2: Choice of Tunnel system, Alignment & Shape/Size of cross-section

1.0 An adequate geological-geotechnical exploration and a thorough description of the ground in the early planning stages is important for deciding:

a) Choice of tunnel system
b) Choice of the alignment
c) The shape of the cross-section

a) **Choice of tunnel system**

The tunnel system comprises all underground works that are necessary to achieve the planned use and ensure the safety of persons and material assets. Besides the main tunnel tube(s), the tunnel system may comprise, e.g. 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.

b) **Choice of alignment**

The vertical and horizontal alignment of the tube(s) also depends on several factors such as:

- **Maximum ruling gradient:** Ruling grade permissible in tunnels should be smaller than in the open air, owing to:
  - Reduced rail-wheel adhesion in tunnels mainly due to presence of moisture in tunnels: This causes decrease in the traction force in tunnels.
  - Increased air resistance in tunnels: The magnitude of air resistance is known to depend on the relative velocities of wind and train, greater resistance upwind than downwind, as wells as on the relative cross-section areas of tunnel and train. Resistance is especially large in single track tunnels which are comparatively narrow.

If possible, maximum grades in straight tunnel should not exceed 75% of the ruling gradient of the line. Grades in curved tunnels should be compensated for curvature in the same manner as for open sections (i.e. outside the tunnel).

- Permitted maximum degree of curve
- The drainage considerations during construction and operation
- The accessibility and natural hazards in the portal areas
- 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. This is particularly true for long tunnels.

c) Shape & dimensions of the cross section

The shape and the dimensions of the cross section of tunnel are determined essentially by

i. The serviceability requirements associated with the use of the tunnel

ii. The geological-geotechnical conditions and

iii. Construction aspects

i. The serviceability requirements associated with the use of the tunnel

The required dimensional/clearance profile is a key factor in the determination of the cross section of the tunnel. Railway Schedule of Dimensions is to be followed to ensure required clearances.

Other serviceability criterions relevant for making choice of cross section are:

- Additional space requirements for operating and safety equipment (cable installations, signaling systems, signage, lighting, ventilation, etc.)
- Aerodynamic requirements
- Drainage requirements
- Maintenance requirements
- Requirements arising from the safety and rescue concept (escape routes within the tunnel, availability of the facilities in emergencies)

ii. The geological-geotechnical conditions

The shape and the size of the cross section depend also on the ground conditions, as the latter determine the extent of the 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 must be avoided by means of additional excavation to account for ground deformations and corresponding support measures.
iii. **Construction aspects**

- Economic considerations and the 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.

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

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.

2.0 **The main shapes used for railway tunnels are:**

- **D Shape:** Economical (as its profile suits the requirements of SOD), Used for hard rocks.

![D Shape Tunnel](image1)

- **Horseshoe cross section:** The horse-shoe shape has a semi-circular roof together with arched sides and curved invert. This cross-section offers a good resistance to external ground pressure and is suitable for soft rocks.

![Horseshoe Cross Section](image2)

- **Elliptical Shape:** Suited for water bearing soils or soft grounds, economical choice (due to its profile) compared to Circular cross section.

![Elliptical Shape](image3)
Circular cross section: This shape is 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. The circular section is often uneconomical for railways as more filling will be required for obtaining flat base. It is best suited for tunnels driven by the TBM/shield method.

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 3: Tunnel excavation methods

1.0 Selection of a particular tunnel excavation depends largely on:

- Type of ground
- Size of tunnel
- Availability of resources (machinery/equipment, funds & time)

Commonly used Tunnel excavation methods can be grouped under the following categories:

A. Excavation Methods for Rock Tunnels
B. Excavation Methods for Soft Ground Tunnels

These are discussed below:

A. **Excavation Methods for Rock Tunnels:**

The 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) TBM (Full Face)

These methods are generally used separately but may also be used in combination.

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

This excavation method has been used since a long time & still remains the conventional method for noncircular cross sections and also for short length circular tunnels. This excavation method involves the cycle of drilling, loading, blasting, scaling & mucking).

A key decision to be taken by engineers is whether to go for “Full Face Excavation” or “Partial Face Excavation”.

**Full Face Excavation & Partial Face Excavation:**

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

**Partial Face Excavation:** Large openings or openings in weak rocks are less stable than small ones. 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.

The various types of partial face excavations are:

a) **Heading and Benching.** This method involves excavation of top heading first. Excavation of the bench is done only after securing the top heading. With the stand-up time problem eliminated (unless there will be a problem with wall stability), longer increments of bench can be excavated.

![Diagram of heading and benching](image.png)

1. Top Heading (also called calotte) 2. Benching

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

![Diagram of ramp and wall plates](image.png)

Installation of rock reinforcement in a top heading presents 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 taken. 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 bench 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 having to establish, undercut, and reestablish support.

**Some well-established schemes of multiple drift excavation are:-**

a) **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). 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.

b) **Old Austrian Tunneling Method:** This method is schematically represented in Figure below:-
The characteristic feature of this method is the crown slot. The simultaneous work in several excavation faces allows a fast advance.

**c) Sidewall drift:** The side galleries are excavated and supported first. 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 three, many other variations of multiple drift sequence are also in practice.

Drill-and-blast are best suited for hard rocks, non-circular cross sections & relatively short tunnels (where use of TBM/Road header is uneconomical) & 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 & minimize damage outside the minimum excavation line.
ii) **Excavation by Road header:**

A road header is a piece of excavating equipment consisting of a boom-mounted cutting head, a loading device usually involving a conveyor, and a crawler travelling track to move the entire machine forward into the rock face.

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 jackhammer 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. One has to provide for measures against dust (suction or water spraying). 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 on road headers is that they currently are usable only in rock of less than about 12,000 psi compressive strength. Somewhat stronger rock can be cut, or chipped away, if it is sufficiently fractured.

This method could also be called partial face mechanization. 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.
iii) **Excavation by Tunnel Boring Machine:**

Tunnel boring machines are used as an alternative to drilling and blasting (D&B) methods in rock. TBMs excavate circular cross sections with a rotating cutter head equipped with disc cutters.

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.

TBM tunnels have very high start-up (pre-excavation) costs and accompanying long lead time; 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.

**B. Excavation Methods for Soft Ground Tunnels**

Two principal Methods for tunneling in soft ground are:

1. **Multiple Drift Method:** This method has already been under “Excavation methods for rock tunnels”. Fore poling is normally done before doing excavation particularly in heading portion.

2. **Excavation by Tunnel Shields:** The shield is a steel tube with a (usually) circular cross section. Its front is equipped with cutters. The shield is pushed forward into the ground by means of jacks.

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.
Chapter 4: Tunneling Methods

Tunneling methods are normally classified on the basis of excavation method under the following categories:-

A) Conventional Tunneling
B) Mechanized Tunneling

The definition of what is “Conventional Tunneling” is rather arbitrary, and subject to variations, depending also on the context. For the present guidelines, the term conventional tunnel applies to any tunnel that is not excavated by a Tunnel Boring Machine (TBM)/Shield. It may however be noted that in tunneling the TBM method has these days become very common and thus could also be regarded as “conventional” in many countries.

Another popular classification is “Rock Tunneling Methods” & “Soft Ground Tunneling” Methods.

Yet another classification is sometimes made into “NATM” & “Non NATM” Tunnels; the former following NATM philosophy.

For these guidelines, we will be discussing tunneling methods under categories of Conventional Tunneling & Mechanized Tunneling. Features/principles of NATM (& NTM) Tunneling are being discussed under separate section.

A) Conventional Tunneling:

Conventional Tunneling in the context of this report means the construction of underground openings of any shape with a cyclic construction process of:

- Excavation, by using the drill and blast methods or mechanical excavators except any full face TBM
- mucking
- Placement of the primary (initial) support elements such as
  - Steel ribs or lattice girders
  - Soil or rock bolts
  - sprayed concrete (shotcrete) or cast in situ concrete, not reinforced or reinforced with wire mesh or fibres
- final or secondary lining (if required)
1.0 **Principles of Conventional Tunneling**

Conventional Tunneling is carried out in a cyclic execution process of repeated steps of excavation followed by the application of relevant primary support, both of which depend on existing ground conditions and ground behaviour.

The Conventional Tunneling Method mainly using standard equipment and allowing access to the tunnel excavation face at almost any time is very flexible in situations or areas that require a change in the structural analysis or in the design and as a result of this also require changes in the support measures.

A standard set of equipment for execution of Conventional Tunneling may consist of the following items:

- Jack Hammers/Drilling jumbo to drill holes for blasting, rock bolting, water and pressure release, grouting etc.
- Road header or excavator in cases where blasting is not possible or not economic.
- Lifting platform allowing the workers to reach each part of the tunnel crown and of the tunnel face.
- Lifting equipment for handling/erection of steel sets.
- Loader or excavator for loading excavated material onto dump trucks.
- Dump trucks for hauling excavated ground.
- Shotcrete machines (robotic/non-robotic).

Using this standard set of equipment the following changes can easily be applied during construction if ground conditions change or if monitoring results require action:

- Increase or decrease of support, e.g. the thickness of shotcrete, number and/or lengths of rock bolts per linear meter of tunnel, spacing and dimensions of steel arches, number and lengths of forepoles, application of shotcrete at the tunnel face, bolting the face etc.
- Variation of ring closure time.
- Variation of explosives charge per blasting round and variation of detonator sequences.

Other variations in the design enable one to react to changes in the stand-up time of the ground encountered:

- Increased or decreased length of excavation round (common round lengths vary from 0.5 m to 4.0 m).
- Partial excavation by splitting the excavation face into the crown, bench, and invert excavation steps or even further in pilot and sidewall galleries.
In case exceptional ground conditions are encountered - regardless of whether predicted or not - the Conventional tunneling Method can react with a variety of auxiliary construction technologies like

- **Grouting**: consolidation grouting, fissure grouting, pressure grouting, compensation grouting.

- **Technologies to stabilize and improve the ground ahead of the actual tunnel face** like forepoling, pipe umbrella, horizontal jet grouting, ground freezing etc.

Conventional Tunneling in connection with the wide variety of auxiliary construction methods enables making the most appropriate choice to achieve safe and economic tunnel construction even in situations with changing or unforeseen ground conditions. It allows reacting in both directions - depending on the ground - either changing to the less favourable or towards the more favourable side. This flexibility makes Conventional tunneling the most advantageous tunneling method in many projects.

The Conventional tunneling Method is preferred choice for tunneling under highly variable ground conditions or for projects with variable shapes.

### 1.1 Conventional tunneling enables:

- A greater variability of the shapes.
- Better knowledge of the ground by using systematic exploratory drillings at tunnel level ahead of the face.
- Greater variability in the choice of excavation methods according to the ground conditions.
- Greater variability in the choice of excavation sequences according to the ground conditions.
- Easier optimization of the primary support using the observational method in special cases.
- A greater variability in the choice of auxiliary construction methods according to the ground conditions.

### 1.2 Conventional tunneling is especially convenient for:

- Difficult ground with highly variable ground conditions
- Projects with highly variable shapes of cross section
- Projects with a higher risk of water inflow under high pressure
- Projects with difficult access
2.0 Construction Methods for Conventional Tunneling:

2.1 Excavation Methods

The excavation methods for Conventional tunneling are:

- Drilling and blasting mainly applied in hard rock ground conditions
- Mechanically supported excavation mainly used in soft ground and in weak rock conditions (using road headers, excavators with shovels, hydraulic breakers etc.)

Both excavation methods can be used in the same project in cases with a broad variation of ground conditions. In both excavation methods the excavation is carried out step by step in rounds. The round length generally varies from 4 m in good conditions to 1 m or less in soil and poor ground conditions. The round length is the most important factor for the determination of the advance speed.

The engineer shall prescribe or limit the choice of the method of excavation only if there are compelling reasons based on project restrictions. The responsibility of the selection of the excavation method should best be left to the contractor, based on the engineer’s description of the ground conditions and the design limits.

2.1.1 Excavation Sequence

Conventional tunneling allows full-face and the partial excavation of the tunnel cross section. Besides the structural analysis an important criterion for selecting the adequate excavation sequence is the length of the individual excavation-steps/rounds, which depends on the stand-up time of the ground without support. In good ground conditions the maximum round length is limited by the acceptable tolerance for over break, which is mainly an economic criterion when over break has to be filled up to the design line of the tunnel circumference.

Both excavation types (full-face and the partial excavation) allow exploratory drillings from the face at any time.

Full-face excavation is used for smaller cross sections and in good ground conditions with long stand-up times. Since a high degree of mechanization of the work and the use of large, high performance equipment has become common, also bigger cross sections (70 to 100 m² and more), even in difficult rock conditions are sometimes excavated with the full-face method. In any case, face stability shall be given serious consideration and often face support - bolting, shotcrete etc. - becomes necessary.
Full-face excavation allows the immediate closure of the primary support ring, close to the excavation face.

Partial excavation is mainly used for big cross sections in soils and unfavorable ground conditions. There are several types of partial excavation as discussed earlier.

Partial excavation, allows the combination of the different excavation methods in the same cross section, e.g. blasting in the top heading and excavating the bench by using a mechanical excavator e.g. a road header.

The choice whether full-face or partial excavation is preferable depends on ground properties but also sometimes on environmental aspects, on the magnitude of settlements at the surface and economic considerations. In special cases both excavation sequences can be used. However, frequent changes in the type of excavation are uneconomical.

2.1.2 Primary Support

The purpose of the primary support is to stabilize the underground opening until the final lining is installed. In many cases it may become necessary to apply the support system in combination with auxiliary constructional measures.

The most common elements for the primary support are

- Rock bolts

![Local Rockbolting and Systematic Rockbolting]
- **Shotcrete** (not reinforced and reinforced- with fibres or wire mesh)

- **Steel ribs and lattice girders**
Lattice Girder placed at site after assembly

Lattie Girders

- **Lagging & back fill concrete**

  These elements are applied individually or in combination in different types of support depending on the assessment of ground conditions by the responsible site engineers and by taking into account the corresponding design.

  In each round, elements of the primary support have to be placed up to the excavation face for reasons of safety and analysis/ assessment of the actual ground conditions.

2.1.3 **Auxiliary Construction Measures**

  In special cases, the excavation work can only be carried out by means of additional auxiliary construction measures. The auxiliary construction measures can be classified in the following categories:

  a. **Ground improvement**
  b. **Ground reinforcement**
  c. **Dewatering**

  a) **Ground improvement**

  Ground improvement means the application of methods that improve the mechanical or hydraulic properties of the ground.

  The main methods are:

  i. Grouting
  ii. Jet grouting
  iii. Ground freezing

  Ground improvement has normally to be carried out alternately to the excavation and leads to interruptions of the excavation work. In special cases ground improvement can be carried out from the surface or pilot tunnels outside the future tunnel cross section.
i) **Grouting:**

The different techniques for grouting are consolidation grouting, fissure grouting, pressure grouting and compensation 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. 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.

![Diagram of grouting](image)

ii) **Jet Grouting**

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.

iii) **Ground freezing**

The following ground freezing techniques are known to waterproof or stabilize temporarily the ground:

- **Continuous frozen bodies** which provide long-term load-bearing. The freezing is achieved by a drilled tube system, through which coolant is pumped.

- **Short-term, immediately effective local freezing** of damp zones close to the face or in the immediate vicinity outside the excavated cross section. This is achieved by means of injection lances with liquid nitrogen cooling.

b) **Ground Reinforcement:**

Ground reinforcement involves the application of methods that use the insertion of structural elements with one predominant dimension. Bolts, anchors, micro piles and spiles are such elements. The main methods of application are pipe umbrellas; face bolting or radial bolting from a pilot bore.

**Pipe umbrella:**

Pipe umbrellas are specified to supplement the arch structure in the roof and spring line regions as well as stabilization of the face and in advance of the face immediately after the excavation. The pipe umbrella should extend at least 30% beyond the face of the next excavation.
Spiles:

Spiles are steel rods left in the ground for the local short-term stabilization of the roof section and at the working face on the boundary of the excavation.

The spiles rest on the first steel arch in front and should be at least 1.5 times as long as the subsequent advance in the excavation. Depending on the type of soil, the spiles can be jacked, rammed or inserted in drill holes.

Face bolts:

Face bolts are often necessary to stabilize or reinforce the face. Depending on the anticipated ground condition/behaviour, the relevant bolt type and length have to be determined in the design. Practically any bolt type or length is possible. As a protection against rock fall, spot bolts may be sufficient whereas in difficult ground conditions (e.g. squeezing rock and soils) systematic anchoring with a high number of long, overlapping steel or
fibre glass bolts may be necessary. Face bolts are placed during the excavation sequence, if necessary in each round or in predefined steps.

c) **Dewatering and drainage:**

In some cases the tunnel construction is only possible with the application of special dewatering measures. According to the ground conditions and other boundary conditions conventional vertical or horizontal wells or vacuum drains can be used.

In the design of the dewatering measures environmental aspects have to be considered, such as limits on lowering the ground water table, settlements, etc.

In the case of low overburden, dewatering measures can be carried out from the ground surface. In the other cases, dewatering has to be done from the tunnel cross section or from pilot tunnels.

### 2.1.4 Placement of final (secondary) lining:

An underground space excavated by the Conventional tunneling Method **may need a final (secondary) lining** in addition to the primary lining according to the requirements of the project to:

- Cater for all the final load cases
- Fulfill the final safety margin
- Include the necessary protection measures (e.g. water tightness)
- Guarantee the required service life time

**Generally two options** exist to construct the final (secondary) lining:

- Installation of an independent secondary lining that is normally dimensioned to withstand all the final load cases. The secondary lining can consist of shotcrete or cast in situ concrete. According to the requirements of the project the final lining consists of unreinforced concrete or reinforced concrete.
- Installation of additional layers of shotcrete to strengthen the primary lining for all the final load cases.

### 3.0 Instrumentation and Monitoring

Field monitoring is an indispensable element of modern tunneling. The purpose of the instrumentation may be:

- Checking the structural behaviour with respect to safety and/or serviceability criteria, mainly during construction and in some cases during service life.
- The quantification of structural response to a specific method of construction and checking the effectiveness of specific support
measures.
- The comparison of theoretical predictions with the actual structural behaviour and the assessment of the material parameters of the ground.
- Checking adjacent structures and facilities for their safety and serviceability as a result of the construction of the tunnel.

Due to its quality, monitoring data can also be used for clearing disputes between contractual partners or between the client and third parties. Therefore a further objective is:

- Documentation related to the tunnel construction

B) Mechanized tunneling:

1.0 Mechanized tunneling offers many advantages; some of these are:-

- industrialization of the tunneling process, with ensuing reductions in costs and construction times
- the possibility some techniques provide of crossing complex geological and hydrogeological conditions safely and economically,
- the good quality of the finished product
- 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.

Detailed analysis of the conditions under which the project is to be carried out should substantially reduce this risk. The experience and technical skills of tunneling machine operators are also an important factor in the reduction of risks.

There are many options available these days for the construction of tunnels. The selection of which 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. Strictly speaking, TBM’s are “unshielded” (i.e. are of Open type) and are used in hard rock whereas Shields have “shield/s” under the protection of which the ground is excavated and the tunnel support is erected. Shields are used in soft ground.

But quite often these two terms are used interchangeably as distinction between two is getting blurred.
1.1 Tunneling shields:

In its simplest form a tunneling shield is a steel frame with a cutting edge on the forward face. 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.

There are two main types of tunnelling shield, one with partial and one 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 only considered for very special applications, e.g. very short advances, due to the low advance rate. This type of tunneling is called the manual shield technique.

Full face Shield are discussed under Shielded TBMs & Soft Ground TBMs; however basic working principle of the same is being briefly described below:-

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 shaft 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 between approximately 50 and 250 mm. 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.

1.2 Tunnel boring machines:

1.2.1 Introduction:

TBMs exist in many different diameters, ranging from micro tunnel boring machines with diameters smaller than 1 m to machines for large tunnels, whose diameters are greater than 15 m (the largest TBMs are now in excess of 19 m).

TBMs are available for many different geological conditions. This means that, for example, the type of tunnel face support required and the excavation procedure as well as numerous other technical requirements can be solved in many different ways. Every tunnel is different and hence there are often frequent technical advancements in this field.

Although TBMs are often designed for specific projects, i.e. with a specific diameter and in order 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.

Tunnel boring machines (except ‘Gripper’ type TBM for hard rock), have a ‘shield’. Although it should be noted that even a Gripper TBM can have a small shield around the cutter wheel to avoid it catching on any rock as the ground deforms under high pressures.

One of the general requirements for the use of a TBM is a 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 being developed). 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. As the tunnel segments are erected within the tunnel shield, there is a gap between the segments and the excavated ground. In order to achieve a rigid connection between the ground and the tunnel lining, thus preventing the ground from moving, the gap is injected with cement slurry.
TBM are often grouped under categories of “Hard Rock TBMs” & “Soft Ground TBMs”.

1.2.2 Hard Rock TBMs:

Hard rock TBMs comprise four key sections, which make up the complete machine. These are the boring section, consisting of the cutter head, the thrust and clamping section, which is responsible for advancing the machine, the muck removal section, which takes care of collecting and removing the excavated material, and the support section, where the tunnel support is erected.

Hard rock TBMs comprise following four key sections:

(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.

Hard Rock TBMs primarily fall under following categories:

ii) 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 drilling system, i.e. 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 conveyer belt. The excavated material is brought outside the tunnel by conveyers.
Typical advance of cutter head is approximately 0.7 to 1.2 m. 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 largish 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.

1.2.3 Soft ground TBMs/Shields

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 rearward 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:

a) Mechanical-support TBM/Shield

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.

**b) Compressed-air TBM/Shield**

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.

**c) Slurry shield TBM/Shield**

A slurry shield TBM has a full-face cutter head. Support of the face is accomplished by pressurized slurry; which in most cases is bentonite suspension.

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.

**d) Earth pressure balance TBM/Shield**

An earth pressure balance machine has a full-face cutter head. 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.

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.

In addition to above, there are Special Purpose TBMs also. Some of these are:-

**Reaming Boring Machines** - allows a tunnel made using a TBM (pilot tunnel) to be widened (reaming).

**Raise Borer** - used for shaft excavation which enables the top-to-bottom reaming of a small diameter pilot tunnel created using a drilling rig.

**Mixed Face TBMs** – allows tunneling under mixed face conditions.

**Multi-Mode TBMs** – can operate in different modes with appropriate modifications to configuration & support techniques.

**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 spaces necessary for auxiliary facilities behind the machine and around the access tunnels
- Safety of tunneling and other related works.
Chapter 5: NATM

1.1 Background:

The 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’.

Although Rabcewicz, Müller and Pacher used techniques and knowledge which were already well known, this was the first time these techniques had been put together in a new and almost revolutionary tunnelling method. Instead of fighting the overburden by a thick lining they acknowledged that the ground, not the lining is the main support of the tunnel. Consequently they reduced the lining thickness radically using sprayed concrete instead of the brick lining, which was common at the time. This gave the benefit of a tight and firm coupling between the lining and the ground; whereas the brick lining left a space between the support and the surrounding ground.

Loosening pressure was conventionally considered to be the main active force to be reckoned with in tunnel design. NATM approach emphasized the importance of avoiding loosening almost entirely. Whereas the earlier methods of temporary support were bound to cause loosening and voids by yielding of the different parts of the supporting structure, a thin layer of shotcrete together with a suitable system of rock bolting applied to the rock face immediately after blasting entirely prevents loosening and reduces decompression to a certain degree, transforming the surrounding rock into a self-supporting arch. A layer of shotcrete with a thickness of only 15cm applied to a tunnel of 10 m diameter can safely carry a load of 45 tons/m$^2$ corresponding to a burden of 23 m of rock.

Furthermore, it was important that the sprayed concrete lining (SCL) was supported by systematic anchoring. Rabcewicz, Müller and Pacher used a flexible approach with respect to the excavation sequence and amount of support. They observed the reaction of the ground as a result of the tunneling process and used this information to determine the required support and construction sequences. The calculation techniques available at that time could not confirm the stability of this thin lining. Therefore, they used displacement monitoring to prove the efficiency of their support.

1.2 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.”
Unfortunately, this definition did not prove to be unique to NATM since nearly every excavation method tries to preserve the supporting ability of the ground. Rokahr (1995) gave a definition which summarizes the original intention of Rabcewicz, Müller and Pacher and makes it easy to distinguish the NATM from other tunnelling methods: ‘NATM is a support method to stabilize the tunnel perimeter by means of sprayed concrete, anchors and other support, and uses monitoring to control stability’.

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”.

1.3 Principles of NATM

The main principles of NATM are:

- The main load-bearing component of the tunnel is the surrounding rock mass. The sprayed concrete has only a secondary supporting function. Maintain strength of the rock mass and avoid detrimental loosening by careful excavation and by immediate application of support and strengthening means. Shotcrete and rock bolts applied close to the excavation face help to maintain the integrity of the rock mass.

- The support must not be installed too early or too late. It should not be too stiff or too weak.

  The level of deformation in the ground should be:

  - on the one hand kept small so that the ground does not lose more of its initial stability than unavoidable and;
  - on the other hand must be large enough in order to activate the support of the ground as a closed arch and to optimize the usage of the support measures and the excavation.

- The tunnel is to be seen as a composite system consisting of the ground, and the support and stabilizing measures, e.g. sprayed concrete, anchors, steel ribs and similar.

- **Rounded tunnel shape**: avoid stress concentrations in corners where progressive failure mechanisms start.

- **Flexible thin lining**: The primary support shall be thin-walled in order to minimize bending moments and to facilitate the stress rearrangement process without exposing the lining to unfavourable sectional forces. Additional support requirement shall not be added by increasing lining thickness but by bolting. The lining shall be in full contact with the exposed rock. Shotcrete fulfills this requirement.
• The closing of the ring is important, i.e. the total periphery including the invert must be applied with shotcrete.

• **In situ measurements**: Observation of tunnel behaviour during construction is an integral part of NATM. The choice of support and construction sequence is made on the basis of monitoring of tunnel behavior. Observational method requires:

  - Determination of acceptable limits for the behaviour of a construction
  - Verification that these limits will (with sufficient probability) not be exceeded
  - Establishing a monitoring programme that gives sufficient warning of whether these limits are kept
  - Providing measures for the case that these limits are exceeded.

1.4 **Construction Sequence**:

The construction process is 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 lining.

1.5 **Excavation & Supporting sequence is illustrated below**:-

| Excavating a length of tunnel, (here with a roadheader) | Applying layer of shotcrete on reinforcement mesh |
1.6 Limitation:

NATM assumes that the ground has sufficient stand-up time itself for the construction cycle. 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.

1.7 Pre-requisites for NATM

- Balanced Contract Structure:- providing for equitable risk sharing & 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.

1.8 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.

2.0 NMT

A variant of NATM using steel-fiber reinforced shotcrete (SFRS) instead of mesh-reinforced concrete is referred to as the **Norwegian Method of Tunnelling (NMT)**.
Chapter 6: Tunnel Design Methods

1.0 Tunnel Design

Tunnel design must achieve functionality, stability, and safety of the tunnel opening during and after construction and for as long as the underground structure is expected to function. Tunnel design includes design of initial ground support as well as permanent support system.

There are two basic methodologies employed in selecting initial ground support, and one or both of these approaches may be used.

- **Empirical Approach:** constructed from experience records of satisfactory past performance.

- **Analytical and/or numerical approach:** based on theoretical or semi-theoretical analysis and/or involving a fundamental approach, with study/investigation of potential modes of failure and a selection or design of components to resist these modes of failure.

Tunnel design approaches for non-NATM & NATM are discussed below:

1.1 Non-NATM

(A) Empirical Approach for Selection of Initial Support for rocks

In past centuries, ground support was always selected empirically. The miner estimated, based on his experience, what timbering was required, and if the timbering failed it was rebuilt stronger. Written rules for selecting ground support were first formulated by Terzaghi (1946). The development of the RQD as a means to describe the character or quality of the rock mass led to correlations between RQD and Terzaghi’s rock loads. This development also led to independent ground support recommendations based on RQD. The RQD is also used in two other rock mass characterization schemes used for initial ground support selection, the Geo-mechanics Classification: Rock Mass Rating (RMR), Bieniawski 1979), and the Norwegian Geotechnical Institute’s Q-system (Barton, Lien, and Lunde 1974).
### 1.1.1 Terzaghi’s rock loads & support recommendations

Terzaghi’s qualitative classification of rock mass/conditions is tabulated below:

<table>
<thead>
<tr>
<th>Rock Condition</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact rock</td>
<td>Contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a spalling condition. Hard, intact rock may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.</td>
</tr>
<tr>
<td>Stratified rock</td>
<td>Consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.</td>
</tr>
<tr>
<td>Moderately jointed rock</td>
<td>Contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.</td>
</tr>
<tr>
<td>Blocky and seamy rock</td>
<td>Consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.</td>
</tr>
<tr>
<td>Crushed but chemically intact rock</td>
<td>Has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.</td>
</tr>
<tr>
<td>Squeezing rock</td>
<td>Slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and submicroscopic particles of micaceous minerals or clay minerals with a low swelling capacity.</td>
</tr>
<tr>
<td>Swelling rock</td>
<td>Advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.</td>
</tr>
</tbody>
</table>

Terzaghi estimated rock loads acting on roof of tunnel supports for various rock mass classifications/conditions. He described these loads in terms of the height of a loosened mass of rock weighing on the support. The height is a multiple of the width of the tunnel or of the width plus the height.
Terzaghi’s recommendations are tabulated below (giving Rock Load $H_p$ in feet of rock of support in tunnel with Width $B$ (ft) and Height $H_t$ (ft) at depth more than 1.5C, Where $C = B + H_t$):

<table>
<thead>
<tr>
<th>Rock Condition</th>
<th>Rock Load $H_p$ in feet</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Hard and intact$^a$</td>
<td>Zero</td>
<td>Light lining, required only if spalling or popping occurs.</td>
</tr>
<tr>
<td>2. Hard stratified or schistose$^b$</td>
<td>0 to 0.5B</td>
<td>Light support.</td>
</tr>
<tr>
<td>3. Massive moderately jointed</td>
<td>0 to 0.25B</td>
<td>Load may change erratically from point to point.</td>
</tr>
<tr>
<td>4. Moderately blocky and Seamy</td>
<td>0.25B to 0.35C</td>
<td>No side pressure.</td>
</tr>
<tr>
<td>5. Very blocky and seamy</td>
<td>(0.35 to 1.10)C</td>
<td>Little or no side pressures.</td>
</tr>
<tr>
<td>6. Completely crushed but chemically intact</td>
<td>1.10C</td>
<td>Considerable side pressure. Softening effect of seepage toward bottom of tunnel requires either continuous support for lower end of ribs or circular ribs.</td>
</tr>
<tr>
<td>7. Squeezing rock, moderate depth</td>
<td>(1.10 to 2.10)C</td>
<td>Heavy side pressure, invert struts required.</td>
</tr>
<tr>
<td>8. Squeezing rock, great depth</td>
<td>(2.10 to 4.50)C</td>
<td>Circular ribs are recommended.</td>
</tr>
<tr>
<td>9. Swelling rock</td>
<td>Up to 250 feet irrespective of value of C</td>
<td>Circular ribs required. In extreme cases use yielding support.</td>
</tr>
</tbody>
</table>

$a$ The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by 50%.

$b$ Some of the most common rock formations contain layers of shale. In an 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.

Today rock tunnels are usually designed considering the interaction between rock and ground, i.e., the redistribution of stresses into the rock by forming the rock arch. However, the concept of loads still exists and may be applied early in a design to “get a handle” on the support requirement. The concept is to provide support for a height of rock (rock load) that tends to drop out of the roof of the tunnel (Terzaghi, 1946).

2.0 Rock Quality Designation (RQD)

In 1966 Deere and Miller developed the Rock Quality Designation index (RQD) to provide a systematic method of describing rock mass quality from the results of drill core logs. Deere described the RQD as the length (as a percentage of total core length) of intact and sound core pieces that are 4 inches (10 cm) or more in length.

It has been shown that a qualitative relationship exists between the RQD and the support required for tunnels in rocks. Several proposed methods of using the RQD for design of rock tunnels have been developed. Support recommendations (Deere et al., 1967) for tunnels having 6 m (i.e. 20 ft) to 12m (i.e. 40ft) dia are tabulated below:-
### Support Recommendations for Tunnels in Rock (6 m to 12 m diam) based on RQD (after Deere et al. 1970)

<table>
<thead>
<tr>
<th>Rock Quality</th>
<th>Tunneling Method</th>
<th>Steel sets³</th>
<th>Rockbolts ³</th>
<th>Shotcrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent¹  RQD&gt;90</td>
<td>Boring machine</td>
<td>None to occasional light set. Rock load (0.0–0.2)B</td>
<td>None to occasional</td>
<td>None to occasional local application</td>
</tr>
<tr>
<td>Good¹  75&lt;RQD&lt;90</td>
<td>Boring machine</td>
<td>Occasional light sets to pattern on 5 to 6 ft center. Rock load (0.0 to 0.4)B</td>
<td>Occasional to pattern on 5 to 6 ft centers</td>
<td>None to occasional local application 2 to 3 in.</td>
</tr>
<tr>
<td>Fair 50&lt;RQD&lt;75</td>
<td>Boring machine</td>
<td>Light to medium sets, 5 to 6 ft center. Rock load (0.4 to 1.0)B</td>
<td>Pattern, 4 to 6 ft centers</td>
<td>2 to 4 in. crown</td>
</tr>
<tr>
<td>Poor²  25&lt;RQD&lt;50</td>
<td>Boring machine</td>
<td>Medium circular sets on 3 to 4 ft center. Rock load (1.0 to 1.6) B</td>
<td>Pattern, 3 to 5 ft center</td>
<td>4 in. or more on crown and sides. Combine withbolts.</td>
</tr>
<tr>
<td>Very poor³ RQD&lt;25 (Excluding squeezing or</td>
<td>Boring machine</td>
<td>Medium to heavy circular sets on 2 ft center. Rock load (1.3 to 2.0)B</td>
<td>Pattern, 2 to 4 ft center</td>
<td>6 in. or more on crown and sides. Combine with bolts.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 3.0 Support Selection based on Rock Mass Rating (RMR) System:

The geomechanics classification also known as **Rock Mass Rating (RMR)** was developed by Bieniawski (1973, 1974, 1979).

The following six parameters are used to classify a rock mass using the RMR system:

1. Strength of Intact Rock Material (Point Load Strength Index or Uniaxial compressive strength of rock material)
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

**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 and 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.

The major use of the RQD in modern tunnel design is as a major factor in the Q or RMR rock mass classification systems.
Ratings for each of the six parameters listed above are summed to give a value of RMR.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions.

IS Code 13365 Part 1 may be referred to for more details of this rating system.

Recommendations for excavation and support of 10 m Span Rock Tunnels in accordance with the RMR System are tabulated below:

<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Rock bolts (20 mm diameter, fully grouted)</th>
<th>Shotcrete</th>
<th>Steel sets</th>
</tr>
</thead>
<tbody>
<tr>
<td>I – very good rock</td>
<td>Full face 3 m advance</td>
<td>Generally no support required except spot bolting</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RMR: 81-100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II – Good rock</td>
<td>Full face, 1-1.5m advance. Complete support 20 m from face</td>
<td>Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wiremesh</td>
<td>50 mm in crown where required</td>
<td>None</td>
</tr>
<tr>
<td>RMR: 61-80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>III – Fair rock</td>
<td>Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.</td>
<td>Systematic bolts 4m long, spaced 1.5-2 m in crown and walls with wire mesh in crown.</td>
<td>50-100 mm in crown and 30 mm in sides</td>
<td>None</td>
</tr>
<tr>
<td>RMR: 41-60</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IV – Poor rock</td>
<td>Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.</td>
<td>Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.</td>
<td>100-150 mm in crown and 100 mm in sides</td>
<td>Light to medium ribs spaced 1.5 m where required</td>
</tr>
<tr>
<td>RMR: 21-40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V – very poor rock</td>
<td>Multiple drifts. 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.</td>
<td>Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.</td>
<td>150-200 mm in crown, 150 mm in sides, and 50 mm on face</td>
<td>Medium to heavy ribs spaced steel lagging and forepoling if required. Close invert.</td>
</tr>
<tr>
<td>RMR: &lt; 20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.0 Support Selection based on Q-System of rock mass classification:

The NGI Q-System (Barton, Lien and Lunde 1974) is generally considered the most elaborate and the most detailed rock mass classification system for ground support in underground works. The value of the rock quality index Q is determined by

\[ Q = \left( \frac{\text{RQD}}{J_n} \right) \left( \frac{J_r}{J_a} \right) \left( \frac{J_w}{\text{SRF}} \right) \]

where:
- RQD = Rock Quality Designation
- Jn = joint set number
- Jr = joint roughness number
- Ja = joint alteration number
- Jw = joint water reduction factor
- SRF = stress reduction factor

Numerical range of the six parameters used in calculation of Q value is large, and the combined effect of their use results in Q Value having an extremely large range (approx. from 0.001 to 1000). Accordingly, the Q System accommodates the entire spectrum of rock mass conditions ranging from sound, un-jointed rock to heavy squeezing ground.

A high Q-value means good stability, whereas low values indicate poor stability. The Q-values together with the span width or height of walls and safety requirements are used as a basis for an evaluation of permanent support.

Implementation of Q system is done through calculating Q and effective span. The effective span is Barton’s method of applying a risk/safety factor to the application. It is obtained by dividing the actual span by Excavation Support Ratio (ESR). A lower ESR provides a higher degree of safety. An ESR of 1.0 is used for civil work projects including railway tunnels.

Standard literature may be referred to for more details of the rock quality rating for Q-System and its application for design purposes. One good document on different Rock Mass Classifications & their applications for tunnel design is available at following URL:

5.0 Empirical Approach for Selection of Initial Support for soft ground

Anticipated ground behavior in soft ground tunnels was first defined by Terzaghi (1950) by means of the Tunnel man’s Ground Classification, a classification system of the reaction of soil to tunneling operation. Terzaghi described the representative soil types and the predicted behavior of these ground types to the tunneling construction methods in use in 1950s. Heur (1974) modified the Tunnel man’s Ground Classification, as shown in table
below to present the classification system in engineering terms that reflect current terminology and usage.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Behavior</th>
<th>Typical Soil Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Firm</td>
<td>Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.</td>
<td>Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.</td>
</tr>
<tr>
<td>Raveling</td>
<td><strong>Slow raveling</strong>&lt;br&gt;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 to over-stress and “brittle” fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground).</td>
<td>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.</td>
</tr>
<tr>
<td></td>
<td><strong>Fast raveling</strong>&lt;br&gt;In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.</td>
<td></td>
</tr>
<tr>
<td>Squeezing</td>
<td>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.</td>
<td>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 excavation surface and squeezing at depth behind surface.</td>
</tr>
<tr>
<td>Running</td>
<td><strong>Cohesive running</strong>&lt;br&gt;Granular materials without cohesion are unstable at a slope greater than their 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.</td>
<td>Clean, dry granular materials. Apparent cohesion in moist sand, or weak 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.</td>
</tr>
<tr>
<td></td>
<td><strong>Running</strong>&lt;br&gt;A mixture of soil and water flows into the tunnel like a viscous fluid.</td>
<td>Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also</td>
</tr>
</tbody>
</table>


The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases. These events can 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 containing significant percentages of montmorillonite. |

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. Standard technical literature may be referred to for more details in this regard.

A simplified system of determining the load on the initial support for circular and horseshoe tunnels in soft ground is tabulated below. The loads are patterned after Terzaghi’s original recommendations (1950) but have been simplified. In all cases, it is important that the experience and judgment of the engineer also be applied to the load selection.

**Table: Initial Support Loads for Tunnels in Soft Ground**

<table>
<thead>
<tr>
<th>Geology</th>
<th>Circular Tunnel</th>
<th>Horseshoe Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Running ground</td>
<td>Lessor of full overburden or 1.0 B</td>
<td>Lessor of full overburden or 2.0 B</td>
</tr>
<tr>
<td>Flowing ground in air free</td>
<td>Lessor of full overburden or 2.0 B</td>
<td>Lessor of full overburden or 4.0 B</td>
</tr>
<tr>
<td>Raveling ground</td>
<td>Same as running ground</td>
<td>Same as running ground</td>
</tr>
<tr>
<td>- Above water table</td>
<td>Same as flowing ground</td>
<td>Same as flowing ground</td>
</tr>
<tr>
<td>- Below water table</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Squeezing ground</td>
<td>Depth to tunnel springline</td>
<td>Depth to tunnel springline</td>
</tr>
<tr>
<td>Swelling ground</td>
<td>Same as raveling ground</td>
<td>Same as raveling ground</td>
</tr>
</tbody>
</table>

* Modified by Heuer (1974) from Terzaghi (1950)*

6.0 **Limitations in the use of empirical ground support selection systems**

a) The empirical methods of ground support selection provide a means to select a ground support scheme based on facts 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.

b) A major flaw of all the empirical systems is that they lead the user directly from the geologic characterization of the rock mass to a recommended ground support without the consideration of possible failure modes. A number of 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.

c) The empirical systems 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.

(B) **Analytical and Numerical Approach:** This approach makes use of various theoretical, Semi-theoretical & numerical Methods to design individual support components like rockbolts, shotcrete, steel ribs, etc. It may also involve a fundamental approach, with study/investigation of potential modes of failure and a selection or design of components to resist these modes of failure (using methods like finite elements, finite differences etc.).
1.2 Design of tunnels for NATM: The design process for NATM tunnels is represented in figure below:

**Concept**

- **Analytical route**
  - Engineering analysis leading to design
  - Commence construction
  - Observe and monitor support behaviour, Back analysis if appropriate
  - Confirm design or if appropriate design strengthened support and/or redesign future support
  - Continue construction

- **Empirical route**
  - Initial support selection based on experience and empirical methods
  - Commence construction
  - Observe and monitor support behaviour
  - If appropriate strengthened support and/or amend future support based on empirical assessment of monitoring results
  - Continue construction

It can be seen that design can follow either the "Empirical" approach or the "Analytical" Approach.

**Empirical Approach:**
and support installation sequencing frequently associated with NATM tunnels depending on the basic types of ground encountered, i.e. rock and soft ground respectively.
Elements of Commonly Used Excavation and Support Classes (ESC) in Rock

<table>
<thead>
<tr>
<th>Ground Mass Quality - Rock</th>
<th>Excavation Sequence</th>
<th>Rock Reinforcement</th>
<th>Initial Shotcrete Lining</th>
<th>Installation Location</th>
<th>Pre-Support</th>
<th>Support Installation influence progress</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Intact Rock</strong></td>
<td>Full face or large top heading &amp; bench</td>
<td>Spot bolting (fully grouted dowels, swellex(R))</td>
<td>Patches to seal surface in localized fractured areas</td>
<td>Typically several rounds behind face or directly near face to secure isolated blocks, slabs/wedges</td>
<td>None</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td><strong>Stratified Rock</strong></td>
<td>Top heading &amp; bench</td>
<td>Systematic doweling or bolting in crown considering strata orientation (fully grouted dowels, swellex(R), rock bolts)</td>
<td>Thin shell (fibre reinforced) typically 4in. (100 mm) to bridge between rock reinforcement in top heading, alternatively chain link mesh; installed with the rock reinforcement.</td>
<td>Two to three rounds behind face</td>
<td>None</td>
<td>No or eventually</td>
<td></td>
</tr>
<tr>
<td><strong>Moderately Jointed Rock</strong></td>
<td>Top heading &amp; bench</td>
<td>Systematic doweling or bolting in top heading considering joint spacing (fully grouted dowels, swellex, rock bolts)</td>
<td>Systematic shell with reinforcement (welded wire fabric or fibres) in top heading and potentially bench; dependent on tunnel size thickness of 6 in. (150 mm) to 8 in. (200 mm); installed with the rock reinforcement.</td>
<td>One to two rounds behind face</td>
<td>Locally to limit over break</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Blocky and seamy Rock</td>
<td>Top heading &amp; bench</td>
<td>Systematic doweling or bolting in top heading &amp; bench considering joint spacing</td>
<td>Systematic shell with reinforcement (welded wire fabric or fibres) in top heading and bench; depending on tunnel size thickness 8 in. (200 mm) to 12 in. (300 mm)</td>
<td>At the face or maximum one round behind face</td>
<td>Systematic spiling in tunnel roof or parts of it</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>------------------------</td>
<td>---------------------</td>
<td>---------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------</td>
<td>------------------------------------------------</td>
<td>------------------------------------------------</td>
<td>-----</td>
<td></td>
</tr>
<tr>
<td>Crush ed but chemically intact rock</td>
<td>Top heading, bench, invert</td>
<td>N/A</td>
<td>Systematic shell with reinforcement (welded wire fabric or fibres) and ring closure in invert; dependent on tunnel size thickness 12 in. (300 mm) and more; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required</td>
<td>After each round</td>
<td>Systematic grouted pipe spiling or pipe arch canopy</td>
<td>Support installation dictates progress</td>
<td>If water is present, ground water draw down or ground improvement is required</td>
</tr>
<tr>
<td>Squeezing Rock</td>
<td>Top heading, bench, invert</td>
<td>Systematic doweling or bolting in top heading &amp; bench considering joint spacing; extended length</td>
<td>Systematic shell with reinforcement (welded wire fabric or fibres) and ring closure in invert; dependent on tunnel size thickness 12 in. (300 mm) and more; potential use for yield elements; for initial stabilization and to prevent desiccation, a layer of</td>
<td>After each round</td>
<td>Systematic grouted pipe spiling or pipe arch canopy</td>
<td>Support installation dictates progress</td>
<td>-</td>
</tr>
<tr>
<td>Ground Mass Quality-Soil</td>
<td>Excavation Sequence</td>
<td>Initial Shotcrete Lining</td>
<td>Installation Location</td>
<td>Pre-Support</td>
<td>Support Installation</td>
<td>Remarks</td>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
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<td>----------------------</td>
<td>---------</td>
<td></td>
</tr>
<tr>
<td>Stiff/hard cohesive soil above groundwater table</td>
<td>Top heading, bench &amp; invert; dependent on tunnel size, further sub-divisions into drifts may be required</td>
<td>Systematic reinforced (welded wire fabric or fibres) 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</td>
<td>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</td>
<td>Typically none; locally spiling to limit over-break</td>
<td>Support installation dictates progress</td>
<td>Overall sufficient stand-up time to install support without pre-support or ground modification</td>
<td></td>
</tr>
</tbody>
</table>

**Table Elements of Commonly Used Soft Ground Excavation and Support Classes (ESC) in Soft Ground**
| Cohesive soil – above groundwater table | Bench & invert; dependent on ground strength, smaller drifts required than above | Reinforced (welded wire fabric or fibres) 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; frequently more invert curvature than above | 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 less than one tunnel diameter behind excavation face; typically earlier ring closure required than above | None; locally pre-spiling to limit over-break | Installation dictates progress | Stand-up time at sufficient to safety install support without pre-support or ground improvement |
| Well consolidated non-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 fibres) 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 is 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 less than one tunnel diameter behind excavation face | Frequently systematic pre-support required by ground pipe spiling or ground pipe arch canopy, alternatively ground improvement | Support installation dictates progress | Stand-up time insufficient to safety install support without pre-support or ground improvement |
| Well consolidated | Top heading, bench & invert | Systematic reinforced | Installation of shotcrete support | Frequently systematic pre-support required by ground pipe spiling or ground pipe arch canopy, alternatively ground improvement | Support installation dictates progress | Stand-up time at sufficient to safety install support without pre-support or ground improvement |
**Analytical & Numerical Approach:**

The following steps typically are used in the analyses to model the construction process:

- Excavation of the ground.
- Establishment of steady state flow conditions.
- Relaxation of the ground to represent deformations in advance of support.
- Installation of the primary lining.
- Analysis of the model and solving to equilibrium.
- Installation of the secondary lining.
- Analysis of the model and solving to equilibrium

The results obtained from the analysis are checked to confirm their acceptability. In addition to the static loadings, the performance of the secondary lining is checked for dynamic loading.

Numerical modeling is a particularly useful tool for design of sequentially excavated, shotcrete-lined tunnels in soft ground. For numerical modeling of tunnel linings in soft ground, software such as Flac or Plaxis may be used. It is necessary to know the theory behind the software and to correlate the analysis with tunnel behavior in the field.
Chapter 7: **Structural Design Issues: Tunnel Lining**

1. **Design Philosophy**

   Tunnel Lining refers to systems installed after excavation of tunnel to provide permanent support and durable, maintainable long term surface finishes. Various types of systems employed for the purpose are as under:

   (i) Shotcrete  
   (ii) Cast in place concrete  
   (iii) Precast segments  
   (iv) Lattice Girder and rolled steel sets.  
   (v) Ribs and Laggings.

   These systems are briefly described hereinafter.

1.1 **Shotcrete**

   It is also used some times as permanent lining. Permanent shotcrete linings are normally built in layers with surface layer containing wire mesh to provide long term ductility. In case, toughness and ductility are desirable, shotcrete reinforced with randomly oriented steel/synthetic fibres may be used as alternative. Occasionally, structural plastic fibres are used in lieu of steel fibres when shotcrete is expected to undergo high deformation and ductility post-cracking is of importance.

   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.

   The adhesion of shotcrete depends on condition of ground surface, dampness and presence of water and 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.
1.1.1 Initial Shotcrete Lining:

Initial shotcrete lining typically consists of 100 to 400mm thick shotcrete 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. This is the case where the shotcrete lining is expected to undergo high deformations and hence ductility post cracking is of importance. 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. This process generates subgrade reaction of the ground that provides support for the lining. 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 tunnel applications and beneath surface structures that are sensitive to deformations such as buildings, 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 only taken into account 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 however, in a generally low stress environment due to the shallow overburden. 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.
Rock reinforcement installed in rock tunnels augments the strength of the surrounding ground, controls deformation and limits the ground loads acting upon the shotcrete initial lining. 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.

The effect of the shotcrete is heavily dependent on the radial and tangential subgrade reaction generated by the surrounding ground. Therefore, shape, shotcrete thickness and installation time have to be designed in accordance with the ground conditions and the capacities of the surrounding ground and the support system. Site personnel should assess the support requirements and, if necessary, adjust the designed support system based on observations in the field. Notwithstanding the need for reaction to site conditions, the designer should always be party to the decision making process prior to changing any support system on site. The design intent and philosophy must be taken into consideration when adjustments of the support system are made.

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. Even the shotcrete arch not forming a closed ring provides substantial support to the ground, if tight contact between the sprayed concrete and the ground is maintained.

The requirement for a ring closure, be it temporary or permanent, is governed by the size of the underground opening and the prevailing ground conditions. In a good quality rock mass, no ring closure is required. In low quality ground (weak rock and soil), it has been reportedly proven that the time of support application after excavation, length of excavation round and time lag between the excavation of the top heading and the invert closure rules the ground and lining deflection. To reduce ground deflection and the potential for ground /lining failure, the excavation and support sequence must be designed such that an early ring closure of the shotcrete support in soft ground is achieved. Also the timely (immediately after excavation) installation of the shotcrete support members is of utmost importance.

An importance 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.

1.1.2 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 300 mm 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. The final shotcrete layer allows for the addition of micro polypropylene (PP) fibres that enhance fire resistance of the final lining.

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. As in final lining applications, contact grout is accomplished with cementitious grouts but the grout requirements are much higher. To ensure a proper grouting around the entire lining circumference it is customary to use longitudinal grout hoses arranged radially around the perimeter.

Major factor that will influence the quality of the shotcrete final lining application is workmanship. While the skill of the shotcrete applying nozzlemen (by hand or robot) is at the core of this workmanship, it is important to address all aspects of the shotcreting process in a method statement. This method statement becomes the basis for the application procedures, and the applicator’s and the supervisor’s Quality Assurance/Quality Control (QA/QC) program. Minimum requirements to be addressed in the method statement are as follows:

- Execution of Work (Installation of Reinforcement, Sequence of Operations, Spray Sections, Time Lag)
- Survey Control and Survey Method
- Mix Design and Specifications
- QA/QC Procedures and Forms
- Testing (Type and Frequency)
- Qualifications of Personnel
- Grouting Procedures

General trends in tunneling indicate that the application of shotcrete for final linings present 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.

1.1.3 **Shotcrete Analysis:**

By its capacity to accept shear and bending and its bond to rock surface, shotcrete prevents displacement of blocks of rock that can potentially fall. It can
also act as shell and accept radial loads. It is possible to analyze these modes of failure only if the loads and boundary conditions are known. Various theories of analysis provides only a crude approximation of stresses in shotcrete. When shotcrete is used in NATM, computer analysis can be used to reproduce construction sequence, including the effect of variations of shotcrete modules and strength with time.

1.2 **Cast in Place Concrete Lining:**

These are generally installed some time 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.

Flexible membrane based water proofing is advantageous in unreinforced lining as lack of bond reduces shrinkage cracking in final lining.

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. The loads can be directly transformed radially due to direct contact.

Cast in place 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. Use of water impermeable cast in place concrete lining as an alternative to membranes is generally not considered due to high demands on construction quality and elaborate measures required to prevent cracking, provision of construction joint etc.

1.2.1 **Advantages of a cast-in-place concrete lining:**

- 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.
1.2.2 Disadvantages of a cast-in-place concrete lining:

- Concrete placement, especially around reinforcement 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 when used is subject to corrosion and resulting deterioration of the concrete unless stainless steel reinforcement or treated reinforcement is used. 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.

1.2.3 Design Considerations:

In order to maximize flexibility and ductility, a cast-in-place 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. 25cm is considered the practical minimum thickness for a cast-in-place concrete lining.

Reinforcing steel in a thin section can also be problematic. The reinforcement inhibits the flow of the concrete making it more difficult to consolidate. 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 forms congested and concrete placement more difficult. Self consolidating concrete has been in development in recent years and has been used in unreinforced concrete linings in Europe with some success. Self consolidating concrete may prove useful in reinforced concrete linings, however, it is recommended that an extensive testing program be made part of the construction requirements to ensure that proper results are, actually obtained.

Cast-in-place 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 when it is practical, cast-in-place concrete linings should be designed and constructed as plain concrete, that is 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 behavior 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. This spalling is caused by the vaporization of water trapped in the concrete pores being unable to escape. Spalling is also caused by fracture of aggregate and loss of strength of the concrete matrix at the surface of the concrete after prolonged exposure to high temperatures. Reinforcing steel that is heated will lose strength. 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.

1.2.4 Materials:

Mixes for cast-in-place concrete should be specified to have a high enough slump to make placement practical. Air entrainment should be used. The moist environment in many tunnels combined with exposure to cold weather makes air entrainment important to durable concrete.

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.

1.3 Precast Segmental Lining:

1.3.1 Description:

Precast segmental linings are used in circular tunnels excavated using a 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 number of segments used to form the ring is a function of the ring diameter and to a certain respect, construction agency’s preferences. The segments are relatively thin, 20 to 30 cm and typically 1 to 1.5 m wide measured along the length of the tunnel.

Precast segmental linings can be used as initial ground support followed by a cast-in-place 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. Segments used as initial linings are generally lightly reinforced, erected without bolting them together and have no waterproofing. The segments are erected inside the tail of the TBM. The TBM pushes against the segments to advance the tunnel excavation. Once the shield of the TBM is passed the completed ring, is jacked apart (expanded) at the crown or near the spring lines. Jacking the segments helps fill the annular space that was occupied by the shield of the TBM. After jacking, contact grouting may be done for filling the annular space and to ensure complete contact between the segments and the surrounding ground. A waterproofing membrane is installed over the initial lining and the final concrete lining is cast in place against the waterproofing membrane. Horizontal and vertical curvature in the tunnel alignment is created by using tapered rings. The curvature is approximated by a series of short chords.

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. As with segments used for initial lining, horizontal and vertical tunnel alignment is achieved through the use of tapered segments.
1.3.2 Advantages of a precast segmental lining:

- 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.

1.3.3 Disadvantages of a precast segmental lining:

- 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.
- Reinforcement when used is subject to corrosion and resulting deterioration of the concrete.
- Cracking that allows water infiltration can reduce the life of the lining.
- Chemical attack in certain soil can reduce lining life.

1.3.4 Design Considerations:

1.3.4.1 Initial Lining Segments:

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. The segments are usually cast in a yard set up specially for manufacturing the segments.

When using a structural analysis program for analysis, the structural model should include hinges (points where no bending moment can develop) at the locations of the joints in the ring. Using hinges at the joint locations provides the ring with the flexibility required to adjust to the loads, resulting in the predominant loading being axial load or thrust. This is an approximation of the behavior of the lining since joints will transfer some moment. The actual
behavior of a segmental lining can be bounded by two models that have zero fixity at the joints and full fixity at the joints.

Radial joints in between segments can be flat or concave/convex. Convex/concave joints facilitate rotation at the joint, allowing the segment to deform and dissipate moments. Flat joints are more efficient in transferring axial load between segments and may result in less end reinforcement. In either case, the 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 amount of reinforcement used should consider the type of joint and the resulting load transfer mechanism. Handling and erecting the segments are also sources of damage at the joints. Reinforcing can mitigate this damage.

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.

Handling, storage, lifting and erecting the segments also impose loads. The segments should be designed and reinforced to resist these loads also. When designing reinforcement for these loads, appropriate codal provisions should be used (refer clause No. 1.7 of this chapter). 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 ground loads applied to the lining.

Initial lining segments are considered to be temporary support, therefore, long term durability is not considered in the design of the linings.

1.3.4.2 Final Lining Segments:

Segments used as a 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. Appropriate codal provisions should be used to design the segments. The segments are manufactured in a yard set up specifically for manufacturing the segments.
Since the segments are cast and cured in a controlled environment, higher tolerances can be attained than in cast-in-place concrete construction.

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 axis 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.

Joint design should consider the configuration of the gaskets. The gasket can eliminate much of the bearing area for load transfer between joints. 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. 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 be damaged and require replacement. These forces are unique to each tunnel and are a function of the ground type and the operational characteristics of the TBM. Reinforcement along the jacking edges of the segments may be required to resist this force. The segments should be checked for bearing, compression and buckling from TBM thrust loads.

Lifting and erecting the segments also impose loads. The segments should be designed and reinforced to resist these loads. When designing reinforcement for these loads, the provisions of relevant codes should be used.

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.

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

Concrete behavior 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 to emergency response personnel responding to the incident. This spalling is caused by the vaporization of water trapped in the concrete pores being unable to escape. Spalling is also caused by fracture of aggregate and loss of strength of the concrete matrix at the surface of the concrete after prolonged exposure to high temperatures. Reinforcing steel that is heated will lose strength. 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.

1.3.5 **Materials:**

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. These strengths are easily attainable in precast shops and casting yards. Curing is performed by in enclosures and is well controlled. 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.

Steel fiber reinforced concrete has become a topic of discussion and research for precast tunnel linings. Theoretically, steel fibers can be used in lieu of steel reinforcing bars. The fibers can potentially eliminate the need for fabricating the steel bars to very tight tolerances, provide ductility for the concrete and make the segments tougher and less damage prone during construction. Unfortunately, there is no design code for the design of steel fiber reinforced concrete. The recommended practice, until further research is conducted and design codes are developed, is to use steel fibers in segments where the design is conducted and the lining is found to be adequate without reinforcing. The steel fibers then can be included in the concrete to improve handling characteristics during construction. A testing program is required by the specifications to have the contractor prove via field testing that the fiber reinforced segments can withstand the handling loads imposed during construction. The fibers then can be used lieu of reinforcement that would be installed to resist the handling loads.
Concrete mixes for one pass lining segments have strengths ranging from 35 to 48 MPa. Higher strengths are easily obtainable in precast shops and assist in resisting handling and erection loads.

### 1.4 Lattice Girders and Rolled Steel Sets

Lattice girders are lightweight, three-dimensional steel frames typically fabricated of three primary bars connected by stiffening elements. Lattice girders are used in conjunction with shotcrete and once installed locally act as shotcrete lining reinforcement. Lattice girders are installed to provide:

- Immediate support to the ground (in a limited manner due to the low girder capacity)
- Control of tunnel geometry (template function)
- Support of welded wire fabric (as applicable)
- Support for fore poling pre-support measures.

The girder design is defined by specifying the girder section and size and moment properties of the primary bars. To address stiffness of the overall girder arrangement the stiffening elements must provide a minimum of five percent of the total moments of inertia. The arrangement of primary bars and stiffening elements is such as to facilitate shotcrete penetration into and behind the girder, thereby minimizing shadows.

In particular cases where, for example, immediate support is necessary for placing heavy spilling for pre-support, the use of rolled steel sets may be appropriate. In such instances steel sets are used for implementation of contingency measures.

### 1.5 Ribs and Lagging

Ribs and lagging are not used as much now as they were a couple of decades ago. However, there are still applications where their use is appropriate, such as unusual shapes, intersections, short starter tunnels for TBM, and reaches of tunnel where squeezing or swelling ground may occur.

#### 1.5.1 Design Issues:

The design approach typically assumes the ribs are acted upon by axial thrust and by bending moments, the latter a function of the spacing of the lagging or blocking behind the ribs. In today’s applications, steel ribs are often installed
with shotcrete being used instead of wood for the blocking (lagging) material. When shotcrete is used, it often does not fill absolutely the entire void between steel and rock. Hence, with properly applied shotcrete appropriate maximum blocking point spacing be taken in the design.

1.6 **Principles of Ground Structure Interaction:-**

1.6.1 **Structural Design Issues (General):-**

(i) The processes of ground pretreatment, excavation and ground stabilization after the pre existing state of stress in ground before lining comes in contact with ground.

(ii) A 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 are ill defined and its behavior is governed by properties of surrounding ground.

(iii) Design of tunnel lining is basically not a structural problem but a ground-structure interaction problem with ground playing predominant role.

(iv) Tunnel lining behavior is a four dimensional problem. During construction, ground conditions at tunnel head involve transverse as well as longitudinal arching, cantilevering from unexcavated face. Most importantly all ground properties are time-dependent especially in short term, leading to commonly observed phenomenon of stand-up time. Thus timing of lining installation is an important variable.

(v) The most serious structural problems encountered with actual lining behavior are related to absence of supports due to inadvertent voids left behind lining rather than on intensity and distribution of load.

(vi) In most of the cases the bending strength and stiffness of structural lining are small compared with those of the surrounding ground. The change in lining properties does not significantly affect deformation of lining, which primarily depends on ground properties. Therefore, lining behavior should be judged by adequate ductility to conform to imposed deformation rather than strength to resist bending stresses.

(vii) Structural analysis of tunnel lining has been a subject of numerous papers and theories. Great disparity of opinion exist on accuracy and usefulness of these analyses eg. Beam-spring models, three dimensional models, (more
sophisticated analysis), empirical methods (for soft ground) (2D general purpose structural analysis program) and numerical methods (modeling lining and surrounding ground as a continuum using 3D FEM).

1.6.2 **Structural Design Issues: NATM**

1.6.2.1 **Ground-Structure Interaction:**

The NATM realizes excavation and support in distinct stages with limitations on size of excavation and length of round followed by the application of initial support measures. In particular the shotcrete lining has an interlocking function and provides an early, smooth support. To adequately address this sequenced excavation and support approach the structural design shall be based on the use of numerical, i.e. finite element, finite difference, or discrete element methods. These numerical methods are capable of accounting for ground structure interaction. They allow for representation of the ground, the structural elements used for initial and final ground support, and enable an approximation of the construction sequence.

1.6.2.2 **Numerical Modeling:**

- **Two (2)-Dimensional and Three (3)-Dimensional Calculations:**
  In general, use of two-dimensional models is sufficient for line structures. Where three-dimensional stress regimes are expected, such as at intersections between main tunnel and cross passages, or where detailed investigations at the tunnel face are undertaken, three-dimensional models should be used.

- **Material Models:**
  In representing the ground, the structural models shall account for the characteristics of the tunneling medium. Material models used to describe the behavior of the ground shall apply suitable constitutive laws to account for the elastic, as well as inelastic ranges of the respective materials.

1.6.2.3 **Ground Loads – Representation of the NATM Construction Sequence:**

Tunnel excavation causes a disturbance of the initial state of stress in the ground and creates a three-dimensional stress regime in the form of a bulb around the advancing tunnel face.

It is important to portray excavation and support sequencing closely in the numerical analyses. For shotcrete lining structural assessments it is important to
distinguish between a “green” shotcrete when installed and when it has hardened to its 28-day design strength. Green shotcrete is typically simulated using a lower modulus of elasticity in the computations. A value of approximately 1/3 of the elastic modulus of cured shotcrete is commonly used to approximate green shotcrete in 2-D applications. In 3-D simulations the shotcrete may be modeled with moduli of elasticity in accordance with the anticipated strength gain in the respective round where it is installed.

The installation of the final tunnel lining generally occurs once all deformations of the tunnel opening have ceased. To account for this fact the calculations perform installation of the final lining into a stress-free state. The final lining becomes loaded only in the long-term resulting from a (partial) deterioration of the initial support (shotcrete initial lining and rock bolts in any), rheological long-term effects and ground water if applicable. Although modeling of the final lining is often undertaken by embedded frame analyses its analysis within a ground-structure interaction numerical model will be most appropriate and can follow directly after the initial support is installed.

In addition to the ground loads, the concrete lining will be loaded with hydrostatic loads in un-drained or partially drained waterproofing systems. This load case generally occurs well before the final lining is loaded with any ground loads and shall be considered separately in the calculations.

Final lining calculations consider the existence of the waterproofing system, which is embedded between the initial shotcrete lining and the final lining. A plastic membrane will act as a de-bonding layer in terms of the transfer of shear stresses. Therefore simulation techniques should be used to simulate this “slip” layer. This is accomplished by only allowing the transfer of radial forces from the initial lining onto the final lining.

1.6.2.4 **Lining Forces:**

Section forces and stresses for beam (2-D) or shell (3-D) elements are ascertained from modeling. Section force and moment combinations are used to evaluate the capacity of the initial shotcrete and final concrete linings using accepted concrete design codes. (Refer Clause No. 1.7 of this chapter).

Based on this evaluation the adequacy of lining thickness and its reinforcement (if any) is assessed. If the selected dimensions are found not to be adequate then the model must be re-run with increased dimensions and/or reinforcement. The process is an iterative approach until the design codes are satisfied.
These calculations do not distinguish between the type of lining application and therefore shotcrete and cast-in-place final linings are treated in the same manner within the program using the material properties and characteristics of concrete.

1.7 **Application of Codes:**

Structural codes should be used cautiously. Most codes have been written for above ground structures on the basis of assumptions that do not consider ground-lining interaction. Accordingly, the blind application of structural design codes is likely to produce limits on the capacity of linings that are not warranted in light of the substantial contributions from the ground and the important influence of construction method on both the capacity and cost of linings. American Road Tunnel Manual specifies use of American Concrete Institute (ACI), ACI 318 and AASTHO LRFD Bridge Design Specifications with mention of use of German Industry Standard DIN 1045 for cast-in-place concrete final lining by some authorities.
Chapter 8: Instrumentation & Monitoring

1.0 General

It is recommended that a geo-technical instrumentation and monitoring program should be planned as an integral part of the tunneling work. For such a program to be successful, it is essential that following basic requirements are met:

- must be carried out for well-defined purposes
- must be well planned & executed
- must be well supported by competent staff throughout the construction period (& subsequent period if required) for:
  - analysis & interpretation of data/results from the monitoring program
  - taking decision for the required follow up action/corrective action based on data/results from the monitoring program

Thus, the essential components of a successful instrumentation & monitoring program are:

- Definition of need and objective.
- Planning and design.
- Execution of program.
- Interpretation of data.
- Action based on monitoring results.

All of these components must be carefully planned in advance. If the data obtained cannot be properly interpreted in a timely fashion, or if no action is foreseen to be taken based on the data, the instrumentation program will have no purpose and should not be implemented.

1.1 Purpose of instrumentation and monitoring program

Geotechnical instrumentation & monitoring programs are carried out for one or more of the following purposes:

1. Characterization & determination of initial site conditions

2. **Design & Design Verification** :-

- NATM monitoring of displacements and loads is an essential part of the construction process, providing input to the ongoing process of design and verification during construction.
- Where initial ground support is selected based on conditions encountered, monitoring can verify the adequacy of the support and indicate if more support is required.
May be used to obtain data from pilot tunnels, or shafts which can be used for design of tunnel.
Decisions regarding design of final lining may be taken.

3. **Construction Control:**
   - Early monitoring during construction can help in planning of later construction procedures or help decide whether contingency plans need to be used.
   - Monitoring can be used to diagnose flaws in the contractor’s construction methodology and indicate the required improvements.

4. **Safety:** In the process of determining the adequacy of ground support, monitoring also serves a safety function, warning of the potential for ground failure.

5. **Regulatory/Environmental requirements:** Monitoring may be required to ensure compliance with regulatory/environmental requirements (e.g., groundwater lowering, ground settlements, vibrations)

6. **Performance Monitoring:** Instruments are used to monitor the in-service performance of a structure. For example, monitoring loads on rock bolts and movements within a tunnel can provide an indication of the stability of tunnel.

7. **Contractual Documentation:** Monitoring data can also be used for avoiding/settling disputes between the Railways and the contractor/s.

### 1.2 Items to consider in planning a successful Instrumentation and Monitoring Program -

1. **Define the project conditions:** The official/s planning and designing should be/should become familiar with project conditions including type and layout of the tunnel or shaft, subsurface stratigraphy and engineering properties of subsurface materials, groundwater conditions, status of nearby structures or other facilities, environmental conditions, and planned construction method.

2. **Predict 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. For example, if the purpose is to monitor safety, hypotheses must be established for mechanisms that could lead to rock or support failure.

3. **Define the purpose of the instrumentation and monitoring and the questions that need to be answered:** Every instrument should...
be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation.

4. **Select the parameters to be monitored**: The parameters that are typically monitored are:
   - Convergence
   - Crown settlement
   - Floor heave
   - Load in rockbolts/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

   It is important to consider which parameters are most significant for each particular situation. For example, if the question is “Is the support overloaded?”, stress or load in the support is likely to be the primary parameter of interest. However, recognizing that stress is caused by deformation of the rock, it may also be necessary to monitor deformation. By monitoring both cause and effect, a relationship between the two can often be developed, and action can be taken to remedy any undesirable effect by removing the cause.

5. **Predict magnitudes of change and set response values for action to be taken**: An estimate of the maximum possible value or the maximum value of interest will determine the instrument range, and the minimum value of interest determines the instrument sensitivity or accuracy. Accuracy and reliability are often in conflict since highly accurate instruments may be delicate and/or fragile. A predetermination should be made of instrumentation readings that indicate the need for remedial action. The concept of green, yellow, and red response values is useful. Green indicates that all is well; yellow indicates the need for cautionary measures including an increase in monitoring frequency; and red indicates the need for timely remedial action.

6. **Devise remedial actions and arrange for implementation**

7. **Assign duties and responsibilities for all phases**: Duties during the monitoring program include planning, instrument procurement, calibration, installation, maintenance, reading, data processing, data presentation, data interpretation, 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.
1.3 Instrument selection and locations:

(1) Reliability is the most desirable feature when selecting monitoring instruments. Lowest first cost of an instrument should not dominate the selection of an instrument. A comparison of the overall cost of procurement, calibration, installation, maintenance, reading, and data processing of the available instruments should be made. The least expensive instrument may not result in least overall cost because it may be less reliable since cost of the instruments themselves is usually a minor part of the overall cost.

(2) Users need to develop an adequate level of understanding of the instruments that they select and often benefit from discussing the application with the manufacturers staff before selecting instruments. During the discussions, any limitations of the proposed instruments should be determined.

(3) Choosing locations for the instruments should be based on predicted behavior of the tunnel or shaft. The locations should be compatible with the questions and the method of analysis that personnel will use when interpreting the data. A practical approach to selecting instrument locations involves following three steps:

- First, identify areas of particular concern, such as structurally weak zones or areas that are most heavily loaded, and locate appropriate instrumentation.

- Second, select zones where predicted behavior is considered representative of behavior as a whole. These zones are regarded as primary instrumented sections.

- Third, because the primary zones may not be truly representative, 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.

1.4 Plan recording of factors that affect measurements:

(1) For proper interpretation of virtually all site instrumentation data, it is essential to monitor and record site activities and climatic conditions that can have an effect on the measurements obtained. These include at least the following:

- Progress of excavation (e.g., distance of advancing tunnel face from installation).
Excavation of adjacent openings, including effects of blasting.
Installation of lining or other ground support.
Installation of drains or grouting.
Unusual events (ground instability, excess water inflows, etc.).
Continued monitoring of groundwater inflow into the tunnel.

(2) Usually, variations in the geology or rock quality have a great effect on monitoring data. While it is generally recommended to map the geology along an important underground facility during construction, it is especially important in the vicinity of extensive monitoring installations.

(3) **Establish procedures to ensure 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.

(4) **Plan redundancy, regular calibration and maintenance**

(5) **Plan data collection and data management:** The plan of data collection includes details about frequency of readings, data transmission and data storage. Readings may be taken at intervals, continuously (real time), depending on specific construction stages or time events. Written procedures for collecting, processing, presenting, interpreting, reporting, and implementing data should be prepared before instrumentation work commences in the field.

1.5 **Typical Instruments for monitoring tunneling work:**

The most important parameters to be monitored can be subdivided in following groups:

- Deformations (displacements, strains, changes in inclination etc.)
- Stresses
- Forces on structural elements (bolt force, normal load on a compression element or steel arch)
- Piezometric levels

1.6 Following instruments are typically used to measure the above parameters:

1. Bore hole extensometer to measure deformations of rock mass and adjacent or surrounding soil.
2. Load cell to measure load in rock bolts
3. Load cell to measure load in ribs and struts
4. Strain gauge to measure strain in ribs and struts
5. Strain gauge to measure strain in shotcrete
6. Shotcrete stress cell to measure stress in shotcrete
7. Piezometer to measure pore water pressure around tunnel
8. Convergence measurement by mechanical methods-tape extensometer
9. Convergence measurement by optical methods- Prism Targets
10. Inclinometer and magnetic settlement devices: to monitor lateral movements (x and y directional movements) and settlements (z directional movements) around a tunnel excavation or on a slope.
11. Measuring Anchor or rock bolt extensometer to measure distribution of load exerted on grouted rock bolts
12. Geophone: to determine seismic response

A typical arrangement of installation of instruments for a tunneling project is shown below:-

2.0 Organizational Issues

In contract documents, responsibilities for installation and commissioning, calibration, monitoring, information flow, data interpretation and reporting etc. must be clearly defined.

It may be advantageous to appoint an independent monitoring contractor who carries out the monitoring work and, on a real-time basis, delivers the results to all parties involved in the execution of the tunnel project (Railways, designer, contractor, Proof Consultant etc). The evaluation of the monitoring results should also be carried out on a real-time basis by a suitably experienced third party (either the Railways with their Proof Consultant or the design engineer responsible for the detailed design of the tunnel). The contract conditions should provide the necessary empowerment to persons to implement immediate works as per the results of the monitoring.
Chapter 9: Drainage

The planning and design of a tunnel drainage system should begin from geotechnical exploration phase itself with an assessment of the potential sources and volumes of water expected during and after construction.

The drainage arrangements for the keeping off and removing of water may be classified broadly in following categories:

i. **Pre-drainage** or preventing the entry of excess water from entering the tunnel before starting the tunnel construction work. This includes arrangements like ground water pumping, curtain grouting, specialty chemical grouting etc.

ii. **Dewatering of the tunnel** i.e. removing the water that has entered the tunnel during the construction of tunnel. During construction, water may come in the tunnel from the following sources:

- Wash water, which is used for washing drill holes and water from other construction related activities
- Ground or sub-soil water

Dewatering pumps (located at sumps created for the purpose) are normally used for dewatering of tunnels during construction if the natural drainage cannot be achieved. Water from open approach areas should not be allowed to enter the tunnel either by providing side drains taking water away from tunnel face or by use of dewatering pumps.

iii. **Permanent drainage** i.e. removing the water from the tunnel after its completion. After construction, water may come in the tunnel from the following sources:

- Tunnel seepage (through tunnel lining)
- Weep Holes
- From open approach areas
- From openings to surface

Drainage can be accomplished either by a gravity flow system or a pumped system; the former being the preferred option. Suitable drainage system needs to be designed for and provided in the tunnel for the estimated quantity of flow. Weep holes may be constructed through the tunnel lining to tap the ground water. Geosynthetic based drainage system may also be used for drainage as well as sealing purposes. Lining of a sealed tunnel has to be designed appropriately.
Chapter 10: Ventilation, lighting & Fire safety

1.1 Tunnel Ventilation

Ventilation is required in railway tunnels primarily to:

1. To remove the heat generated by locomotive units
2. To flush the tunnel of air contaminants

Ventilation can be either in form of natural ventilation or mechanical/forced ventilation.

Ventilation systems (both temporary & permanent) shall be designed to supply sufficient fresh air to all parts of the tunnel; maintain exhaust pollutants within prescribed limits; to provide adequate visibility from the accumulation of smoke & to provide a hazard free environment for tunnel users.

Many factors have an influence on the choice of ventilation system and should be taken due account of in accordance with their relative importance to a particular scheme. The end result for a ventilation system shall be an aerodynamically sound system, one that provides satisfactory environmental conditions inside the tunnel and adjacent to it, controls smoke in the event of fire & has acceptable capital and running costs while providing ease of operation & maintenance.

The parameters of a suitable and economical ventilation system for a particular tunnel layout will be determined by:

- The purpose, length, gradient, cross-section, topography and general configuration of the tunnel.
- Its location and general impact on the local environment, Environmental considerations at portals and shaft outlets (if provided)
- Traffic volume & direction, number of tracks, speed & traffic Composition
- Type of traction
- Fires and their likely severity
- Capital investment and running costs, maintainability.

1.2 Tunnel lighting

Tunnel lighting is required for the following purposes:

- To provide sufficient illuminance to enable passengers and staff to detrain and to make their way safely out of the tunnel
- 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.

- To let the inspectors or workers at track level could clearly see the track elements/condition or go through their routine inspections without using flashlights.

1.2.1 Design of tunnel lighting systems: The design of the lighting system should also:

a) Take full account of the possible conditions of the tunnel following an emergency (for example, fire and smoke)
b) Consider when and how the lighting should function. Options include:
   i) Permanently switched on
   ii) Switched on automatically following an incident or condition
   iii) Manually controlled
c) Consider how the maintained illuminance will be provided in the event of failure of the normal power supply.

1.3 Fire Safety in Tunnels

Safety in the event of a fire is of paramount importance in a tunnel. For planning purposes, it is important to understand the fire safety issues/risks relevant for a tunnel and to make appropriate plans for the same based on risk perception. Some of the key aspects to be considered are:-

- Fire risk mitigation plan: Sources of flammable materials and potential sources of ignition should be identified. A plan to minimize risk should be drawn up
- Choice of Structural Elements
- Fire-detection and surveillance systems, including heat and smoke monitoring Systems
- Communication System
- Traffic Control Systems
- Fire Protection (i.e., fire hydrants, water supply, portable fire extinguisher, etc.)
- Emergency Egress: Emergency walkways, refuges & exits and Emergency lighting
- Emergency response plan.
Chapter 11: Tunnel Survey

1.0 Tunnel Survey

Surveying plays a major role in the overall engineering and construction of tunnels right from the planning stage to the final completion. Good competence in Surveying is necessary for:

- Proper initial planning
- Integration of geo-technical and geographical data with topographical mapping (and utility mapping if located in urban area)
- Actual alignment and guidance of tunnel, adit and shaft construction

It is therefore necessary that survey engineers should be involved right from the inception of planning stage to design and final construction stage. The results of these surveys would provide more cost-effective existing-conditions data, ranging from topographic mapping to detailed utility surveys; the use of appropriate coordinate systems tailored to meet the specific needs of the project; optimized alignments; more accurate surface and subsurface horizontal and vertical control net-works properly tied to other systems and structures; precise layout and alignment of tunnel, adit & shaft.

1.1 Stipulations recommended to be included in contract documents:

Except in rare instances, the contractor should be entrusted with responsibilities for all surveying conducted for the construction work, 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.

It is therefore very important that provisions of contract documents are properly 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. This includes the following:

1. Mathematized line and grade drawings, overlain on profiles and topography from the mapping efforts. Designers will use a working line as a reference, usually the invert of the tunnel or some other defined line.
2. All parts of the cross section along the tunnel are referenced to the working line.
3. Drawings showing alignment posts and benchmarks to be used as primary controls. These should be verified or established for the project.
4. Drawings showing existing conditions as appropriate, including all affected utilities, buildings, or other facilities.
5. Interfaces with other parts of the project, as required.
6. Specifications stating the accuracy requirements and the required quality control and quality assurance requirements, including required qualifications for surveyors. Minimum requirements to the types and general stability of construction benchmarks and alignment posts may also be stated.

7. Benchmarks and alignment posts are sometimes located where they may be affected by the work or on swelling or soft ground where their stability is in doubt. Such benchmarks and alignment posts should be secured to safe depth.

8. Where existing structures and facilities may be affected by settlements or groundwater lowering during construction, preconstruction surveys should be conducted to establish a baseline for future effects. Such surveys should preferably be supplemented by photographs.

### 1.2 Surveying Steps in alignment control of Tunnels:

Setting out of centre line of tunnel at exact location and elevation is to be done in various steps as mentioned below:

(i) Establishment of temporary benchmarks and alignment posts: The contractor’s surveyor will establish temporary benchmarks and alignment posts as required for the work and will ensure their stability. In case stability of temporary benchmarks and alignment posts is in danger or they have been disturbed, contractor shall take prompt measures to transfer & re-establish these temporary benchmarks and alignment posts. Proper documentation for the same should be maintained.

(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: care should be taken to eliminate cumulative error

(v) Transferring levels underground (underground leveling): care should be taken to eliminate cumulative error

Railway officials should be properly trained in operation of surveying equipments (particularly the Total Station; which is presently the most commonly used surveying equipment for alignment control of tunnels) GPS is not suitable for locations where GPS antenna cannot be exposed to the satellite via direct line of sight. As such, GPS is not used for survey in interior of tunnels.

Theodolites, Total Stations & 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.
Chapter 12: Problems in Tunnelling

1.0 Common problems encountered during tunneling are:

- Daylighting,
- Underground collapse,
- Rock burst,
- Inrush of water and
- Portal collapse

These are briefly described below:

- **Daylighting**: In this case the ground is unravelled to the surface, mainly due to erosion by underground water. Propagation of the failure to the surface can be extremely quick. This is often termed as Chimney/cavity formation. Daylighting is normally caused due to reason/s like weakness in the crown of a tunnel, insufficient cover to overlaying permeable water bearing strata, insufficient cover to surface or insufficient cover to overlaying deposit materials.

![Fig: Daylighting due to weakness in the crown of a tunnel](image1)

![Fig: Daylighting due to Insufficient cover to overlaying permeable water bearing strata](image2)
Underground collapses: This type of collapse does not reach the surface. These can be subdivided into:-

- Roof collapse: breaking down and caving in of competent or loose rock at the tunnel roof.
- Stope: local progressive upward roof collapse in thick but fractured rock strata.
- Side-wall caving: Breaking and caving in of the side walls of a tunnel - takes place often in conjunction with roof caving.
- Heading hurrying on ahead of the driving: affects the area of several meters in advance of the heading & involves caving of the roof as well as caving of the side walls- occurs mainly during tunneling by mechanized means or as a result of standstill or disruptions in work.

Rock burst: Rock burst is a term used to describe rock failures ranging in magnitude from the explosion of small fragments of rock from underground excavation faces or side walls to sudden collapse of a large section of a
tunnel or an excavation. A burst is defined as a sudden and violent explosion of rock in or around an excavation. Failure is normally associated with high stress and brittle or brittle-elastic materials.

The occurrence of rock bursts in deep mining tunnels is well known and requires special precautions during construction and adoption of appropriate support measures. Basically, the most common form of rock burst manifestation is the sudden ejection into the excavation of fractured and detached rocks from the tunnel periphery due to a rapid strain relief process.

**Ingress of water / Inrush of water:** Water inflow causes various degrees of difficulty in tunnels. 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 ingress of water in underground excavations may result from geologic and manmade conditions. Groundwater inflow influences the construction procedure, tunnel stability, and the environment.

**Portal collapses:** The portal areas frequently represent some of the most problematic points during the excavation of a tunnel. Several factors, for example the direction of excavation, the morphology of the site, the geomechanical 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.

### 2.0 Common causes:

**2.1 Geological & Geo-mechanical conditions:**

a) **Rock alteration:** The natural processes of weathering produce rock alteration which can be of major importance to tunnelling. Weathering reduces the strength of the rocks and can extend to considerable depths by the action of groundwater movement. Pockets of highly weathered
rocks usually contain water and can be under appreciable hydrostatic pressure head. Consequently they can possess the ability to rapidly flow into an excavation if disturbed by underlying or adjacent tunneling activities.

b) **High rock stress**: Instability due to excessively high rock stress is also generally associated with hard rock and can occur when mining at great depth or when very large excavations are created at reasonably shallow depth. Tunnelling in steep mountain regions or unusually weak rock conditions can also give rise to stress-induced instability problems.

c) **Faults**: Collapses due to adverse structural geology tends to occur in hard rocks which are faulted and where several sets of discontinuities are steeply inclined.

d) **Joints**: Where a rock fracture results in no significant visible displacement at the plane of fracture, then this is commonly referred to as a joint. Their frequency and orientation are related to the nature of the stress field with tensional and compressive states coupled with folding and faulting playing important roles. Appreciable intensification of jointing can be expected in close proximity to the axis of severely folded rocks and adjacent to major faults. Joint patterns as observed at surface rock exposures may not necessarily exhibit close similarity with those encountered in the tunnel, with the exception of the portal areas. Joints observed in surface rocks may have developed primarily due to climatic and weathering effects, as for example, due to expansion and contraction cycles, and therefore are most likely to be of limited depth of penetration below the surface.

Essentially, joint patterns represent structural weaknesses in rock masses, and can substantially influence the stand-up time of different rock types. They are likely to influence the mode of rock failure and character of its collapse potential during tunnel excavation operations. Stability problems in blocky jointed rock are generally associated with gravity falls of blocks from the roof and sidewalls. Consequently, the joint patterns require special considerations when giving attention to the choice and application of support and particularly for temporary measures. Rock joint patterns should be taken into account at the tunnel design stage and when considering selection of the permanent support system.

e) **Folded rock masses**: The most common form of deformation of rock masses is that of folding and is especially conspicuous in layered rock structures, although folding occurs in all rock types. Folds occur on widely differing scales ranging in wavelengths from the order of centimetres to kilometres. The intensity of folding reflects the degree of localized distortion and relative slipping within different parts of the affected rock mass. Folded strata allow natural traps to form which attract accumulation in significant quantities of natural gas and water. Severely distorted folds are frequently accompanied by plastic behaviour of rocks especially in the softer sediments and in metamorphic rocks.
f) **Groundwater aspects**: The presence of groundwater in very large quantities is recognized as a major hazard in addition to causing operational difficulties in respect of tunnel construction works. Potential problems from groundwater inflow during tunnelling can be predicted to a large extent in many situations by a comprehensive site investigation employing deep boreholes. Predicting with accuracy the likely water inflow quantities is, however, difficult, and detailed monitoring and regular review of conditions together with the adoption of special measures such as de-watering or injection programs need consideration.

g) **Karst formations**: Karst development is the occurrence of solution features in limestone and related carbonate rocks, and gives rise to natural caverns and sinkholes through which water can flow.

h) **Squeezing ground conditions**: Squeezing ground commonly refers to weak, plastic rock materials which displace into the tunnel excavation under the action of gravity and from the effect of stress gradients around the tunnel opening (Barla, 2002).

Plastic and semi-plastic rocks which are sensitive to deformation and failure at relatively low stress levels are likely to exhibit squeezing behaviour. Squeezing rock slowly advances into the tunnel without perceptible volume increase. It has been noted that visible deformation of the tunnel boundary is most obvious in the portion below springing line. Heaving of invert is commonly observed especially in non-circular tunnels.

Squeezing occurs when the unconfined compressive strength of the rock mass is less than the increased tangential stress (due to tunnel excavation). Rate of squeeze and rock loads are somewhat dependent on tunnel size and rate of advance. It is essential in squeezing (or swelling) conditions to establish a program of convergence point installation that will be used to monitor the amount and rate of movement of the tunnel walls. This information would be useful in determining the support requirements as tunneling progresses.

Except in extreme conditions, squeezing is almost always self-limiting and will not occur vigorously, or at all, once the intruding material has been removed. It may well be that the re-excavation of a failed tunnel opening will encounter much less difficult conditions.

An empirical approach can be used for predicting ground conditions (Goel et al., 1995b). This approach suggests that squeezing ground will be encountered when:

\[ H \cdot a^{0.1} > 260 N^{0.33} \]

where,

- \( H \) = tunnel depth in metres,
- \( a \) = tunnel radius in metres,
- \( N \) = rock mass number (i.e., Barton’s Q keeping SRF = 1),
\[ = \frac{\text{RQD}/J_n}{J_r/J_a}J_w \]

- **RQD** = rock quality designation,
- **Jn** = joint set number,
- **Jr** = joint roughness number,
- **Jw** = joint water reduction factor

Prediction of the squeezing ground condition alone is not enough. It is also necessary to ascertain the expected support pressure and the tunnel closure for effective handling of squeezing ground. Once the optimum desirable closure is known, flexible supports can be designed to absorb it and accordingly the excavation size can be increased so that re-excavation is not required to accommodate the closure.

1. **Swelling ground conditions**: Swelling behaviour denotes the response of rock to the presence of water. Swelling phenomena are generally associated with argillaceous soils or rocks derived from such soils. Rocks which are rich in clay minerals are likely to have pronounced swelling characteristics.

In the field it is quite hard to distinguish between squeezing and swelling especially since both the conditions are often present at the same time. Swelling may continue as long as free water and swelling minerals are present.

The squeezing ground can be distinguished from swelling ground on the basis that under the former, the movement is visible as soon as the excavation is made, on other hand, swelling ground is relatively slow process and the time factor is involved.

Many European railway & highway tunnels constructed in swelling susceptible formations are still being periodically repaired even a century after their construction. Since the swelling will not passivate in the same way as squeezing generally will in rock, tunnel supports must be designed to resist the swelling pressure.

2. **Sloping ground surface conditions**: Instability is sometimes caused due to the nature of the sloping ground surface overlying the tunnel location. Such sloping ground may be inherently unstable without any disturbance created by construction of the tunnel. In other situations, however, the tunnel may introduce changes in the stability conditions of the slope. Groundwater conditions may also change. The tunnel excavation may prove to be a significant weakening influence in the proximity of a potential landslip area.

2.2 **Inadequate design and specifications**: e.g. Inappropriate support system, inadequate specification of construction materials and of tolerances on profiles etc.

2.3. **Mistakes at the construction stage**: e.g. use of sub-standard materials and tools, violation of design assumptions, specification requirements, lining
not constructed to specified thickness, insufficient shotcrete strength and thickness, belated placing of the means of support.

2.4 Interruptions & breaks

2.5 Delayed decision making

Tunnels require the 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 can be developed to overcome these uncertainties and permit safe tunnel construction, but their successful application requires the proper management of complex technical issues.

Every collapse requires careful analysis because, on the one hand, the cost questions have to be settled, and on the other hand, conclusions for the continuation of work can be drawn from the knowledge gained in this way. Usually, the problem results from a combination of several unfavourable factors.

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