الجمهوريـــة الجزائريـــة الديمقراطيـــة الشعبيـــة
People's Democratic Republic of Algeria
وزارة التعليم العالي و البحث العلمي
Ministry of Higher Education and Scientific Research
جامعة حسيبة بن بوعلي- الشلف
Hassiba Benbouali University of Chlef
كلية الهندسة المدنية و المعمارية
Faculty of Civil Engineering & Architecture
الهندسة المدنية
Civil Engineering Department

Underground Structures Handout

Course and Exercises

Field of Study: Public Works
Specialty: Roads and Engineering Structures
Level: Master II
Year: 2025

Dr. Zohra Boutaraa
Lecturer (MCA) - Civil Engineering Department

SUMMARY

FOREWORD

Underground structures, especially tunnels, are essential elements of civil engineering, particularly within the public works sector, which has been expanding consistently over the last few decades and continues to grow both in our country and around the world. As a result, students in this field are expected to master this aspect of their studies and to be knowledgeable about the techniques involved in excavation, construction, calculation, and maintenance.

In this context, this handbook is dealing with underground structures, initially intended for Master 2 students specializing in Public Works, but is also relevant for professionals in the civil engineering field, such as technicians and engineers. The included chapters provide readers with the knowledge necessary for conducting a dimensioning study of tunnel linings. This handbook is prepared according to the Algerian Ministry of Higher Education and Scientific Research's established syllabus. However, the author has made a few modifications, such as combining two chapters into one.

The design of underground projects fundamentally hinges on reconnaissance recommendations regarding the geology, hydrogeology, and geotechnical attributes of the soil that constitutes the excavation walls. Thus, a comprehensive understanding of rock mechanics is imperative for students to fully appreciate the course content. The handbook, therefore, articulates the principles, primary methodologies, and tools that students can employ to design and dimension underground structures, including tables, figures, and abacuses. At the end of the four chapters, examples of exercises along with their solutions are provided to facilitate a deeper understanding of the program's content.

In this light, this handbook is the outcome of the teacher's personal endeavors and the synthesis of various scientific research articles and books published on websites relevant to the topic. After a wide glossary defining tunnel terminology, the handbook gives definitions and recaps of tunnels and underground structures that were discussed during the fifth semester of the bachelor's degree program. Additionally, it acts as a reference guide that encompasses the chapters of the official curriculum. Nonetheless, interested readers are encouraged to further investigate the various concepts by consulting the bibliography at the end of the handbook.

At the end of this "Underground Structures" handbook, students will have acquired the necessary knowledge relevant to the field of underground construction. This includes essential geotechnical and rock mechanics concepts applied to underground projects. They will understand the computational methods typically employed and will also gain insights into these approaches. Accordingly, students will also obtain practical experience with pre-dimensioning calculations using semi-empirical and analytical methods, such as the convergence-confinement method. Additionally, students will be introduced to numerical calculations through mini-projects and assignments that they must complete during tutorials.

Finally, I would like to express my deepest gratitude to Professor *Badreddine Sbartai* from *Badji Mokhtar* University of Annaba, and Doctor *Abdallah Cherif Taiba* from *Hassiba Benbouali* University of Chlef, who both reviewed this manuscript for any suggestions they may have made, along with their comments and counsels for improving the content of this handbook.

GLOSSARY

Advance Flight: *Length of land excavated in a single phase*

Anchor Bolt: *Bar, usually made of steel, used for the stabilization of rock. Placed into a drilled hole and anchored in the rock at the end (end anchorage) or along its whole length (grouted bolt). The visible extremity is often fitted with a plate.*

Axis: *The lengthwise course of a tunnel, especially along the center line.*

Backfill: *Material placed around the sides and over the top of the tunnel within excavated trench after the tunnel is installed in the trench.*

Bench : *Starting point of the vault and the vault-wall connection.*

Bore: *construction method for tunnels which involves digging a tube-like passage through the earth. Usually refers to mountain tunneling.*

Blasting chamber: *An area of the shield in contact with the face, where the material is blasted.*

Caisson: *A shaft of concrete or steel which can be sunk through the ground by excavating the ground within the perimeter of the lower edge, with the rate of sinking frequently controlled by compressed air. A pressurized, bell-shaped structure which allows construction fully under the water.*

Conduit: *A pipe or liner used as a passage for other pipes or wires.*

Confinement*: Being limited in movement, view, travel, etc.*

Consolidation : *Soil stabilization.*

Convergence : *The diametric narrowing of a tunnel section, measured after excavation. Also used to designate the radial displacement of a point on the wall.*

Course: *In tunnels, the path of a tunnel. In masonry, one horizontal row of blocks.*

Cover: *Perpendicular distance to nearest ground surface from the tunnel.*

Cross Section: *The shape of a tunnel; shapes include ovoid, horseshoe, round or square.*

Crown: *The uppermost part of the tunnel.*

Cut and Cove*: construction method which involves excavating a large trench, building a roof structure, and then covering it with earth. Commonly used for subways and relatively flat locations.*

Cutting face: *Zone where excavation takes place: the temporary end of the tunnel being dug.*

Deconfinement: *The reorganization of stresses around the tunnel on either side of the working face. The ground is fully unconfined when it has reached final equilibrium.*

Dewater*: Removal of water during construction.*

Diaphragm Wall: *Watertight wall formed from cast-in-place reinforced concrete. Commonly uses slurry wall construction.*

Discontinuity: *A collective term for most types of joints, bedding planes, schistosity planes, shear and fault zones.*

Drift: *horizontal, underground passage.*

Drill and Blast: *Construction method in which pilot holes are drilled for explosive charges. The resulting debris is carried out and the process is repeated.*

Drilling: *Making a hole in the soil using a tool with a rotating and/or percussive movement.*

Embedded Wall: *Retaining wall constructed using sections placed side-by-side or interlocking to form a continuous structure.*

Erector: *Mechanical device used to install temporary or permanent support (particularly segment rings in tunnel boring).*

Extrusion: *Axial displacement towards the cavity of the unexcavated soil core located in front of the working face.*

Flight: *The step of advance of a tunnel excavated by blasting. It also corresponds to the*

Full Face Boring Machine: *A tunneling machine which has cutting teeth at its front. It creates the tunnel opening while passing the waste material through the rear. Many types of tunnel boring cut small sections which are progressively enlarged. A full face boring machine cuts the complete cross section of the tunnel at once.*

Gauge: *Cross-sectional area of the tunnel.*

Ground decompression: *The phenomenon that accompanies the modification of natural ground stresses in the area around the excavation.*

Grouting: *Material used to strengthen by the injection of chemicals, cementitious grout unstable rock and soil.*

Half-section: *Half-section tunneling involves excavating a tunnel in two staggered phases. First, the upper half-section is excavated, followed by the lower half-section (or stross).*

Hanger: *Arch-shaped support (generally made of steel: IPE, HEA, HEB, etc.) designed according to the geometry of the tunnel profile and installed against the wall to support the ground.*

Immersed Caisson: *A pressurized, bell-shaped structure which allows construction fully under the water.*

Immersed Tube: C*onstruction method using pre-fabricated tunnel sections.*

Initial Support: *Applied or installed immediately to the interior surface after excavation to maintain the opening. It includes shotcrete, rock bolts and/or steel ribs.*

Injection: *Techniques used to replace and fill voids in soil with a hardening grout. Injections serve two purposes: increasing strength and/or sealing the soil.*

Invert: *The bottom /floor of the tunnel.*

Jacking: *Construction of small diameter passages by forcing pipe through the soil.* ***Joints:*** *A break of geological origin in the continuity of a rock mass along which there has been no visible displacement.*

Key stone: *Section of the arch located in its plane of symmetry.*

Lagging: *Boards placed side-by-side to retain the face of an excavation. Held in place by soldier piles.*

Line: *A description of the location and grade of a tunnel.*

Lining: *A casing of brick, concrete, shotcrete, iron, steel, or wood placed in a tunnel or shaft to provide final everlasting bearing structure of underground space and/or to finish the interior.*

Micro Tunneling: *Construction method for tunnels which are too small for humans to dig inside. May be performed using tethered, remote-controlled drilling machines or pipe jacking.* ***Mining:*** *The process of digging below the surface of the ground to extract ore or to produce a passageway such as a tunnel.*

Muck*: Debris removed during excavation.*

Mucking: *The process of excavating a limited length of the tunnel at a time, allowing the surrounding ground to stabilize before further excavation or the installation of support structures. It also involves removing excavated rock, soil, and debris from a tunnel or shaft site.*

NATM (New Austrian Tunneling Method): C*onstruction method, also known as the shotcrete method. As the excavation progresses, a layer of concrete is immediately sprayed on the interior surface of the tunnel and rock bolts are driven in. This method attempts to maintain the original rock structure by avoiding settling, subsidence and deformation. Additional liners may be added or more layers of shotcrete may be applied.* ***Overburden:*** *The soil and rock supported by the roof of a tunnel.*

Pipe Roof Method: C*onstruction method in which steel pipes are laid along the sides and roof of the tunnel course and the tunnel was then constructed inside.*

Portal: *The entrance from the ground surface to a tunnel. Usually includes a wall to retain the soil around the opening. May also include service and ventilation building.*

Profile: *A side view of a tunnel.*

Rib: *An arched individual frame, usually of steel, used in tunnels to support the excavation.*

Rock Anchor*: Tensioned tendons anchored to the ground over a defined length.*

Rock Bolts: S*teel bar with larger plates at its head. These are driven into the interior surface of the tunnel to stabilize and add strength to the rock.*

Rock dowel*: An intentioned reinforcement element consisting of a rod embedded in a grout-filled hole and bonded to the surrounding ground along their entire length (fully bonded) either by friction or grout.*

Rock mass*: Ground mass built up by in situ pieces of rock material of which are limited by discontinuities.*

Sealing: *Operation designed to limit or eliminate the flow of water through a coating to an acceptable value.*

Segment: *A prefabricated reinforced concrete shell. Several segments form a ring, and*

Settling Trough: *A depression caused by tunnel excavation on the ground surface.*

Shaft: A *vertical, underground passage.*

Sheet Pile *– pre-fabricated sections installed vertically side-by-side to form a retaining wall.*

Shield *– a metal frame used to maintain the opening as the tunnel boring progresses. As forward progress is made, the liner is constructed behind the shield, and the shield is jacked or moved forward.*

Shotcrete: *A quick-setting concrete sprayed onto the bare rock surface immediately after excavation. It forms preliminary tunnel liner.*

Slurry Wall: C*onstruction method used in wet, unstable soil.*

Soldier Pile: *Steel H-shape beam driven vertically into the ground to provide supports for*

Springline: *The line at which the tunnel wall breaks from sloping outward to sloping inward*

Sunken Tube: C*onstruction method using pre-fabricated tunnel sections.*

Support: *Resistant structure placed in the intrados of the tunnel to limit convergence and soil*

Tailings: *In mining, waste material remaining after the valuable minerals have been*

Top heading: *upper part of a tunnel in a half-section excavation (upper section).*

Tunnel Blasting Plan: *Plan of the working face, showing drill holes, various detonator delays and micro-delays, firing lines for sequential blasting and quantities of explosives used.*

Tunnel*: An underground passage for vehicles or pedestrians, created by digging into earth.*
Tunnel Boring Machine (TBM): *A full-face excavator or tunneling machine.*

Tunneling: *All operations involved in cutting and mucking. Broadly speaking, it refers to all operations involved in creating a tunnel.*

Vault: *An enclosing structure formed by building a series of adjacent arches.*

Ventilation: *A system in which fresh air is supplied at one end of the tunnel and the polluted air is expelled at the other.*

Wall: *The side of the tunnel.*

Main Symbols

A: sliding area

B: maximum allowable bolt load

C, the cohesion of the discontinuity

D_e: equivalent dimension

E*:* rock deformation modulus

ESR: Excavation Support Ratio

F: safety coefficient

G: ground stiffness

ɣ: the volumetric weight of the soil

Is: Franklin index

P_i: wall pressure

q: load per unit area

Q: rock quality index

R.M.R: Rock Mass Rating

R.Q.D*:* Rock Quality Designation

R*:* excavation radius

R_c: rock compressive strength

R_t: rock tensile strength

SF: strength factor

T*:* shear force

UCS: Uniaxial Compressive strenght of intact rock core

V_p: wave velocity

W : weight of a block

β: dip of the sliding discontinuity

E*:* Young's modulus

μ_e : pseudo-elastic displacement

μ_{S0} : displacement value at the moment the support is installed

ν : Poisson coefficient

λ(x): the rate of confinement loss

σ_0: initial stress in the massif

σ_1: major principal stress

σ_3: minor principal stress

σ_{1f}: major principal stress at failure

σ_b : the clamping pressure

σ_t : maximum tensile stress

φ: friction angle of the discontinuity

First Chapter: Digging Methods and Equipment

I. Introduction

Tunneling in urban areas is a complex operation because it depends on a number of parameters that can affect the ground and surface structures. Depending on the type of soil encountered, the geometry of the structure and its depth, different excavation methods can be used. Construction techniques must ensure soil stability, economy, speed and safety.

Over the last thirty years or so, new construction methods have been introduced. They make it possible to work on urban sites in difficult geological conditions without causing major surface damage. Technological development has made it possible to increase the size of excavations and to automate the work using a multifunctional machine: the tunnel boring machine.

Tunneling disturbs the initial stress field in the ground as well as the hydrogeological conditions. This change is generally accompanied by an instantaneous displacement of the working face towards the excavation and a convergence of the tunnel walls. This leads to settlement, which can undermine the stability of surface structures.

II. Digging and Excavation Technics

Irrespective of the excavation technique used, displacements occur in the vicinity of the excavation, which propagate through the rock mass and may reach the surface. Depending on their amplitude, magnitude, direction and speed of propagation, these displacements can cause damage to buildings in the vicinity of the tunnel.

The techniques used to select the tunneling methods have evolved considerably over the last three decades:

- Improvement (in quality and quantity) of the geotechnical reconnaissance studies geological and hydrogeological conditions.
- Mechanization of felling and support operations.
- Consideration of environmental issues, especially in urban areas.
- Consideration of operational issues to improve design.

Figures I-1 and I-2 provide, consecutively, the terms commonly associated with shaft and tunnel excavation (The ground is deformed at two points: at the face (extrusion), and at the walls level -convergence), and the terminology of a tunnel cross section.

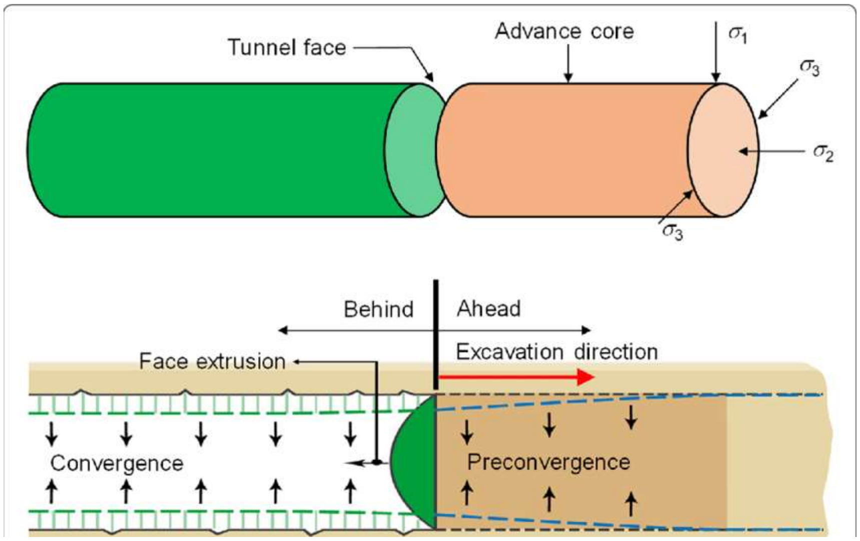


Fig. I-1: Near face and longitudinal tunnel sections *(Prassetyo and Gutierrez*, 2020*).*

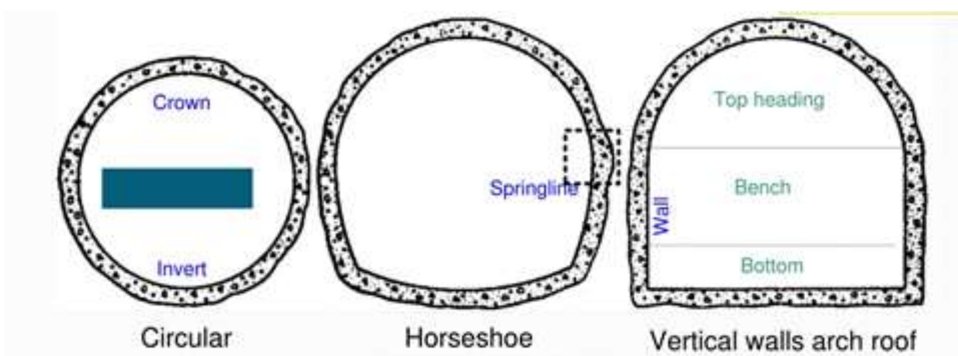


Fig. I-2: Terminology of a tunnel cross section (image source: Slideserve.com Website)

II.1. Process for Choosing an Excavation Technique

The choice of the digging technique of a tunnel project passes through three phases:

a) First phase: In the first phase, the choice is the result of a compromise between the requirements of:

- The surrounding ground,

- Site and environment,
- Tunnel geometry, and
- The construction process itself.

The resulting process of reasoning, proceeding by successive approximations, must lead at each stage to an assessment of the overall economic balance of the investment (including access, expropriations, etc.). This approach, more or less detailed depending on the complexity of the project under study, leads to two or three possible technical variants.

b) Second phase: The rule is then to study which of these two or three construction processes best ensures, in decreasing order of importance, the following criteria:
- The safety of the structure, both during and after construction;
- Uniformity of method over the entire length of the structure (since changes requiring the introduction of new equipment are always long and costly);
- Flexibility of use (so as to adapt to often unforeseeable difficulties); and
- Reduced environmental impact, particularly in urban areas.

c) Third phase: At a later stage, when the companies are consulted and the final choice is made, new criteria (linked to the economic situation, the specific technical skills of the companies, etc.) come into play, such as:
- Conjunction and size of the proposed work package;
- Level of the competitors' technical skills (specialized personnel, available equipment, experience acquired, etc.);
- Insertion of the overall timeframe for the tunnel project within the general execution schedule; and
- Solution Cost and corresponding contingencies.

II.2. Digging Methods

There are three different digging methods: Full section method, half-section method, and divided section method. Each of these methods is described below. It is to highlight that four factors affect the choice of the excavation method, which are:

- The size of tunnel;
- The type of ground;
- The available equipment; and

- The method of sequence of excavation.

II.2.1. Full Face Excavation Method

When using a full face excavation method, the entire cross-section of the tunnel is excavated in a single phase (Fig. I.3a). This method is recommended for ground of good bearing capacity when the section to be excavated is generally less than 12 m^2. Beyond that, more powerful equipment is required, making the method extremely expensive.

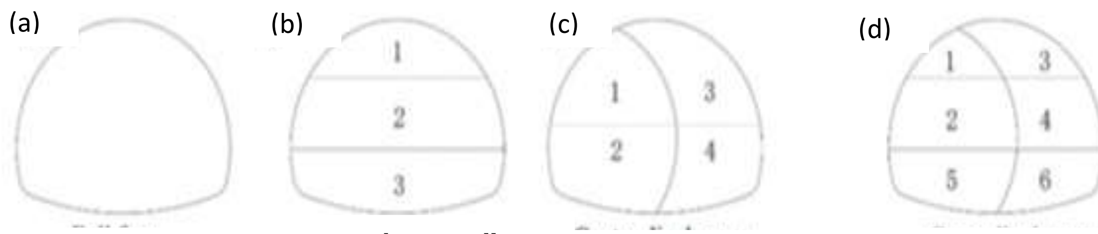
- Advantages of the method: The speed of this method facilitates site organization, as it allows a clear separation between the excavation and lining teams.
- Disadvantages of the method: The removal of the excavated material requires several repeats. In the event of an accident involving this technique requires very difficult adaptation.

II.2.2. Half Section Method

The upper part of the section is excavated first, while the lower part is excavated later. If necessary, the support is reinforced before the section is excavated, both in the upper part (arches, armoring, shotcrete, concrete) and in the lower part (micropiles under the arch supports, pedestal columns). The final lining is installed only after the entire section has been excavated.

II.2.3 Split-Section Method

This method is used when the section to be excavated is large, or when the terrain is poor and cannot be stabilized with a half-section opening. It has an economic disadvantage in that it is very costly and takes a relatively long time to implement. With excavation in scheduled sections, each phase of the work involves excavation of smaller sections of land. This means that the stability of the excavated sections is easier to control, and decompression of the overlying land is more limited. Figure I-3 schematizes the different digging split-sections.



Full face excavation method | Benching tunelling method | Center diaphragm method | Cross diaphragm method

Fig. I-3: Different Tunnel Digging Methods (After *Zhang et al*. 2023)

II.2.3.1 The Belgian Method

The principle of this method, which is used for moderately firm or hard soils, is to rapidly build the vault to protect the construction site from below, and to determine the lining by straight legs. Where the terrain is good, the first step is to build a small-section driving gallery on the upper axis of the tunnel, supported by metal frames. The cap is then cut and progressively supported by metal arches, and the vault is lined. The invert is then excavated to allow installation and lining of the pedestals (walls). Figure I-4 gives an illustration of the construction of a railway tunnels using the Belgian method.

Like all digging methods, the Belgian method has its advantages and disadvantages. The first are :

- The use of Lighter Timber Section;
- We can start work from different points simultaneously.
- Completing first the construction of roof arch which exposed to greater pressure and then constructing side walls and inverts.
- Redirection cost as the muck cars can be taken up to the farthest end in the center trench.

The disadvantages of Belgian method of tunnelling are:

- Settlement and cracks in arch masonry due to system of underpinning of the built arch, particularly when avoidable subsidence of the soil may take place.
- But settlement is reduced by using R.C.C arches.

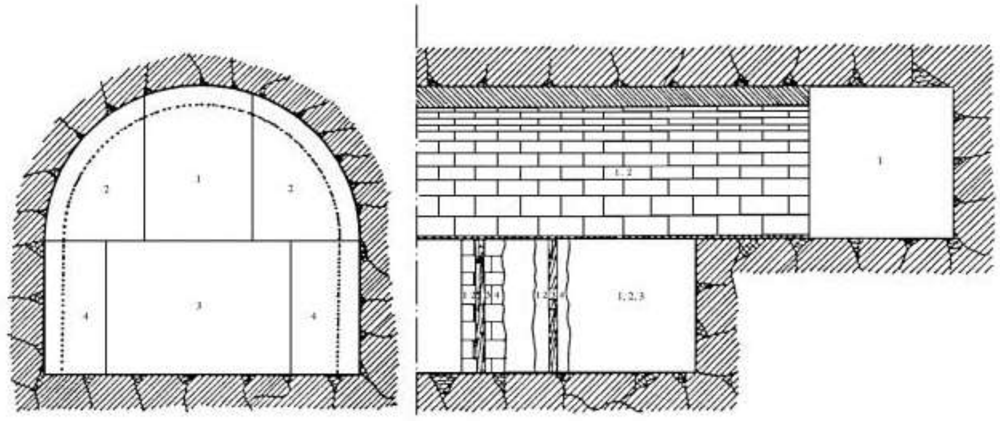


Fig. I-4: Construction of a railway tunnels using the Belgian method (*Silva et al.*, 2002)

II.2.3.2 Two Gallery Method

The difference with the Belgian-French method is the construction of two galleries, one on the upper axis and the other on the lower axis, linked by a shaft to enable evacuation of excavated material.

II.2.3.3. Full-Cap Excavation Method

This method consists of excavating a ridge gallery supported by metal frames, with support provided by metal arches. When the arches are complete, the props can be removed, allowing the use of heavy machinery (for tamping), and the concreting of the vault. Once the vault has been built, the invert is excavated; the pedestals (walls) cut and concreted. The work is completed with the earthwork and concreting of the invert.

II.2.3.3. The Umbrella Arch Technique

The umbrella arch method, also known as the pipe roof support system (Fig. I-5), refers to the use of steel or fiberglass pipes extending from the tunnel face to the front in a canopy or umbrella-like configuration surrounding the excavation zone; these pipes typically measure between 12 to 15 m in length, with an outer diameter that ranges from 70 to 200 mm. Steel pipes are usually positioned at an angle of 5 to 10 degrees to form a type of “umbrella” above the area designated for excavation. The pipes will ultimately be installed with progress that ensures the required overlap for the tunnel face's stability. The pipe umbrella technique is implemented through two distinct approaches: predrilling and cased drilling (*Siad* et al., 2023).

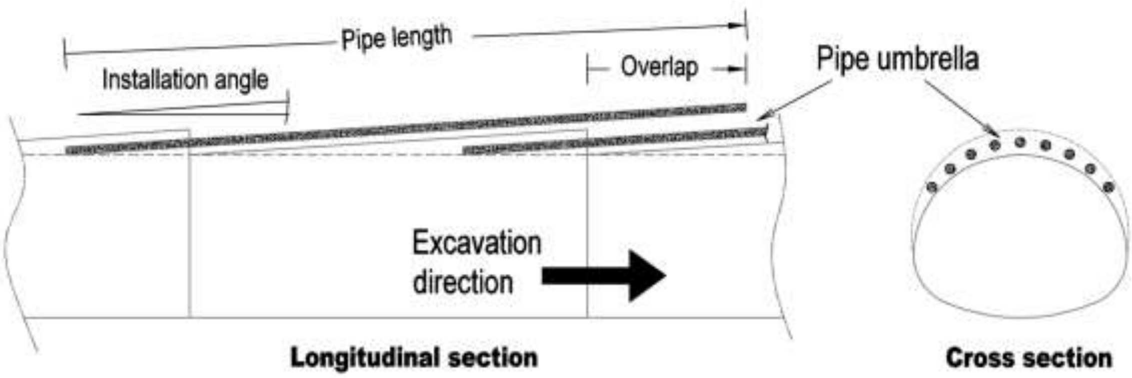


Fig. I-5: Use of umbrella arch technique in supporting the tunnel roof (*Siad et al*., 2023).

III. Tunneling Equipment

Each terrain has its own characteristics, which means that tunnelling must be carried out in the most appropriate way. This situation has led engineering to develop various tunnelling techniques, which can be divided into two categories:

- Excavation in soft ground.
- Excavation in hard ground.

III.1. Excavation in Soft Ground

Excavating an underground structure in soft ground with poor mechanical properties requires the use of a complex machine known as a tunnel boring machine (TBM). It can dig tunnels with diameters ranging from 2 to 15 meters. It is particularly well-suited to excavating long stretches of soft ground (due to its cost). Its advance speed is in the order of 10 to 50 meters per day, which is 20 hours of excavation approximately.

III.1.1. TBM Functions

The TBM performs the following functions on a continuous basis:

- Excavation of the ground;
- Stabilization and support of the working face;
- Temporary lining of the tunnel walls just after excavation;

- Removal of excavated material.
- Installation of temporary support or final surfacing;
- Guidance along the theoretical axis;
- Automatic advance using jacks.

III.1.2. TBM Components

Three parts constitute the TBM components, which are: the cutting head also called the front, the shield (middle), and the trailing gear (rear) that constitutes the middle part of the TBM (WELTY Website). The cutting head (front) rotates and thrusts into the rock surface that lies ahead of it. This trust causes the cutting disc tools to break the rocks. The grippers/braces develop the reaction to applied thrust and torque forces with the help of anchoring.

Next comes the TBM shield (roof shield) and concrete panels also known as the mid-section. As the TBM makes progress, it needs an outer shield to protect itself and the workers inside from the surrounding ground. The concrete panels get installed right behind the shield itself, becoming the outer layer of the tunnel. As the cutter-head makes progress, the panels are picked up and set in place by a rotating vacuum powered lift.

The trailing gear comprised in some cases of over 300 feet of gear that supports the TBM, excavates the dirt and rock at the same time. It includes a conveyor belt that removes all the soil excavated by the cutter head from the tunnel, which gets longer and longer as the TBM makes progress. The trailing gear also holds supplies that the operating crew needs to keep the machine moving forward. Sometimes, up to 25 crew members at a time can be needed to operate a TBM.

III.1.3. Types of TBMs

The Soft Ground TBMs are of three main types:

- Slurry shield,
- Earth pressure balance, and
- Open face type.

III.1.3.1. Slurry Shield TBM

Slurry Shield TBM is used when the ground conditions are granular with very high water pressure. This TBM is useful in cohesionless soils conditions. The tunnel face is stabilized by application of pressurized fluid. Hydrostatic pressure is applied on the excavation by filling the cutter head with pressurized slurry, which is also used as a transporting medium when mixed with the excavated material before it is pumped to the cutter head back and, then to a separation plant outside the tunnel. Particles of spoil from the slurry are filtered using the slurry separation plants to reuse it in the construction process (*Katuwal*, 2023). Figure I-6 shows a schematic illustration of a Slurry Shield TBM.

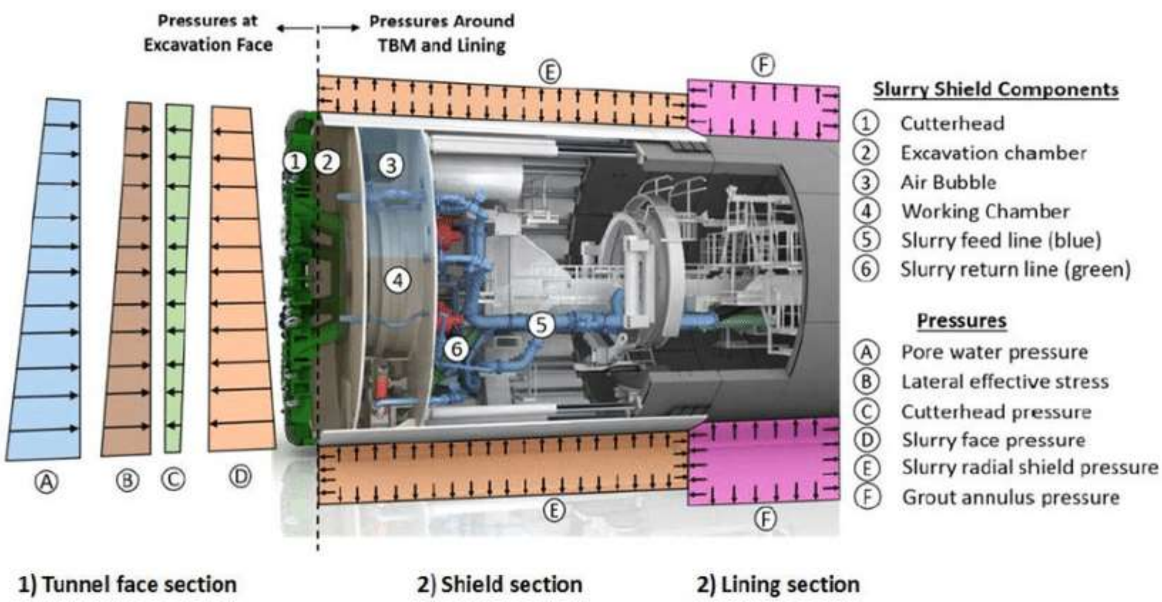


Fig. I-6: Schematic of Slurry Shield TBM and pressure components (*Jaafer*, 2017)

III.1.3.2. Earth Pressure Balance

This TBM is adaptable for weak fine-grained cohesive soil or soft soil and when the soft groundwater is less than 7 bars of pressure. In this type, the cutter head not only uses disc cutter but also a combination of tungsten carbide cutting bits, drag picks and carbide disc cutters. The spoil is then transferred into a TBM with the help of a screw conveyor also known as a "cochlea" that allows the pressure to remain balanced without any use of slurry.

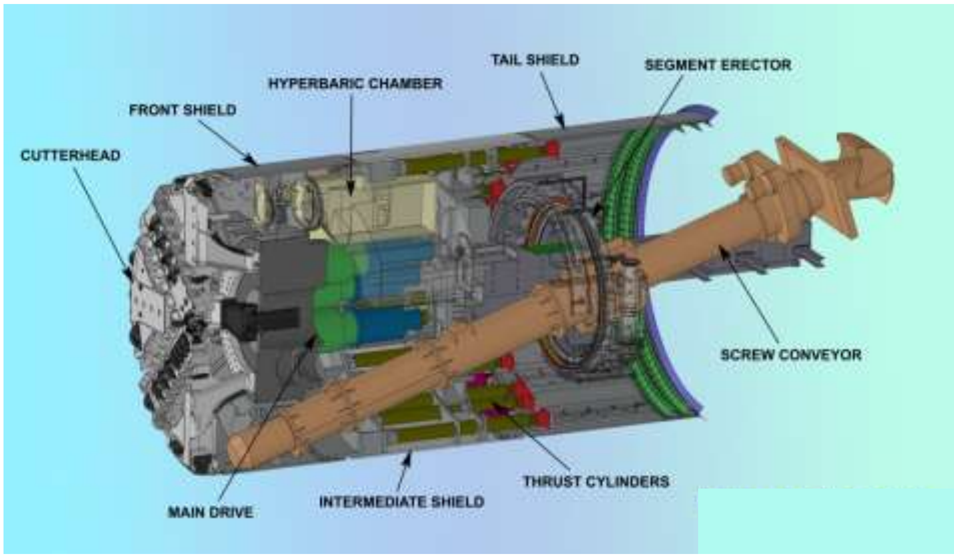


Fig. I-7: Earth Pressure Balance (image source: tunnelpro.it Website)

III.1.3.3. Open Face Type TBM

Open Face TBM is a method of excavating with no frontal support, relying on the fact that the ground being excavated will stand up with no support for a brief period, making it suitable to be used with rock types having strength up to 10MPa. The cutter head excavates the face within 150mm of the edge of the shield. Now, the shield is put forward with the cutter getting rid of the remaining ground in sync with the circular shape. Precast concrete is used to provide the ground support. Finally, Key which is the final segment expands the ring until and unless it gets tight against the circular cut of the leftover ground.

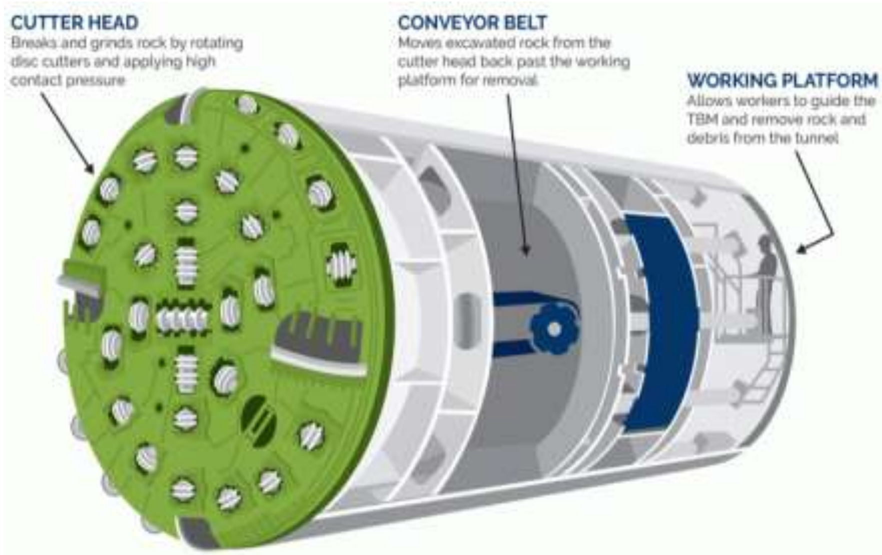


Fig. I-8: Tunnel Boring Machine *Components* (image source: ENCARDIORITE Website)

III.2 Excavation in Hard Terrain

The tunnelling in hard ground is carried out using digging machines so-called point-attack machines in firm soils or boom type machine (Fig. I-9), or using explosives in rocky terrain that is called blasting. Blasting is a cyclical process. First, blast holes are marked out and drilled, then loaded according to a blasting plan (Fig.I-10). Once the blasting has been completed, the fumes are ventilated and the unstable blocks purged.

Fig. I-9: Boom Type Machine (B.T.M) in action (image source: Solid Ground Website)

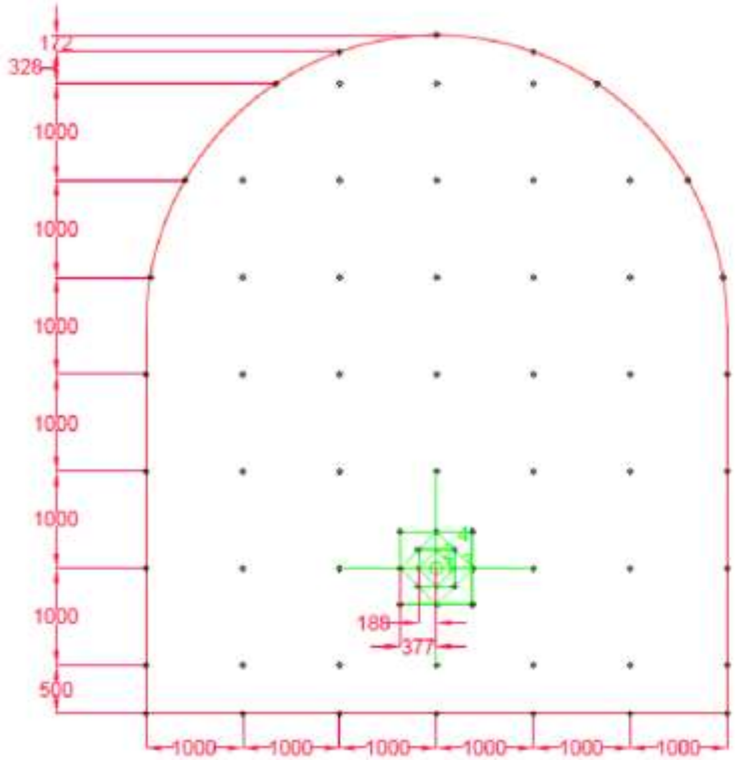


Fig. I-10: Example of a Full-face Tunnel Blasting (*Yano*, 2019)

III.2.1 Mechanical Digging

Mechanical digging is carried out using a point-attack machine, also known as a Boom Type Machine (B.T.M.). The principle of this type of work is to break up the rock by the combined effects of penetrating the cutting face and sweeping the arm. The BTM is able to work on a section of any shape, and the cut is neat and without over-profiling. Its reduced size in relation to the tunnel cross-section enables continuous observation of the working face. What's more, the B.T.M cutting technique does not cause any shaking. However, one of the disadvantages of the BTM is the high cost of replacing the cutting heads, which wear out too quickly, but this is still more advantageous than for the T.B.M.

III.2.2 Blast Method (DBM)

Blast method is one of the oldest and most widely used methods as it was used for the first time in 1627 to dig silver mine in Slovakia (*Katuwal*, 2023). It involves using explosives to break up rocky ground in order to excavate it. However, explosives should be used as rock-cutting tools, not as "bombs. In urban areas, since their use produces adverse psychological effects on the surrounding population (noise, smoke, vibrations, etc.) and poses a risk of destabilizing nearby buildings and services, explosives should be used only when absolutely necessary. Figure I-11 illustrates the blasting operation in tunnel cutting face.

Explosive blasting is carried out cyclically for each advance flight, according to the following basic operations:

- Blasting notations in tunnel cutting face;
- Tracing and perforating the blast pattern;
- Loading blast holes and firing the blast;
- Ventilation and purging of the excavation; and
- Removal of spoil from the face (marining).

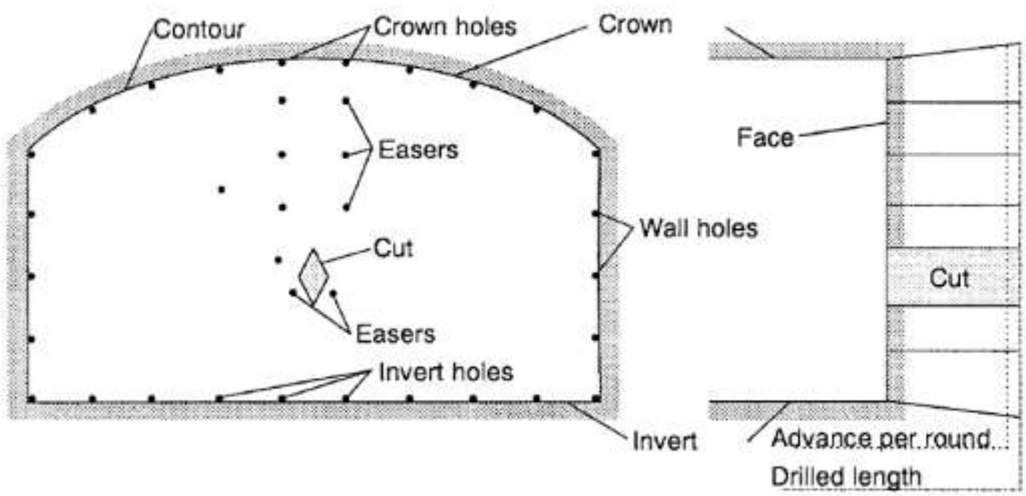


Fig. I-11: Blasting notations in tunnel cutting face (Kutawal 2023)

IV. Tunnel Lining Systems

The operation that immediately follows the excavation of the tunnel or takes place at the same time as the excavation, depending on the method used, is the installation of a supporting system, also called tunnel lining, to stabilize the ground displacement. The lining can be defined as a set of devices that ensure the temporary or permanent stability of the excavation. The lining not only ensures the stability of the tunnel during construction, but also contributes to the final stability of the structure by reducing the stresses carried by the lining layer that will later be applied to the inner walls of the tunnel. On the other hand, the lining also ensures the functions of watertightness or waterproofing and aesthetics of the tunnel.

IV.1. Tunnel Lining Function and Materials

When a tunnel is excavated, the pre-existing stresses in the rock are channeled into the contour of the cavity, creating a "*vault effect*" that allows the cavity to be held. The role of the lining is therefore to allow this equilibrium to be established under good conditions by limiting the expansion of the volume of decompressed soil around the cavity. If the quality of the ground is good (sound rock with few fractures), the role of the lining can be limited to controlling local instabilities (e.g. isolated blocks formed by discontinuities). A well-designed support structure should resist extreme ground pressures and may even improve the resistance of the ground (*Bouvard et al*., 1992).

Tunnel lining provides the structural support of the excavated cavity and waterproofing measures. It may also be described as the temporary lining, inner or permanent support. Materials usually used to make a tunnel support are: timbering, steel ribs, lattice beam elements, reinforced or unreinforced concrete, shotcrete, metal hangers, anchor and rock bolts, precast segmental elements, or a combination of two or more of these systems depending on the ground quality. Each type of these supports has its own method and time of installation, and is appropriate to a certain kind of tunnel project. The structural behavior of the support materials is determined by their deformability or stiffness, the degree of bonding between support and rock mass and the time of installation (*Maidl and al.,* 2013). Table I-1 gives the normal stiffness modulus and the maximum support pressure values of four types of tunnel lining.

Table I-1: Stiffness modulus and support pressure values for different tunnel lining types (After *Bouvard et al*., 1992).

	Formwork Concrete Ring e = 40 cm	Shotcrete e = 10 cm	Arches HEB, e = 1,00 m	Point-Anchored Bolt L= 4,00 m Diameter = 18 mm
Normal stiffness modulus Ec (MPa)	1800	210	170	64
Maximum support pressure P (KPa)	600	100	140	80

A temporary tunnel lining is used during digging, before the permanent lining can be put in place, for on-site safety of workers and equipment until the complete installation of the permanent lining. It can also provide part of the structural function of the permanent lining if it is integrated. Primary tunnel lining is then used to stabilize the tunnel surface before the permanent lining can be put in place. This latter is designed to carry the long-term loads of the tunnel (Fibo Intercon Website). The permanent lining is a critical component in underground construction, playing a pivotal role in ensuring the durability and structural strength of the finished structure. It also serves as a base for essential service installations like ventilation, aeration, and lighting (Maccaferri Website).

IV.2. Criteria for Selecting a Support System

Several criteria are taken into the consideration in the process leading the engineer to choose the support or tunnel lining to use in the project. These factors are (*Bouvard et al*., 1992):

- The strength of the rock mass;
- Discontinuities;
- Weathering;
- Hydrological conditions;
- Natural constraints;
- Cavity dimensions;
- Excavation method; and
- Sensitivity to settlement.

IV.3. Tunnel Lining Classes

There are four main types of retaining structures, depending on how they act in relation to the surrounding terrain (*Bouvard et al.*, 1992):

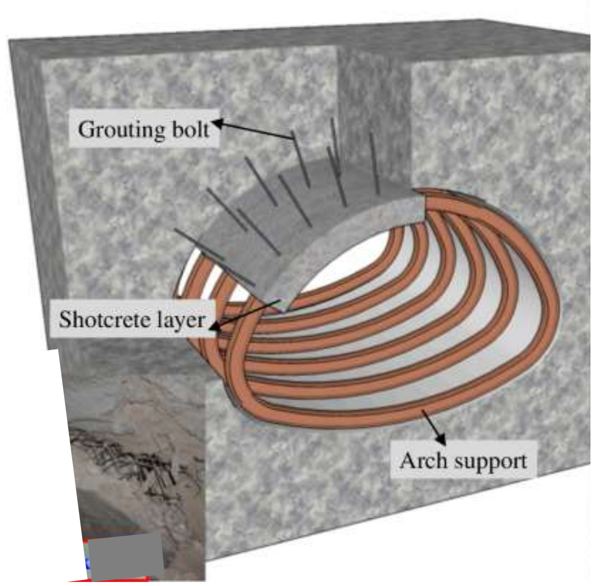
a) Retaining structures acting by confinement of the surrounding soil: these are essentially shotcrete (Fig. I-12) alone and shotcrete combined with light arches (Fig. I-13);

b) Roof supports that act both by confinement and as reinforcement of the surrounding soil: these include bolting in its various forms (Fig. I-14 a, b, and c), whether or not combined with shotcrete, light arches/ribs, or both simultaneously: point-anchored bolts (with shell or resin), and distributed-anchored bolts (sealed with resin or mortar);

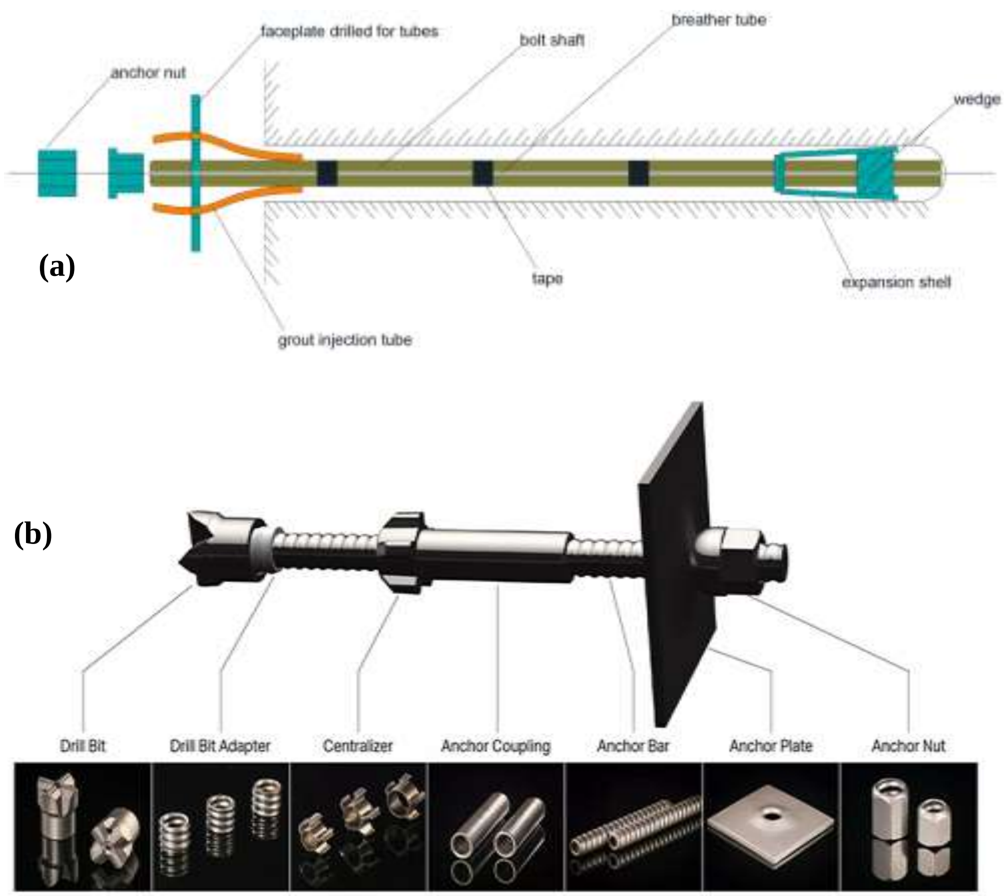
c) Supporting structures: heavy hangers, light hangers, assembled metal plates, precast segmental tunnel lining (Fig. I-15), preferred tubes (umbrella vaults), and shields; and

d) Retaining structures acting by consolidating the soil and modifying its geotechnical or hydrological characteristics: consolidation injections, compressed air, and soil freezing.

Fig. I-12: Tunnel intrados lining with shotcrete (image source :Strade Autostrade Website)



I-13: Tunnel lining support using arches, shotcrete and grouting bolts (Modified from *Sun et al*., 2022)



(c)

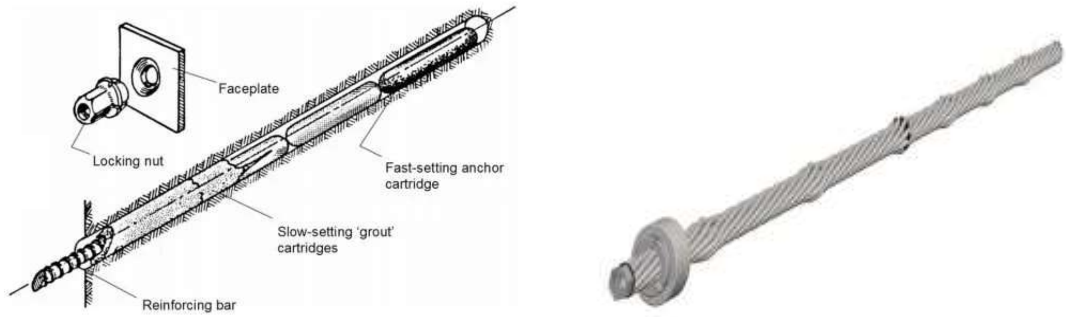


Fig. I-14: Various Forms of Rock Bolts: (a) Mechanically Anchored, (b) Fully Grouted, and (c) Resin-Injection Bolt (left) and Cable Bolt (right) (image source: *Kamlesh Metal & Alloy* Website)

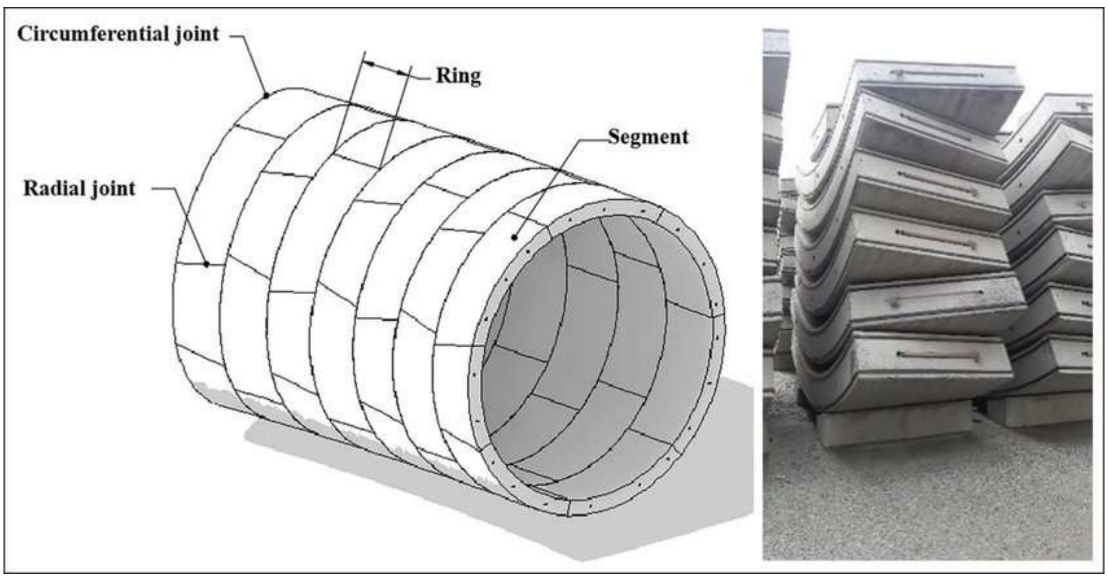


Fig. I-15: Segmental lining elements (left); a pile of prefabricated ring segments ready for TBM feeding (right) (*Getuli et al*, 2021)

Chapter Two: Review of Rock Mechanics Concepts and Site Reconnaissance

I. Introduction

During tunnel excavation, there is a risk of falling boulders formed by discontinuities in the rock. These blocks are often located at the vault level, but they can also be located at the excavation wall level and slide down. This poses a risk to personnel and machinery inside the excavation. Therefore, the civil engineer must take the necessary steps to stabilize any blocks at risk of falling or sliding during digging work. A simple manual calculation is used to determine how many anchor bolts are needed for stabilization. Yet, before getting down to the calculation method, let explain the notion of vault effect.

II. The Vault Effect Notion

The stability of an underground excavation is linked to a natural phenomenon of stress rearrangement known as the vault effect. To fully understand the vaulting effect, we need to examine the state of stress at four points located at different distances from the excavation walls (Fig. II-1).

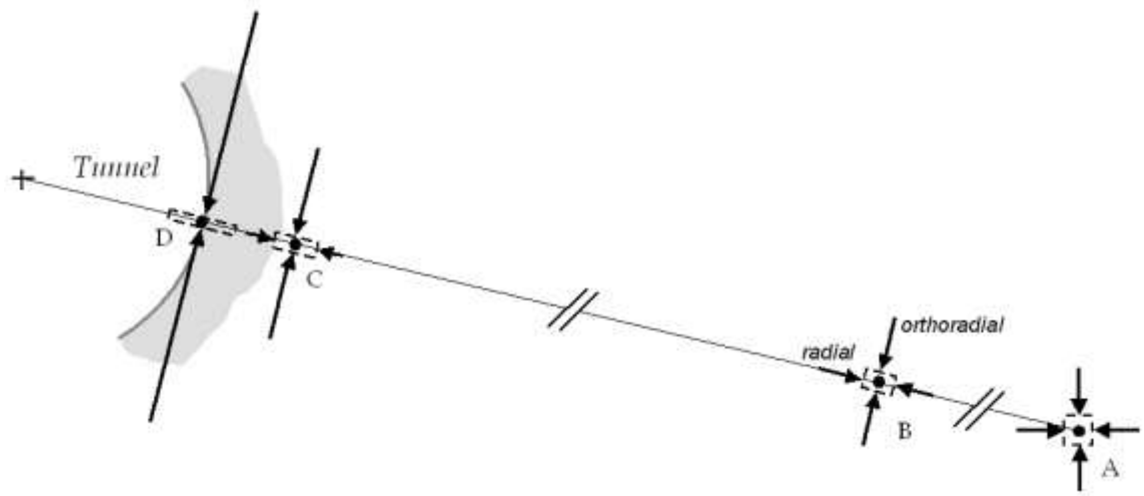


Fig. II-1: Evolution of stress and deformation in a volume element around an excavation (*Martin*, 2012).

- Point A: Located "at infinity" with respect to the tunnel, the stress state (assumed to be isotropic) remains unchanged. The two axes represent the major and minor (here identical) principal stresses in the plane;

- Point B: At around four diameters from the tunnel wall, the effects of excavation are considered to be noticeable. The principal stresses are oriented along the axes of the cylindrical coordinate system. There are two main stresses: radial and orthoradial;
- Point C: Close to the wall, the evolution is as follows: The radial compressive stress decreases while the orthoradial component increases.
- Point D: At the wall, the radial stress is zero due to the boundary condition where there is no support. Conversely, the orthoradial stress is maximum. This is the vault effect. The ground naturally "blocks" as it would in the arch of a self-standing bridge**.**

The stress deviator increases from A to D, reaching zero in the initial configuration. This bounded deviator is referred to as the plasticity criterion. If the stress deviator exceeds a threshold value, instability will occur, followed by collapse. However, if it remains within a certain range of values, the ground will be stable and will not require support. To take full advantage of this mechanism, the cross-section should be as close to circular as possible—the "ideal" cross-section (*Martin*, 2012).

III. Stability of Excavation Walls

III-1. Stability of a Bocks/wedges/Dihedron

The study of dihedral stability comprises four main stages. However, calculation algorithms automatically identify blocks likely to fall. In 2D, for simple cases involving one or two dihedrals, calculations can be carried out by hand, right up to the dimensioning of the bolts using the four following stages (*Martin*, 2012):

Stage 1- Structural analysis: collecting geometric and geomechanical data, and determining the orientation and dip of the main discontinuities. The dip being the angle between the horizontal line and the line of greatest slope. It specifies the direction relative to the horizontal (Fig II-2).

Stage 2- Kinematic analysis: identification of potentially unstable dihedrals that may slip or fall at the contour of the excavation (Fig II-3).

Stage 3- Calculation of the safety factor depending on the mode of failure of the equilibrium.

Stage 4- Calculation of the reinforcement required for each unstable dihedral to achieve an acceptable safety factor.

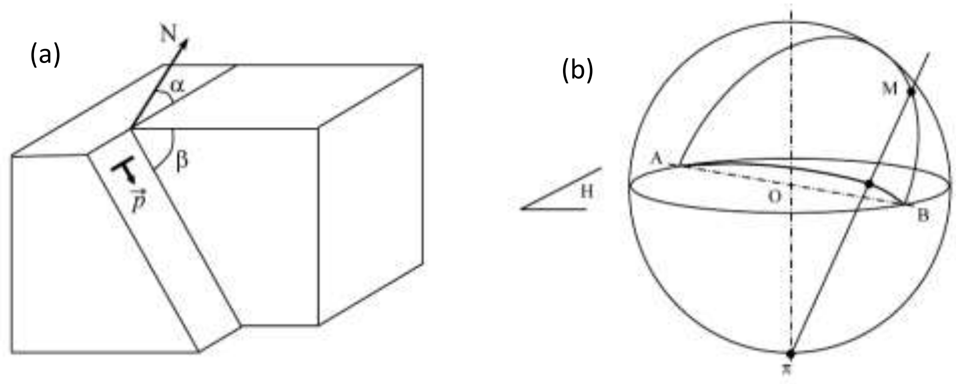


Fig. II-2: Geometric representation of a discontinuity (After *Martin*, 2012).

In figure II-2, two angles define the plane of the discontinuity:

- The angle of the horizontal projection of the vector $\vec{p}$ with respect to north, turning eastward is the Azimuth. It is equal to α +90°, and
- The angle β of the horizontal line and the line of greatest slope, which specifies the direction with respect to the horizontal.

In a moderately fractured massif, a boulder is geometrically defined by the discontinuity planes and the shape of the excavation (Fig. II-3). The block is initially blocked by the massif, with the excavation gradually releasing the abutting forces. The aim of the block or dihedral method is to define the reinforcement (passive or active anchoring) needed to take up the forces and hold the block in place, whether in the vault or in the facing. The failure mechanism results from the action of gravity and the structure of discontinuities. In the case of free-fall on a roof, it is gravity alone that causes instability. In the case of sliding, a law of discontinuity behavior must be taken into account. The balance of driving and resisting actions is used to determine a safety coefficient (*Martin*, 2012).

Figure II-3 illustrates certain instances of poor joint alignment that need for extra care when it comes to bolting. Unstable wedges may be formed in the crown of an excavation by joints with varying dip directions but sub-parallel strike directions to the excavation's length (Fig. II-3 a). Because a sub-horizontal joint may meet the rock mass just above the crown and may not be visible prior to failure, a combination of sub-horizontal and sub-vertical joints may need extra attention (Fig. II-3 b). Longer bolts than those suggested by the Q-system may be the answer in these circumstances. In these situations, it is also advised to change the rock bolts' orientation. Inclined joints intersecting the walls in an underground opening could serve as sliding planes for unstable blocks. In such cases the stability of opposite walls may be quite different depending on the dip direction of the joints (Fig. II-3c). if two intersecting joints form a wedge as shown in Figure II-3 d, a similar situation will occur (NGI, 2022).

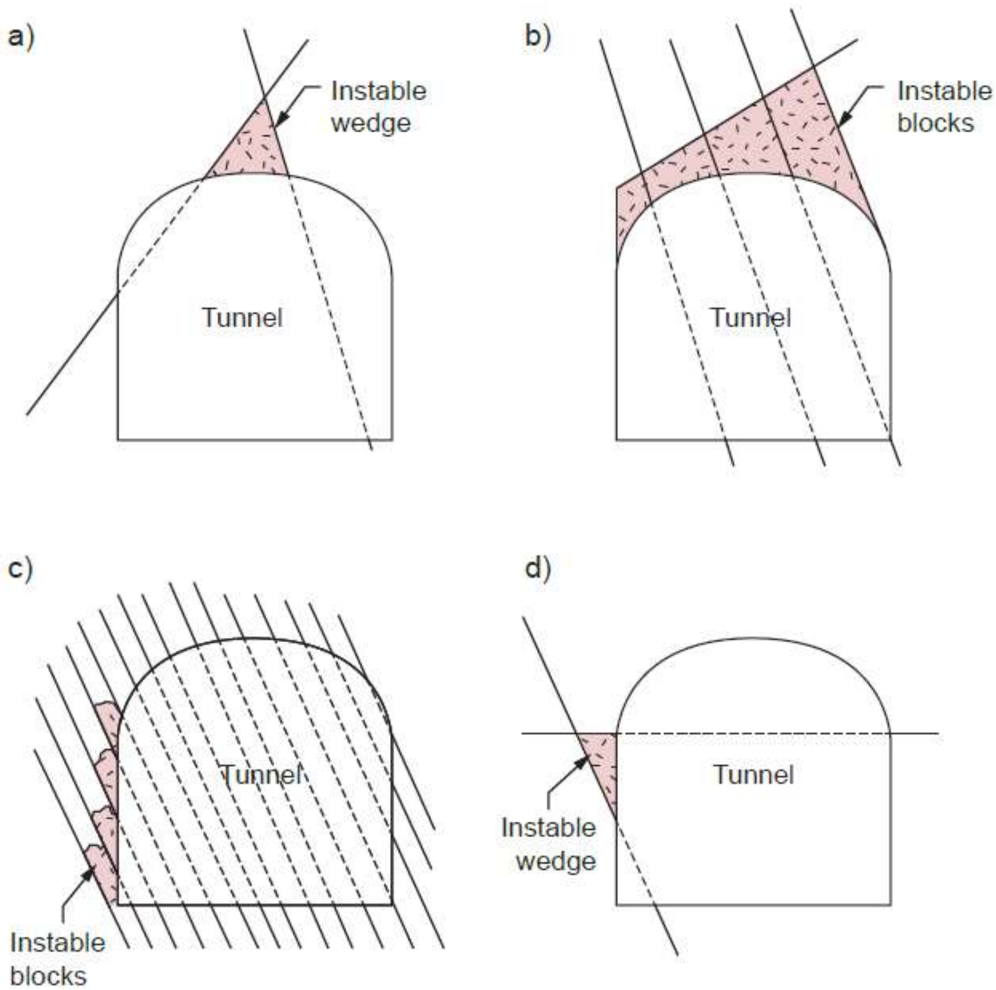


Fig. II-3: Instability of blocks and wedges caused by unfavorable joints orientations (NGI, 2022).

III.1.1 Unstable Dihedron in Vault

In this case, it is not necessary to consider the mechanical properties of the discontinuities when calculating the support. The bolts must protrude well into the sound rock to ensure sufficient anchorage (one meter minimum). The total number of bolts N can be approximated by formula (1):

$$\boldsymbol{N} = \frac{\boldsymbol{W.f}}{\boldsymbol{B}} \quad \text{………………} \text{ (II-1)}$$

Where:

W : the weight of the block;

F: the safety coefficient, often assumed to be between 2 and 5;

B: the maximum allowable bolt load (Technical sheet).

III.1.2 Unstable Dihedron on the Wall

In this case, it is necessary to know the mechanical properties of the discontinuities, in particular their angle of friction and cohesion. If the fracture on which the dihedral can slide is not sufficiently cemented or roughened, movement will be initiated when the tunnel passes through. Maximum shear strength isn't reached immediately, and the block may appear to be "holding" when it's actually on the verge of sliding. Therefore, the dangerous dihedral must be bolted down.

The clamping pressure applied by the bolting system creates an additional normal force on the discontinuity, thus increasing the shear strength of the joint and consequently increasing the safety against sliding. Equation (2) can be used to estimate the number of bolts N required:

$$\boldsymbol{N} = \frac{\mathbf{W}*(\boldsymbol{f}*\boldsymbol{\sin\beta} - \boldsymbol{\cos\beta}*\boldsymbol{\tan\varphi}) - \boldsymbol{c}*\boldsymbol{A}}{\boldsymbol{B}*(\boldsymbol{\cos\alpha}\ \boldsymbol{\tan\varphi} + \boldsymbol{f}*\boldsymbol{\sin\alpha})} \quad \text{………..} \text{ (II-2)}$$

Where:

W is the weight of the dihedron, including that of any other blocks it supports;

f: safety factor between 1.5 and 3 ;

β: the dip of the sliding discontinuity ;

c, the cohesion of the discontinuity;

φ, friction angle of the discontinuity;

A, sliding area;

B, maximum allowable bolt load;

α, the angle formed by the bolts with the normal to the discontinuity.

Note that these values are very often difficult to determine, and the designer will then have to consult the literature to obtain orders of magnitude (φ, c, etc...).

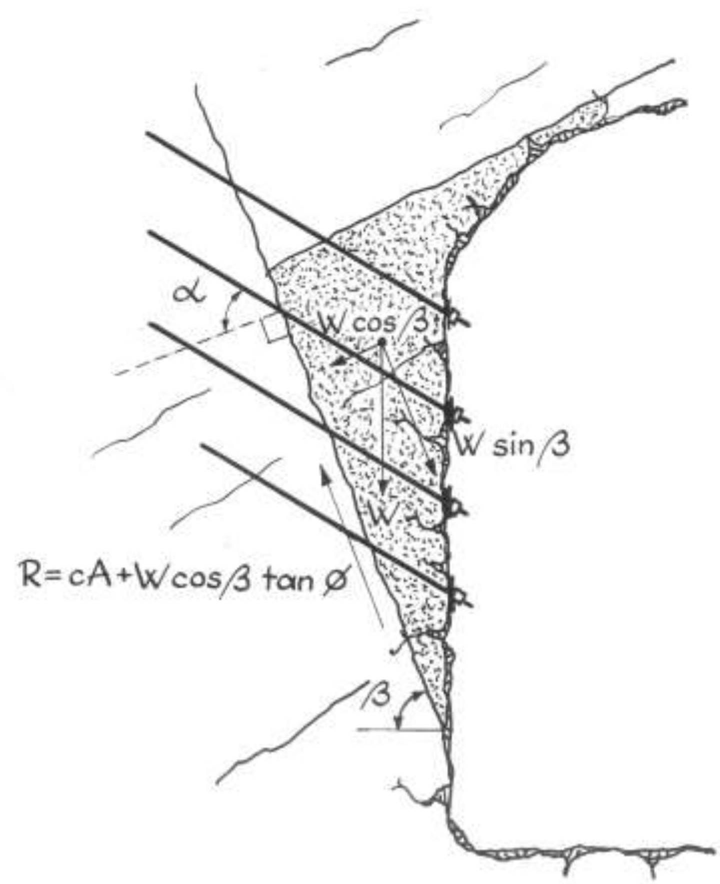


Fig. II-4: Reinforcing a rock dihedral likely to slide under its own weight (*Martin*, 2012).

To elaborate the precedent exposed subsections, an example of a reinforcement study illustrates the schematic cross section of a low overburden tunnel (< 1 D) in hard rock illustrating the interaction of rock reinforcement and rock support in an isotropically jointed rock mass. Rock bolts around the excavation and spilling above the crown hold hard rock blocks together and contribute to rock arching (grey color) to restrain gravitate failure and ground loading from potentially unstable rock (dark brown color).

As illustrated in Figure II-5, both rock arching and the failure mechanics of jointed rock roofs are governed by a complex interplay of rock mechanical parameters such as the joint orientation relative to the tunnel, the shear strength of joints, rock competence, the stress confinement, and the characteristics of the reinforcement. The latter represents a typical rock engineering problem defined by the failure mechanics of jointed rock tunnel roofs (*Almenara et al*., 2024).

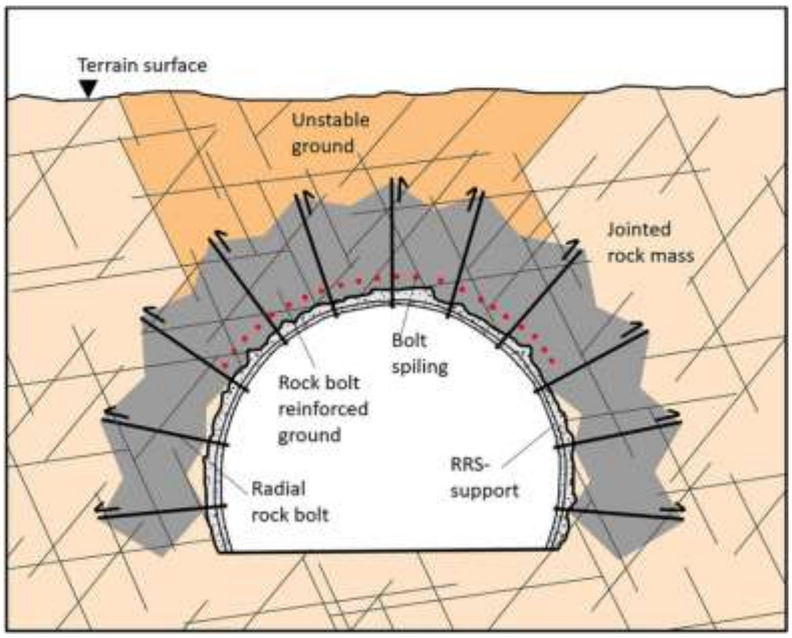


Fig. II-5: Example of tunnel section reinforcement using rock bolts (*Almenara et al*., 2024).

III.2. Stability of rocky banks

This is the special case of structures excavated in the direction of the bedrock, which is clearly horizontally stratified. Their geometry must be slender (square shape, low vaults, etc.) so that a "beam on two supports" operating scheme is relevant (Fig.II-6).

In the case shown in Figure 5, the gallery roof consists of a stack of beams in the sense of RDM. We simply estimate the deflection of these beams and compare the tensile, compressive, and shear stresses with the corresponding resistances. The sizing of the required bolting is based on the evaluation of the normal stress (bolt tension divided by its tributary area) required to "clamp" the beams or more precisely the rocky banks, prevent relative sliding, and limit the tensile stress in the rock, taking into account the coefficient of friction between the banks.

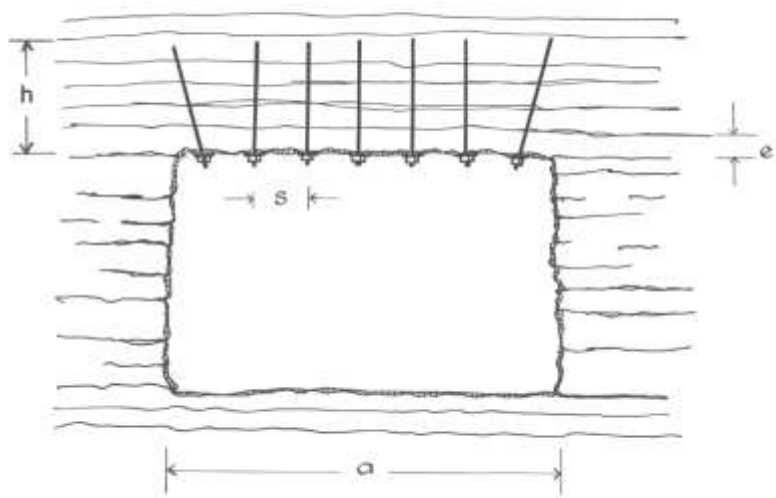


Fig. II-6: Illustration of a directional gallery in a near-zero dip stratified massif (*Martin*, 2012).

Let *q* be the load per unit area above the roof, at be the span of the beam, *h* its effective height (taking into account the action of the bolts) and *t* the tensile strength of the rock. As a first approximation, we assume that the maximum moment is:

$$M = \frac{q*a^2}{8} \dots\dots\dots\dots\dots\dots\dots (II\text{-}3)$$

The associated maximum tensile stress, which must be less than the permissible rock stress, is written as:

$$\sigma_t = 6.\frac{M}{h^2} \dots\dots\dots\dots (II\text{-}4)$$

Then, from equation (II-4) we deduce the minimum bolt length *h*, which is written as:

$$h \geq \frac{a}{2} \cdot \sqrt{\frac{3*q}{2*\sigma_t}} \dots\dots\dots (II\text{-}5)$$

Let φ be the angle of friction between two benches, T the shear force of the section under consideration and σ_b the clamping pressure that must oppose sliding (prestressing relative to the tributary surface). The maximum shear stress is reached at the mid-height of the supports:

$$T_{max} = \frac{3*T}{2*h} \text{.........} \quad (II\text{-}6)$$

Where:

T = $q.a/2$

Then, we need to check that:

$$\sigma_b .tan\varphi \geq T_{max} \text{.........} \quad (II\text{-}7)$$

Finally, we obtain:

$$\sigma_b \geq \frac{3*T}{2*h*tan\varphi} \text{.........} \quad (II\text{-}8)$$

IV. Stress Induced Failure

As the depth of a tunnel becomes greater or where adjacent underground structures exist and the ground condition becomes less favorable, the stress within the surrounding rock mass increases and failure occurs when the stress exceeds the strength of the rock mass. This failure can range from minor spilling or rock fall in the rock surface to an explosive rock burst where failure of a significant volume of rock mass occurs (*FHWA & NHI*, 2009). Figure II-7 illustrates an example of the stress induced failure phenomena and how a set of unstable blocks become stabilized using rock bolts and shotcrete.

Note that the stress induced failure potential can be investigated using the strength factor "*SF*" against shear failure defined as:

$$SF = \frac{\sigma 1_f - \sigma 3}{\sigma 1 - \sigma 3} \quad \text{...............} (II\text{-}9)$$

Where:

σ_1 and σ_3 are major and minor principal stresses;

σ_{1f} is major principal stress at failure;

($\sigma_{1f} - \sigma_3$) represents the strength of the rock mass; and

($\sigma_1 - \sigma_3$) represents the induced stress.

A stress induced failure potential "*SF*" greater than 1.0 indicates that the rock mass strength is greater than the induced stress, i.e., there is no overstress in the rock mass. However, when it is less than 1.0, the induced stresses are greater than the rock mass strength and the rock mass is overstressed and likely to behave in the plastic range.

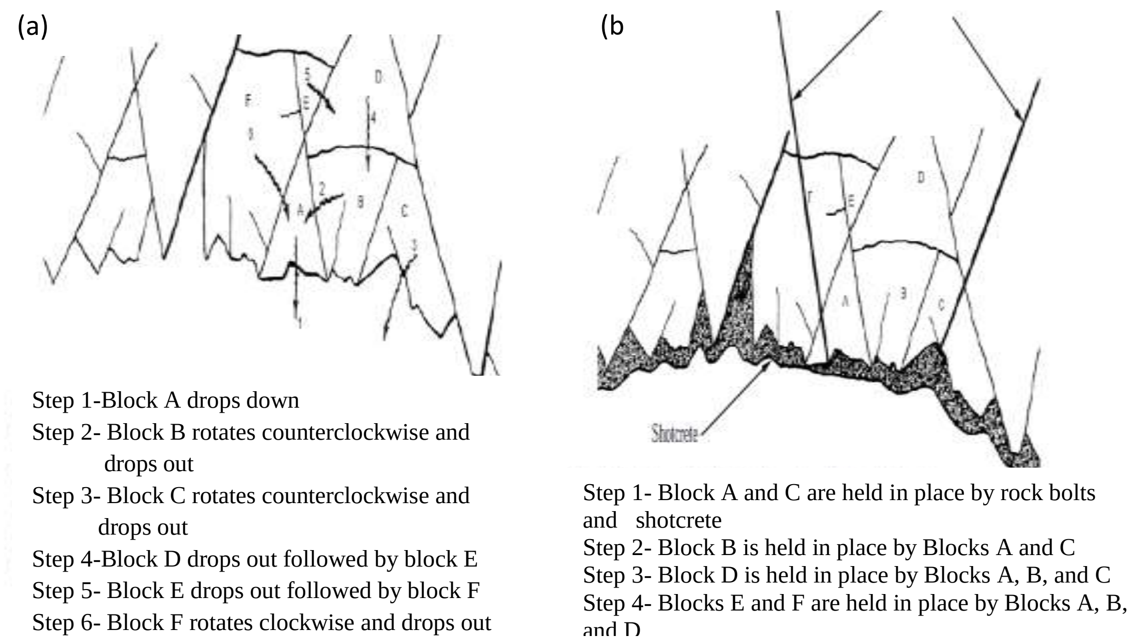


Fig. II-7: Stress induced failure in blocky rock: (a) Progressive failure in unsupported tunnel, and (b) Prevention of progressive failure in supported tunnel (Modified from *FHWA & NHI*, 2009)

V. The Site Reconnaissance Campaign

The site investigation campaign is a multidisciplinary task involving three engineers specialized in fields closely related to civil engineering: the geologist, the hydraulic engineer and the geotechnical engineer. It includes three types of investigation: geological, hydrogeological and geotechnical. Each type has its own specific objectives and uses appropriate means and tools. The three types are complementary and provide the structural engineer with the data needed to design the underground structure to be studied or built.

V.1. Geological Investigation

The geological exploration campaign is the first reconnaissance to be carried out on the site where an underground structure is to be excavated. In fact, the geology of the massif to be excavated determines whether the project will go ahead or whether it will be decided to modify or change the design of the underground structure, or even to cancel it altogether. Indeed, the presence of a seismic fault, for example, running longitudinally or transversely through the trace of the structure, or located in its vicinity, is a factor leading to a radical change in the structure. In such a case, relocating the structure to a site devoid of past seismic activity would be an ideal solution.

V.1.1. Objectives of the Geological Investigation

The aim of a geological survey of the site is to identify the geological elements, especially those related to the tectonic history of the massif. The regional geology and the geological and tectonic history of the massif, as well as the structure of the massif, the location of faults and the description of the terrain encountered (petrographic and mineralogical nature) are the main objectives of this type of reconnaissance (*Bouvard et al*., 1992).

V.1.2 Resources/Tools of the Geological Investigation

There are several types of tools and resources required for geological site reconnaissance. However, before resorting to any of these means, it is always necessary to re-examine existing archives and search for available data from previous studies. However, in the absence of archives or previous studies, the most valuable tool remains core sampling in the field, as this type of investigation provides visible information that can be examined and tested in the laboratory. Unfortunately, it is not always possible to carry out core sampling, especially if the underground structure is very extensive, which would make the survey very costly. A geological site investigation is generally conducted using the following resources:

- Inventory of existing data (geological maps, previous studies, etc.),
- Geological surface survey,
- Geophysical surveys (gravimetry, magnetometry, seismic refraction, etc.),
- Remote sensing, and

- Core boring – Borehole (valuable and visual information).

V.2. Hydrogeological Investigation

Knowledge of the hydrogeological conditions of the soil mass to be excavated for an underground structure is of great importance for the selection of the excavation method, the preparation of the excavation equipment and the parameters used to calculate the dimensions of the support and sealing of the structure. Once the geological survey has been carried out, it is followed or sometimes accompanied by a hydrogeological survey.

V.2.1. Objectives of the Hydrogeological Investigation

The purpose of a site hydrogeological investigation is to obtain clear information allowing us to define the nature of possible water arrivals along the structure's route, the hydraulic loads, the flow rate and the water chemical composition, and the possible treatment to be carried out prior to the works (lowering of the water table, drainage system, etc.). The main objectives of this reconnaissance are to determine (*Bouvard et al*., 1992):

- The hydraulic flow regime of the water,
- The permeability of the soil, and
- Location of aquifer formations (which may contain water) and impermeable soils.

V.2.2. Resources/Tools of the Hydrogeological Investigation

Like geological reconnaissance, hydrogeological reconnaissance involves the use of a wide range of resources, from the simplest - the exploration of existing archives and data from previous studies - to in-situ tests. In general, the resources required for hydrogeological exploration of an underground work site are as follows:

- Inventory of existing data (hydrogeological maps, previous studies, etc.),
- Surface hydrogeological survey,
- Geophysics,
- In situ soil permeability measurements,
- Soundings/Core boring (if necessary).

V.3. Geotechnical Investigation

The final step before digging an underground structure is the geotechnical investigation. This can be done in parallel with the previous two by taking soil samples and testing them in the laboratory. The mechanical parameters can then be determined by appropriate tests.

V.3.1. Objectives of the Geological Investigation

The aim of the geotechnical site investigation is to obtain precise information on the geotechnical characteristics of the ground to be crossed during excavation along the entire tunnel's route. This allows the engineer to predict the behavior of these layers during the construction phase and to prepare for possible disbeliefs due to changes in the terrain to be crossed (*Bouvard et al*., 1992).

V.3.2. Resources/Tools of the Hydrogeological Investigation

The resources required for a geotechnical site investigation include all of the techniques listed below, which can be applied as needed and available. However, laboratory testing remains a mandatory part of any geotechnical investigation. The main laboratory tests on rocks are summarized in table 1. It should be noted that Soundings/Core boring can be used for shallow underground structures where it is important to know the nature and mechanical properties of the overburden layers. However, for deep structures, this means of investigation becomes very costly.

- Inventory of existing data (previous geotechnical studies),
- Geophysics,
- Soundings, and
- Laboratory tests.

Table II-1: Common Laboratory Tests on Rocks (*FHWA-NHI*, 2009)

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

V.4. Exploratory Tunnel

When a major underground structure is planned at great depth, drilling (making holes/penetrating tests) becomes costly and uneconomical. The best solution in this case is to build a reconnaissance drift or tunnel close to the structure to be excavated. Once the work has been completed, this gallery is used as a safety and ventilation exit.

Even if the cost of its construction remains relatively high, since it is an underground work of reduced dimensions compared to the planned structure, this gallery allows us to obtain the most precise information on the different layers of ground traversed during excavation, given the opportunity of their direct visibility. What's more, several in-situ tests and measurements can be carried out inside the exploratory tunnel. These include the following:

- Measurement of natural stresses in rock (using the flat-jack or the stress-relief methods),
- Measurement of rock deformation (plate test) and rock abrasiveness (LCPC test),
- Measurement of ground convergence (using extensometers or invar wire).
- Measurement of seismic wave propagation velocity (using seismic refraction),
- Measurement of rock strength (compressive and tensile (Franklin test),
- Measurement of discontinuity joints in rock, type of filling between joints, etc.

V.5. Reconnaissance during the Construction Phase

In addition to the three main types of investigation mentioned above, reconnaissance tests are often carried out during the excavation of the underground structure, to check that certain data do not fluctuate and to verify the values initially obtained. This is because certain characteristics, particularly those relating to the hydrogeology and geotechnics of the terrain, may change between the test period and the construction period, or from laboratory conditions to actual field conditions (*Bouvard et al*., 1992).

Summarizing what has been said in this chapter about the campaign of reconnaissance of an underground structure, a flow chart can be drawn up to schematically summarize the different types of investigations to be carried out before and during the construction of an underground structure (Fig. II-8).

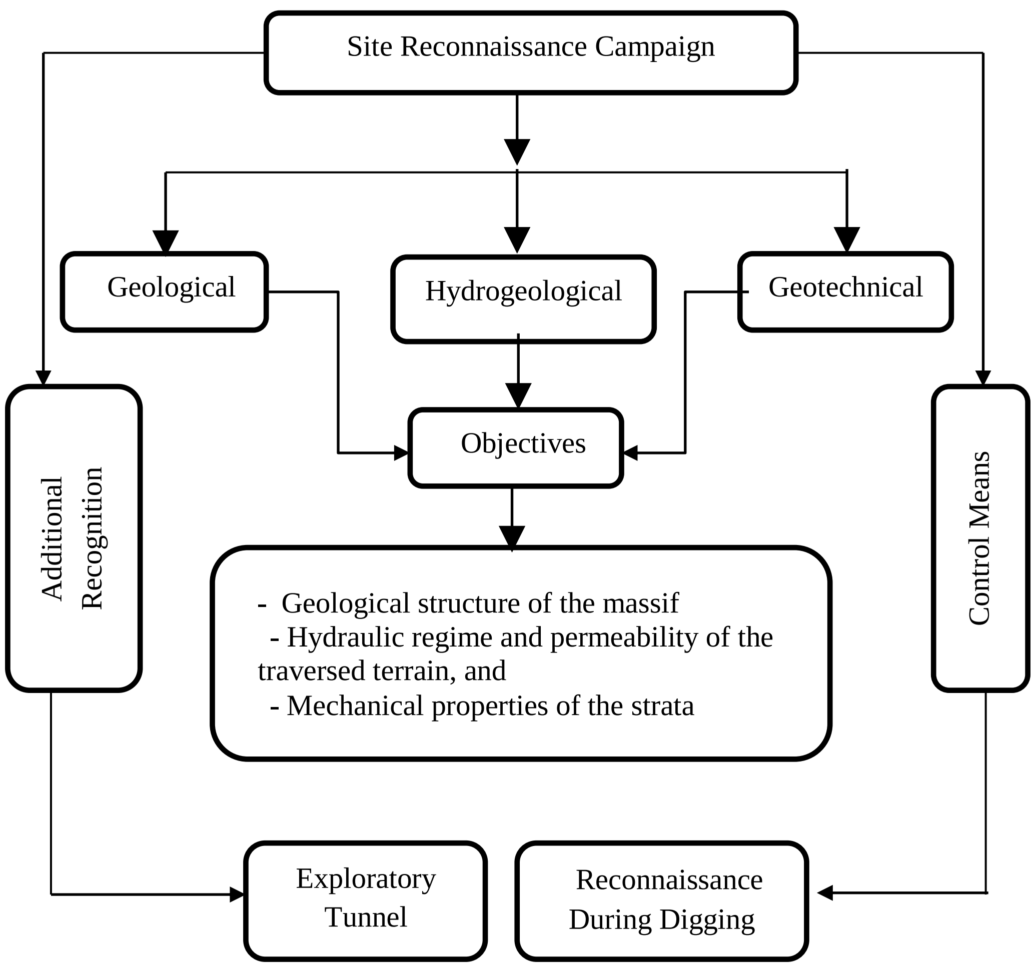


Fig. II-8: Types and objectives of site investigations of underground structures

Chapter Three: Support Design Methods

A. Support Dimensioning Using Empirical Methods

At the preliminary design stage, underground structures are frequently dimensioned using empirical methods based on rock classification. Currently, rock mass classification schemes are utilized alongside numerical simulations, particularly in the initial phases of geotechnical projects, when data is typically limited. Rock mass classifications can be used to determine strength and deformation parameters based on specific constitutive laws such as the Mohr-Coulomb or *Hoek-Brown* material models. These parameters can then be applied in numerical simulations to analyze stability, failure patterns, Factor-of-safety, deformations, etc. several examples of applying rock mass classification schemes in engineering practice, particularly in underground mining and slope stability exist (*Abbas and Heinz Konietzky*, 2017).

I. Empirical Methods

Rock classification methods or empirical methods, which are quick to use and therefore economical, are based on various geotechnical parameters. It is the choice of these parameters and the way in which they are used to design the structure that will make the difference between one method and another. The main empirical methods for dimensioning underground structures are:

- *TERZAGHI* method;
- The AFTES recommendations;
- Z. *BIENIAWSKI* method; and
- *N. BARTON* method.

Rock mass classification has many advantages, such as improving the quality of site investigations by calling for a systematic identification and quantification of input data. Then, a rational, quantified assessment is more valuable than a personal assessment. Furthermore, providing a checklist of key parameters for each rock mass type, i.e. it guides the rock mass characterization process. In addition, it gives results in quantitative information for design purposes and enables better engineering judgment and more effective communication on a project (*Bieniawski*, 1993). Finally, the rock mass classification offers correlations between rock mass quality and mechanical properties of the rock mass. These correlations have been established and are used to determine and estimate its mechanical properties and its squeezing or swelling behavior.

In contrast, rock mass classification has also disadvantages. According to *Bieniawski* (1993), the major pitfalls of rock mass classification systems arise when (*Bieniawski*, 1989):

- Using rock classifications as a definitive empirical "cookbook", i.e. ignoring analytical and observational design methods,
- Using only one rock classification system, i.e. without cross-checking the results with at least one other system,
- Using rock classifications without sufficient input data,
- Using rock classifications without a full understanding of their conservative nature and the limitations that arise from the databases on which they were developed.
- Other important (often dominating) factors are not considered, and
- Rock classification results may be used inappropriately (e.g., to claim that conditions have changed).

I.1 *Terzaghi*'s Method

In 1946, *Terzaghi* proposed a simple classification of rock masses to estimate the loads acting on steel hangers in tunnels. This method and its subsequent developments formed the basis of tunnel design in the USA for many years. To estimate the load, *Terzaghi* assumes that a certain amount of rock or soil will decompress and weigh on the support, which is then given by the general formula:

$$H_p = K\,B + H_t \quad \text{(III-1)}$$

Where:

H_p : Height of soil surcharge evenly distributed horizontally,

B : Gallery width.

H_t : Gallery height, and

K : represents a multiplication coefficient, the value of which depends on the category of the land encountered.

In theory, the formula can be applied to tunnels of medium depth, where the headroom H is greater than 1.5 (B + Ht). On the imprecision of the value obtained, *Terzaghi* states “Even if calculation programs can give more correct values for specific situations, they can only give impractical values for pre-dimensioning, given the uncertainty of the conditions of rocks adjacent to the excavation”.

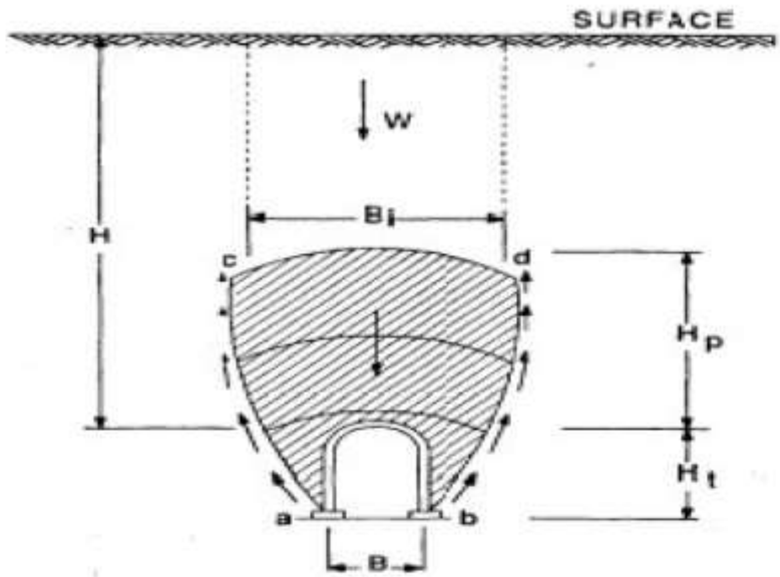


Fig. III-1: Decompressed zone above a cavity (*Terzaghi* 1946).

Table III-1: Height of decompressed ground above a cavity (after *Terzaghi*,1946).

Rock Nature	H_P Load	Remarks
Hard and intact	0 to 0,25 B	a few anchors in case of rockfall
Hard and laminated	0 to 0,50 B	Light support
Solid with some joints	0 to 0,25 B	The load may change suddenly from one point to another
Moderately chipped	0,25 to 0,35 $(B+H_t)$	No lateral pressure
Very loose	0,35 to 1,1 $(B+H_t)$	Little or no lateral pressure
Completely crushed Chemically inert	1,1 $(B+H_t)$	Considerable lateral pressure
Flowing rock at moderate depth	1,1 to 2,1 $(B+H_t)$	High lateral pressure recommended circular hangers
Flowing rock at great depth	2,1 to 4,5 $(B+H_t)$	High lateral pressure recommended circular hangers
Swelling rock	Up to 75 m Independent of $(B+H_t)$	circular hangers, in extreme cases sliding hangers

Concerning the vault effect, *Terzaghi* also studied the influence of rock conditions and the increase in load after the support is in place. To this end, he defines the vaulting period as the time between excavation and the fall of the unsupported part of the vault. This varies from a few hours for swelling rock to a few days for other types of rock, and even infinity for sound rock.
Note that this classification is only valid for tunnels with a rectangular cross-section and supported by ribs.

I.2 The AFTES Recommendations

The AFTES (*Association Francaise des Tunnels et de l'Espace Souterrain*) gives a set of recommendations rather than an actual calculation method. These recommendations are given in the form of a table which, for each rock classification criterion and for each type of support, indicates the feasibility or otherwise of the support under consideration, depending on the value of the parameter characterizing the criterion in question (support highly recommended (clearly advantageous), possible, not suitable or impossible in principle). By superimposing the results for each criterion, the most appropriate type of support can be selected. The most important criteria for the selection of a support system are (AFTES, 2000):
- Rock strength, discontinuities, weathering (disintegrating, the mechanical or chemical breaking down or dissolving of rocks such as erosion), hydrological conditions, and natural constraints.

It should be noted that the AFTES recommendations result in the selection of the type of support rather than its design, and do not incorporate recent developments in pre-support and face reinforcement processes (pre-vaulting, umbrella vaulting, face bolting, jet grouting, etc.). Figure III-2 shows an example of a table relating to the discontinuities criterion.

Discontinuités (cas où l'excavation est faite à l'explosif avec découpage) 1 - Matériaux rocheux (R1 à R4)					Boulons			Cintres		Voussoirs				Procédés spéciaux		
			pas de soutènement	béton projeté	à ancrage ponctuel	à ancrage réparti	barres foncées	lourds	légers coulissants	plaques métal assemblées	béton	tubes préforés	bouclier ou pousse tube	injection	air comprimé	congélation
Nombre de familles	Orientation	Espacement (1)				(2)										
N1			●				X			**X**	**X**	X	X	X	X	X
N2	Or2 ou Or3	S1 à S3	●				X			**X**	**X**	X	X	**X**	X	X
N2 N3 ou N4	Quelconque	S1			●		X			**X**	**X**	X	X	**X**	X	X
		S2			●	●	X			**X**	**X**	X	X	**X**	X	X
		S3		●	Gr	Gr ●	X				**X**	X	X		X	**X**
		S4	**X**	●	Gr ou Bp	● Gr ou Bp	X	● Bl ou Bp	● Bl ou Bp				X		X	**X**
		S5	X	●	**X**	Bp	X	● Bl ou Bp	● Bl ou Bp				**X**		X	**X**
N5			X	●	X	Bp	X	● Bl ou Bp	● Bl ou Bp	●			**X**	●	X	**X**

Fig. III-2: Example of AFTES recommendations: tunnel support based on criterion relating to discontinuities (AFTES, 1993)

Legend:

Gr : Continuous Wire Mesh,
BP : Shotcrete,
BI : wood or metal shielding,

● Particularly recommended (clearly favorable)

Possible if other criteria are particularly favorable

X Very unsuitable, although possibly possible (rather unfavorable)

X Clearly unfavorable.

I.3 *BIENIAWSKI*'s Method

In 1983, Z. *Bieniawski* from the South African Council for Scientific and Industrial Research proposed a method based on the classification of the rock mass for underground tunneling, combining five parameters: the Rock Quality Designation (*RQD*), the rock Compressive Strength / modulus of elasticity, the rock discontinuities spacing (joints spacing), the type of joints and their filling (morphology of walls of rock fractures), and the groundwater pressure and flow.

I.3.1. Rock Quality Designation

The Rock Quality Designation *R.Q.D.* is a parameter used to classify the rock according to its quality. It was proposed by *D. Deere* in 1964 and is determined from observations made on samples taken from a borehole (Fig. III-3). It is defined as the percentage of intact core pieces in the total length of a core having a diameter of at least 50 mm to ensure that pieces of sound rock are not broken during sampling. It is calculated over the length of the borehole as the sum of core pieces lengths over 10 cm divided by the length of the core pass, which is usually assumed to be equal to one meter. For example, core samples several decimeters long can be extracted from mylonite[1]; such a sample cannot be considered as a piece larger than 10 cm. The *R.Q.D.* parameter is often calculated for each meter of coring. Table III-2 gives the rocks classification according to the *R.Q.D.* value.

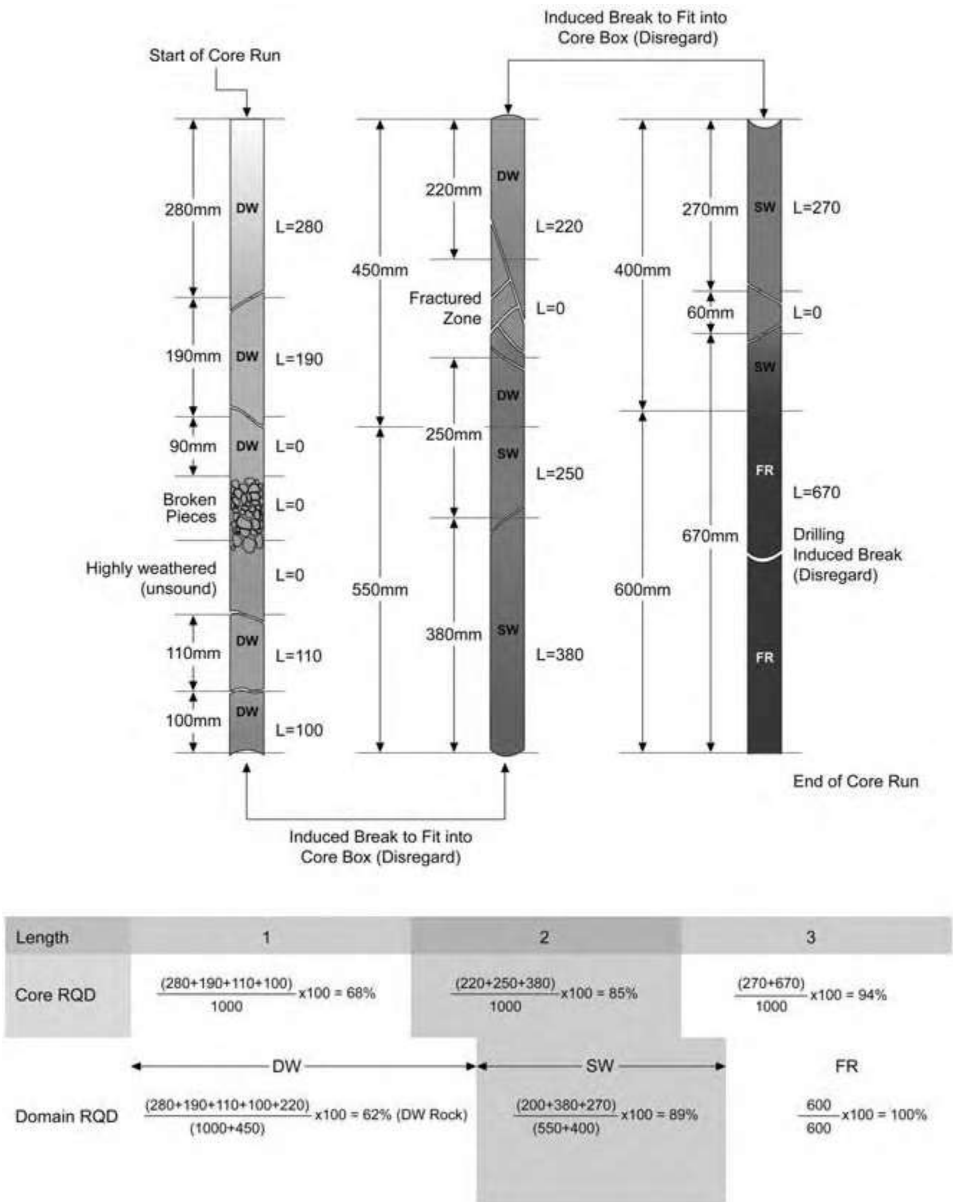
$$RQD(\%) = 100 * \frac{\Sigma\ \text{core pieces of length over } 10\ cm}{\text{length of the core pass } (cm)} \quad \dots\dots \quad \text{(III-2)}$$

Fig. III-3: Core sampling for RQD parameter calculation

Table III-2 : Classification of the rock according to the R.Q.D. parameter (after *Deere*, 1966)

Designation	*RQD* Value (%)
Very poor	0-25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

[1] *The mylonite is a fine-grained metamorphic rock that is formed through intense shearing and crushing of various rocks along tectonic zones influenced by strong pressure or dynamic stress.*



<u>Legend:</u>
DW: Distinctly weathered rock (Rock strength usually changed by weathering); SW: Slightly weathered rock (Rock is slightly discolored but shows little or no change of strength from fresh rock; FR: Fresh rock (Rock shows no sign of decomposition or staining).

Fig.III-4: Example of a *RQD* measurement (*Look*, 2007).

Note that in a very weak and fragile rock environment, the rock mass may have an *RQD* value of zero, meaning that in such a case the behaviour of rock is similar to that of soil. Hoek et al. (1993) proposed a new rock classification approach to overcome the perceived limitation, called Geological Strength Index (*GSI*).Without considering *RQD*, *GSI* is suitable for an environment with a very weak rock mass. It assumes that rock mass is isotropic material and the behaviour of rock mass does not depend on the direction of the applied load (Hussian et al., 2020).

The Geological Strength Index *GSI* can evaluate the quality and deformability of rock mass with limited test data and site characterization information. The estimated rock mass properties and discontinuity features provide reliable input information for numerical analysis. This means once the value of *GSI* is determined; it is employed in some empirical equations to calculate rock mass properties, such as deformation modulus, Poisson's ratio and compressive strength. Under the condition of *RMR* < 15, the relationship between *RMR* and *GSI* can be expressed in an equivalency as: (*GSI* = *RMR* – 5). Under other conditions, different relationships exist and can used to determine GSI based on RMR parameter (Hussian et al., 2020).

I.3.2. Rock Compressive Strength Calculation

For rocks of moderate to high strength, point load index is conventional. *Bieniawski* suggests evaluating the compressive strength of rock using the point load test, in which a core is loaded along a diameter by two steel points (*Broch & Franklin*, 1972). This gives the point load index "Is", also known as the Franklin index (equation III-3).

$$\boldsymbol{Is} = \frac{\boldsymbol{P}}{\boldsymbol{D^2}} \ldots\ldots\ldots\ldots\ldots\ldots \textit{(III-3)}$$

Where : *P* is the ultimate breaking load of the specimen and *D* is its diameter (in mm).

Finally, to estimate the value of the rock's compressive strength, a direct correlation exists between the I_S index and the R_C strength for a coring tool of 50 mm diameter (Eqt. III-4). Table III-3 gives compressive strength values for some rocks.

$$\boldsymbol{R_c = 23.\, I_{s\,(50)}} \ldots\ldots\ldots\ldots\ldots\ldots \textit{(III-4)}$$

Table III-3: Examples of compressive strength values for some rocks (*Bouvard et al*., 1992).

Resistance Description	R_c (MPa)	Rock
Very low resistance	1-25	Chalk, Salt
Low resistance	25-50	Coal, Limestone, Shale
Medium resistance	50-100	Sandstone, Slate, Clay

High resistance	100-200	Marble, Granite, Gneiss
Very high resistance	>200	Quartzite, Dolérite, Gabbro, Basalte

I.3.3. Spacing of Rock Discontinuities

Discontinuities are defined as joints, faults, stratifications and other planes of weakness in the rock (Fig.III-5). *Bieniawski* uses the classification proposed by *Deere* (1964) for this parameter (Table III-4).

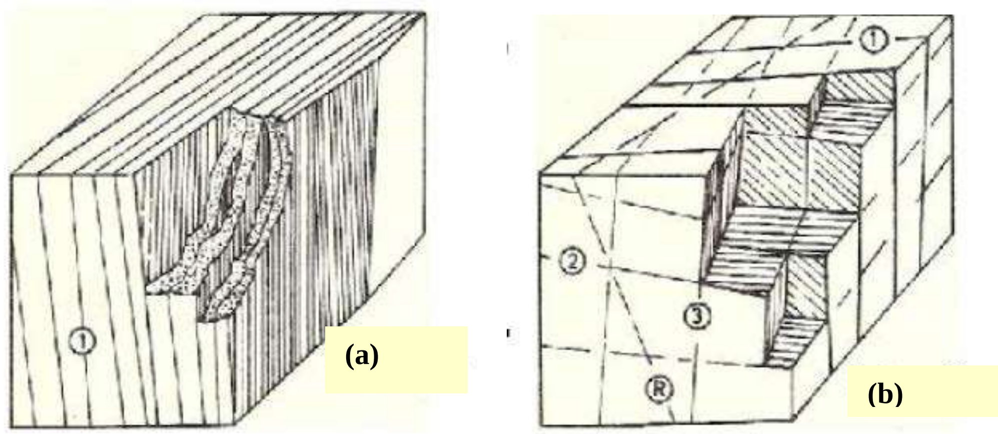


Fig. III-5: Joint set Number in rock mass: (a) one joint set, (b) three joint set (*Bouvard et al*., 1992**)**

Table III-4: Diaclase distance classification (*Deere, 1964*)

Description	Spacing	Massif Conditions
Extra wide	> 3m	Solid
Wide	1 à 3m	solid
Medium tight	0,3 à 3m	A bloc
Tight	50mm à 0,3m	Fractured
Very tight	<50mm	Crushed

Given that the rock quality designation RQD is an indicator of the rock fracturing, a connection exists between the RQD value, the fracture frequency per meter and the discontinuity spacing (Table III-5).

Table III-5: Correlation between *RQD* and discontinuity' spacing (After *Look*, 2007).

RQD (%)	Fracture frequency per meter	Typical mean discontinuity spacing (mm)
0–25	>15	<60
25–50	15–8	60–120
50–75	8–5	120–200
75–90	5–1	200–500
90–100	≤1	>500

I.3.4. Joints Nature (Quality of joint surface and dip of Rock Discontinuities)

This parameter takes into account joint opening, joint continuity, joint roughness and the presence of filler materials between joints (Figures III-6 and III-7). However, the orientation of joints or their dip is dealt with separately, as its influence is marked differently depending on the type of application, i.e. tunnels, embankments or foundations. In the case of tunnels, an adjustment factor is considered. The latter, which is the result of a qualitative estimation, is given in Table III-6. In Figure III-7 below, different configurations

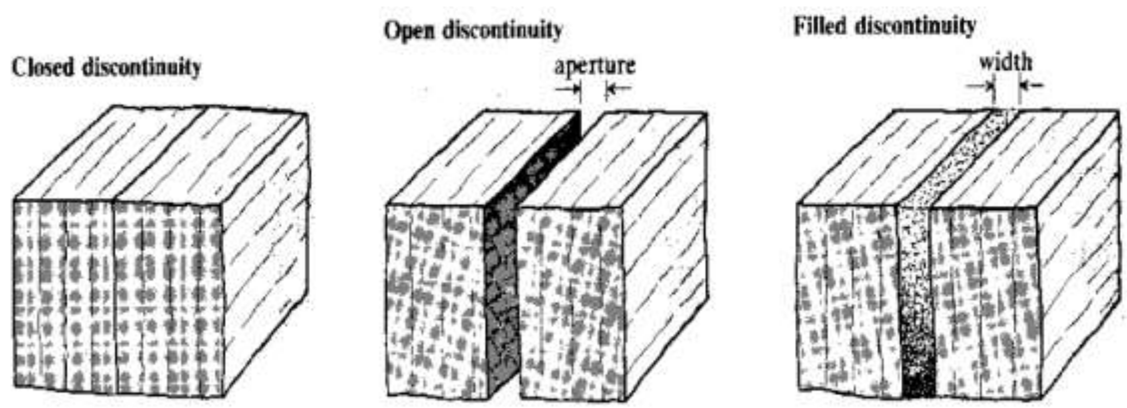


Fig. III-6: Joint nature: (left) closed discontinuity, (middle) open discontinuity, and (right) filled discontinuity (CFMR, 2000)

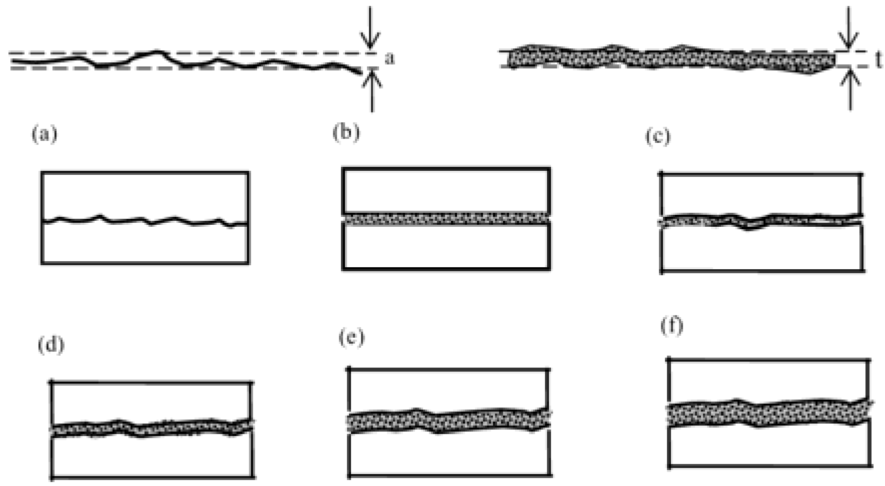


Fig. III-7: Joint nature: (a) rough and interlocked discontinuity surface, (b) planar discontinuity surface with no rock contact, (c) non-planar discontinuity surface with almost immediate rock contact (t << a), (d) non-planar discontinuity surface with possible rock contact after some shearing (t < a), (e) non-planar discontinuity surface with no rock contact (t > a), and (f) discontinuity surface dominated by filling material, and where influence by surface roughness is not possible (t >> a) (CFMR, 2000).

Table III-6: Score Adjustment for Joint Orientation

	Digging perpendicular to tunnel axis				**Digging parallel to tunnel axis**		**Any Orientation**
	Digging along the dip (in the same direction)		**Digging against the dip**				
Dip	45°- 90°	20°-45°	45°-90°	20° - 45°	45° - 90°	20° -45°	0° - 20°
Joints Orientation	Very favorable	Favorable	Medium	Unfavorable	Very unfavorable	Medium	Unfavorable
Score N_{adj}	0	-2	-5	-10	-12	-5	-10

I.3.5. Hydrological Conditions (Water Inflows)

The influence of groundwater on the stability of the excavation can be considered by measuring the flow rate of water entering the structure, the ratio of the water pressure in the joints to the maximum principal stress, or by qualitative observation of the water inflow. *Bieniawski* assigns a weighting to the various rock classification parameters (Table III-7).

I.3.6. Rock Mass Rating Calculation

Finally, after summing the scores obtained for the five main parameters (Tab. III-7), and adding the adjustment score (Tab. III-6) to account for fracture orientation, the overall score obtained to determine the rock class is commonly referred to as the "Rock Mass Rating" or "*R.M.R*" (equation III-5). Using Table III-8, the *R.M.R* can then be used to determine the rock class (from very good to very poor) and the time during which an excavation is stable without support. Figure III-8 can also be used for this purpose.

$$\boldsymbol{R\ M\ R} = \sum_{1}^{5} \boldsymbol{N_i} + \boldsymbol{N_{adj}} \ldots\ldots\ldots (III\text{-}5)$$

Table III-7: Parameters and Weighting Score for Rock Mass Rating System assessment (*Bieniawski*, 1989)

A. Classification parameters and their rating									
1	Strength of intact rock material	Point-load strength index	>10 MPa	4–10 MPa	2–4 MPa	1–2 MPa	Use of uniaxial compressive test preferred		
		Uniaxial compressive strength	>200 MPa	100–200 MPa	50–100 MPa	25–50 MPa	10-25 MPa	1–5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90–100%	75–90%	50–75%	25–50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>2 m	0.6–2 m	0.2–0.6 m	60–200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of joints		Very rough surface Not continuous No Separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces OR Gouge < 5 mm thickness OR Joints open 1–5 mm Continuous joints	Soft gouge > 5 mm thick OR Joints open > 5 mm Continuous joints		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length	None	<10 L/min	10–25 L/min	25–125 L/min	>125 L/min		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0	<0.1	0.0–0.2	0.2–0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Table III-8: Rock class and dwell time of unsupported excavated rock

Rating (RMR)	**100-81**	**80-61**	**60-41**	**40-21**	**< 20**
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Average stand-up time	10 (years) For 5 m span	6 months For 4 m span	1 week For 3 m span	5 hours For 1,5 m span	10 minutes For o,5 m span

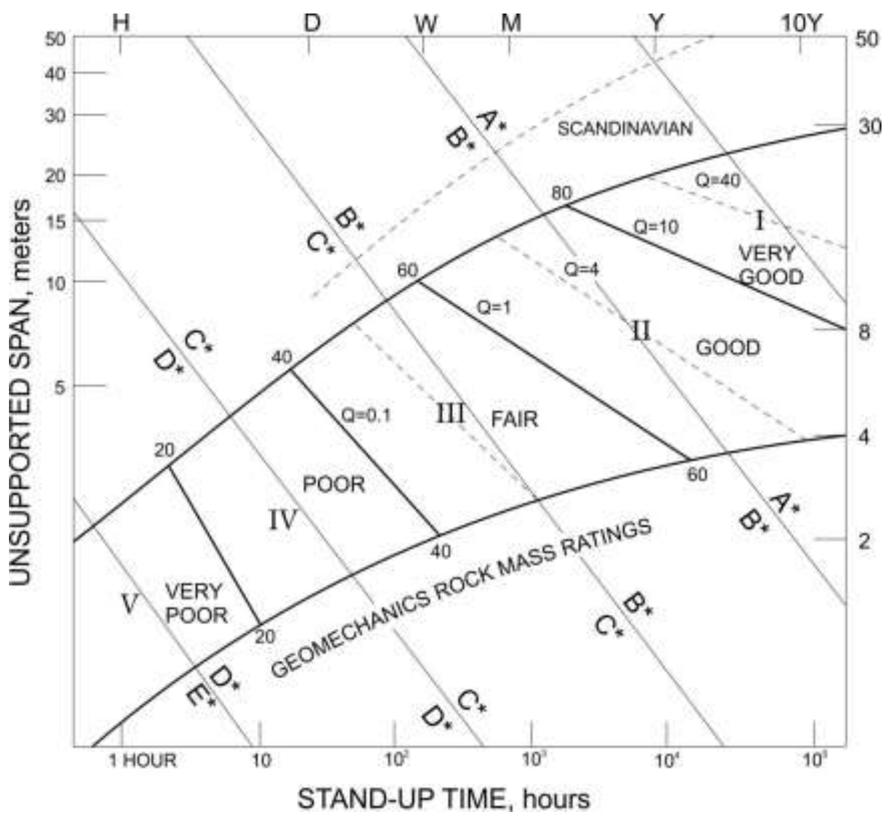


Fig. III-8: Relationship between stand-up time of unsupported span for diverse rock mass classes as categorized by the *RMR* System (*Lauffer*, 1988).

I.3.7. Bieniawski's Support Recommendations

The engineer must choose between these types of support according to the availability of materials (ribs, hangers, anchor bolts, etc.) and the financial framework of the project. In fact, in order to reduce the cost of the support and, consequently, the cost of the project, it is sometimes preferable to use a type of support whose components are produced locally, such as shotcrete (concrete and welded mesh produced in Algeria), rather than an imported type of support, such as anchor bolts.

Table III-9: Bieniawski's Support Recommandations

Rock Class	**Support Type**						
	Anchor Bolts		**Shotcrete**			**Steel ribs (arches)**	
	Spacing (m)	Anchor Complement	Vault (mm)	Walls (mm)	Support Complement	Type	Spacing (m)
I	Generally not required						
II	1-1,5	Occasionally welded mesh in vault	50	None	None	Uneconomical	
III	1-1,5	Welded mesh + 30 mm shotcrete in vault if required	100	50	Occasionally welded mesh + bolts if required	Lightweight ribs/hangers	1,5 - 2
IV	0,5-1	Welded mesh + 30 mm shotcrete in vault and pedestal	150	100	Welded mesh and bolts at intervals of 1.5 to 3 meters	Medium hangers + 50 mm of shotcrete	0,7 - 1,5
V	Not advised		200	150	Welded mesh-bolts and lightweight ribs/hangers	Immediately 80 mm of shotcrete + heavy ribs /hangers on the move	0,7

I.3.8. *R.M.R* Use in Determining Rock Mechanical Parameters

There are correlations between the Rock Mass Rating and the rock deformation modulus *E*. Thus, using *R.M.R* parameter *E* modulus can be obtained using several equations. One example of the existing equations, one can cite the *Serafim and Pereira (1983)* equation:

$$\boldsymbol{E = 10^{\left(\frac{RMR-10}{40}\right)}} \ldots\ldots\ldots\ldots \textit{(III-6)}$$

Besides, using *R.M.R*, the equations below can also be used to estimate the tensile and compressive strengths of the rock mass in situ (*Bouvard et al*., 1992).

$$\boldsymbol{R_{CM} = R_c\sqrt{S}} \ldots\ldots\ldots\ldots \quad \textbf{\textit{(III-7)}}$$

$$\boldsymbol{R_t = \frac{RcM}{2}\left(m - \sqrt{4.S + m^2}\right)} \ldots\ldots\ldots \textbf{\textit{(III-8)}}$$

where :

$S = e^{\frac{RMR-100}{9}}$ and $m = 7.e^{\frac{RMR-100}{28}}$

Furthermore, direct relationships between the rocks mass classification and the mechanical properties (E, φ, c, σ, ν) have been empirically established and summarized by *Aydan et al.,* (2014). Some of them are reported on table III-10.

Note that there is a correlation between the *RQD* values and the rock strength, which can be used in rock foundation bearing capacity assessment. For a *RQD* values ranging from 0 to 70 % (very poor to fair rock), the design compressive strenght to be considered is equal to 0.33 *UCS* (Uniaxial Compressive strenght of intact rock core), which corresponds to a cohesion of 0,1*USC*, and a friction angle of 30^0. For *RQD* vaules >70% (good to excellent rock), the design compressive strenght to be considered ranges between 0.33 to 0.8 *UCS*, corresponding to a cohesion of 0,1*USC*, and a friction angle ranging from of 30^0 to 60^0 (*Look*, 2007).

Table III-10: Relationships between rock mass rating and rock mass mechanical properties (After *Aydan et al.*, 2014)

Rock Mass Properties	Empirical Correlation	Author (s)
Deformation Modulus *E* (GPA)	E=2 RMR-100 (for RMR>50)	*Bieniawski (1978)*
	$E = e^{(4.407+0.081+RMR)}$	*Jasarevic and Kovacevic (1996)*
	$E=0.0876\ RMR$ (if $RMR > 50$) $E=0.0876RMR+1.056(RMR-50)+0.015(RMR-50)^2$ (if $RMR < 50$)	*Galera et al. (2005)*
Deformation Modulus *E* (MPA)	$E=0.0097\mathrm{RMR}^{3.54}$ $E= 0.1\ (RMR/10)^3$	*Aydan et al. (1997)* *Read et. al. (1999)*
Friction Angle *φ* (°)	$\varphi = 20 + 0.5\ RMR$	*Aydan and Kawamoto (2001)*
	$\varphi = \tan^{-1}(Jw \times (Jr/Ja))^{*}$	*Barton (2002)*
Cohesion *C* (MPa)	$C = (\sigma/2)\ (1-\sin\varphi)/\cos\varphi)$	*Aydan and Kawamato (2001)*
	$C = (\frac{RQD}{Jn} * \frac{1}{SRF} * \frac{\sigma}{100})^{**}$	*Barton (2002)*
Uniaxial compressive strength, *σ* (MPa)	$\sigma = 0.0016\ RMR^{25}$	*Aydan et al. (1997)*
Poisson's ratio ν	$\nu = 0.25\ (1+ e^{-\sigma/4})$	*Aydan et al. (1993)*
	$\nu = 0.5-0.2\ \frac{RMR}{(RMR+0.2(RMR-100))}$	*Tokashiki and Aydan (2010)*

* *Jw/Jr is a Ratio that characterizes the shear strength of blocks in relation to each other.*
** *RQD/Jn is a quotient approximating the size of boulders and SRF is the Stress Reduction Factor, respectively.*

In conclusion, it should be noted that classification systems are not appropriate for use in the detailed design stage, especially for intricate underground structures. Further improvement of these systems is necessary for the classification to be used effectively. The rock mass classification systems were created to assist in engineering design and were never meant to replace field observations, analytical thinking, measurements, and engineering judgment (Bieniawski, 1993). It is to mention that Rock mass classification systems have been helpful tools in the planning of various engineering projects, specifically those involving underground constructions, tunneling, and mining (*Hoek*, 2007).

It is also important to mention the advantages and weaknesses of rock mass classification systems. Indeed, through the rock mass classification the quality of site investigations is enhanced, since it offers a set of important parameters for each rock mass type, helping to direct the process of characterizing the rock mass. In addition, this classification provides numerical data for design needs and allows for improved engineering decision-making and clearer project communication (Bieniawski, 1993).

Finally, correlations have been identified between rock mass quality and its mechanical properties, which are utilized to assess its mechanical characteristics. Conversely, according to Bieniawski (1993), the primary difficulties with rock mass classification systems arise when:

- Disregarding analytical or observational design methods and depending only on rock mass classifications as the final practical guide;
- Using only one rock mass categorization system without using at least one other system to confirm the results,
- Using rock mass classifications without fully comprehending their conservative nature and the limitations arising from the database used for their construction;
- Applying rock mass classifications with inadequate input data;
- The findings of rock mass classification are easily abused.

I.4. *BARTON*'s Method

Based on the analysis of more than 200 underground cavities (mainly road and hydroelectric tunnels), *Barton* and his research team from the Norwegian Geotechnical Institute (NGI) have proposed an index for determining the quality of a rock mass with respect to tunneling. *BARTON*'s method, developed in 1974, is an empirical classification of rock masses.

I.4.1. Method Principle

The principle of this classification is the same as that of *BIENAWSKI*'s classification, i.e. "to note the quality of the rock mass by means of parameters". The quality of the rock mass is represented by the Q index, calculated from six parameters. Once calculated, the Q index can be used to define the type of support to be installed, provided that the width of the excavation and its function (road or rail tunnel, storage gallery, etc.) are known. Knowing Q also enables correlations to be made between different parameters, such as equivalent RMR, deformability modulus, pressure exerted on the lining at roof and wall level, and wave velocity

I.4.2 Calculation of the Rock Quality Index *Q*

The value of this coefficient Q is determined from the knowledge of six parameters, and is given by equation (III-9):

$$\boldsymbol{Q} = \frac{RQD}{Jn} \; x \; \frac{Jr}{Ja} \; x \; \frac{Jw}{SRF} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}9)$$

Where the three quotients represent:

$\frac{RQD}{Jn}$ Quotient approximating the size of boulders,

$\frac{Jr}{Ja}$ Quotient characterizing the shear strength of the blocks against each other, and

$\frac{Jw}{SRF}$ Quotient characterizing the active constraints and forces.

Note that the last quotient is an empirical factor representing active stress incorporating water pressures and flows, the presence of shear zones and clay bearing rocks, squeezing and swelling rocks and in situ stress state (*Hoek*, 2007). According to *Singh and Geol* (1999), SRF is a measure of :

- loosening load in the case of an excavation through shear zones and clay bearing rock,
- rock stress in competent rock, and
- Squeezing loads in plastic incompetent rocks. The quotient increases with decreasing water pressure and favorable in situ stress ratios.

Detailed descriptions and values of the various parameters used to calculate the Q index are given below (*Barton* et al., 1974):

- RQD: a parameter that stands for Rock Quality Designation. It is determined on the basis of a cored borehole with a diameter of around 50 mm and calculated over the length of the borehole pass.
- Jn: represents the joint set number (number of discontinuity sets). It expresses the number of main families of discontinuities. Its description and values are given in Table III-11.
- *Jr*: a factor representing the Joint roughness number for critically oriented joint set (roughness of discontinuity surfaces). Its description and values are given in Table III-12,
- *Ja*: a factor representing the degree of alteration or weathering and filling of discontinuity surfaces for critically oriented joint set (joint thickness and type of filling material. Its description and values are given in Table III-13
- *Jw*: a factor representing the pressure and inflow rates of water within discontinuities, it specifies hydrogeological conditions, including the size and pressure of water inflows. Its description and values are given in Table III-14, and
- *SRF:* Stress Reduction Factor representing the presence of shear zones, stress concentrations, squeezing or swelling rocks). It gives an idea about the state of stress in the massif (Fig. III-9). Its description and values are given in Table III-15.

Table III-11: Description and values of J_n parameter

Joint Set Number		*Jn*
A	Massive rock, no or few joints	0,5 -1,0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint set	4
E	Two joint set plus random joints	6
F	Three joint set	9
G	Three joint set plus random joints	12
H	Four or more joint set, random joints, very dense fracturing, etc…	15
J	Crushed rock, earthlike	20

Note: (i) For intersections, use (3.0 × Jn).
(ii) For portals, use (2.0 × Jn).

Table III-12 : Description and values of J_r parameter

Joint roughness Index		*Jr*
(a) Rock-wall in contact, and (b) Shear contact of less than 10 cm		
A	Discontinuous joints	4
B	Rough or irregular, undulating joints	3
C	Undulating and smooth joints	2
D	Slickensided, undulating joints	1,5
E	Rough or irregular, planar joints	1,5
F	Planar and smooth joints	1
G	Slickensided and planar joints	0,5
(c) : Rock-wall not in contact after sheared		
H	Clay zone of sufficient thickness to prevent rock-wall contact	1
J	A sandy, gravelly or crushed area of sufficient thickness to prevent rock-wall contact	1

Note: (i) Descriptions refer to small and intermediate scale features, in that order.

(ii) The value of parameter J_r is increased by 1 if the average distance of all the joints involved is ≥ 3 m.

(iii) Jr = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

Table III-13 : Description and values of *Ja* parameter

Joint Alteration		*Ja*
Rock-wall in contact		
A	Waterproof, resistant and highly consolidated filling	0,75
B	Unaltered joint walls, surface staining only	1,0
C	Extensively altered joint walls. Non-softening mineral coating, sandy particles, clay-free disintegrated rock	2,0
D	Silty or sandy coat with low clay content	3,0
E	Coating with low-friction minerals, i.e. kaolinite, mica, gypsum, chalk, talc, graphite... with a small quantity of swelling clays.	4,0

Rock-wall in contact after a slight shear		
F	Disintegrated rock, with sandy parts but no clay	4,0
G	Strongly over-consolidated clay filling	6,0
H	Weakly to moderately over-consolidated clay filling	8,0
I	Clay filling with swelling minerals e.g. montmorillonite, Ja value depends on percentage of swelling clay size particles and hydration conditions.	8,0-12,0
Rock-wall not in contact		
K,L,M	Zones or bands of disintegrated or crushed rock with clay conditions identical to cases G, H and I	6,0 -8,0 Or 8,0-12,0
N	Zones or bands of silty- or sandy-clay,with small clay fraction	5,0
O,P,R	Thick, continuous zones or bands of clay with identical conditions to cases G, H and I	10,0 -13,0 Or 13,0 -20,0

Table III-14 : Description and values of *Jw* parameter

Description of water flow or pressure	***Jw***	**Approximative water pressure (Kg/cm²)**
A. Dry Excavation or with a very low flow rate (5 l/min) locally	1,0	<1,0m
B. Medium inflow or pressure, joint fillings occasionally bathed	0,66	1,0-2,5
C. High pressure or flow in competent rock without filling in discontinuities	0,5	2,5-10,0
D. High pressure or high flow largely bathed fillings	0,33	2,5-10,0
E. Pressure or flow rate exceptionally high at the time of cutting, then decreasing over time	0,2-0,1	> 10,0
F. Exceptionally high pressure or flow remaining constant over time.	0,1-0.05	>10,0

Table III-15 : Description and values of the *SRF* factor

Stress Factor	*SRF*
(a) Underground structures encountering altered zones likely to induce rock mass instability during excavation.	
A. Frequent zones of weakness, containing clay or chemically decomposed rock	10,0
B. Isolated zones of weakness containing clays or chemically degraded material, at a depth < 50 m	5,0
C. Same as B, but depth > 50 m	2,5
D. Multiple shear zones in competent rock (clay-free) (excavation depth ≤ 50 m)	7,5
E. Isolated shear zone in competent rock (clay-free) (excavation depth ≤ 50 m)	5,0
F. Same as E, but depth > 50 m	2,5
G. Open discontinuities, highly fractured rock (all depths)	5,0
(b) Competent rocks - Stress conditions	
H. Low stress, shallow depth	2,5
I. Medium stress, favorable stress condition	1,0
J. High stress, very tight structure. Usually favorable to stability, may be unfavorable to wall stability	0,5-2
K. Moderate flaking	5-10
L. Important flaking	10-20
(c) Pushing rock (plastic deformation of rock under the action of strong natural stresses)	
M. Average deformation pressure	5-10
N. High deformation pressure	10-20
(d) Swelling rocks (chemical action depending on the presence of water)	
O. Medium swelling pressures	5-10
P. High swelling pressures	10-15

Note that the *Q* index can vary from 0.001 to 1000, and is grouped into 9 classes corresponding to a rock quality described in Table III-16.

Table III-16 : Classification of rock quality as a function of *Q*

***Q* Index Value**	**Rock Quality Class**
400-1000	Exceptionally good
100-400	Extremely good
40-100	Very good
10-40	Good
4-10	Fair
1-4	Poor
0,1-1	Very poor
0,01-0	Extremely poor
0,001-0,01	Exceptionally poor

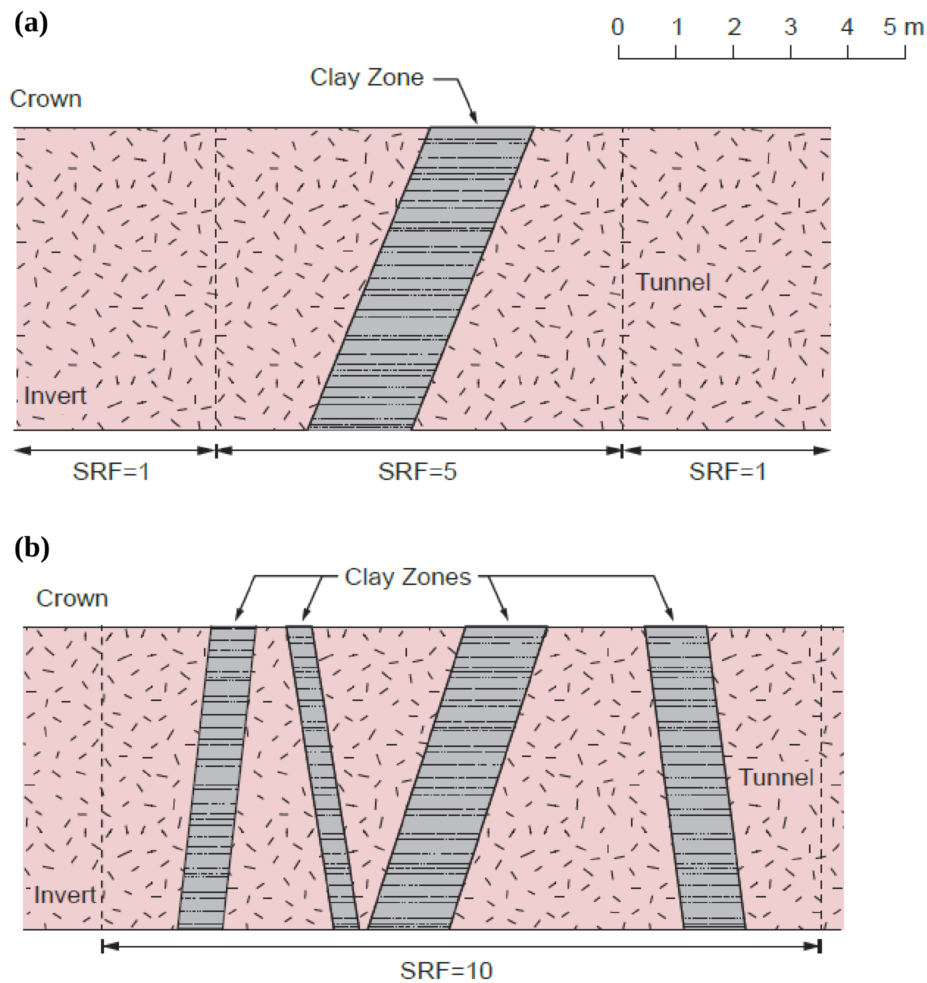


Fig. III-9: SRF values related to: (a) single, and (b) multiple weakness zones (NGI, 2022).

I.4.3 Excavation Equivalent Dimension

After calculating the quality index Q of the rock using equation (III-9), the dimensions and purpose of the structure will lead to one of 38 categories of support (Appendix A). To this end, and in order to be able to relate the index *Q* to support recommendations in underground structures, *Barton* et al. (1974) defined an additional quantity they called the "equivalent dimension *De* of the excavation:

$$De = \frac{\textbf{Excavation span}(s)/\ \textbf{diameter}\ (D)\ \textbf{or height}\ (m)}{ESR} \quad \text{...........(III-10)}$$

Where:

ESR (Excavation Support Ratio) being a factor depending on the required excavation degree of safety (for stability), i.e.. It is a dimension correction coefficient ranging from 0.8 to 3.5. Note that in equation (III-10), the Span/diameter is used for analyzing the roof support, and height of the wall is used in case of wall support. Suggested ESR values are shown in Table III-17.

Table III-17. Values of Excavation Support Ratio, *ESR* (*Barton et al*. 1974)

	Excavation Category	**ESR**
A	Temporary mine openings, etc…	3.5
B	Vertical shafts : - circular cross-section - rectangular / square section	 2.5 2.0
C	Permanent mine openings, water tunnels for hydro power (excluding high-pressure penstocks), pilot tunnels, drifts and headings for large excavation	1.6
D	Storage rooms, water treatment plants, minor road and railway tunnels, access tunnels.	1.3
E	Power stations, major road, and railway tunnels, civil defense chambers, portal inter-sections.	1.0
F	Underground nuclear power stations, railway stations, sports and public facilities, factories	0,8

I.4.4 Support Category

Once the quality index *Q* of the rock and the equivalent dimension *De* of the structure are determined, the support category number can be read from Figure III-10 that represents *Q* index on a logarithmic scale. Finally, *Barton*'s method gives a set of recommendations guiding the engineer in choosing the type of support to be adopted (Appendix A).

In addition to the recommendations outlined in annex A, *Barton* et al. (1980) suggested calculating the rock bolt length *L* in meters using the excavation width *B* and the maximum unsupported span S_{max}, respectively as follows:

$$L = (2 + 0.15 * B)/ESR \qquad \text{(III-11)}$$

$$S_{max} = 2 * ESR * Q^{0.4} \qquad \text{(III-12)}$$

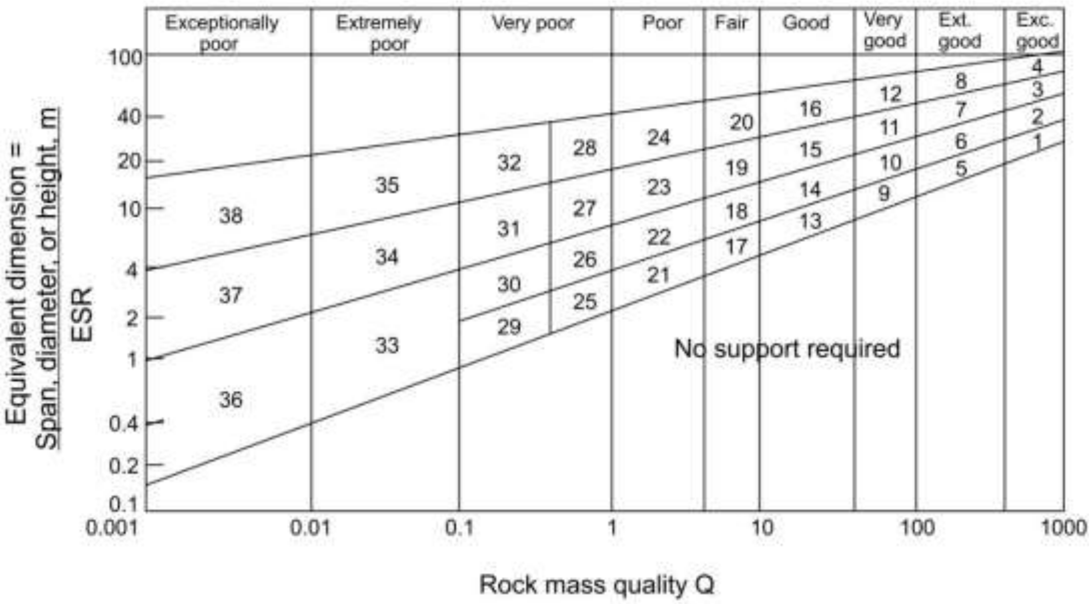


Fig. III-10: Relationship between *Q* Index, *De* and the 38 support categories according to *Barton et al*. 1974

On the other hand, the rising utilization of wet mix steel fiber reinforced shotcrete (SFRS) alongside rock bolts since the early 1980s has led *Grimstad and Barton* (1993) to recommend a revised support design chart that features SFRS, depicted in Figure III-11. However, this chart is advised for use in tunneling operations within suboptimal rock conditions, as noted by *Singh and Geol* (1999).

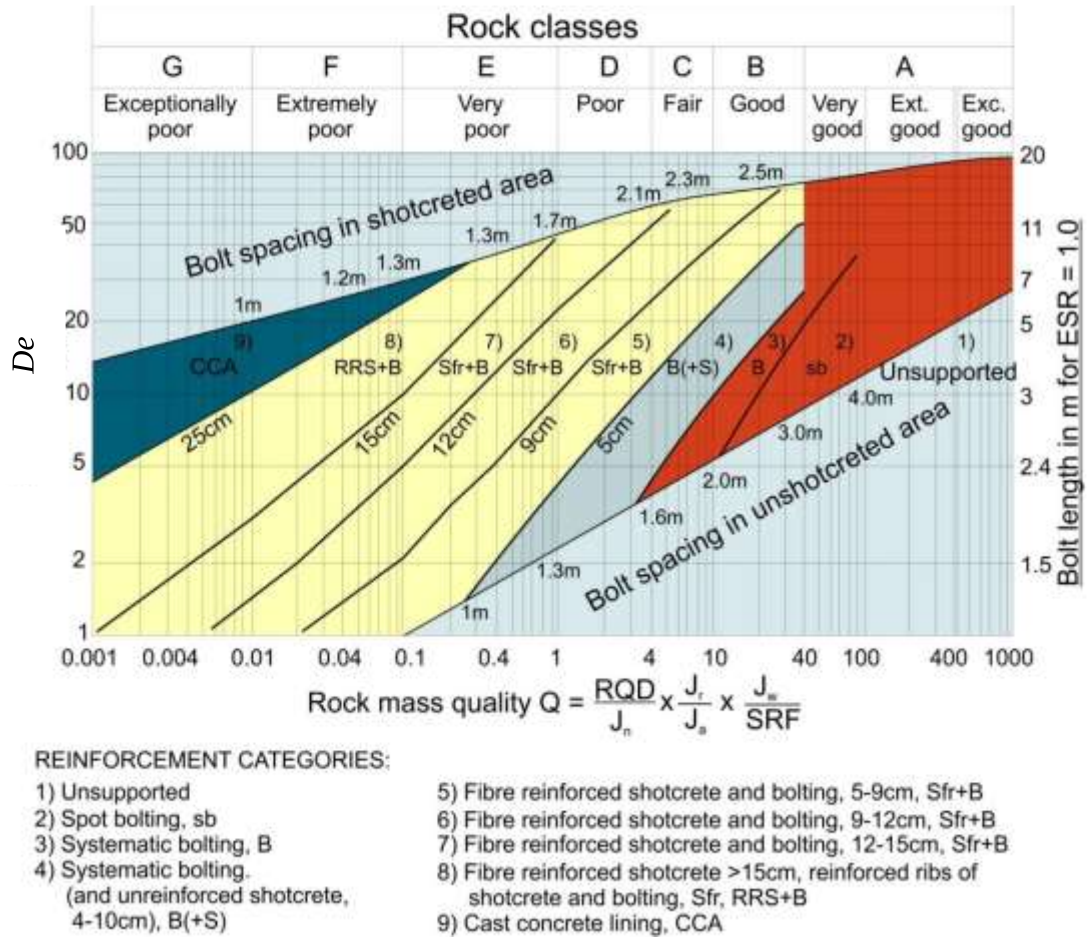


Fig. III-11: Different Support Categories for different rock mass classes (*Grimstad and Barton, 1993*)

Correlations between the *RMR* and the *Q* index exist in the form of semi-log equations. Among them, one can cite that developed by *Bieniawski* (1976):

$$\boldsymbol{RMR = 9 \log Q + A} \quad \text{………………….}(III\text{-}13)$$

Where:

A is derived from 111 case histories in tunneling. It varies between 26 and 62, and its average value is 44. However, *Bieniawski* advises to relate *Q* and *RMR* cautiously (*Bieniawski*, 1989). Another correlation has been proposed by Abad et al. (1983) on the basis of 187 case histories in petroleum mining:

$$\boldsymbol{RMR = 10.5 \log Q + 42} \quad \text{………………….}(III\text{-}14)$$

I.4.5. Uses of the *Q* Index

a) To Determine Forces Applied on the Support

The *Q* value is used to calculate the pressure exerted on the support. At the vault level, the relationship between Q value and the permanent roof support pressure is expressed as follows:

$$P_{roof}(MPa) = \frac{2\sqrt{J_n}Q^{-1/3}}{3J_r} \quad \text{……………….(III-15)}$$

At the level of the wall, observations have shown that the pressure exerted on the support is equal to one third of the pressure exerted at the level of the vault, assuming a "normal" state of stress:

$$\begin{cases} \sigma_1 = \sigma_v \\ \sigma_3 = \sigma_h \\ \frac{\sigma h}{\sigma v} = 0.5 \end{cases} \quad \text{……………….(III-16)}$$

A new index called *Qp* is calculated from *Q*. It corresponds to the *Q* index, but at the wall level, and is called "Wall Quality Index". Depending on *Q* index value, the wall quality index is equal to:

$$Qp = 5*Q \text{ when } Q > 10$$
$$Qp = 2.5*Q \text{ when } 0.1 < Q < 10 \quad \text{……………….(III-17)}$$
$$Qp = Q \text{ when } Q < 0.1$$

b) To Determine the Rock Mass Deformability Modulus and to Predict Strength Parameters

The *Q* index allows calculating the deformability modulus of the rock mass of good quality. There are three deformability modulus values: a minimum modulus E_{min}, a maximum modulus E_{max}, and an average or medium modulus E_{med}. These three moduli can be calculated using the index *Q* as follow:

$$E_{min} = 10*\log Q,$$
$$E_{med} = 25*\log Q, \text{ and} \quad \text{……………….(III-18)}$$
$$E_{max} = 40*\log Q$$

Barton also presents the following formula for calculating E_m from the discontinuity spacing S, the normal joint stiffness Kn and the deformability modulus of intact rock E_i:

$$E_m = E_i \frac{S * K_n}{S * K_n + Ei} \ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}19)$$

Note that all the above formulas can be used to calculate E_{max} in the case of a supported excavation. For excavations without support, the deformability modulus is calculated using the excavation span and the *ESR* coefficient as follow:

$$E_{max} = 100 * \log\left(\frac{Span}{2 * ESR}\right) \ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}20)$$

The relationship between Q index values and the RQD parameter are also used to predict the rock mass mechanical parameters, which are the cohesive strength, the frictional angle, the compressive strength, the wave velocity, and the Young modulus of the rock mass. According to *Barton* (2006), the following values are obtained from these relationships (Table III-18).

Table III-18: Rock Mechanical characteristics versus *RQD* and *Q* values (Adapted from *Look*, 2007).

RQD	***Q***	**Cohesive strength (MPa)**	**Frictional Angle φ (°)**	**σ (MPa)**	**V_p (Km/s)**	**E (GPa)**
100	100	50	63	100	5,5	46
90	10	10	45	100	4,5	22
60	2.5	2.5	26	55	3,6	10,7
30	0.13	0.26	9	33	2,1	3,5
10	0.008	0.01	5	10	0,4	0,9

B. Support Dimensioning Using Analytical Methods

Analytical methods allow design parameters to be quantitatively determined based on a model that represents how the structure behaves under applied stresses. These methods can be used to quickly estimate orders of magnitude for design parameters and to evaluate the impact of certain parameters on the response of the soil-supporting structure. However, their direct field of application is limited due to the restrictive calculation assumptions on which they are based.

These methods are based on simple assumptions designed to simplify the modeling of the encountered problem. These simplifications relate to the following assumptions (*Bouvard et al., 1992*):

A. Geometry: The tunnel is assumed to have a circular cross-section and a horizontal axis.

B. Stratigraphy: A single layer of soil is assumed.

C. Soil Behavior Law: The soil is assumed to behave linearly elastically or in an elasto-plastic behavior.

D. The initial state of stress is assumed to be isotropic and homogeneous.

E. The formulas are expressed in the framework of small deformations and in the plane.

Analytical methods can be divided into two groups: i) Analytical elastic methods; and ii) Elasto-plastic analytical methods. The latter take into account the development of a plastic zone around the underground cavity due to tunnel excavation. Elasto-plastic analyses include the 1950's Limit Theorem, and Convergence-confinement method (*Panet*, 1995). In this course, we will focus on two analytical methods belonging to the second group: elastoplastic analytical methods. These methods are the hyperstatic reaction method and the convergence-confinement method.

I. The Hyperstatic Reaction Method

The hyperstatic reaction method is relatively old compared with finite element calculations and more realistic convergence-confinement concepts. However, it is simple to understand and use, which means it remains a common (and economical) estimate in its field of application. It should be noted that this method is very useful for structural fire resistance, as modeling is much simpler than using finite elements, for example. It's worth remembering that ground pressure doesn't apply along the entire length of the support, and that contacts are often localized at certain points on the bends. In this case, it's difficult to accurately predict the stresses on the structure.

I.1. Field of Application of the Method

In general, this method is suitable for structures that are excavated in loose ground or fractured rock with low cover and heavy support techniques, such as backfilled tunnel heads. However, as you go deeper, you need to apply concepts derived from the convergence-confinement method. However, it is impossible to model soil/support coupling at live load levels, which is a significant limitation of NATM. Yet, in special cases, the hyperstatic reaction method can be useful for studying impacts on tunnel structures, such as the impact of falling boulders in discontinuous rocky environments or a vehicle's impact on internal structures (e.g., bulkheads, running slabs).

I.2. Principle of the Method

The principle is to study the behavior of the support (or lining) under the action of external loads. We therefore perform a classic structural calculation that any sophisticated RDM software can carry out. The geometry of the support is entered precisely for a linear meter of gallery, in the form of 2D beams, and then a load is applied. A distinction is then made between so-called active loads, which are independent of the state of deformation, and so-called passive loads, which are the hyperstatic reactions resulting from the deformation of the support.

The former category includes the pressure applied by the weight of the soil (vertical and horizontal), hydrostatic pressure if the tunnel crosses a water table, possible swelling, detachment of a block, the weight of the lining, road traffic at shallow depths, etc. The latter category includes the pressure applied by the weight of the soil (vertical and horizontal), hydrostatic pressure if the tunnel crosses a water table, possible swelling, detachment of a block, the weight of the lining, road traffic at shallow depths, etc.

The second loads are the ground's abutment reactions (Fig. III-12). These are considered to be linearly related to the displacement, which means they can be modeled by a series of springs, whose stiffness K (reaction modulus) is derived from the mechanical properties of the surrounding rock or soil. Once the structure's equilibrium has been established, the support forces (M, N, and T) and maximum convergence can be accessed.

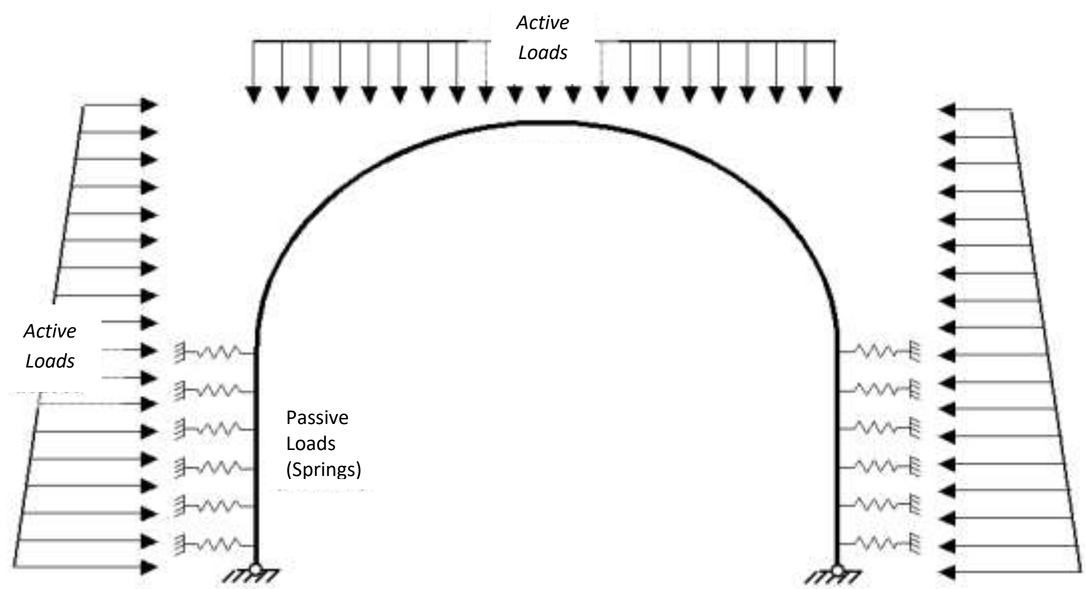


Fig-III-12: Classic diagram of a hyperstatic reaction model, with active and passive forces (springs).

I.2.1. Determining "Active" Loads

In the proposed model (Fig.III-12), the active loads (*q*) constitute the "external load" which will not be modified by the displacement of the structure. They depend on a number of parameters, such as depth, tunnel dimensions, rock quality, the gap left between the face and the lining, etc. There are many theories for assessing the pressures applied to a support structure.

a) **Continuous Media:** Since the hyperstatic reaction method is better suited to shallow tunnels, the *Terzaghi* or *Caquot* formulas are primarily used. These formulas are based on wedge-shaped soil failure at the wall of the tunnel and the weight of the relief vault. *Terzaghi*'s formula is expressed as follows (Fig. III-13):

$$\boldsymbol{q} = \frac{\boldsymbol{b}\left(\gamma - \frac{\mathbf{2\,C}}{\mathbf{b}}\right)}{\mathbf{2\,tan\varphi}}\left(\mathbf{1} - \boldsymbol{e}^{\frac{-\,2\,H\tan\varphi}{b}}\right) \ldots\ldots\ldots\ldots\ldots\ldots (III-21)$$

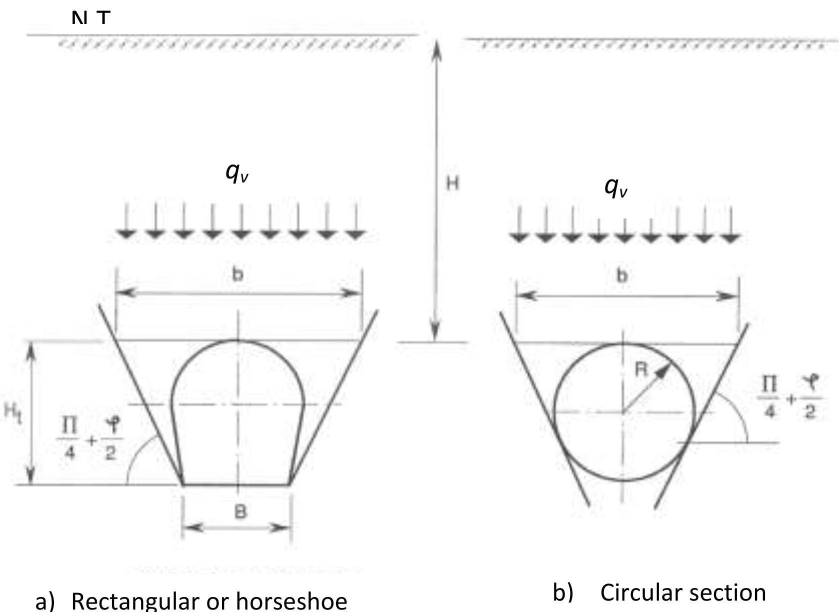


Fig.III-13: Geometric representation of variables used in *Terzaghi* formulas

In Figure III-13 above the following parameters represent:

H and b: the tunnel depth and the width of the estimated collapse cone in keystone, respectively;

C and φ: the Coulomb parameters; and

γ: the volumetric weight of the soil.

The pressure q_h on the side walls, which can be triangular, is estimated using q_v and the coefficient *Ka* (thrust coefficient) or K_0 (coefficient of earth pressure at rest). Other formulas, based on rock classifications, provide approximate pressure values. All should be used with the utmost caution.

b) **Discontinuous media**: In some cases, it may be useful to model the impact of a falling boulder on a support or lining structure. In such cases, verify the structure's stability under two types of active load:

- Fall of a boulder from a vault: We only consider the boulder's weight at the keystone (see chapter two).

- Fall of a boulder from a wall side: This case of asymmetrical loading is particularly unfavorable. The block should be assumed to slide over one or two discontinuities (see chapter two).

I.2.2 Determining "Passive" Loads

Apart from the difficulty of knowing where to place the springs, it is often challenging to determine the value of the modulus K without testing the material. The analytical expression of displacement μ in the wall of a circular tunnel in an elastic, isotropic, and linear rock mass provides an approximate value of this modulus.

$$\boldsymbol{K} = \frac{\boldsymbol{E}}{(\mathbf{1} + \boldsymbol{\nu})\boldsymbol{R}} \ldots\ldots\ldots\ldots\ldots (III-22)$$

$$\boldsymbol{q = K\,\mu} \quad \ldots\ldots\ldots\ldots\ldots (III\text{-}23)$$

q: being the passive pressure applied to the wall. In the software, it is sufficient to consider the abutment as a series of normal elastic supports.

The contact between the support and the ground is never perfectly slippery, and tangential friction exists. Alternatively, we can model them as springs, tangential to the support.

II. The Convergence-Confinement Method

The convergence-confinement analytical method was born of the success of NATM. The concept behind the method, which dates back to the early 80s, is to study, for an excavation in which a support is installed, the convergence or displacement of the ground on the one hand, and on the other hand, the confinement which is the radial pressure applied to the perimeter of this excavation by the support.

Instead of a three-dimensional problem, the Convergence-Confinement method addresses a two-dimensional plane strain problem of the ground/support interaction. By analyzing this behavior, we can use explicit formulas to draw characteristic curves for the ground and the support, known as convergence and confinement curves respectively (*Panet*, 1995).

The power of the convergence-confinement method lies in the simple representation of the two curves characterizing the behavior of the soil and the support. Convergence is linked to the release of soil containment caused by decompression after excavation. Confinement, on the other hand, is the pressure exerted by the support to prevent ground convergence or displacement. This method makes it possible to optimize the support to be used.

II.1. Method's Assumptions

The problem is considered to be one-dimensional, i.e. the following assumptions must be satisfied:

- Assumption of plane deformations;
- Assumption of isotropy of initial stresses (K_0 = 1) and isotropy of the massif; this is often verified at medium and great depths.
- The cavity studied has a cylindrical shape. In the case of a quasi-circular cross-section, an equivalent radius, calculated on the basis of an identical circular cross-section, is used.
- The initial stress state is defined by the isotropic stress state. If H is the height of cover and ɣ the weight by volume of the overlying terrain, the initial stress in the massif is given by:

$$\sigma_0 = \Sigma_i^n . \gamma_i H \quad \text{………………..(III-24)}$$

Note: When calculating σ0, the weight of any structures on the ground surface, such as buildings, roads, or railroads, must be considered[2].

II.2. Method's Applications

The convergence-confinement method is mainly used for the predimensioning of tunnel supports. In order to apply the theories of continuum mechanics, the surrounding rock must be represented as a homogeneous, isotropic and continuous medium at the scale of the structure. Furthermore, in order to simplify the three-dimensional problem to a one-dimensional one, the terrain must be identical on both sides of the currently investigated section over a distance of a few tens of meters. This excludes the study of tunnel heads. The cover height (distance between the tunnel keystone and the topographic surface) must be at least 4 times the tunnel

[2] The overloads considered for buildings are: 20 KN/m² for the ground floor plus 10 KN/m² multiplied by the number of floors.

diameter. Digging conditions must also be identical over a linear distance of at least 1 diameter in front of and 2 diameters behind the section being investigated.

II.3. Method Principle

II.3.1. Convergence Curve

To move from a three-dimensional state, with the ground progressively decompressing around the cutting face, to a state of flat deformation (traditionally encountered in a section far from the cutting face), a fictitious wall pressure is introduced (Fig.III-14). This pressure, uniformly distributed around the perimeter of the excavation, has a value that decreases with distance from the front. *Pi* thus varies from σ_0 to 0, from the initial state of stress to the fully decompressed state. The evolution of P_i is therefore governed by the distance x, which allows us to locate ourselves in relation to the cutting face (where x = 0). We can then write:

$$Pi = (1 - \lambda_{(x)}).\ \sigma 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}25)$$

Where λ(x) is the rate of confinement loss characterizing the state of the rock at the considered section situated at a distance *d* from the cutting face. It is also described by the displacement release coefficient at the excavation face.

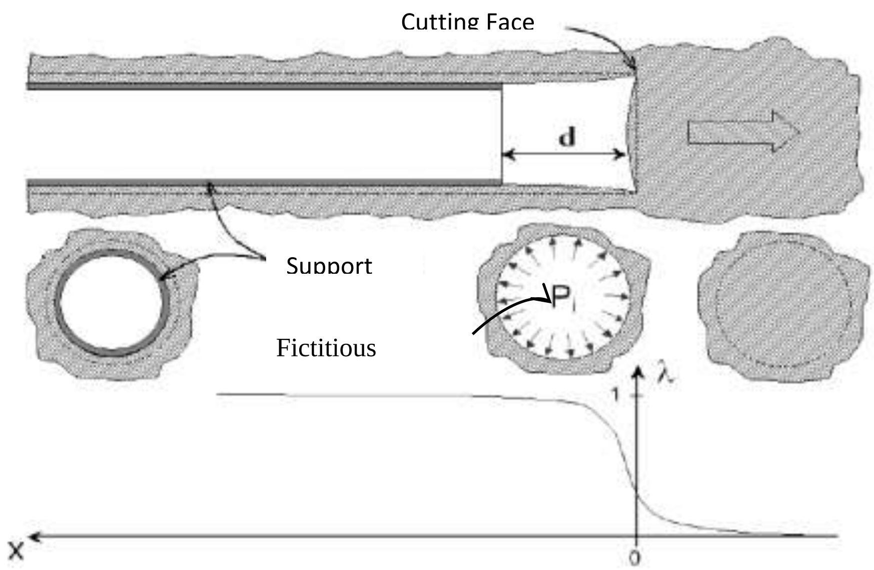


Fig.III-14: Schematic representation of the notion of fictitious pressure and loss of confinement around the cutting face.

In the initial state (in front of the cutting face), λ(x) is zero, while in the fully loss confinement state (away from the cutting face), λ(x) is equal to 1. This confinement loss starts at about one diameter from the working face (Fig.III.14). The convergence-confinement method requires knowledge of this confinement loss rate at the time the support is placed and starts interacting with the ground. This is the most critical point in the Convergence-Confinement method. Several authors have proposed formulas to define λ(x). If the soil remains elastic, λ(x) is given by:

$$\lambda_{(X)} = \alpha + (1-\alpha)\left[1 - \left(\frac{R.m_0}{(R+X)m_0}\right)^2\right]\ldots\ldots\ldots\ldots\ldots \textit{(III-26)}$$

Where α and m_0 are two constants whose values are 0.25 and 0.75 respectively, and R the excavation radius. Hence equation (V-6) can be written as follow:

$$\lambda(x)= 1- 0.75.\left(\frac{0.75\,R}{0.75\,(R+x)}\right)^2\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textit{(III-26-bis)}$$

Once the ground begins to behave plastically, the rate of confinement loss $\boldsymbol{\lambda_a}$ is calculated using equation III-27:

$$\lambda_{ic} = \sin\varphi + \frac{C}{\sigma_0}\cos\varphi \qquad \textit{(III-27)}$$

The convergence curve (Fig. III-15a) is a parametric curve (with parameter x), represented in a reference frame (Pi, u). In the absence of support, it gives the value of the displacement u in the excavation wall as a function of the fictitious pressure *Pi*. The theory of elasto-plasticity enables us to obtain the equation of this curve for simple criteria (*Mohr-Coulomb*, for example).

Two phases can be distinguished (Fig. III-15a):

a) An elastic phase, from μ = 0 to $\mu = \mu_{ic}$. The ground loss gradually its confinement state in a linear way. By extending the straight line along the abscissa axis, we obtain the pseudo-elastic displacement μ_e, which can be determined by:

$$\mu_e = R.\,P_0\,\frac{(1+\nu)}{E}\ldots\ldots\ldots\ldots \textit{(III-28)}$$

Where:

v and *E* are the soil Poisson's ratio and Young's modulus, respectively;

R is the radius of a circular-section tunnel; and

P_0 is the natural stress (pressure) in the ground.

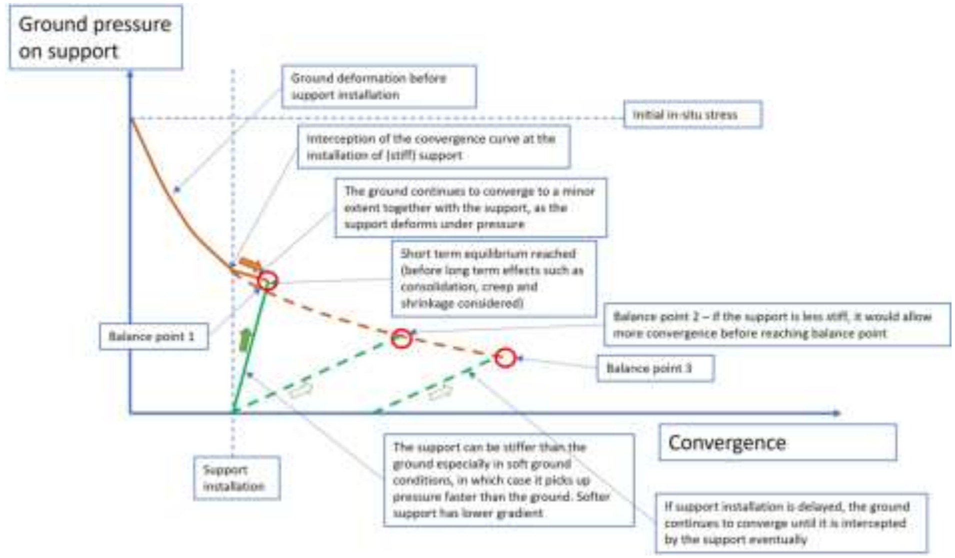


Fig.III-15: Convergence (ground) and confinement (support) curves (Si Shen, 2022)

b) A plastic phase, from $\mu = \mu_{ic}$ to $\mu = \mu_\infty$. The soil in the wall enters a state of irreversible deformation. An excessive compression or crushing causes ground failure. Sometimes the ground curve does not intersect the abscissa axis (very large deformations). The plastic state should be avoided, as this is one of the roles of support. For the *Mohr-Coulomb* criterion, the plastic curve equation is written as follow:

$$\mu = R.\frac{(1+\nu)}{E}.\left(C_1 + C_2\left(\frac{R}{R_P}\right)^{KP-1} + C_3\left(\frac{RP}{R}\right)^{\beta+1}\right) \quad \text{(III-29)}$$

Where:

$$C_1 = -(1-2\nu)(P_0 + H) \quad \text{(III-30)}$$

$$C_2 = \left(\frac{(1-\nu).(1+\beta.KP)}{KP+\beta} - \nu\right)\frac{2.(P0+H)}{KP+1} \quad \text{(III-31)}$$

$$C_3 = 2(1-\nu)\frac{(KP-1)(P0+H)}{K_P+\beta} \quad \text{(III-32)}$$

$$R_P = \left(\frac{2\,(P0+H)}{KP+1}\,\frac{R^{KP-1}}{H+P_i}\right)^{\frac{1}{KP-1}} \quad \text{(III-33)}$$

$$H = \frac{C}{\tan\varphi} \quad \text{(III-34)}$$

The following formulas are used to determine the *Mohr-Coulomb* constants, the expansion angle ß, and the bearing capacity coefficient *Kp*:

$$\boldsymbol{\beta} = \frac{\mathbf{1+\sin\psi}}{\mathbf{1-\sin\psi}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots.(III\text{-}35)$$

$$\boldsymbol{K_P} = \boldsymbol{tan^2}(\frac{\boldsymbol{\Pi}}{\mathbf{4}} + \frac{\boldsymbol{\varphi}}{\mathbf{2}})\ldots\ldots\ldots\ldots(III\text{-}36)$$

To calculate the maximum soil displacement μ_∞, we use the above formulas and simply assume $Pi = 0$.

Finally, in order to draw the convergence curve, the peak pressure at which plasticity occurs must be calculated:

$$\boldsymbol{Pic} = \frac{\mathbf{2}\ \boldsymbol{P0-H(KP-1)}}{\boldsymbol{KP+1}} \ldots\ldots\ldots..(III\text{-}37)$$

A second formula allowing the calculation of the pressure *Pic* exists:

$$\boldsymbol{P_{ic}} = \boldsymbol{\sigma_0}\ (\mathbf{1} - \boldsymbol{sin\varphi}) - \boldsymbol{C\ cos\varphi} \qquad (III-37-bis)$$

The convergence (displacement) corresponding to the beginning of ground plasticity can be calculated as follows:

$$\boldsymbol{\mu_{ic}} = \boldsymbol{\lambda_a}.\,\mu_{\mathbf{e}} \ldots\ldots\ldots\ldots\ldots.(III\text{-}38)$$

If the ground quality is good, for example in hard rock, the ground can remain elastic throughout the whole excavation process. The stability factor *F* is a criterion widely used in underground workings to determine whether the surrounding massif is at risk of plasticity. This factor equals:

$$\boldsymbol{F} = \frac{\mathbf{2}.\boldsymbol{P_0}}{\boldsymbol{R_C}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(III\text{-}39)$$

Where Rc is the compressive strength of the rock massif equals to:

$$\boldsymbol{R_C} = \mathbf{2}.\boldsymbol{C\ tan}\left(\frac{\boldsymbol{\Pi}}{\mathbf{4}} + \frac{\boldsymbol{\varphi}}{\mathbf{2}}\right) \ldots\ldots\ldots\ldots..(III\text{-}40)\ ;$$

$$\boldsymbol{R_c} = \frac{\mathbf{2.C.\cos\varphi}}{\mathbf{1-\sin\varphi}}\ldots\ldots\ldots\ldots..(III\text{-}40\text{-}bis)$$

- For $Rc > 2P_0$, the ground remains entirely in the linear elastic domain, even with complete deconfinement.

- For $Rc < 2P_0$, it is impossible for $2P_0$ to be greater than Rc for complete deconfinement. This means that the terrain is in the elastic domain close to the working face (with low deconfinement) and gradually reenters the plastic domain farther from the working face (with increasing deconfinement).

Thus, if $F > 1$, there is a risk of instability; the soil enters the plasticity domain and risks failure. Otherwise, the ground behaves elastically and the risk of instability is avoided.

II.3.2. **Confinement Curve**

A second curve is required for the method (Fig. III-15b). This is the confinement curve characterizing the support behavior under loading. The loading considered is purely radial, i.e. a pressure applied to the entire outer perimeter of the structure. By calculating the radial displacement µs as a function of the applied pressure P_s, the confinement curve can be plotted on a graph identical to that of the convergence curve. There are also two phases in this curve:

- An elastic phase, from $u_s = 0$ to $u_s = \mu_{max}$. The support behaves linearly.

- A plastic phase, after μ_{max}. This zone corresponds to the failure of the support, and is therefore forbidden.

To draw the characteristic curve of the support, we need to determine the stiffness Ks of the chosen support and the maximum pressure P_{max} developed by the latter.
Several types of support can be chosen: formwork concrete, shotcrete, metal bends, anchor bolts and a combination of two or more types of support. The formulas for calculating stiffness and maximum pressure are given below for each type of support.

a) Case of a support composed of formwork concrete, or shotcrete

- If the support consists of a thin concrete ring, such as a shotcrete lining, of thickness e and permissible limit stress σ_{bmax} , the stiffness K_b of the support and the maximum pressure P_{bmax} developed by it are given by formula (III-41) and (III-42), respectively :

$$\boldsymbol{K_b} = \frac{\boldsymbol{E_b\, e}}{\boldsymbol{(1-\nu b^2).R}} \text{..................}(III\text{-}41)$$

$$\boldsymbol{P_{bmax}} = \frac{\boldsymbol{\sigma_{bmax}\cdot\ e}}{\boldsymbol{R}} \text{..................}(III\text{-}42)$$

Where:

E_b *:* Young's modulus (7000 - 15000 MPa),

ν_b : Poisson coefficient of the concrete, and

R: the excavation radius.

Then, since the value of (1 - νb^2) is close to 1, equation (III-41-bis) can be written as:

$$K_b = \frac{E_b\, e}{R} \text{..................(III-41-bis)}$$

- If the support is thick (case of formwork concrete), the stiffness K_b of the support and the maximum pressure P_{bmax} developed by it are given by formula (III-43) and (III-44) respectively:

$$K_b = \frac{E_b\,(R^2 - R_i^2)}{(1-\nu b).\ [R^2\,(1-2\nu b) + R_i^2)} \text{..............(III-43)}$$

$$P_{bmax} = \frac{R^2 - R_i^2}{R^2 + R_i^2}\ .\ \sigma_{bmax} \text{..................(III-44)}$$

Where:

R_i *is the* intrados radius of the support, and

E_b: Young's modulus of the formwork concrete (between 15,000 and 30,000 MPa).

The maximum confinement value generated by the support is then as follows:

$$\mu_{Sbmax} = \frac{P_b \times R}{K_b} \qquad \text{...............(III-45)}$$

b) Case of a support composed of metal arch

A metal arch support of cross-section *S*, longitudinal spacing *e*, strength σ_a and modulus of elasticity E_a (E_a = 200 000 MPa) develops a stiffness K_c, a maximum pressure P_{cmax} and maximum confinement value given by formulas (III-46), (III-47), and (III-48), respectively:

$$K_c = \frac{S.E_a}{R.e} \text{.................(III-46)}$$

$$P_{cmax} = = \frac{S.\ \sigma_a}{R.e} \text{.................(III-47)}$$

$$\mu_{cmax} = \frac{P_b \times R}{K_b} \text{.................(III-48)}$$

Note that if the arch is embedded in a shotcrete lining, which is a common case, then the stiffness of the support will be the sum of the two stiffnesses: that of the arch and that of the shotcrete.

C) Case of a support composed of anchor bolts

In the case of a support composed of a set of bolts with point anchors of circumferential spacing e_c and longitudinal spacing e_l, the stiffness K_S of the underpinning is given by equation (III-49):

$$K_S = \frac{R}{e_c.e_l}\ \frac{1}{(Q+\frac{4l}{\Pi.d^2.E_a})} \text{.................}(III\text{-}49)$$

Where:

l : Free bolt length between plates,

d : Bolt diameter,

Ea : Young's modulus of the bolt material, and

Q : The value related to the load-deformation characteristics of the bolt components (the anchor, bearing plate, and bolt head). Q can be determined during a pull-out test and is given by eq. (III-50).

$$Q = \frac{S_b}{T_b} \text{.................}(III\text{-}50)$$

Where:

S_b is the difference between bolt and rod elongation, and T_b is the load acting on the bolt.

Note that, since anchor bolts are often used in conjunction with another type of support, the support's stiffness is the sum of the stiffness of the different used supports. The maximum pressure P_{Smax} resulting from point-anchored bolt retention is given by equation (III-51) :

$$P_{Smax} = \frac{T_{br}}{e_c.e_l} \text{.................}(III\text{-}51)$$

Where:

T_{br} is the ultimate strength load of the bolt, determined by a pull-out test performed on the rock for which the bolting system is intended, and

e_c and e_l have the same meanings as above.

Then, as for the previous support types, the maximum confinement value generated by the support is given by Eq. 52:

$$\mu_{smax} = \frac{P_{bsmax} \times R}{K_s} \text{ …..(III-52)}$$

II.3.3. Final Balance between Ground and Support

The interaction between the two curves, one representing soil behavior and the other representing the support behavior can be studied by coupling them after they have been traced. However, this requires introducing a new parameter: us_0, the displacement of the excavation wall at the time the tunnel support is installed. This is because the support cannot be laid immediately at the face, but a few decimeters behind, when the ground is already partially deconfined. The displacement us_0 is, of course, closely related to the rate of deconfinement λ_d when the support is laid, and its value corresponds to the starting point of the support characteristic curve.

Note that the point of intersection should neither be beyond the minimum value, denoted by point D in Fig. III-16, of the required support pressure, nor be much ahead of this value. In the former case, the support will be unsafe and in the latter, uneconomical. In fact the curves may not intersect at all in the former case.

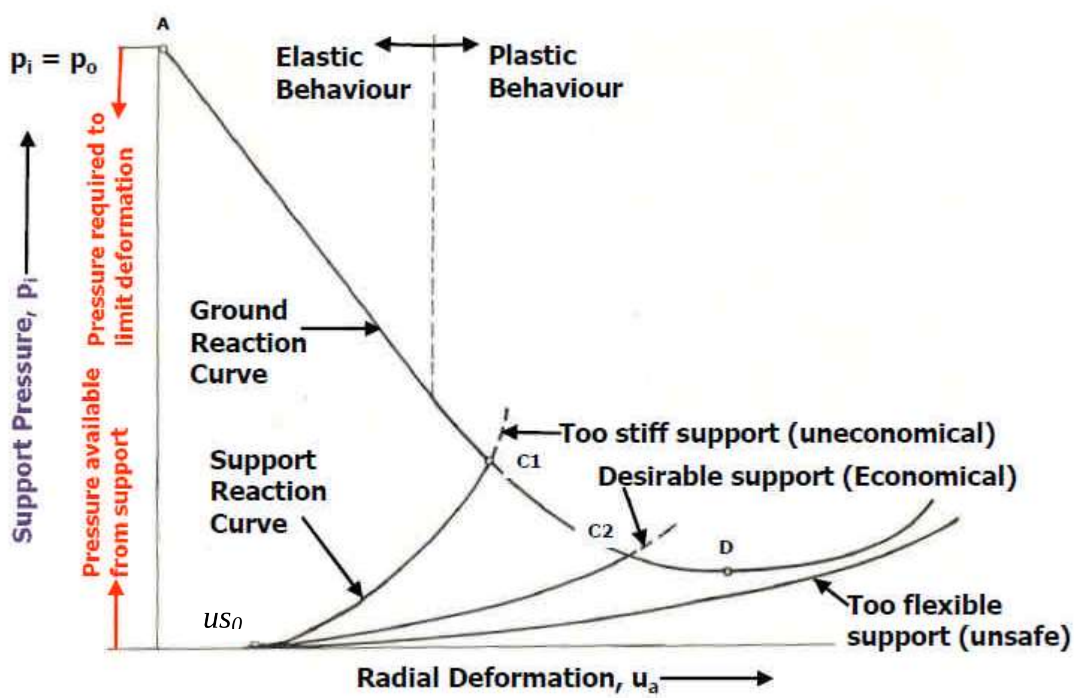


Fig.III-16: Convergence-Confinement Curves (Modified from *Verman*, 2009)

Furthermore, studies have shown that when the support is installed near the working face, the deconfinement rate λ_d is 0.265. This value increases to one-third under actual installation conditions and rises quickly with distance from the face. Therefore, we can adopt a displacement value us_0 when the support is installed that is generally greater than one-third of the elastic displacement u_e (equation III-53).

$$us_0 \approx 0{,}33\ u_e \ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}53)$$

Once the two curves have been assembled, the point at which they intersect corresponds to the equilibrium state between the soil and the support. If the pressure exceeds the support's capacity, another support with different characteristics (dimensions, spacing, or thickness) is considered until one that meets this condition is found. If the ground behaves elastically, meaning the convergence curve is a straight line, the support pressure P_s corresponding to this condition is given by equation (III-50):

$$P_s = \frac{K_S}{K_S + 2G}.(1 - \lambda_{S0}).P_0 \ldots\ldots\ldots\ldots\ldots\ldots (III\text{-}54)$$

Where:

G: the stiffness of the ground, and

λ_{s0}: the rate of confinement achieved at the time the support is installed.

Ks and σ_0 have the same meanings as above.

C. Support Dimensioning Using Numerical Methods

First introduced in the 1970s, numerical methods allow us to accurately determine displacement and stress fields. These methods are favored for their speed and ease of use. The finite element method, in particular, is widely used in civil engineering.

I. General Presentation of Numerical Method

The choice of numerical methods used to study and solve problems in underground structures depends on the environment in which the tunnel is excavated. The existence of joints is an important factor to consider. A rock mass is characterized by a certain number of joints that form it; this number can range from one to several. Figure III-17 illustrates the types of rock according to the number of joint they comprise.

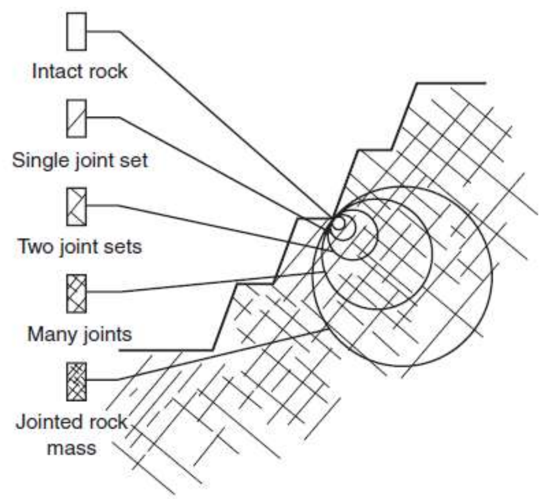


Fig. III-17: Difference between intact rock and rock mass according to the number of joint they comprise (*Wyllie et al.*, 2004)

I.1. Continuous Media

Only a continuous massif can be modeled. Single localized discontinuities can still be represented, but require a refinement of the mesh and a good knowledge of their shear behavior. In general, there are two similar calculation methods for continuous media based on a spatial discretization of the rock mass: the finite element method and the finite differences method. It is to highpoint that a tunnel, especially near the face, is a three-dimensional problem. Thus, only 3D calculations can provide an effective understanding of the deformation state of the massif, but they are longer and more costly than 2D calculations. However, we can simplify the problem by using convergence-confinement concepts.

I.2. Discrete Media: Distinctive Element Method (DE)

The case of continuous media does not apply to underground structures or rock mechanics. Rock is fundamentally a fractured, or discontinuous, medium. It would be a serious mistake to reduce it to a continuous medium using homogenization and correlation techniques because this could lead to false conclusions. This is why numerical calculation methods have been developed for discrete media.

The Distinctive Elements method enables us to consider large displacements, rotations, and instabilities at an advanced stage. It models the behavior of discrete media ranging from weakly fractured (dihedral method) to strongly fractured and nearly continuous. Block assemblies (modeled as either rigid or deformable) interact through joints represented by contacts. It should be noted that this method is rarely used for dimensioning embankments, and even less for verifying tunnel lining.

II. Modeling principle

The modeling principle starts by the consideration of the problem in two or three dimensions (depending on the tool we are using and the results quality we need). After this we create a model by dividing the continuous medium into a number of finite elements. The higher the numbers of elements are, the more accurate the results are. When creating a model, we have to fix the initial and boundary conditions and choose the behavior laws.

The model's elements are assumed to be interconnected by a finite number of nodal points that transmit forces from one element to another. The displacements of these points are the unknowns of the problem. An interpolation function is used to define the displacement field inside each FE as a function of the displacements of its nodes. The stress state of the element is then defined at one or more points of the element (integration points). From the displacement functions and rheological laws chosen, a stiffness relationship is determined, so that each displacement field corresponds to a stress field.

Where K is a square matrix called the stiffness matrix and U is the displacement matrix. Finally, we solve the linear system:

$$\{F\}=[K].\{U\} \quad \ldots\ldots\ldots\ldots\ldots.(III\text{-}55)$$

II.1. Initial and Boundary conditions

The quality of the result depends on the model's initial and boundary conditions. Displacements are assumed to be zero at the model's left, right, and bottom boundaries. These boundaries should generally be located at a distance equal to four or five times the tunnel diameter. Initial state stress is considered zero: $\sigma = \sigma_0$. The initial state can be calculated using M.E.F. on the model itself by applying gravity. However, it is important to consider displacement as free at the topographic surface; displacement must not be blocked at the model surface. Figure III-18 shows an example of a tunnel section model.

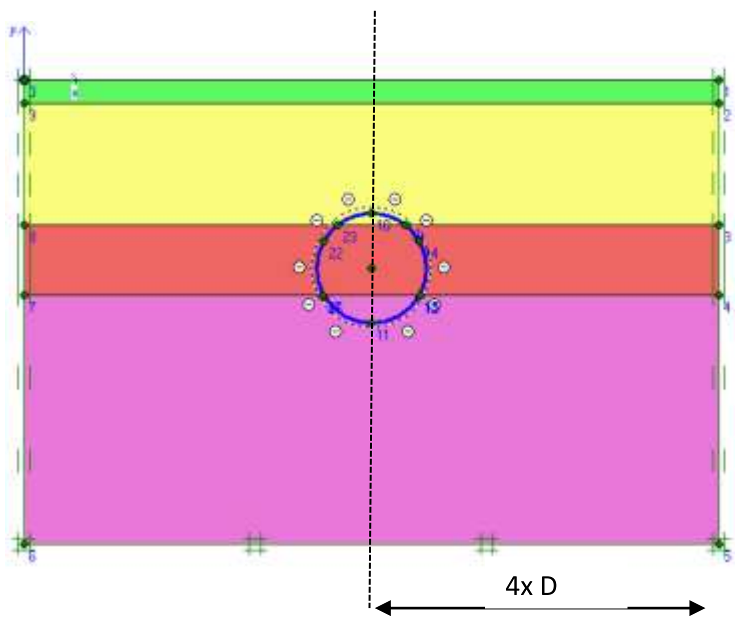


Fig. III-18 : Example of a tunnel section model

II.2. Behavior Laws

The behavior laws used can be very complex, requiring the measurement of a large number of parameters. In 90% of cases, the Mohr-Coulomb law (five parameters with dilatancy) or the *Hoek-Brown* law (six parameters with dilatancy) is used. A finer mesh size is needed in areas of high stress variation (such as the corners of an excavation).

II.3 Finite Element Examples

There are many possible shapes for an element. A few of the more common element types are shown in Figure III-19. Nodes attached to the element are shown in red. When using a two dimensions model, we generally use elements that are either triangular or rectangular. However, when using a three dimensions model, the elements are generally tetrahedral, hexahedra or bricks (https://www.brown.edu/).

The number of nodes is grater in the latest; this is the reason behind the more accurate quality of the result of a 3D model compared to a 2D model. Figure III-20 presents an illustration of a mesh employing 6-node triangle elements in 2D (left) and 3D (right).

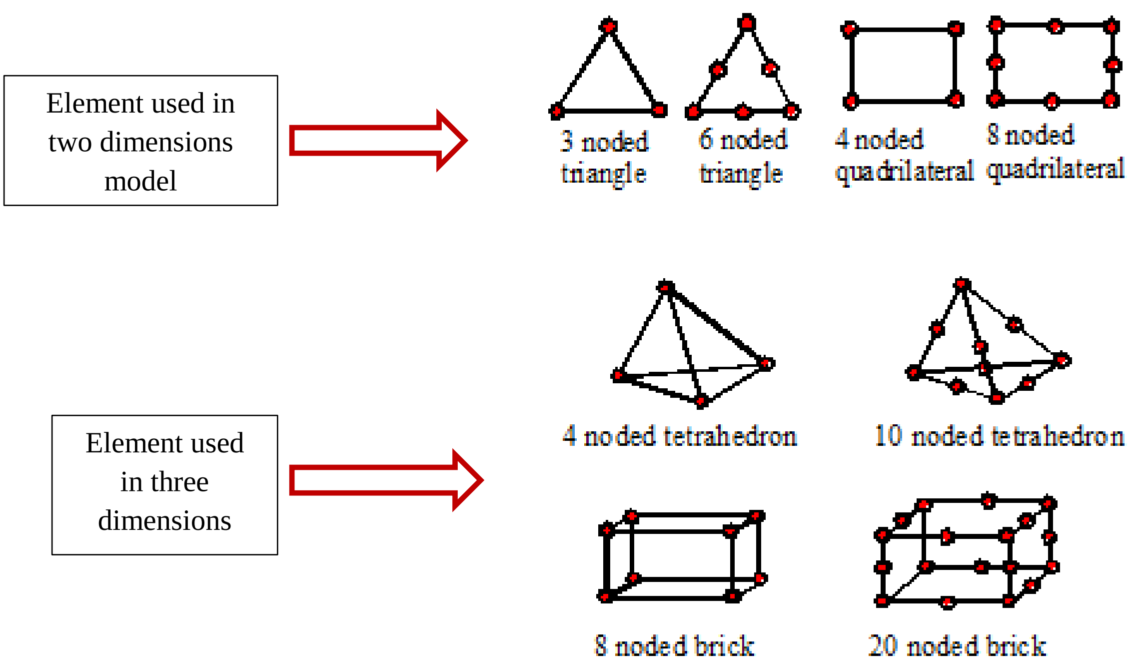


Fig- III-19 : Common Used Element Types (modified from https://www.brown.edu/)

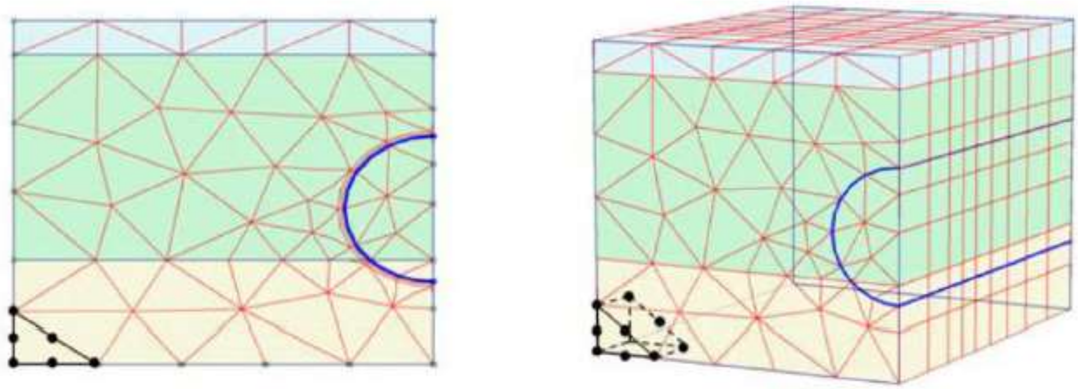


Fig.III-20: Example of triangulated element mesh

II.4. 2D Plane-Deformation Model

Plane deformation calculations, in the profil across a tunnel section, account for 98% of the numerical calculations performed in engineering offices. These calculations employ the concepts of the convergence-confinement analytical method and offer many advantages over the latter, such as: any cross-section, anisotropic stress state, split-section calculations. The calculation is performed in successive phases, at progressive states of deconfinement. Figure III-21 shows an example of the result of a 2D Plane deformation calculation model.

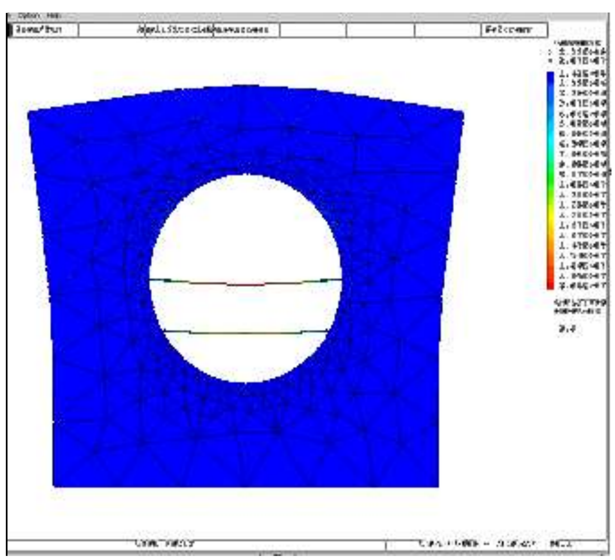


Fig.III-21: Example of a 2D Plane deformation calculation model

II.5. 2D Axisymmetric Model

Axisymmetric models represent the tunnel along its longitudinal axis. These models, which are based on the assumptions of the convergence-confinement method, have the advantage of enabling us to study the mechanical state of the ground around the cutting face. Through this modeling, we can approach the famous law of evolution of the coefficient “λ ». Figure III-22 shows an example of a study carried out adopting a 2D calculation axisymmetric model.

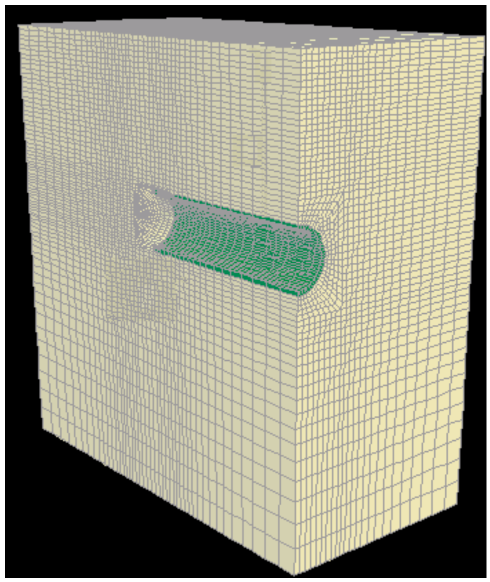


Fig.III-22: Example of an axisymmetric model

In practice, 2D cross-sectional analysis is most commonly used. However, convergence-confinement is the most common method to simulate the three-dimensional effects of the advancing working face. Two-dimensional analysis is quicker than three-dimensional analysis and allows for the use of more complex behavioral models if sufficient field data is available. However, cross-sectional modeling does not allow us to analyze the stability of the working face. Therefore, longitudinal modeling parallel to the tunnel axis may be necessary.

III. Settlement Calculation

Estimating the settlement caused on the surface by the excavation of a cavity represents one of the most important issues of studying the excavation of an underground structure. Of course, this study should be carried out before work begins, in order to estimate settlement and plan pre-support techniques to avoid excessive settlement of constructions built on land. The vertical displacement of the natural ground level at tunnel level is the result of the surrounding rock displacements (convergence and extrusion) caused by the excavation, drainage, and vibrations emitted by the machines. These displacements depend not only on the geometric and mechanical configuration, but also on the chosen excavation method (TBM with open or closed face, confinement pressure, face support with bolts or precast, lining very close to the face, etc.).

III.1. Settlement Causes

Although there are a large number of sources or causes of settlement, they can be conveniently lumped into two broad categories: those caused by ground water depression and those caused by lost ground.

III.1.1 Groundwater Depression

Ground water depression may be caused by intentional lowering of the water during construction or by the tunnel itself (or other construction) acting as a drain. When either of these occurs the effective stress in the ground increases. Basic soils mechanics can then be applied to estimate the resulting settlement. For tunnels in granular soil the settlement due to this increase in effective stress is usually reflected as an elastic phenomenon requiring knowledge of the low stress modulus of the ground and calculation of the change in effective stress. Unless the soil contains silt or very fine sand, this elastic settlement will typically represent the majority of the total but its absolute value will also be relatively small (Tunnel Manual).

III.1.2. Lost Ground

Lost ground has a number of root causes (at least nine) and is usually responsible for the settlements that make the headlines. By definition, lost ground refers to the act of taking (or losing) more ground into the tunneling operation than is represented by the volume of the tunnel. Thus it is highly reflective of construction means and methods. As will be discussed, modern machines can be a great help in controlling lost ground but in the end it usually comes down to quality of workmanship.

As a general rule, settlement should only be considered for shallow urban underground structures, and for those passing close to other structures at risk, such as railroad tracks, viaduct piers, etc. Figure III-23 shows an example of the result of estimating the settlement of a railroad track under which a tunnel is dug. The settlement basin that is a consequence of the subsidence of the tunnel vault is represented by a *Gauss* curve. More or less empirical formulas are used to estimate the volume of soil compacted, as well as the amplitude of this compaction (see the AFTES recommendation: TOS, 1995).

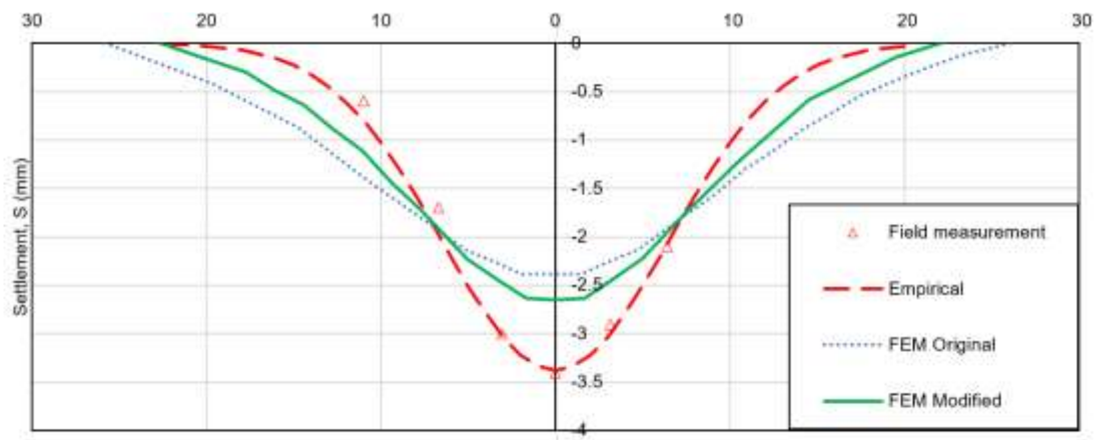


Fig.III-23: Settlement values due to tunnel excavation using three assessment methods (*Le & Nguyen*, 2021)

IV. Modeling Tools

Several software packages are used in underground structures. The U.S departement of transportation of the federal highway administration provides a set of the numerical modeling programs used in tunnel design and analysis (Table III-19).

Table III-19: Numerical Tools used in underground structures studies (FHWA & NHI, 2009)

Program	Description	Applications
FLAC 2D FDM	A two dimensional finite difference code. Widely used in general analysis and as a design tool applied to a broad range of problems. It enables dynamic analysis, thermal analysis, creep analysis, and two-phase flow analysis. As a weakness, it requires high running time.	Mechanical behavior of soils and rock mass; Coupling of hydraulic and mechanical behavior of soils; Well suited for tunneling or excavation in soil; Seismic analysis.
FLAC 3D FDM	A three-dimensional version of FLAC; Meshing generation software is recommended for complicated geometry.	Complex three-dimensional behavior of geometry; Suitable for interaction study for crossing tunnels
PLAXIS FEM	A finite element packages for two-dimensional and three-dimensional analysis; Automatic finite element mesh generator; User Friendly.	Tunneling and excavation in soil; Coupling of hydraulic and mechanical behavior of soils; Modeling of hydraulic and non-hydraulic pore pressures in soils.
PHASE2 FEM	Two dimensional elasto-plastic finite element stress analysis; Well suited for rock engineering; Automatic finite element mesh generator; Easy to use.	Tunneling and excavation in rock; Global overview of engineering solution in rock mass.
SEEP/W	A finite element code for analyzing groundwater seepage and excess pore-water pressure dissipation problems within porous materials. Available from simple saturated steady state to sophisticated problems saturated-unsaturated time-dependent problems, Both saturated and unsaturated flow.	Steady state and transient groundwater seepage analysis for tunnels and excavations. Equivalent properties of the rock mass should be properly evaluated.
MODFLOW MDM	A modular finite difference groundwater flow model, Most widely used tool for simulation groundwater flow	Three dimensional steady state and transient flow, Modeling of heterogeneous anisotropic aquifer system, Fate and transport modeling for geo-environmental problems with available package.
UNWEDGE	Pseudo-three dimensional wedge generation and stability analysis for tunnels,	Conceptual analysis tool for tunnel support design,

	Simple safety factor analysis, Three joint sets are required to form wedges.	A parametric study for wedge loading diagrams for tunnel.
SWEDGE	Pseudo-three dimensional surface wedge analysis for slopes and excavations, An easy to use analysis tool for evaluating the geometry and stability of surface wedges, Wedges formed by two intersecting discontinuity planes and a slope surface.	Conceptual design of slopes, A parametric study for wedge loading diagrams for slopes and excavations.
LSDYNA	A general purpose transient dynamic finite element program, Coupling of Euler-Lagrange non-linear dynamic analysis Optimized for shared and distributed memory Unix-, Linux-, and Windows-based platforms.	Impact analysis, Blast/explosion analysis, Seismic/vibration analysis, Modeling of computational fluid dynamics.
AUTODYN	A finite difference, finite volume and finite element- based Hydrocode, Coupling of Euler-Lagrange non-linear dynamic analysis Convenient material library, Widely used in dynamic analysis.	Blast/explosion analysis, Impact analysis, Seismic/vibration analysis, Modeling of computational fluid dynamics.

Chapter Four: Tunnel Inspection, Maintenance and Repair

I. Introduction

The process of inspecting, maintaining, and repairing tunnels requires a diverse group of specialists who are well-versed in various functional areas of tunnel operations. This encompasses civil/structural, mechanical, electrical, drainage, and ventilation systems, in addition to operational elements like signals, communication, fire-life safety, and security systems. This chapter outlines these interconnected operations, adhering to the recommendations provided by the U.S. Federal Highway Administration and the Federal Transit Administration (FHWA & NHI, 2009).

II. Tunnel Inspection

It is expected that tunnel inspections must be carried out by skilled personnel who are well-versed in the types of materials encountered in tunnels, have a basic comprehension of the behavior of tunnel structural systems, and possess experience in the inspection of transportation structures. The following sub-sections will elaborate on the standard parameters for inspection.

II.1. Inspection Parameters

Standardized inspection parameters are required to expedite the processing and evaluation of the observed data. The utilization of standardized coding for information, essential for consistent reporting, also plays a role in ensuring quality control by providing guidelines for inspection teams and standardizing visual observations. For instance, the letter F symbolizes a continuous water flow that requires volume measurement. Tabe IV-1 shows some of the common tunnel leakage items, symbols and descriptions.

Table IV-1: Common description of tunnel leakage (FHWA & NHI, 2009).

Item	Symbol	Description
Moist	M	Discoloration of the surface of the lining, moist to touch
Past Moisture	PM	Area showing indications of previous wetness, calcification etc
Glistening Surface	GS	Visible movement of a film of water across a surface
Flowing	F	Continuous flow of water from a defect ; requires voume measurement
Dry	D	Structural defect illustrates no signs of moisture

II.2. General Notes in Field Books

All general field inspection and repair notes, which include a chronology of events, are required to be maintained in a bound field book. Each member of the field team is obligated to carry a bound field book at all times while on site. The information recorded in the field book should encompass notes regarding safety issues as well as discussions with contractors, operations personnel, and other relevant parties. Entries in the field book must be organized chronologically by date and time, and should provide clear, concise, and factual notifications of events along with appropriate sketches. Field records, notes, and the inspection database must be stored in a single location. Field books should be duplicated on a weekly basis to avoid data loss. In contemporary practice, electronic notebooks and/or specialized laptop computers are frequently utilized to digitally record field data and sketches, which may also include digital photographs and/or videos with embedded date, time, and GPS location information.

II.3. Field Notes

The three types of field notes required for effective inspection of roadway tunnels are:

- General notes in field books;
- Documentation of defects on field data forms; and
- Documentation of defects by photographs/video.

II.3.1. Field Data Forms

Field data forms are utilized to capture the information essential for a given project. Generally, these forms are designed specifically for the project and cater to its unique needs. They set a standard for the arrangement of data gathered from inspections. This information is subsequently sent to the data management staff for inclusion in the project database.

II.3.2. Photographic Documentation

The best way to document tunnel defects is by employing a digital camera. Photographs should capture both typical and atypical conditions. Additionally, these photographs should be utilized as documentation for any special or unique conditions. The photographs must:

- Show the project number, date, time, location, photographer, and a general description of the item;

- Be systematically cataloged and stored for future access.

II.3.3. Survey Control

All condition surveys demand a definitive baseline for location (survey) purposes. In general, most highway systems have a recognized survey baseline. The post-construction baseline survey of the highway system is typically carried out for the maintenance of the roadway and tunnel structure. Such stationing systems are usually clearly defined with permanent markers situated on the tunnel walls. Some tunnels may already possess a baseline condition established via laser scanning techniques. The tunnel inspection documentation must be connected to the existing baseline stationing system for the following reasons:

-Allow inspection data to be employed for long-term monitoring of the tunnel structure by the owner's engineering/maintenance staff;

-Allow defects to be easily located for future inspections or repairs;

-Facilitate the quick start-up of inspection teams; and

-Reduce project costs and eliminate confusion.

II.3.4. Groundwater Intrusion

Groundwater intrusion can be managed by either treating the ground surrounding the tunnel or by sealing the tunnel's interior. The selection of the right repair product for the specific conditions encountered in the project is essential for the success of a leak containment initiative. Each site has its own unique environmental and physical characteristics. The pH, hardness, chemical composition, and turbidity of the groundwater entering the tunnel all affect the capability of chemical or particle grouts to effectively seal the leaking defect. The physical conditions that led to the defect, including the movement of the crack or joint, the potential for freezing, and the volume of water flow, are all site-specific limitations that must be taken into account when choosing the repair material, and all these parameters must be carefully evaluated.

In an ideal situation, if any movement of the crack or joint is suspected, it is advisable to monitor the defect for a sufficient duration to allow for an accurate assessment of actual movement. The choice of the appropriate grout to seal a tunnel liner is determined by the extent of leakage into the tunnel from the defect. Generally, the tunnel defects that result in leakage include construction joints, liner gaskets, and cracks that extend through the entire depth of the liner. Standardized terminology that has been developed to describe water inflow is useful in selecting the grout, as it allows all personnel, including those who have not visited the tunnel, to be aware of the level of water inflow. Figure IV-1 below shows the effect of high water pressure erosion mechanism and its influence on the mechanical performance deterioration of submarine shield tunnel concrete segments (*Feng et al*., 2023).

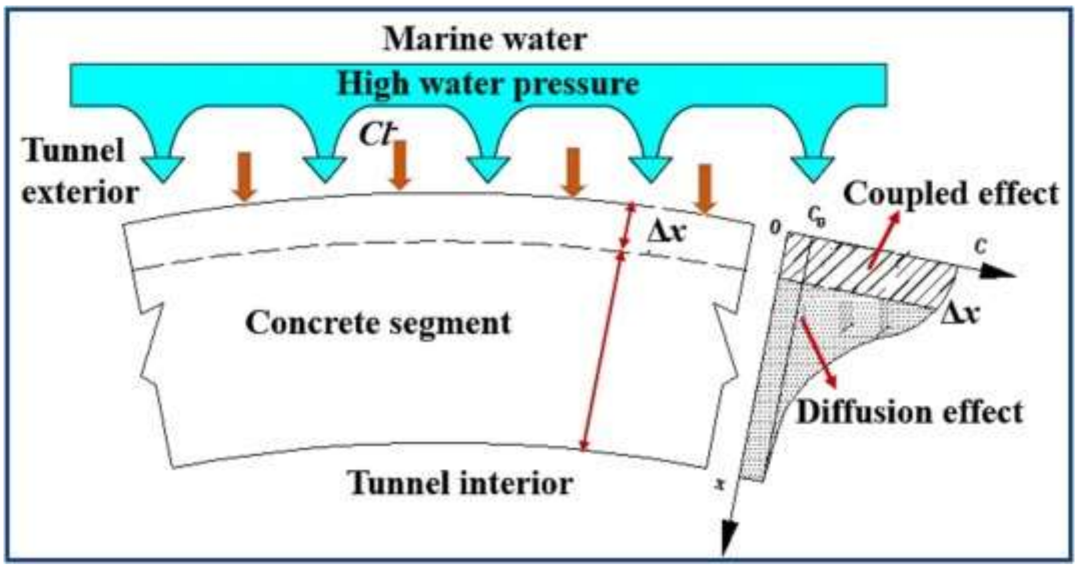


Fig. IV-1: Deterioration of tunnel concrete segments due to water pressure (*Feng et al*., 2023)

III. Tunnel Maintenance and Repair

The repair of concrete delaminations and spalls in tunnels has historically been executed using the form-and-pour technique for concrete placement, or through the manual application of cementitious mortars enhanced with polymers. Neither of these approaches is particularly suitable for highway tunnels that operate continuously on a daily basis. Such daily operations typically allow for very brief periods during which the tunnel can be out of service. Consequently, the repair process must be swift, should not interfere with the daily traffic operating envelope, and must result in a durable, long-lasting monolithic repair.

Currently, the repair of concrete structural components is generally carried out using two methods: hand-applied mortars for minor repairs and shotcrete for more extensive structural repairs. In both scenarios, the substrate preparation remains consistent; only the material type varies. Shotcrete involves the pneumatic application of cementitious materials, which can be utilized to restore concrete structures. This technique has been employed for several decades in the construction and repair of concrete structures.

III.1. Surface Preparation

The process of preparing the surface for concrete repair requires the removal of all unsound concrete, which can be done using either chipping hammers or hydro-demolition. The unsound concrete must be taken out to its full depth. Hydro-demolition requires initial testing on-site at the beginning of the project to determine the pressures needed to excavate the unsound concrete without damaging the sound substrate (Fig. IV-2). Hydro-demolition should not be employed in areas that contain electrical equipment, cables, or other mechanical devices that could be affected by the excavation process. The repair area must not have featheredges and must feature a vertical edge of at least 1/8 inch in height. This vertical shoulder is necessary to prevent spalling at the edge of the new repair. After the unsound concrete has been removed, any leaking cracks or construction joints must be sealed before applying the reinforcing steel coatings and shotcrete. This sealing should be performed with a chemical grout that is appropriate for the type and extent of the leakage. In general, single-component polyurethane grouts are the most effective in sealing most tunnel leaks.

Fig. IV-2: Hydro-demolition process used to selectively remove the damaged and decaying concrete (image source: forconstructionpros Website)

III.2. Steel Reinforcement

After the removal of the unsound concrete, it is essential to clean the reinforcing steel. If any loss of section is observed, the damaged reinforcing steel must be taken out and replaced. All rust and scale should be eliminated from the reinforcing steel, as well as from any exposed steel liner sections or other structural steel components. The cleaning process typically achieves a white metal commercial grade standard. Once the cleaning is completed, the reinforcing steel should be assessed for any loss of section. If the loss exceeds 30%, an analysis is required. Should the analysis reveal that the remaining reinforcing steel does not provide sufficient strength for the lining, the damaged steel must be replaced. Mechanical couplers are utilized when connecting new reinforcing steel to the existing structure. These couplers remove the necessity for lap splices in the reinforcing steel, thus minimizing the extent of lining removal needed for the replacement of the reinforcing steel (Fig. IV-3).

Fig. IV-3: Mechanical coupler used for reinforcing steel in tunnel repair (image source: regbar Web Site)

After the steel has been cleaned, a protective coating must be applied to prevent accelerated corrosion resulting from the formation of an electrolytic cell. A variety of products are available for this purpose, including epoxy and zinc-rich coatings. Zinc-rich coatings are more suitable for this application because they do not create a bond-breaker, as many epoxies do. This is significant since these materials are applied with a paintbrush, making it difficult to avoid inadvertently coating the concrete surface. The zinc-rich coating should be applied within 48 hours of cleaning and no more than 30 days before the shotcrete is applied.

III.3. Shotcrete Repairs

There exist two distinct processes for the application of shotcrete: the Dry Process and the Wet Process. Both processes have been employed for numerous years and are equally relevant for tunnel rehabilitation efforts. The wet process produces minimal dust and is applicable in tunnels when partial closures permit traffic to continue within the tunnel during repair work. On the other hand, the dry process generates substantial dust and is not suitable for partial tunnel closures due to the limited visibility that the dust creates.

The successful execution of shotcrete, regardless of the method selected, is contingent upon the skill of the nozzle man (the individual who applies the shotcrete by operating the nozzle). A successful repair program requires that both the nozzle man and the other members of the shotcrete crew be proficient and tested on-site using mock-ups that closely mimic the areas to be repaired. These mock-ups should accurately reflect the shapes and surfaces that need repair. This testing program is frequently used to certify the skill level of the shotcreting crew and provides improved quality control throughout the work process.

The testing program fosters a mutual understanding between the Engineer, Owner, and Contractor, which defines an acceptable standard for the work. After the reinforcing and structural steel elements have been cleaned and coated, welded wire mesh is to be placed over the area designated for shotcreting (Fig. IV-4). The mesh is to be positioned within 2 inches of the edge of the repair. The wire mesh is secured to the existing reinforcing and to the substrate using 'J' hooks.

Fig. IV-4: Nozzle man applying Wet Process Shotcrete in tunnel repair
(image source: civildigital Website)

Once the entire area designated for patching is filled with shotcrete, the material should be allowed to cure for a period of 20 to 30 minutes. During this time, the mix is screeded and troweled to achieve the desired finish. Attempting to manipulate the shotcrete before this duration may lead to surface tearing, complicating the finishing process. It is essential to monitor the drying rate of the shotcrete, as the specified times may fluctuate based on wind conditions and relative humidity. After the repair has been troweled to the preferred finish, a curing compound must be applied to the surface of the new shotcrete to prevent rapid drying. The manufacturer of the premixed shotcrete will suggest a curing compound that is most appropriate for the jobsite conditions.

III.4. Structural Injection of Cracks

Cracking represents the most frequent defect encountered in concrete tunnel liners. While the majority of these cracks result from thermal activity, some are caused by structural stresses that were not factored into the design. It is essential to understand that cracks may also develop due to shrinkage and thermal stresses within the tunnel structure. Cracks exhibiting thermal stresses should not be structurally rebonded, as they will merely shift and re-crack. However, structural cracks that arise from structural movement, such as settlement, and are no longer active should be structurally rebonded. Any crack being evaluated for structural rebonding must be monitored to determine if any movement is occurring. A structural analysis of the tunnel lining should be carried out to ascertain if the crack in question requires rebonding. There are three types of resin typically available for the injection of structural cracks in tunnels. They are:

-Vinyl Ester Resin,

-Amine Resin; and

-Polyester Resin.

Since most tunnel cracks are damp or wet, vinyl ester resin, the most popular kind of resin used for bridge repair, is typically not appropriate for use in tunnels. Both surface-saturated concrete and damp or moist cracks cannot be structurally rebonded by vinyl ester resin. However, this epoxy will offer an adequate rebonding of the concrete if the fracture is completely dry throughout the injection procedure.

For the structural rebonding of tunnel cracks, amine and polyester resins work best. Both resins will adhere to surface-saturated concrete and are unaffected by moisture during installation. To guarantee correct resin installation, cracks with flowing water must be carefully injected, and the manufacturer's advice must be sought. The manufacturer's instructions must always be followed when injecting epoxy resins, especially when installing them overhead. An example of a typical epoxy installation is shown in Figure IV-5.

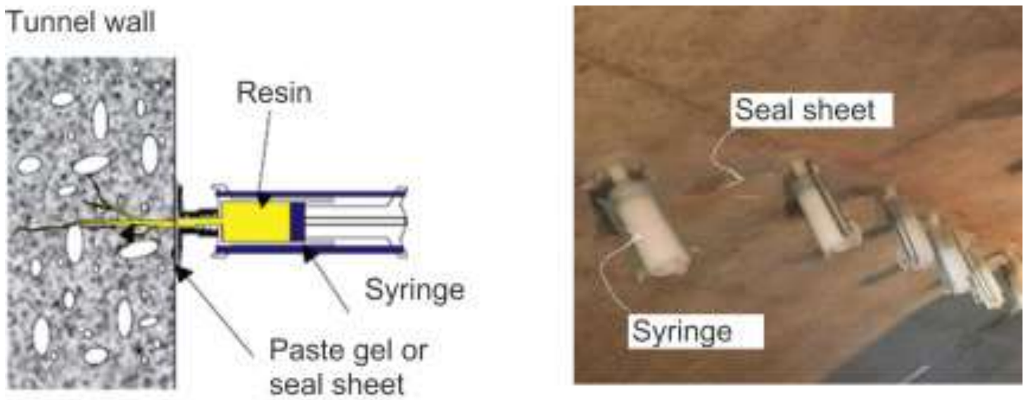


Fig. IV-5: Tunnel crack repairs using epoxy resin (*Issa*, 2009)

III.5. Segmental Linings Repair

Steel, cast iron, or precast concrete can be used for segmental lining, as covered in Chapter I. Typically; a tunnel's segmental lining serves as its main liner. The segments are either keyed or fastened together. Precast liners are the only segmental liners that are keyed. The most frequent issues with segmental liners are corner spalling of precast concrete segments and flange deformation in the case of steel and cast iron liners. Precast segment spalling and steel/cast iron segment flange deformation typically happen during installation or as a result of vehicle impact damage. Additionally, there are times when the liner plate of steel or cast iron segments rusts through.

III.5.1 Precast Concrete Segmental Liner

Spills in precast concrete liner segments are repaired using a high-performance polymer-modified repair mortar that is designed to restore the segment's original lines. If the segment gasket is damaged, the above-described injection of a polyurethane chemical grout restores the gasket's waterproofing function. Precast concrete liner segments with damaged bolt connections can be fixed by carefully removing the damaged bolt and replacing it with a new bolt, washer, waterproof gasket, and nut. Using a torque wrench, make sure the bolts are torqued to the original specification.

III.5.2. Steel/Cast Iron Lining

The kind of liner material affects how steel or cast iron liners are repaired. Deformed flanges and rust-induced liner segment penetration are frequent flaws in these kinds of liners. Flanges that are deformed can be fixed by reshaping them using heat or hammers. Welding on a new plate can fix holes in steel liner segments. Bolted connections frequently need to have the entire bolted connection replaced due to galvanic corrosion, which is brought on by dissimilar metal contact. A nylon isolation gasket is used to keep the high-strength bolt from coming into contact with the liner plate when the bolted connection is replaced. Figure IV-6 shows the repair operation conducted for a tunnel in Hefei city, (China) that suffers from several leakage at the level of segment circumferential and longitudinal joints (a and b), at the level of the bolt hole and the grouting hole (c and d).

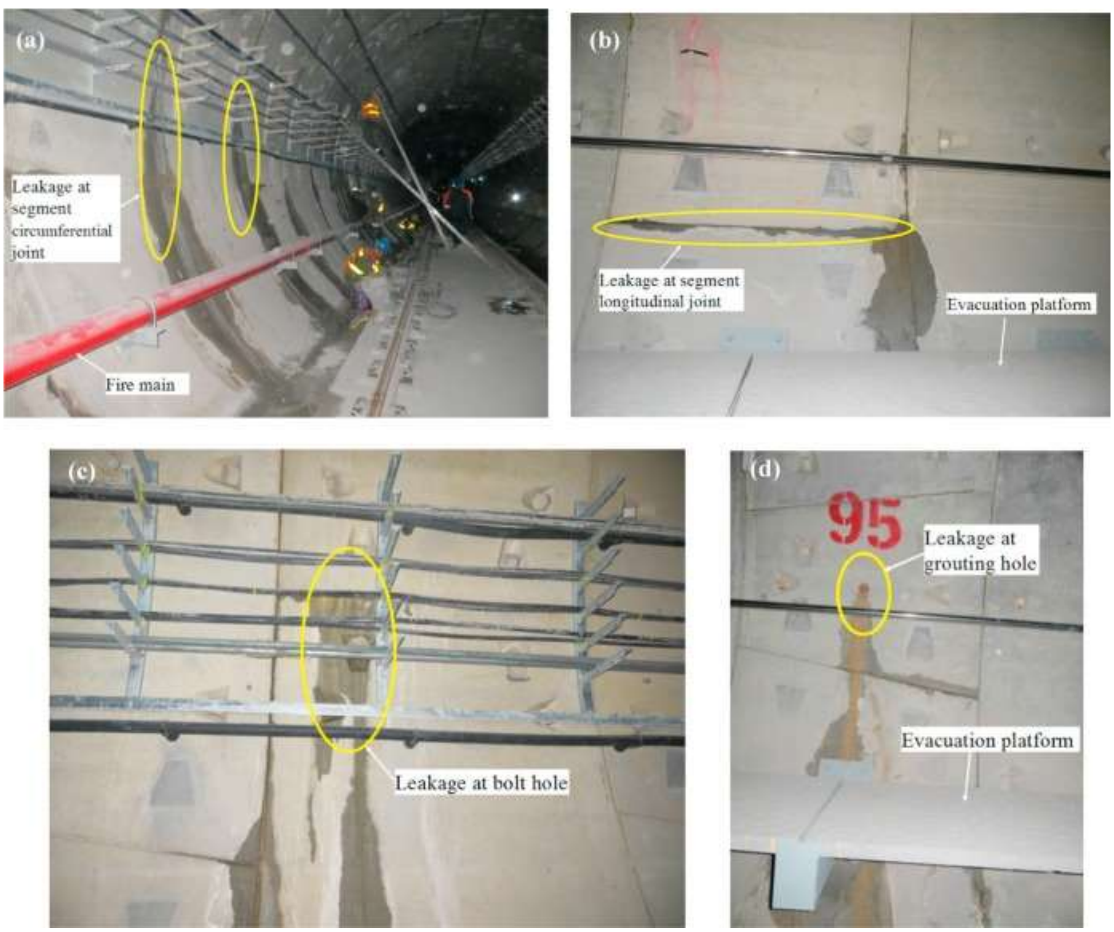


Fig. IV-6: Repair work conducted for a tunnel in Hefei city, China (*Jin-Long et al*., 2018)

III.6. Repair Materials

The key to successfully repairing a tunnel leak is choosing the right repair product for the site-specific situation. Injecting cementitious grout or a chemical is the most popular method of sealing a tunnel liner. It is possible to apply grout to the tunnel's outside to form a "blister" repair, which covers the damaged area with cement and stops the leak. The chemical characteristics of the soil and water, as well as groundwater influx, influence the grout choice. Injecting a chemical or particle grout straight into the crack or joint is the most popular technique for caulking leaky joints and cracks.

Drilling holes through the flaw at a 45-degree angle accomplishes this. On either side of the flaw, the holes are arranged alternately at intervals of ½ the structural element's thickness. The path for injecting grout into the defect is formed by the intersection of the drill holes. To guarantee that the grout is well bonded to the concrete, all holes must be flushed with water to remove any debris and to clean the sidewalls of the crack or joint before injection. In Figure V-7, typical injection ports are displayed. Grout is injected in the field in Figure V-8. The normal position of injection ports and details of repairing leaky cracks are shown in Figure IV-9.

Fig. IV-7: Typical Injection Ports for Chemical Grout

Fig. IV-8: Tunnel field leak injection (image source: Civilenggblitz Website)

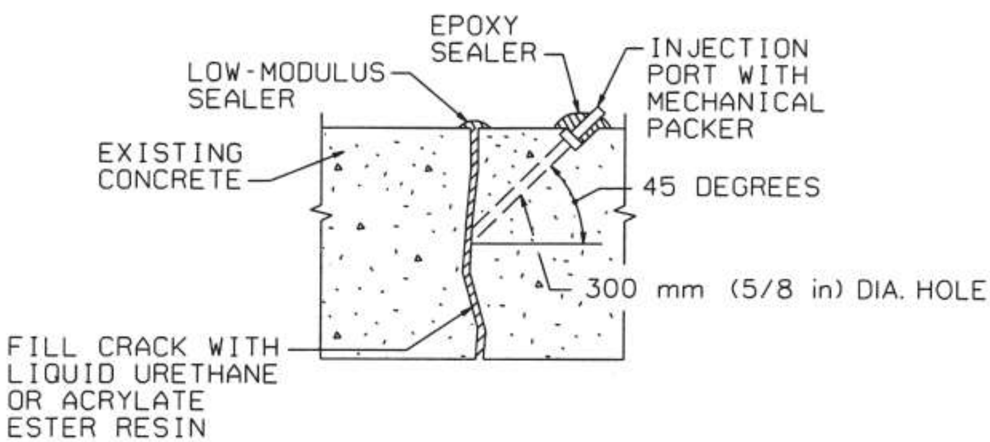
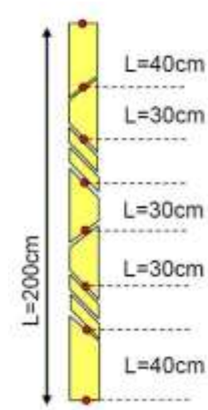


Fig. IV-9: Repair Detail: Typical location of injection ports and leaking crack (*FHWA & NHI*, 2009).

Exercises

Rock Quality Designation

1): Using the obtained measurements shown in the figure bellow calculate the RQD parameter.



2) : During the survey campaign for a 1,200 meters long tunnel project, surveys are carried out along the entire profile of the structure in five profiles spaced 300 meters apart.00 m and descending to depths ranging from 8 m to 32 m. The length of the core pass varies from 1 to 2 meters. The scooped samples measured the recovered pieces over 10 cm in length. The table below summarizes this data.

	Profil				
	1	**2**	**3**	**4**	**5**
Length of the core test pass (m)	1,00	1,50	1,20	1,80	2,00
Σ pieces ≥ 10 (cm)	73,50	111,99	93,00	166,98	186
Borehole Depth (m)	8	15	19	25	32

a. What visual information do we get from these surveys?
b. Calculate the RQD parameter for each of the five profiles!
c. What is the quality of rock in each of the five profiles?
d. Based on the results found, what are the excavation methods to be used to excavate the rock mass according to the advance of the excavation?

e. What conclusion can you draw from these R.Q.D results?

3) : During the reconnaissance campaign for a 10-meter-diameter, one-kilometer-long tunnel project, boreholes are drilled in five profiles spaced 250 meters apart. These profiles descend to different depths along the tunnel's longitudinal profile. The length of the core pass varies for each profile. The observations made on the samples taken from the core holes are shown in the table below.

Profil	Length of sounding pass (m)	Collected Samples Sample Length (cm)	Borehole depth (m)
1	1,00	 19 9,5 15 9 8 16 23,50	6
2	1,5	 19 3 x 8 22 18 2 x 7 23 30	9
3	2,00	 32 3 x 9 28 24 2 x 9,5 30 40	15
4	1,20	 21 3 x 5 18 11 2 x 6 18 25	18
5	1,80	 34 7 31 32 4 30 40	25

- What information do these surveys provide?
- Calculate the R.Q.D parameter for each of the five profiles!
- - How do you estimate the quality of the rock to be excavated in each of the five boreholes?
- What conclusions can you draw from these results?

  Based on the calculated RQD values, what means should be used to excavate the rock mass in question?

Support Dimensioning Methods:

4) During the preliminary study of a main road tunnel, the reconnaissance campaign resulted in the following data:

- Strength index obtained from Franklin test *Is* = 2.5 Mpa,

- *RQD* parameter = 38%,

- The joints are spaced approximately 145 mm apart, with a slightly rough surface, and a thickness of 0.9 mm.

- General consideration of the groundwater revels a dripping, and a water inflow of 35 l/min over a distance of 10 m.

The excavation is performed perpendicular to the tunnel axis, in the opposite direction to the dip of the discontinuities, estimated by the geologist at 60°.

a)- Calculate the rock's compressive strength *Rc*?
b)- Determine the overall rating of the five parameters used to calculate the *RMR*?
c)- Determine the orientation of the joints? What is the adjustment grade in this case?
d)- Calculate the *RMR* coefficient?
e)- Give the class of the rock crossed?
f)- Determine the average holding time of the excavation without support?
g)- Propose appropriate recommendations for the following three types of support: (Anchor bolts, shotcrete and metal hangers).

5) In a preliminary study of a main railway tunnel with a diameter of D=12 m, the reconnaissance campaign revealed a geological composition formed by a waterproof, resistant and highly consolidated filling with irregular, planar joints spaced at around 35 cm, a slightly weathered surfaces with separation less than 1 mm. The rock strength, measured in-situ, is equal to 68 Mpa, an *RQD* of 65%, and no water pressure is recorded. The excavation is perpendicular to the tunnel axis and the orientation of the joints is very unfavorable.

a) Calculate the *RMR* coefficient,
b) Give an evaluation of the deformation modulus and the various mechanical parameters of the rock mass (φ, c, E, ν)?

6) During the preliminary study of a major road tunnel of 10,5 m diameter. The site reconnaissance campaign gives the following data:

- An *RQD* value of 45%,
- A rock with three undulating and smooth joint set. The joint alteration is characterized by a Silty coat with low clay content infillings.
- An approximate water pressure value of 5 (Kg/cm²) without filling in discontinuities.
- Finally the tectonic state of the rock massif indicates the existence of isolated zones of weakness containing chemically degraded material, at a depth < 50 m.

Your asked to :

- Determine the six parameters to assess the *Q* index, and then calculate it?
- In function of the tunnel purpose, find the *ESR* value?
- Calculate the equivalent dimension of the excavation D_e?
- What is the tunnel Support category number?
- Describe the type of the support to be adopted?

7) We wish to dimension the support of the intersection zone of two major twin railway tunnels, each with a diameter of 8 meters. The tunnels pass through a massive rock characterized by only few joints. The intersection is 3 meters wider on each side. The following geological and hydrogeological data are available:

- *RQD* parameter equal to 78%;
- Discontinuous undulating joints approximately spaced about 2 meters apart, with a not continuous very rough surface;
- The joint walls are unaltered, with only a surface staining;
- The stress state in the massive rock is characterized by favorable conditions;
- Concerning the water pressure, according to the hydrogeological report, it seems to be moderate with joint fillings occasionally bathed.

You are asked to:

a) Following *Barton* method, calculate the *Q index?*
b) Calculate the minimum, maximum, and medium Young modulus of the mass rock?

c) Using a correlation formula and based on Q index, estimate the RMR factor and the deformation modulus E of the rock to be excavated?

d) Compare the E modulus values obtained from the two indexes?

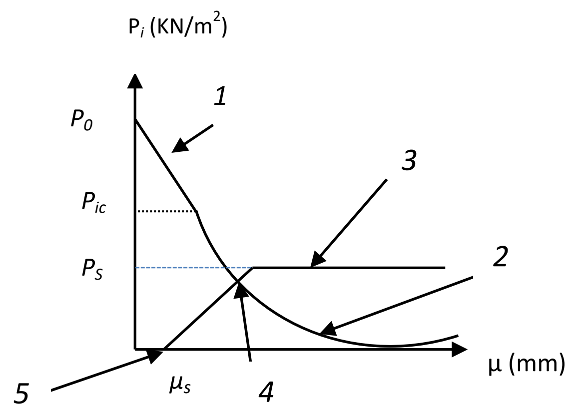
e) Determine the pressure applied on the support at the level of the vault and the walls?

f) What is the equivalent dimension of the intersection of these twin railway tunnels?

g) Describe the mass rock class and give the support category number?

h) Suggest the appropriate recommendations about the support to be adopted according to both methods? Compare the two recommendations?

8) The convergence-confinement method is used to study a 11-meter-diameter circular tunnel excavated in a homogeneous rock mass with a density of 2,600 kg/m³ and an average cover of 20 meters. The earth coefficient at rest is assumed to be 1 ($K_0 = 1$). The estimated modulus of elasticity is 1100 MPa, and the Poisson's ratio is 0.3. Figure below shows the interaction between the excavation walls and the support.



a. Justify the applicability of the convergence-confinement method to this tunnel.

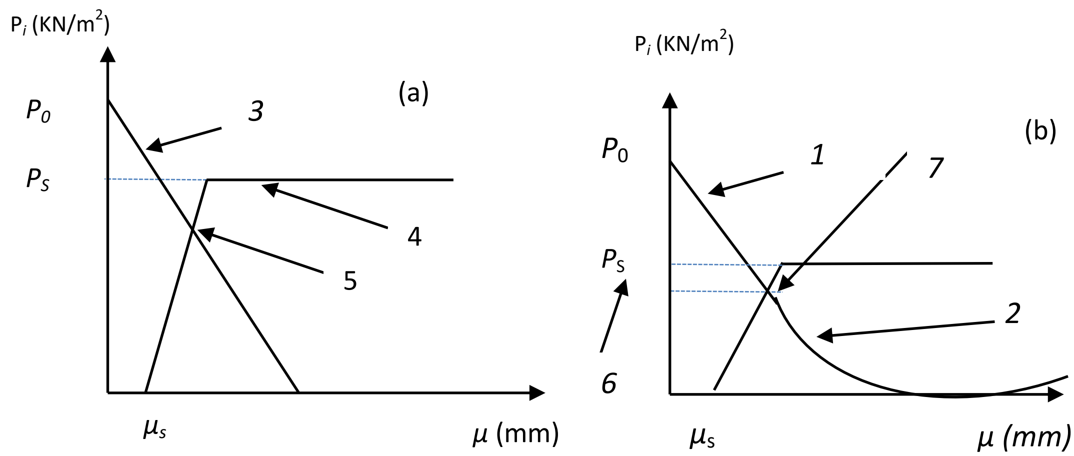
b. Identify the elements indicated by numbers 1 through 5 in the Figure above.

c. Give a title to this figure, according to the convergence-confinement method lexicon.

d. Calculate the initial stress (P_0) in (kN/m²) after excavation, knowing that the density of the soil in this section is 2,850 kg/m³.

e. Calculate the elastic displacement of the excavation walls in millimeters (mm), and then determine their displacement when the support is in place.

9) The convergence-confinement method is used to study a 12 -meter-diameter circular tunnel excavated in a homogeneous and isotrop rock mass with a density of 26 KN/m³.Two different sections spaced 60 meters apart are considered: the first one is excavated under a cover of 28 meters, and the second cover is 30 meters. The estimated modulus of elasticity is 1200 MPa, and the Poisson's ratio is 0.3. Figure below shows for both sections the two curves: that of ground convergence and that of support confinement.



a) Justify the applicability of the convergence-confinement method to this tunnel.
b) Complete Figure 2(a) and (b) by naming the elements from 1 to 7
c) How do the excavation walls behave in both sections (a and b)?
d) Calculate the initial stress (P_0) in (kN/m²) after excavation, in both sections, assuming that the soil density is equal to 26 KN/m³.
e) Calculate in both sections the elastic displacement in millimeters (mm), and then determine the displacement of the excavation walls at the moment of support laid.
f) What can you say about the used support in each section regarding criterion of economic and de security!?

10) We are interested in studying a horseshoe-shaped tunnel (Fig.1), whose section S is equal to 94.98 m^2 under a cover of variable height (Figure 1). The main constraints are isotropic (K0 = 1), and the terrain consists of two homogeneous layers, the parameters of which are cited in Fig.2.

The overloads on the surface of the land consist in a road (10 KN/m 2) and neighboring buildings (R+6). (20kN/m2 for the ground floor, plus a value of 10 KN/m^2 multiplied by the number of floors). Given that the convergence – confinement method applies to tunnels of circular shape and not horseshoe-shaped, a radius equivalent to a circular tunnel (**R_{eq}**) is to be calculated.

1. Is the convergence-confinement method applicable in this case of tunnel?
2. By classifying the results in a table, you are asked to:
    a. Calculate the tunnel equivalent radius?
    b. Calculate the initial ground pressure in the four sections?
    c. Calculate the compressive strength of the massif around the excavation?
    d. Check for the appearance of plasticity in the walls of the excavation for each section?
3. Calculate, in millimeters, the value of pseudo-elastic displacement μe for the four sections?
4. Calculate the pressure at the time of onset of plasticity for the plasticized sections?
5. Calculate the displacement at the time of onset of plasticity for the plasticized sections?
6. Plot the convergence curve of the massif for the sections that do not plasticize (if applicable)?
7. Plot the convergence curve of the massif for one plasticized section?
8. If we opt for a shotcrete support of 350mm thickness and a Young modulus Eb= 10000 Mpa, and a Poisson coefficient of 0.2. Calculate the maximum pressure of the support, the displacement at the time of its installation (μs), and at the onset of plasticity (μ_{Sbmax})?

Be given: $Deq = \sqrt{\frac{4S}{\pi}}$

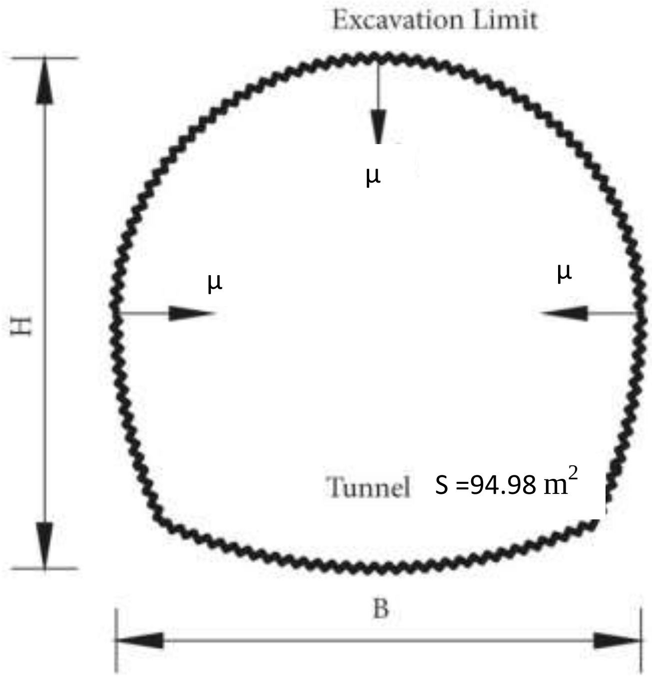


Fig.1- Tunnel Cross Section

$E_1 = E_2 = 3x10^6$ Mpa,
$\nu = 0{,}3$
C1= C2 = 0,35 Mpa
$\varphi 1 = \varphi 2 = 22°$,
$\gamma 1 = 21$ KN/m3,
$\gamma 2 = 22$ KN/m3,

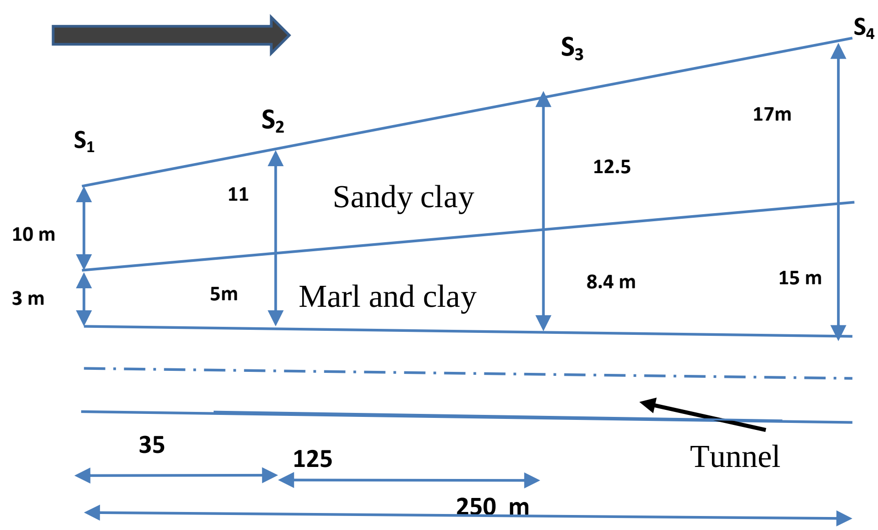


Fig.1- Tunnel longitudinal profile

11) During the excavation of a 5-meter-diameter underground gallery, two fractures formed a dihedral at the vault level, threatening to destabilize it (see Fig. 1a). The joints formed a 35-degree angle with the horizontal and are 1.35 meters apart. Assuming the joints are straight and parallel, and that the cross-section of the dihedral at the intrados of the vault is an isosceles triangle, as shown in Fig. 1b:

a) Calculate the surface area of the dihedral that is at risk of instability?
b) What is the joints dip in this rock mass?
c) Calculate the depth, H, of the dihedral at risk of instability, assuming that .
d) Calculate the weight (W) of the dihedral, assuming the rock's density is 30 kN/m³.
e) Assuming the utmost safety for workers and machinery on site (i.e., f = 5), calculate the number of bolts needed to stabilize the dihedral. Assume that the available bolts can withstand a maximum force of 15 KN.
f) In your opinion, what should the length of the anchor bolts be to maintain the stability of the dihedral when driven into the rock mass?

Note: To simplify the calculations, neglect the shape of the arch at the vault level.

We give: The dihedral volume *V*= 1/3 S*H

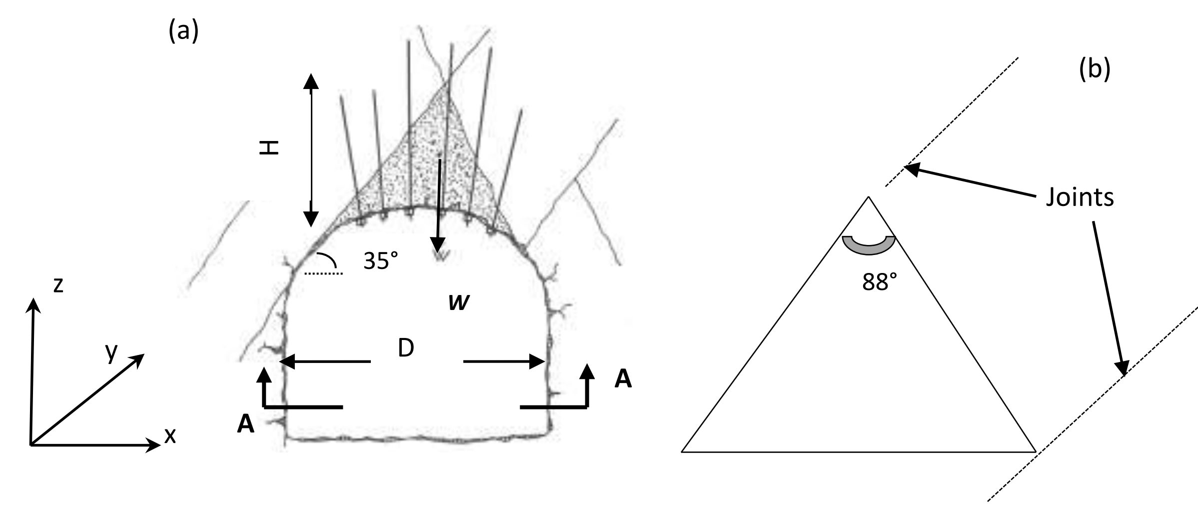


Fig.1 : (a) Tunnel cross-section and (b) Section A-A (bottom view of the dihedral at the vault intrados)

12) When an underground excavation of 2.5 meters of diameter is excavated, two fractures in the rock at the vault level form a dihedral (Figure 1), which can be destabilized. We will assume that the discontinuities are parallel with a dip of 45 degrees. The rock density is equal to 24 KN/m^3.

If the bolts available on site can withstand a maximum force of 15 KN, and assuming that the dihedral can be assimilated to a cone, you are asked to:

a) Calculate the dihedral lower surface, its depth H, its volume, and its weight W?

b) Calculate the number of bolts required and the anchoring bolts length to keep the dihedron stable in the rock.

c) What will be the bolt distribution density on the dihedral surface?

Note that a reasonable degree of safety is required.

Note: To simplify calculations, the arch shape at the vault level is considered as straight.

We give: The dihedral surface S and volume:

$S = \sqrt[2]{p(p-a)(p-b)(p-c)}$, $P\,\frac{a+b+c}{2}$, $V= 1/3\ S*H$.

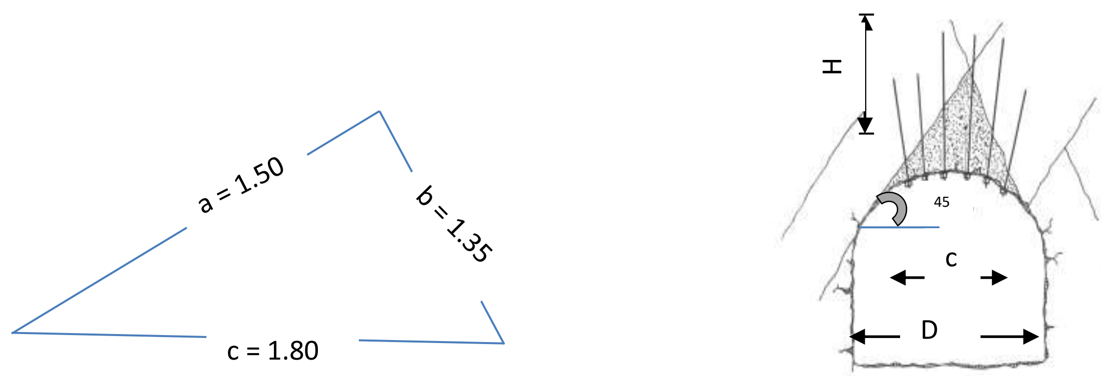


Fig.1: (a) Bottom view of the dihedral and (b) cross-section of the tunnel

13) A road tunnel of 10 m diameter is dogged in a mass rock composed of two soli layers under variable covers (Fig.1). the mechanical and geotechnical parameters are as follow:

E_1=900 Mpa, E_2=1100 Mpa ; C_1= 0,2 Mpa , C_2 =1,1 Mpa ; φ_1=45°, φ_2= 22°, γ_1= 18 KN/m^3, γ_2= 14 KN/m^3, ν = 0,3 for both layers.

Presenting the results in a table (see template), you are asked to calculate in each of the three sections:

a) The initial pressure P_0 ?

b) The compressive strength R_c in the excavation walls?

c) Check for wall plasticity?

d) Calculate, in millimeters, the value of the elastic displacement of the walls?

e) What material should be used to excavate this tunnel? Why

Table template for results presentation.

Section	P_0 (KN/m^2)	Rc (KN/m^2)	F	plasticity Risk	Elastic Displacement μ_e (mm)
S1					
S2					
S3					

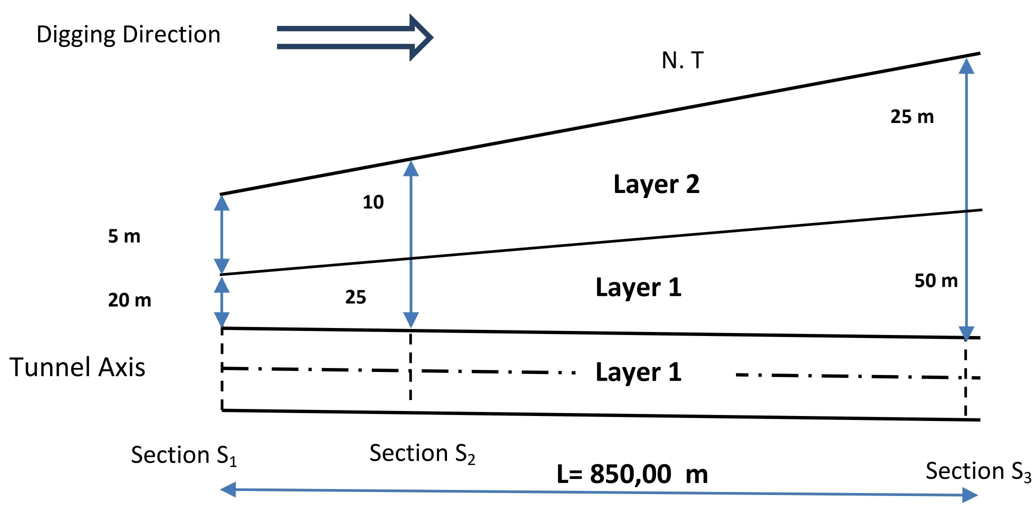
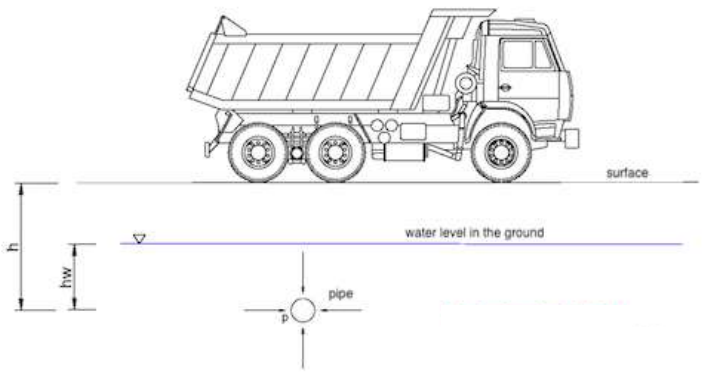


Fig. 1 : Longitudinal profile of the tunnel

14) We want to study the pressure exerted on an underground pipe that is buried two meters deep (see figure). The water table is located one meter below the surface. The soil above the pipe consists of two layers: the first has a density of 1,800 kg/m³ and the second with a density of 1,000 kg/m³. Calculate the pressures to which the pipe is subjected, including ground, water, and vehicle pressure.



15) Write an algorithm/program to resolve convergence problems during tunnel digging using the convergence-confinement method. Use whatever advanced (C, C++,…).or interpretive language (Paython, R) you master.

<u>Solution :</u>

1) *RQD = 85%*

2)

a): *Number of joints, rock quality, and geological formation.*

b): *73.5, 74.66, 77.5, 92.76, 93*

c): *Medium, Medium, Good, Excellent, Excellent*

d): *Mechanical digging, Mechanical digging, Mechanical digging, Blast, Blast*

e): *The quality of the rock improves with increasing drilling depth.*

3)

a) : *The rock quality, the number of joints, and the geological composition*

b): *: 73,5% ; 74,66% ; 77% ; 77,5% ; 92,77%*

c): *Medium quality: profiles 1 and 2, good quality: profiles 3 and 4, and excellent: profile 5.*

d): *The quality of the rock improves with increasing drilling depth.*

e): *BTM from the start of the project to 250 m and blasting from 250 m to the rest of the project.*

4)

a) : *Rc = 23.* I_s *= 57.5 MPa*
b) : *N1 =7, N2=8, N3=8, N4=20, N5=4*
c): *Medium joint orientation ,* N_{adj} *= -5*
d): *RMR=* 42
e): *Fair rock (class III). The average holding time of the excavation without support is one week for a 3 meters span.*
f): *Support Recommendations for the three types of support:*

- Anchor Bolts Spaced 1-1,5 m + Welded mesh + 30 mm shotcrete in vault if required

- Shotcrete: 10 cm on the vault and 5 cm on the walls + occasionally welded mesh + bolts if required

- Lightweight metal hangers spaced 1,5 – 2 m

5)

a) : *RMR=* 60
b) : *E≈ 18 GPa,* $\nu \approx 0.27$, $\varphi = 28^0$, *C = 6.8 MPa*

6)

a) : *RQD = 45%,* J_n *=9, Jr = 2, Ja = 3, Jw=0.5, and SRF = 5, Q = 0.34*
b) : *ESR = 1*
c): *De= 10,5m*
d): *Tunnel Support category number 32*
e): *Span/ESR > 20**,** systematic bolting of 1 m spacing + shotcrete with mesh reinforced of 40 to 60 cm thickness.*

7)

a) : *Q = 47.45*

b) : E_{min} *= 16.77 GPa,* E_{med} *= 41.90 GPa,* E_{max} *= 67 GPa*

c): *RMR = 59.34, E =17.12 GPa, E = 18.68 GPa (using two different formulas)*

d): *Using RMR parameter, the obtained E modulus is equivalent to the minimum E value assessed using Q index.*

e): P_{roof} *= 0.17 MPa,* P_{Wall} *= 0.13 MP.*

f): *De = 10.77*

g): *Very good rock class.*

h): *Untensioned, grouted bolting spaced from 1.5 to 2 m + Chain Link Mesh. (Barton's Recommendations)*

Bolt spaced from 1 to 1.5 m + shotcrete: e = 10 cm on the vault, e = 5 cm on the walls. (Bieniawski Recommendations).

The two support recommendations are quite similar. However, Bieniawski's recommendations are more conservative and therefore safer.

8)

a): *The Convergence-Confinement method is applicable to this tunnel case.*
Justification :
-Circular tunnel;
-Homogeneous rock mass,
-Stress isotropy ($K_0 = 1$), and
- Tunnel excavated below medium depth (H =20 m)

b): *1: Elastic domain of the soil convergence curve; 2: Plastic domain of the soil convergence curve; 3: Confinement curve; 4: Balancing point, 5: Displacement at the time the support is laid.*

c): *Convergence-Confinement Connection Curves.*

d) : $P_0 = 560$ kN/m²

e): $\mu_e = 3.64$ mm, $\mu_S = 1.2$ mm.

9)

a): *The Convergence-Confinement method is applicable to this tunnel case.*

Justification :
-Circular tunnel;
-Homogeneous and isotrop rock mass,
- Tunnel excavated below medium depth (H =28 and 30 m)

b): *1: Elastic domain of the soil convergence curve; 2: Plastic domain of the soil convergence curve; 3: soil convergence curve (elastic behavior), 4: Confinement curve; 5: Balancing point, 6: maximum support pressure, 7: plasticity onset point.*

c): *In section (a), the excavation walls behave in an elastic manner.*

In section (b), the excavation walls behave in an elastic manner till the plasticity onset point, when they behave in a plastic manner.

d): *Section (a):* P_0 *=728* KN/m², *Section (b):* P_0 *= 780* KN/m²

e): *Section (a):* μ_e *= 4.73 mm,* μ_S *= / mm (elastic behavior of the terrain). Section (b):*μ_e *= 5.07 mm,* μ_S *= 1.67 mm.*

f): There is no need to support excavation (a); a waterproof coating is sufficient.

Section (b), however, needs to be supported due to the elasto-plastic behavior of the excavation walls. Thus, the support ensures the safety criterion.

10)

1- *No, the convergence confinement method is not applicable in this case of tunnel because the shape is not circular.*

2-

   a- *The tunnel Equivalent radius :*

   $Deq = \sqrt{\frac{4s}{\pi}} = \sqrt{\frac{(4\times 94.98)}{\pi}}$ *= 11m, Thus* $R = \frac{11}{2} = 5.5m$

   b- *The initial ground pressure in the four sections:*

   $P_o = \sum \gamma i \times hi + Q$

   $P\,1: P_o = (21\times 3) + (22\times 10) + 10 + 20 + (10\times 6) = 373 KN/m^2$

   $P\,2 : P_o = (21\times 5) + (22\times 11) + 90 = 437\ KN/m^2$

   $P\,3 : P_o = (21\times 8.4) + (22\times 12.5) + 90 = 541.4\ KN/m^2$

   $P\,4 : P_o = (21\times 15) + (22\times 17) + 90 = 779\ KN/m^2$

c- The compressive strength in the walls around of the excavation:

$$Rc = 2 \times C\ tang\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) = 2\times 0.35 \times 10^3\ tang\left(\frac{180}{4} + \frac{22}{2}\right) = 1037.79\ KN$$

d-The stability factor:

$F = \frac{2\times Po}{Rc}$

P 1: F= 0.72; P 2 : F = 0.91; P 3 : F= 1.04 ; P 4 : F = 1.50

3- *Elastic displacement* μ_e*:*

4- $\mu_e = R\times P_0 \times \left(\frac{1+\nu}{E}\right)$

P1: $\mu_e = 88 \times 10^{-5}$ *mm ; P2 :* $\mu_e = 11 \times 10^{-4}$ *mm ; P 3 :* $\mu_e = 13 \times 10^{-4}$ *mm ; P 4:* $\mu_e = 18 \times 10^{-4}$ *mm*

4-The pressure and displacement values at the moment of terrain's plasticity:

Pic= $\frac{2Po-H(Kp-1)}{Kp+1}$; $\mu_{ic} = \lambda_{ic} \times \mu_e$; $\lambda_{ic} = \sin\varphi + \frac{C}{Po}\cos\varphi$

H=$\frac{C}{TangQ} = \frac{0.35\times10^3}{tang22}$ =866.28 KN/m^2 =0.866MPa

Kp= tang^2 $(\frac{\pi}{4}+\frac{Q}{2})$ = tang^2 $(\frac{180}{4}+\frac{22}{2})$= 2.197

Profile 3:

Pic = $\frac{2\times541.4-0.866\times10^3(2.197-1)}{(2.197+1)}$ =14.45 KN/m^2 ;

λ_{ic} = sin22 + $\frac{0.35\times10^3}{541.4}$ cos22= 0.97

μ_{ic} = 0.97 ×0.0013 = 0.00126 m

Profile 4:

Pic =$\frac{2\times779-(0.866\times10^3)(2.197-1)}{(2.197+1)}$ = 163.09 KN/m^2 ; $\mu_{ic} = \gamma_{ic} \times \mu_e$ = 0.79 × 0.0018= 0.00142 m

λ_{ic} = sin22 + $\frac{0.35\times10^3}{779}$ cos22= 0.79

μ_{ic} = 0.97 ×0.0013 = 0.00126 m

8- Pressure and displacement in the support:

$P_{bmax} = \frac{\sigma\, bmax\,.\, e}{R}$; $\sigma_b = \frac{0.85\times25\times10^3}{1.15}$= 18478.26 KN/m^2

$P_{bmax} = \frac{18478.26\ \times0.20}{5.5}$ =671.93 KN

$K_b = \frac{Eb\times e}{R} = \frac{10000\times10^3\times0.20}{5.5}$ = 363636.36 KN

$\mu_{bmax} = \frac{Pb\times R}{Kb} = \frac{671.93\ \times5.5}{363636.36}$ = 0.010 m = 1 mm

Profile	P_0 (KN/m^2)	F	Plasticity (yes/no)	μ_e (mm)	P_{ic} (KN/m^2)	μ_{ic}(mm)
P1	373	0.72	no	88 x 10^{-5}	/	/
P2	473	0.91	no	11 x 10^{-4}	/	/
P3	541.4	1.04	yes	13 x 10^{-4}	14.45	126 x 10^{-6}
P4	779	1.50	yes	18 x 10^{-4}	163.09	142 x 10^{-5}

11)

a): *S =0.90* m^2*.*

b): *The joints dip is* 35^0

c): *H = 1.75 m.*

d): *W = 47.26* KN/m^2

g): *N = 16 bolts*

e): *The length of the anchor bolts must exceed the dihedral depth "H" in the mass rock. So 2 m of anchoring is selected.*

11):

a): *S = 0.99* $m^2 \approx 1\ m^2$*, H = 1 m , V = 1* m^3 *, W=24 KN.*

b): *N = 6 bolts (if f= 3.5)*

c): *D= 6* $Bolt/m^2$

12)

Section	P_0 (KN/m^2)	Rc (KN/m^2)	F	Plasticity Risk	Elastic Displacement μ_e (mm)
S1	430	965.68	0.89	No	3.10
S2	590	965.68	1.22	Yes	4.26
S3	1250	965.68	2.58	Yes	9.03

e): *Due to the soil nature, which cannot stand unsupported during the digging works, the appropriate material to be used to excavate this tunnel is a TBM (Tunnel Boring Machine).*

13)

p_{soil} *= [(1800 x 9.81 x 1 m) + (1100 x 9.81 x1 m)]*
= 28449 Pa = $\underline{28.5}$ *kPa*
p_{water} *= (1000 x 9.81* m/s^2 *x 1 m)*
*= 9810 Pa = * $\underline{9.8}$ *kPa*
$p_{transport}$ *= read from the diagram (Fig.VI-) =* $\underline{20}$ *kPa*
Total pressure
p = 28.5 + 9.8 kPa + 20 kPa = $\underline{58}$ *kPa = 58* kN/m^2

Appendix A: Rock Massifs Support Using *Barton*'s method

Category	Critical Factors			Support	
	RQD/Jn	Jr/Ja	Span/ESR		
1	-	-	-	Sb (utg)	
2	-	-	-	Sb (utg)	
3	-	-	-	Sb (utg)	
4	-	-	-	Sb (utg)	
5	-	-	-	Sb (utg)	
6	-	-	-	Sb (utg)	
7	-	-	-	Sb (utg)	
8	-	-	-	Sb (utg)	
9	≥20	-	-	(A)	Sb(utg)
	<20	-	-	(B)	B(utg)2,5-3 m
10	≥30	-	-	(A)	B(utg) 2-3 m
	<30	-	-	(B)	B(utg) 1,5-2m+ clm
11	≥30	-	-	(A)	B(tg) 2-3 m
	<30	-	-	(B)	B (tg) 1,5-2m+ clm
12	≥30	-	-	(A)	B(tg) 2-3 m
	<30	-	-	(B)	B (tg) 1,5-2m+ clm
13	≥10	≥1,5	-	(A)	Sb(utg)
	≥ 10	<1,5	-	(B)	B(utg) 1,5-2 m
	<10	≥1,5	-	(C)	B(utg) 1,5-2 m
	<10	<1,5	-	(D)	B(utg) 1,5-2m +S 2-3cm
14	≥10	-	≥15 m	(A)	B(tg) 1,5-2 m +clm
	<10	-	≥15 m	(B)	B(tg) 1,5-2 m +S (mr) 5-10 cm
	-	-	<15 m	(C)	B(utg) 1,5-2 m + clm
15	<10	-	-	(A)	B(tg) 1,5-2 m +clm
	≤10	-	-	(B)	B(tg) 1,5-2 m +S (mr) 5-10 cm
16	>15	-	-	(A)	B(tg) 1,5-2 m +clm
	≤15	-	-	(B)	B(tg) 1,5-2 m +S (mr) 10-15 cm
17	>30	-	-	(A)	Sb(utg)
	≥10, ≤30	-	-	(B)	B(utg) 1-1,5 m
	<10	-	≥6 m	(C)	B(utg) 1-1,5 m +S 2-3 cm
	<10	-	<6 m	(D)	S 2-3 cm
18	>5	-	≥ 10m	(A)	B(tg) 1-1,5 m +clm
	>5	-	<10 m	(B)	B(utg) 1-1,5 m +clm
	≤5	-	≥10 m	(C)	B(tg) 1-1,5 + S 2-3 cm
	≤5	-	<10 m	(D)	B(utg) 1-1,5 m + S 2-3 cm
19	-	-	≥20 m	(A)	B(tg) 1-2 m +S (mr) 10-15 cm
	-	-	<20 m	(B)	B(tg) 1-1,5 m +S (mr) 5-10 cm
20	-	-	≥35 m	(A)	B(tg) 1-2 m +S(mr) 20-25cm
	-	-	<35 m	(B)	B(tg) 1-2m +S(mr) 10-20 cm
21	≥12,5	≤0,75	-	(A)	B(utg) 1m +S 2-3 cm
	<12,5	≤0,75	-	(B)	S 2,5-5 cm
	-	>0,75	-	C)	B(utg) 1m

22	>10 , <30 ≤ 10 <30 ≥ 30	>1 > 1 ≤ 1 -	- - - -	(A) (B)	B(utg) 1m + clm S 2,5-7,5 cm B(utg) 1 m + S(mr) 2,5-5 cm B(utg) 1 m
23	- -	- -	≥ 15 m < 15 m	(A) (B)	B(tg) 1-1,5 m +S(mr) 10-15 cm B(tg) 1-1,5 m +S(mr) 5-10 cm
24	- -	- -	≥ 30 m < 30 m	(A) (B)	B(tg) 1-1,5 m +S (mr) 15-30 cm B(tg) 1-1,5 m +S (mr) 10-15 cm
25	>10 ≤ 10 -	> 0,5 > 0,5 ≤ 0,5	- - -	(A) (B)	B(utg) 1m + mr ou clm B(utg) 1m + S(mr) 5 cm B(tg) 1m + S (mr) 5 cm
26	- -	- -	- -	(A) (B)	B(tg) 1m + S (mr) 5-7,5 cm B(utg) 1m + S 2,5-5 cm
27	- - - -	- - - -	≥ 12 m < 12 m > 12 m < 12 m	(A) (B)	B(tg) 1 m+S (mr) 7,5 -10 cm B (utg) 1m +S (mr) 5-7,5 cm CCA 20-40 cm + B(tg) 1m S (mr) 10-20 cm +B(tg) 1 m
28	- - - -	- - - -	≥ 30 ≥20m,<30m <20 m -	(A) (B)	B(tg) 1 m +S (mr) 30-40 cm B (tg) 1m + S(mr) 20-30 cm B (tg) 1 m +S(mr) 15-20 cm CCA(sr) 30-100 cm +B(tg) 1m
29	>5 ≤ 5 -	> 0,25 > 0,25 ≤ 0,25	- - -	(A) (B)	B (utg) 1 m +S 2-3 cm B (utg) 1 m +S (mr) 5 cm B (tg) 1 m+ S (mr) 5 cm
30	≥ 5 < 5 -	- - -	- - -	(A) (B)	B(tg) 1 m + S 2,5-5 cm S (mr) 5-7,5 cm B(tg) 1m + S (mr) 5-7,5 cm
31	>4 ≤4 , ≥1,5 <1,5 -	- - - -	- - - -	(A) (B) (C) (D)	B(tg) 1 m +S (mr) 5-12,5 cm S(mr) 7,5-25 cm CCA 20-40 cm +B(tg) 1m CCA(sr) 30-50 cm +B(tg) 1m
32	- - -	- - -	≥20 m <20 m -	(A) (B) (C)	B(tg) 1 m+ S(mr) 40-60 cm B(tg) 1 m +S(mr) 20-40 cm CCA(sr) 40-120 cm+ B(tg) 1m
33	≥ 2 < 2 -	- - -	- - -	(A) (B) (C)	B(tg) 1m + S(mr) 2,5-5 cm S(mr) 5-10 cm S(mr) 7,5-15 cm
34	≥ 2 < 2 - -	≥ 0,25 ≥ 0,25 < 0,25 -	- - - -	(A) (B) (C) (D)	B(tg) 1m +S(mr) 5-7,5 cm S(mr) 7,5-15 cm S(mr) 15-25 cm CCA(sr) 20-60 cm +B(tg) 1m
35	- - - -	- - - -	≥ 15 m ≥ 15 m <15 m < 15 m	(A) (B) (C) (D)	B(tg) 1m +S(mr) 30-100 cm CCA(sr) 60-200 cm +B(tg) 1m B(tg) 1m+ S(mr) 20-75 cm CCA(sr) 40-150 cm +B(tg) 1m

36	- -	- -	- -	(A) (B)	S(mr) 10-20 cm S(mr) 10-20 cm+ B(tg) 0,5-1 m
37	- -	- -	- -	(A) (B)	S(mr) 20-60 cm S(mr) 20-60 cm+ B(tg) 0,5-1 m
38	- - - -	- - - -	≥10 ≥10 <10 <10	(A) (B) (C) (D)	CCA(sr) 100-300 cm CCA(sr) 100-300 cm+ B(tg) 1m S(mr) 70-200 cm S(mr) 70-200 cm + B(tg) 1m

Legend:

Sb : Spot bolting.

B : systematic bolting followed by bolt spacing in m (bolting diameter ϕ = 20mm).

Utg : untensioned, grouted.

Tg : tensioned (for resistant massifs, grouting and post-tensioning for very poor foundations).

S : Shotcrete followwed by the thickness in cm.

mr : mesh reinforced.

clm: Chain Link Mesh.

CCA : Cast concrete Arch (followed by the thickness in cm).

Sr : steel reinforced.

Appendix B: Values of Young's Elastic Modulus and Poisson's Ratio

In an isotropic elastic medium.

a) Young's modulus E:

- Loose soils: E = 20 to 100 MPa

- Compact soils: E = 100 to 300 MPa

- Rocky soils: E = up to several times 1000 MPa

b) Poisson's ratio ν :

- Fine, compact, saturated soils: ν = 0.5 in the short term

- Other soils: ν = 0.25 to 0.30

Other materials Young's modulus and Poisson coefficient values are given in the table below:

Typical Values of Young's Elastic Modulus and Poisson's Ratio for Pavement Materials

Material	Young's Elastic Modulus (E or M_r), psi	Poisson's Ratio (μ or ν)
Asphalt concrete (uncracked) 32 F 70 F 140 F	2,000,000 – 5,000,000 300,000 – 500,000 20,000 – 50,000	0.25 – 0.30 0.30 – 0.35 0.35 – 0.40
Portland cement concrete (uncracked)	3,000,000 – 5,000,000	0.15
Extensively cracked surfaces	Similar to granular base course materials	Similar to granular base course materials
Crushed stone base (clean, well-drained)	20,000 – 80,000	0.35
Crushed gravel base (clean, well-drained)	20,000 – 80,000	0.35
Uncrushed gravel base Clean, well-drained Clean, poorly-drained	 10,000 – 60,000 3,000 – 15,000	 0.35 0.40
Cement stabilized base Uncracked Badly cracked	 500,000 – 2,000,000 40,000 – 200,000	 0.20 0.30
Cement stabilized subgrade	50,000 – 500,000	0.20
Lime stabilized subgrade	20,000 – 150,000	0.20
Gravelly and/or sandy soil subgrade (drained)	10,000 – 60,000	0.40
Silty soil subgrade (drained)	5,000 – 20,000	0.42
Clayey soil subgrade (drained)	3,000 – 12,000	0.42
Dirty, wet, and/or poorly-drained materials	1,500 – 6,000	0.45 – 0.50
Intact Bedrock Note: Values greater than 500,000 have negligible influence on surface deflections.	250,000 – 1,000,000	0.20

Note: Exceptions to the typical values given above **often** occur. High fines contents and/or high moisture contents tend to reduce Young's modulus and increase Poisson's ratio. Unusually low fines contents and/or low moisture contents have the opposite effect. Poisson's ratio for a completely saturated material will be close to 0.50. Well-cured, asphalt emulsion stabilized gravel, and reclaimed asphalt pavements, will have moduli slightly less than asphalt concrete.

References:

Abbas, S.M. & Heinz Konietzky, H. (2017). Rock mass classification systems. In book: Introduction to Geomechanics (pp.43). Edition: 09/2017. Publisher: Department of Rock Mechanics, Technical University Freiberg, Germany. Editors: Heinz Konietzky.

A.F.T.E.S. (1993). Recommandations relatives au choix du soutènement en galerie, TOS hors-série.

A.F.T.E.S. (1995). Recommandations relatives aux tassements liés au creusement des ouvrages souterrains, TOS.

A.F.T.E.S. (2000). Recommandations relatives aux choix des techniques d'excavation mécanisées, TOS.

Almenara J.T., Skretting E., and Holter K. G. (2024). Design of Rock Support in Low Overburden and Hard Rock Conditions with the use of Rock Mass Classifcation Systems and Numerical Analyses: a Study Based on the Construction of the Hestnes Railway Tunnel, Norway. Rock Mechanics and Rock Engineering, DOIhttps://doi.org/10.1007/s00603-024-04126-8

Aydan, O., Ulusay, R., and Tokashiki, N., 2014. A New Rock Mass Quality Rating System: Rock Mass Quality Rating (RMQR) and Its Application to the Estimation of Geomechanical Characteristics of Rock Masses: *Rock Mechanics Rock Engineering*, Vol.. 47, pp: 1255-1276.

Barton, N. (2006) Rock Quality, Seismic Velocity, Attenuation and Anisotropy. Taylor & Francis Group, London, UK. ISBN 978-0-415-39441-3.

Barton, N., (2002). Some new Q-value correlations to assist in site characterization and tunnel design: International Journal of Rock Mechanics and Mining Sciences, v. 39, pp: 185-216.

Barton, N., Løset, F., Lien, R., and Lunde, J., (1980), Application of the Q-system in design decisions. , *In:* Bergman, M., ed., Subsurface space, Volume 2: New York Pergamon., pp: 553-561.

Barton, N.R., Lien, R., and Lunde, J., (1974), Engineering classification of rock masses for the design of tunnel support: Rock Mechanics, Vol. 6, pp: 189-239.

Maidl, B., Thewes, M. and Maidl, U. (2013). Handbook of Tunnel Engineering: Volume I: Structures and Methods. Ed. Wiley Ernst & Sohn. 482 p. DOI:10.1002/9783433603499

Bieniawski, Z.T., (1989). Engineering rock mass classifications: a complete manual for engineers and geologists in mining, civil, and petroleum engineering: New York, Wiley, xii, p.251

Bieniawski, Z.T., (1993). Classification of rock masses for engineering: The RMR system and future trends, *In:* Hudson, J.A., ed., Comprehensive Rock Engineering, Volume 3: Oxford; New York, Pergamon Press, pp: 553-573.

Bouvard-Lecoanet, A., Colombet, G., et Esteulle, F. « ouvrage souterrains. Conception, réalisation, entretien ». Presse des ponts et chaussées, 2ème édition 1992, p. 285

Broch, E.& Franklin, J.A.. (1972). The point-load strength test. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*,Vol. 9(6),, PP: 669-676. https://doi.org/10.1016/0148-9062(72)90030-7

CFMR, Manuel de mécanique des roches : Tome 1, Ed. Les Presses de l'École des Mines, Paris, (2000).

Deere D.U. (1964). Technical Description of Rock Cores for Engineering Purposes. *Rock mechanics and rock engineering*, vol1, pp, 17-22.

Deere D.U. and Miller R.P. (1966), "Engineering Classification and Index Properties for Intact Rock'' Technical Report AFWL-TR-65-115, Air Force Weapons Laboratory, New Mexico.

Deere, D. U., and Deere, D. W. (1988). "The Rock Quality Designation (RQD) Index in Practice, in Rock Classification Systems for Engineering Purposes," L. Kirkaldie, ed., ASTM 1984. doi.org/10.1533/9781845697037.2.169.

Federal Highway Administration & National Highway Institute. Technical Manual for Design and Construction of Road Tunnels - Civil Elements. U.S. Department of Transportation Publication No. FHWA-NHI-10-034, December 2009.

Feng K. and al., (2023). Experimental study on the high water pressure erosion mechanism and its influence on the submarine shield tunnel concrete segments. *Construction and Building Materials*, Volume 408. https://doi.org/10.1016/j.conbuildmat.2023.133577.

FHWA-NHI. Technical Manual for Design and Construction of Road Tunnels – Civil Elements. Report No. 10-034.(2009).,P. 702.

Getuli V. et al., (2021). "On-demand generation of as-built infrastructure information models for mechanised Tunnelling from TBM data: A computational design approach". *Automation in Construction*, Vol. 121, 103434, https://doi.org/10.1016/j.autcon.2020.103434.

Grimstad, E., Barton, N. 1993. Updating of the Q-system for NMT. In: International Symposium on Sprayed Concrete. Fagernes, Proceedings, pp. 46–66

Handbook of Tunnel Engineering. Volume I: Structures and Methods. Maidl, B., Thewes, M. & Maidl U.1st Edition. Ed. Erest & Sohn, A Wilwy Brand, 2013

Hoek, E. (2007). Practical Rock Engineering. P. 341. available from: https://www.rocscience.com/assets/resources/learning/hoek/Practical-Rock-Engineering-Full-Text.pdf

Hoek, E.; Kaiser, P.K.; Bawden,W.F. Support of Underground Excavations in Hard Rock; CRC Press: Boca Raton, FL, USA, 1993.

Hussian, S.; Mohammad, N.; Ur Rehman, Z.; Khan, N.M.; Shahzada, K.; Ali, S.; Tahir, M.; Raza, S.; Sherin, S. (**2020).** Review of the geological strength index (GSI) as an empirical classification and rock mass property estimation tool: Origination, modifications, applications, and limitations. Adv. Civ. Eng., 2020, 6471837.

Issa C.A. (2009). Methods of crack repair in concrete structures. *Civil and Structural Engineering Failure, Distress and Repair of Concrete Structures*, Woodhead Publishing, pp: 169-193.

Jaafer M. (2017). Numerical Modelling for Circle Tunnel Under Static and Dynamic Load for Different Depth. *Research Journal of Mining,* Vol. 1 Issue 1, pp: 1-11

Jin-long, L., Hamza, O., Davies-Vollum, S., & Jie-qun, L. (2018). Repairing a Shield Tunnel Damaged by Secondary Grouting. *Tunnelling and Underground Space Technology*, *80*, 313-321. https://doi.org/10.1016/j.tust.2018.07.016

Katuwal T. B. (2023). Selection of Tunnel Construction Methods. In Conference: NORHED II PROJECT 70141 6th Internal Conference at: IGP (NTNU), Trondheim, Norway. DOI:10.13140/RG.2.2.31837.59361/1

Lauffer H., (1988). Zur Gebirgsklassifizierung bei Fräsvortrieben. Felsbau, 6(3), pp :137–149.

Le, B. T., Nguyen, N. T., Divall, S., Goodey, R. & Taylor, R. N. (2021). Modified Gap Method for prediction of tunnelling-induced soil settlement in sand - a case study. In: Geotechnical Aspects of Underground Construction in Soft Ground. Abingdon, UK: Taylor & Francis. ISBN 9780429321559 doi: 10.1201/9780429321559

Look B.G., (2007). Handbook of Geotechnical Investigation and Design Tables. Ed. Taylors & Francis Group, London, UK.

MARTIN F. (2012). Mécanique des Roches et Travaux Souterrains : Cours et exercices corrigés. Centre d'Études des Tunnels Adrien SAÏTTA. BG Ingénieurs Conseil. Polycopié, ENS Cachan 2012, p.88

NGI. Using the Q-System: Rock Mass Classification and Support Design. Handbook. Norwegian Geotechnical Institute. 2022.

Panet, M. « Le calcul des tunnels par la méthode Convergence-Confinement ». Edition Presse des Ponts et Chaussées 1995, p. 177

Prassetyo, S.H., Gutierrez, M. Hydro-mechanical response of excavating tunnel in deep saturated ground. *Geo-Engineering* **11**, 21 (2020). https://doi.org/10.1186/s40703-020-00128-x

Russo, A.; Hormazabal, E. Correlations Between Various Rock Mass Classification Systems, Including Laubscher (MRMR), Bieniawski (RMR), Barton (Q) and Hoek and Marinos (GSI) Systems. In Geotechnical Engineering in the XXI Century: Lessons Learned and Future Challenges; IOS Press: Amsterdam, The Netherlands, 2019.

Serafim, J.L. and Pereira, J.P. (1983). Considerations on the Geomechanical Classification of Bieniawski. Proceedings of International Symposium on Engineering Geology and Underground Openings, Lisbon, Portugal, 1983, 1133-1144.

Si shen (2022). Convergence-Confinement method (2). Available from : https://www.si-eng.org/post/convergence-confinement-method-2. Accessed on November, 17, 2025

Siad I., Akchiche M., and Spyridis P. (2023). “Effect of the Umbrella Arch Technique Modelled as aHomogenized Area Above a Cross Passage”. Appl. Sci.Vol, 13, 1588.doi.org/10.3390/ app13031588

Silva C., Pereira M., and E Sousa, L. R.“Safety Control of Old Railway Tunnels”. In ISRM Symposium on Rock Engineering for Mountainous Regions At: Funchal, Madeira Island, pp. 417-425

Singh, B., and Geol, R.K., (1999), Rock mass classification: a practical approach in civil engineering: Amsterdam, Elsevier Science, 282 p.

Sun, H., Song, S., Lu, W. et al. (2022). « Safety design of tunnel lining structure considering bond-slip failure mechanism. Arab J Geosci 15, 489 https://doi.org/10.1007/s12517-022-09832-7

Terzaghi, K., (1946). Rock defects and loads on tunnel supports, in Proctor, R.V., and White, T.L., eds., Rock tunneling with steel support, Vol.1: Youngstown, Ohio, Commercial Shearing and Stamping Company p. 17-99.

Verman, M. (2009). Interpretation of Tunnel Instrumentation Data”. IGC 2009, Guntur, INDIA. Available from: https://www.researchgate.net/publication/267684751

Wyllie, C. and Mah, W. (2004) Rock Slope Engineering Civil and Mining. In: Hoek, E. and Bray, J.W., Eds., Rock slope Engineering, Taylor & Francis Group, London and New York, 431 p.

Yano. Y.I. (2019). Creusement des Galeries. available from : https://www.researchgate.net/publication/337907248_CREUSEMENT_DES_GALERIES_Creusement_conventionnel_des_galeries

Zhang Y-J, Cao J and Xu H (2023). A design analysis method and its application in the cover arch of cut-and cover tunnels. *Front. Earth Sci*. 11:1092928.doi: 10.3389/feart.2023.1092928

Web sites:

https://www.kpstructures.in/2020/12/belgian-method-of-tunneling.html. Accessed on April, 28, 2025

https://www.encardio.com/blog/all-about-tunnel-boring-machine-components-types-advantages Accessed on April, 28, 2025

https://thinkwelty.com/how-does-a-tunnel-boring-machine-tbm-work/ accessed on May, 2, 2025

https://solidground.sandvik/fr/abattage-adaptable/ Accessed on May, 2, 2025

https://www.maccaferri.com/solutions/final-lining/

https://www.kamleshmetalalloy.com/blog/types-of-rock-bolting-used-in-tunneling/ Accessed on May, 4, 2025

https://www.stradeeautostrade.it/en/infrastructures-road-network/the-project-of-the-castello-tunnel-draco/ Accessed on May, 4, 2025

https://nebraskapublicmedia.org/en/news/news-articles/tunnel-repairs-show-vulnerability-of-old-infrastructure/). Accessed on May, 8, 2025

https://www.brown.edu/Departments/Engineering/Courses/En1750/Notes/FEA_Intro/FEA_Intro.htm. Accessed on June, 4, 2025

https://tunnel.ita-aites.org/en/component/seoglossary/1-main-glossary. Accessed on June, 7, 2025

https://regbar.com/products/rebar-couplers/fastcoup/. Accessed on June, 12, 2025

https://www.forconstructionpros.com/concrete/article/12055242/hydrodemolition-method-repairs-damaged-concrete-at-working-rail-tunnel. Accessed on June, 12, 2025

https://civildigital.com/monolithic-dome-construction-advantages-disadvantages-materials-profiles-ppt/application-of-shotcrete/. Accessed on June, 12, 2025

https://civilenggblitz.com/pressure-grouting-for-dams-and-tunnels/. Accessed on June, 13, 2025

https://www.slideserve.com/afaucette/tunnelling-powerpoint-ppt-presentation. Accessed on June, 13, 2025

https://www.engineeringtoolbox.com/underground-pipe-pressure-soil-transport-d_2145.html. Accessed on June, 21, 2025

https://mohua.gov.in/upload/uploadfiles/files/Part-A-Chapter-11-PIPES-AND-PIPE-APPURTENANCES.pdf. Accessed on June, 23, 2025

Majority of the glossary is taken from: international tunneling and underground space association:

https://tunnel.ita-aites.org/en/component/seoglossary/1-main-glossary/B