

PDHonline Course C973 (8 PDH)

Tunnel Basics

2026

PDH Online • PDH Central

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Course Description

Through mountains, under rivers, beneath cities... tunnels are no boring story! In this course, we will take you underground to explore the history and ingenuity behind some of the world's tunneling marvels. Besides the challenges of tunneling through various types of soil/rock, we will discuss in detail the three steps to a tunnel's success: excavation, support and lining. In addition, we will address how to reduce fire hazards during tunnel operations. Course highlights include the Hoosac Tunnel, London Underground, Holland Tunnel, Channel Tunnel (Chunnel), Marmaray Tunnel, and Gotthard Base Tunnel.

This course includes a multiple-choice quiz at the end, which is designed to enhance the understanding of course materials.

Learning Objectives

At the conclusion of this course, students will:

- Know the history of tunnels dating back to the Roman Empire;
- Understand the challenges of tunneling through various types of soil/rock;
- Learn the stories behind some of the most significant tunnel projects in the world; and
- Get familiar with the fundamentals of tunnel design and construction techniques.

Intended Audience

This course is designed for architects, engineers, geologists, land surveyors, contractors or anyone who would like to learn about the tunnel basics.

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Table of Contents

Introduction 4
Tunnel Basics 4
Choice of Tunnels versus Bridges 5
Types of Tunnels 7
Tunnel Planning 7
Types of Ground 10
Tunnel Excavation 10
Cohesionless Granular Soils 11
Cohesive Granular Soils 12
Nonswelling Stiff to Hard Clays 12
Soft to Stiff Saturated Clays 13
Tunnel Shapes 13
Tunneling Terms 14
The Tunnel Shield 14
Tunnel Construction: Soft Ground and Hard Rock 18
Tunnel Construction: Soft Rock and Underwater Tunnel Excavation by Mechanical Means 20
Initial Ground Support 25
Sequential Excavation and Support 41
Portal Construction 45
Shaft Construction 48
Options for Ground Improvement 50
Drainage and Control of Groundwater 54
Construction of Final, Permanent Tunnel Linings 56
Ventilation of Tunnels and Shafts 61
Fire Hazards in Tunnels 63
The Future of Tunneling 65
List of Longest Tunnels 65

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Tunnel Basics

Introduction



At its most basic, a tunnel is a tube hollowed through soil or stone. Constructing a tunnel, however, is one of the most complex challenges in the field of civil engineering. Many tunnels are considered technological masterpieces and governments have honored tunnel engineers as heroes.

In the UK, a pedestrian, cycle or animal tunnel beneath a road or railway is called a subway, while an underground railway system is differently named in different cities, the "Underground" or the "Tube" in London, the "Subway" in Glasgow, and the "Metro" in Newcastle. The place where a road, railway, canal or watercourse passes under a footpath, cycleway, or another road or railway is most commonly called a bridge or, if passing under a canal, an aqueduct. Where it is important to stress that it is passing underneath, it may be called an underpass, though the official term when passing under a railway is an underbridge. A longer underpass containing a road, canal or railway is normally called a "tunnel", whether or not it passes under another item of infrastructure. An underpass of any length under a river is also usually called a "tunnel", whatever mode of transport it is for.

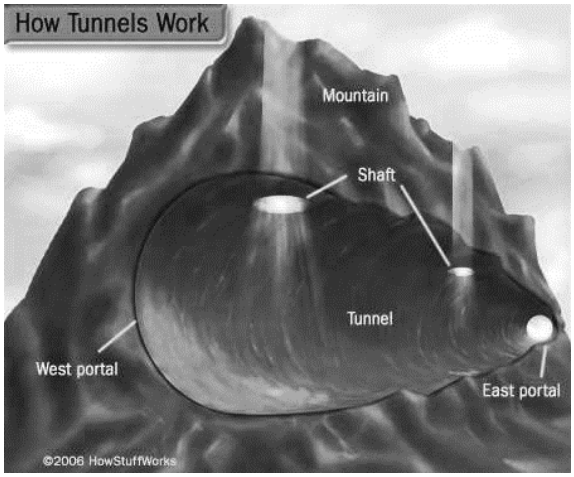
In the US, the term "subway" means an underground rapid transit system, and the term pedestrian underpass is used for a passage beneath a barrier. Rail station platforms may be connected by pedestrian tunnels or footbridges.

Tunnel Basics

A tunnel is a horizontal passageway located underground. While erosion and other forces of nature can form tunnels, this course covers man-made tunnels, tunnels created by the process of excavation. There are many different ways to excavate a tunnel, including manual labor, explosives, rapid heating and cooling, tunneling machinery or a combination of these methods. Some structures may require excavation similar to tunnel excavation, but are not actually tunnels. Shafts, for example, are often hand-dug or dug with boring equipment. But unlike tunnels, shafts are vertical and shorter. Often, shafts are built either as part of a tunnel project to analyze the rock or soil, or in tunnel construction to provide headings, or locations, from which a tunnel can be excavated.

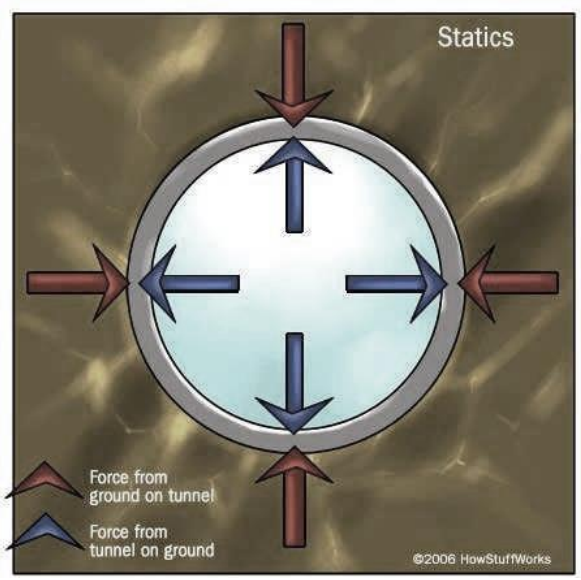
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The diagram below shows the relationship between these underground structures in a typical mountain tunnel. The opening of the tunnel is a portal. The "roof" of the tunnel, or the top half of the tube, is the crown. The bottom half is the invert. The basic geometry of the tunnel is a continuous arch. Because tunnels must withstand tremendous pressure from all sides, the arch is an ideal shape. In the case of a tunnel, the arch simply goes all the way around.



Tunnel engineers, like bridge engineers, must be concerned with an area of physics known as statics. Statics describes how the following forces interact to produce equilibrium on structures such as tunnels and bridges:

- Tension, which expands, or pulls on, material
- Compression, which shortens, or squeezes material
- Shearing, which causes parts of a material to slide past one another in opposite directions
- Torsion, which twists a material



The tunnel must oppose these forces with strong materials, such as masonry, steel, iron and concrete.

In order to remain static, tunnels must be able to withstand the loads placed on them. Dead load refers to the weight of the structure itself, while live load refers to the weight of the vehicles and people that move through the tunnel.

To build a stable and functioning tunnel, engineers and contractors need to follow three basic steps:

1. Excavation: dig through the earth with a reliable tool or technique.
2. Support: support any unstable ground within tunnel while digging.
3. Lining: install lining, construct roadway, add ventilation, lighting and other monitoring equipment.

Choice of Tunnels versus Bridges

For water crossings, a tunnel is generally more costly to construct than a bridge. However,

navigational considerations may limit the use of high bridges or drawbridge spans intersecting with shipping channels, necessitating a tunnel.

Bridges usually require a larger footprint on each shore than tunnels. In areas with expensive real estate, such as Manhattan and urban Hong Kong, this is a strong factor in favor of a tunnel. Boston's Big Dig project replaced elevated roadways with a tunnel system to increase traffic capacity, hide traffic, reclaim land, redecorate, and reunite the city with the waterfront.

The 1934 Queensway Tunnel under the River Mersey at Liverpool was chosen over a massively high bridge for defense reasons; it was feared that aircraft could destroy a bridge in times of war. Maintenance costs of a massive bridge to allow the world's largest ships to navigate under were considered higher than for a tunnel. Similar conclusions were reached for the 1971 Kingsway Tunnel under the Mersey. In Hampton Roads, Virginia, tunnels were chosen over bridges for strategic considerations; in the event of damage, bridges might prevent US Navy vessels from leaving Naval Station Norfolk.

Water-crossing tunnels built instead of bridges include the Holland Tunnel and Lincoln Tunnel between New Jersey and Manhattan in New York City; the Queens-Midtown Tunnel between Manhattan and the borough of Queens on Long Island; the Detroit-Windsor Tunnel between Michigan and Ontario; and the Elizabeth River tunnels between Norfolk and Portsmouth, Virginia.

Other reasons for choosing a tunnel instead of a bridge include avoiding difficulties with tides, weather, and shipping during construction (as in the 51.5-kilometre or 32.0-mile Channel Tunnel), aesthetic reasons (preserving the above-ground view, landscape, and scenery), and also for weight capacity reasons (it may be more feasible to build a tunnel than a sufficiently strong bridge).

Some water crossings are a mixture of bridges and tunnels, such as the Denmark to Sweden link, the Chesapeake Bay Bridge-Tunnel in Virginia, and the Hong Kong–Zhuhai–Macau Bridge-Tunnel link (55-kilometre or 34 mile) in China.



Immersed Tunnel of the Hong Kong-Zhuhai-Macau Bridge

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Types of Tunnels

There are three broad categories of tunnels: mining, public works and transportation. From geological point of view, there are also three types of tunnels: soft-ground tunnels, rock tunnels, and underwater tunnels.



A coal miner standing on the back of a car in a mine tunnel in the early 1900s. Notice that the sides of the tunnel are shored up with timber.

Mine tunnels are used during ore extraction, enabling laborer's or equipment to access mineral and metal deposits deep inside the earth. These tunnels are made using similar techniques as other types of tunnels, but they cost less to build. Mine tunnels are not as safe as tunnels designed for permanent occupation, however.

Public works tunnels carry water, sewage or gas lines across great distances. The earliest tunnels were used to transport water to, and sewage away from, heavily populated regions. Roman engineers used an extensive network of tunnels to help carry water from mountain springs to cities and villages. These tunnels were part of aqueduct systems, which also comprised underground chambers and sloping bridge-like structures supported by a series of arches.

Before there were trains and cars, there was transportation tunnels such as canals -- artificial waterways used for travel, shipping or irrigation. Just like railways and roadways today, canals usually ran above ground, but many required tunnels to pass efficiently through an obstacle, such as a mountain. Canal construction inspired some of the world's earliest tunnels.



A Roman aqueduct that runs from the Pools of Solomon to Jerusalem

By the 20th century, trains and cars had replaced canals as the primary form of transportation, leading to the construction of bigger, longer tunnels.

Tunnel Planning

Almost every tunnel is a solution to a specific challenge or problem. In many cases, that challenge is an obstacle that a roadway or railway must bypass. They might be bodies of water, mountains

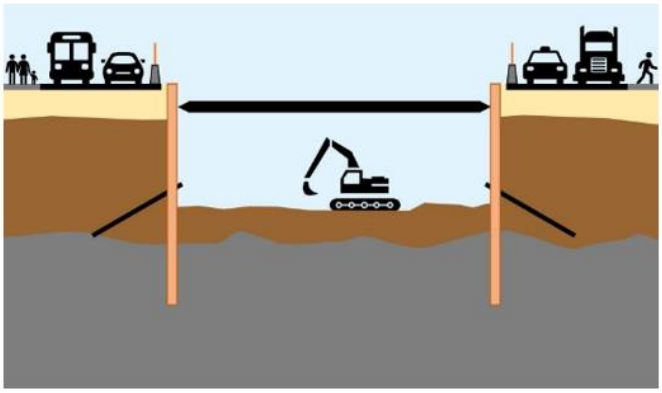
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or other transportation routes. Even cities, with little open space available for new construction, can be an obstacle that engineers must tunnel beneath to avoid.

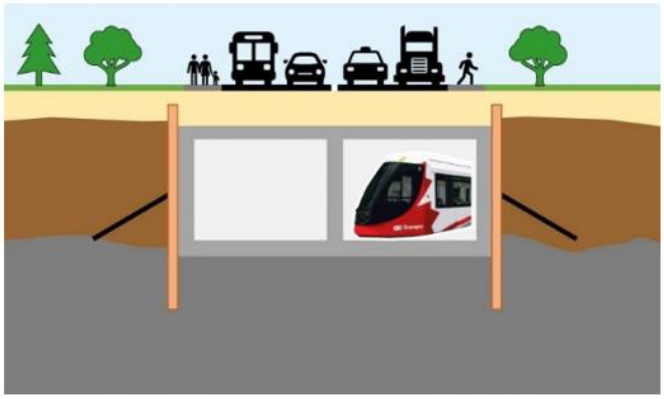
How a tunnel is built depends heavily on the material through which it must pass. Tunneling through soft ground, for instance, requires very different techniques than tunneling through hard rock or soft rock, such as shale, chalk or sandstone. Tunneling underwater, the most challenging of all environments, demands a unique approach that would be impossible or impractical to implement above ground. Early engineers used pressurized excavation chambers to prevent water from gushing into tunnels. Today, tunnel engineers are using a much faster method, called the immersed tube technique, to construct tunnels under water.

There are three basic types of tunnel construction in common use:

1. **Cut-and-cover tunnel**, constructed in a shallow trench and then covered over. Cut and cover tunneling is one of the oldest methods of constructing a tunnel. The basic concept involves digging a trench, constructing the tunnel walls and roof, and then backfilling it at the surface. Each cut and cover tunnel section will be constructed using the following steps:



Cut-and-Cover Tunnel



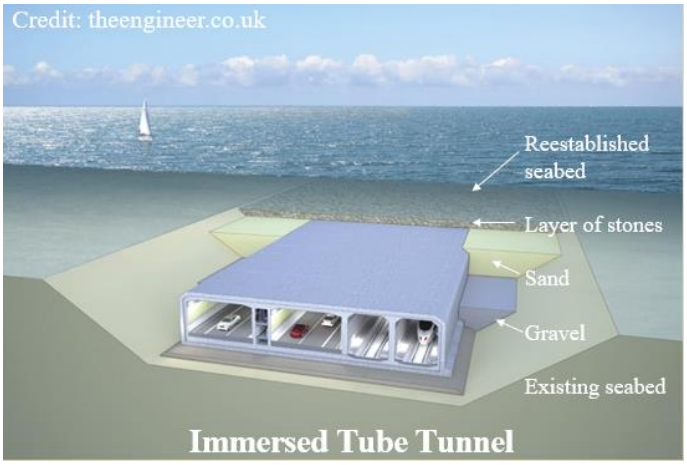
Credit: baywardbulletin.ca

   Step 1 – Prepare: Relocate utilities, walkways, and traffic as needed.
   Step 2 – Support: Support walls are installed for reinforcement during tunnel excavation.
   Step 3 – Cut: Excavate the tunnel trench.
   Step 4 – Construct: Tunnel foundation, walls, and roof are constructed using steel struts and concrete walls.
   Step 5 – Cover: Backfill and reinstate surface.

2. **Bored tunnel**, constructed in situ, without removing the ground above. They are usually of circular or horseshoe cross-section. Early engineers used a support structure, called a tunnel shield, to bore through soft ground. Modern techniques include Shotcrete used in the New Austrian tunneling method (NATM), and use of a tunnel boring machine (TBM).



3. **Immersed tube tunnel**, sunk into a body of water and laid on or buried just under its bed. Immersed tube tunnels are composed of prefabricated sections placed in trenches that have been dredged in river or sea bottoms. The sections are usually constructed at some distance from the tunnel location and made watertight with temporary bulkheads. They are then floated into position over the trench, lowered into place, and joined together underwater. The temporary bulkheads are removed, and the trench is backfilled with earth, gravel and stone to protect the tubes from damage caused by passing ships.



Before selecting the type of tunnel construction, engineers conduct a thorough geologic analysis to determine the type of material they will be tunneling through and assess the relative risks of different locations. They consider many factors, but some of the most important include:

- Soil and rock types
- Weak beds and zones, including faults and shear zones
- Groundwater, including flow pattern and pressure
- Special hazards, such as heat, gas and fault lines

Often, a single tunnel will pass through more than one type of material or encounter multiple hazards. Good planning allows engineers to plan for these variations right from the beginning, decreasing the likelihood of an unexpected delay in the middle of the project.

Once engineers have analyzed the material that the tunnel will pass through and have developed an overall excavation plan, construction can begin. The tunnel engineers' term for building a

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tunnel is driving, and advancing the passageway can be a long, tedious process that requires blasting, boring and digging by hand.

Types of Ground

The type and extent of the soil mass is critical in the determination of the conditions and techniques of tunneling to be applied. The variability and heterogeneity of the soil deposits must be accurately and as thoroughly uncovered. It is the recognition of these differences and probable variations in ground behavior to be encountered that directs the design and subsequent construction. A site investigation must include an understanding of the geologic history and forces that have acted on the area.

Soils are generally formed from some form of degradation process, either mechanical/physical or chemical in nature. Most soils are derived from the degradation of rock materials or decomposition of organic material. Residual soils remain in place where they are formed. Soil transported from where it is formed, is called transported soils. During the course of geologic time, transport, deposition, and weathering may continue to occur.

Tunnel Excavation

a. Stand-up Time. Stand-up time is the amount of time a newly excavated cavity can support itself without any added structures. Knowing this parameter allows the engineers to determine how far an excavation can proceed before support is needed, which in turn affects the speed, efficiency, and cost of construction. Generally, certain configurations of rock and clay will have the greatest stand-up time, while sand and fine soils will have a much lower stand-up time.

b. Excavation Problem. Soft ground tunnels may be excavated by a variety of means, either mechanized or by hand:

(1) Hand mining - clay spades, knives, and shovels
(2) Shield - open face, closed face, with or without breasting tables or boards, forepoles
(3) Tunnel boring machine (TBM)
(4) Earth pressure balance (EPB) or slurry face machine

Generally tunneling in soil is performed with the protection of a shield. Shields are provided to support the excavated perimeter and to, when necessary, support the tunnel face until the initial or initial/final support system can be erected.

The criteria for selecting the appropriate tunneling method and lining system are based on the properties and expected behavior of the ground and are based primarily on practical experience.

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A descriptive name is given to various ground conditions which correspond to the face stability and working conditions at the face for various soil types.

c. Stability of the Tunnel Face. Stability of the tunnel face is a function of many variables of which the more important appear to be:

(1) type of soil and variability,
(2) size and geometry of opening,
(3) existing hydrostatic condition,
(4) past and existing state of stress,
(5) Excavation method and support.

The stability of the unsupported tunnel excavation and the face of the excavation determine the methods and means of construction and generally dictate the time at which tunnel support must be applied. The construction of every soft ground tunnel is associated with some change in the state of stress in the ground surrounding the tunnel with corresponding induced strains and displacements. These induced strains and displacements are not necessarily bad for they allow the mobilization of the soils inherent strength to support the excavation. However, if they exceed the strength of the soil, they can result in excessive movements or failure of the soil itself if these movements are allowed to continue without the support of the opening, threatening the stability of the excavation allowing large movements of surrounding ground. In less extreme cases the instability of the face and sidewalls may manifest itself as cave-ins or as a slow creep and plastic flow into the excavation.

The stability of the face (stand-up time) may be examined in terms of four principal groupings of soil; granular soils with little or no actual cohesion, cohesive granular soils, nonswelling stiff to hard clays, and stiff to soft saturated clays.

Cohesionless Granular Soils

The stability of a tunnel face in cohesionless materials such as uncemented sand, silts, and gravels is essentially controlled by the groundwater conditions and effects of the construction method used. Excavations in this material can be carried out only by providing complete protection to the face and excavated parameter of the tunnel. Above the groundwater table these soils will not generally stand unsupported but will ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the soil material in such a loose state. In many instances granular soils above the moisture table contain enough soil moisture to create a small apparent cohesion which may be sufficient to allow erection of an initial support system if the erection time is small enough to prevent drying and the mechanically induced vibration of the construction process from destroying this effect, otherwise full breasting or forepolling may be required to support the

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tunneled face. Failure to do so will allow runs to develop.

Runs may also develop into cavities and unfilled voids outside the initial/final support system which remain open temporarily but may collapse following excavation and lead to surface subsidence at a later time. Unless groundwater is adequately drained from ahead of such soil masses, even small seepage gradients may induce large ground movements or runs which completely invade the heading. The control of groundwater then becomes paramount. Dewatering may be applied to drain the soil; however, the general stratified and lenticular nature of most soil deposits makes the complete drainage of all zones unlikely allowing imperfect drainage of others. Coarser and more permeable zones will be well drained and tunneling advance satisfactory until a poorly drained area is encountered, and a run may develop. Fine grained soils may also be easily transported through even the smallest of cracks in lagging or initial support systems or poorly cast joints in the final lining for the smallest of flows. Loss of such ground around the lining system once erected may be result in a run which may deprive the support system of the necessary restraining ground reaction, inducing failure of the lining system.

Cohesive Granular Soils

Soils of this category include soil types ranging from clayey sands to sandy clays to cohesive silts. Residual soils and hydrochemically altered soils possessing cohesive bonds or cementation may also be included. In most instances these soils behave admirably with sufficient stand-up time to allow support of the excavation. Losses of ground are typically associated with the infilling of the annular space behind the tunneling shield once the support system is erected and emerges from the shield. Raveling may develop if support is not provided or installed in sufficient time. The use of a shield is considered a prudent precautionary measure to forestall such problems. Where stand-up time is long enough to allow erection of an initial support system behind the shield, expansion or immediate grout injections may be beneficial to minimize the infilling of the annular space.

Raveling of such ground may, if allowed to begin, continue to persist and where these soils exhibit sensitivity to adverse seepage gradients, they must be predrained in advance of tunneling to minimize catastrophic ground losses. Ground losses due to travelling may be further troublesome because settlements so induced may be delayed for years as overlaying soils ravel or slough slowly into the voids created during tunneling.

Nonswelling Stiff to Hard Clays

These grounds have the desirable properties of those in the preceding category and are not likely to be adversely influenced by seepage gradients towards the tunnel face or subjected to raveling.

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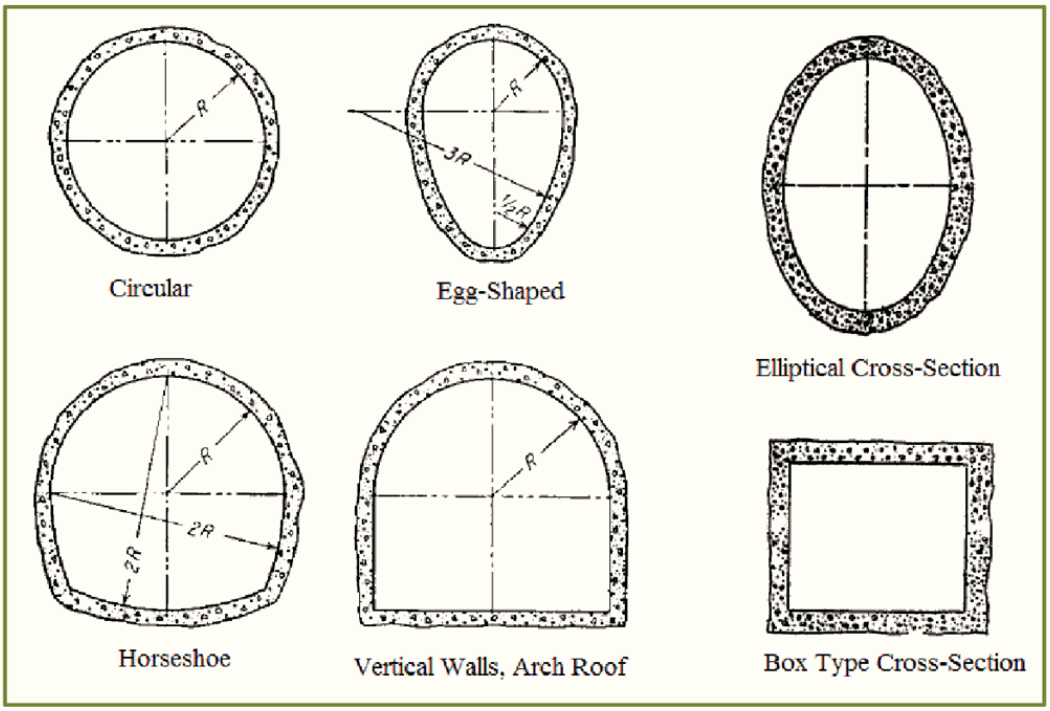
Soft to Stiff Saturated Clays

These soils are characterized by undrained shear strengths ranging from 1.4 to 14 Mpa (203 to 2,030 psi) and comprise a relatively large number of naturally occurring clay strata generally found at shallower depths and which are also generally impervious. Movement of the ground during tunneling occurs as a longitudinal or inward movement into the shield without any visible signs of distress or soil raveling. This process continues for the duration of the jacking and excavation process itself and generally seems to diminish once these activities are stopped. The minimization of ground movements may best be established by filling the annular void created during the tunneling process.

Tunnel Shapes

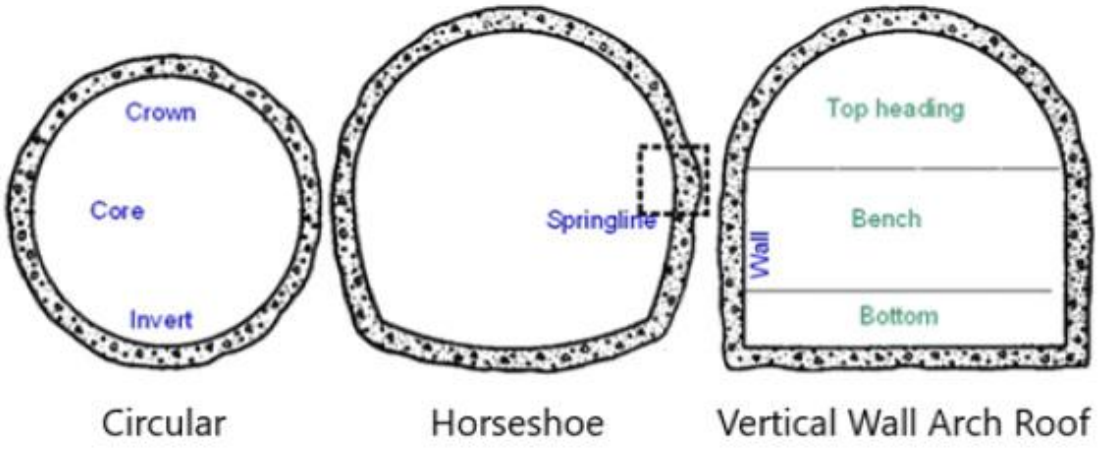
The shape of the cross section of a tunnel depends upon the pressure of the ground which the tunnel lining can resist and purpose for which the tunnel is constructed. There are six common tunnel shapes:

1. Circular
2. Egg Shaped
3. Elliptical
4. Horseshoe
5. Vertical walls with arch roof, and
6. Box section

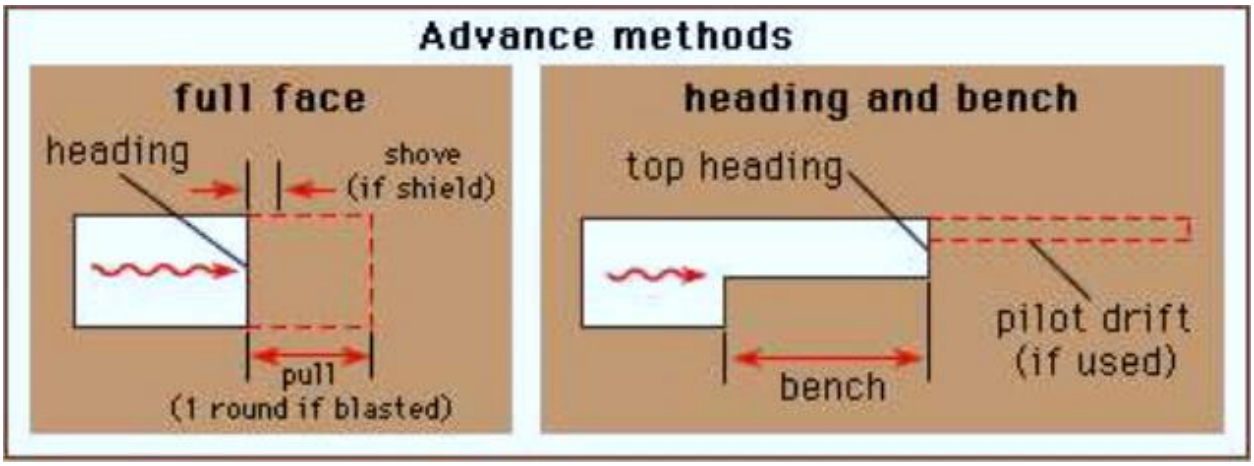


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Tunneling Terms



Crown: The uppermost part of the tunnel or the "roof" of the tunnel tube.
Drift: A horizontal excavation.
Heading: The excavated face of the tunnel.
Invert: The bottom (floor) of the tunnel.
Wall: The side of the tunnel.
Portal: The tunnel entrance.
Station: The distance measured from the portal (chainage)
Springline: The line at which the tunnel wall breaks from sloping outward to sloping inward toward the crown. Usually it is along the point where the curved portion of a tunnel roof meets the top of the wall.
Bottom, bench and top heading are related to excavations in multiple drifts (see diagrams below).



The Tunnel Shield

This device is often compared to a cookie cutter turned horizontally and propelled from the rear. The shield, besides providing a cutting head of the correct geometry, provides safety and a stable

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opening for the miner until the initial support system can be erected. In times past, tunneling was performed without the aid of a shield, using cribbing, heavy timbering, and lagging. Such tunneling proved both time consuming and cumbersome and all too often restricted the heading, limiting excavation and subsequent advance rate.

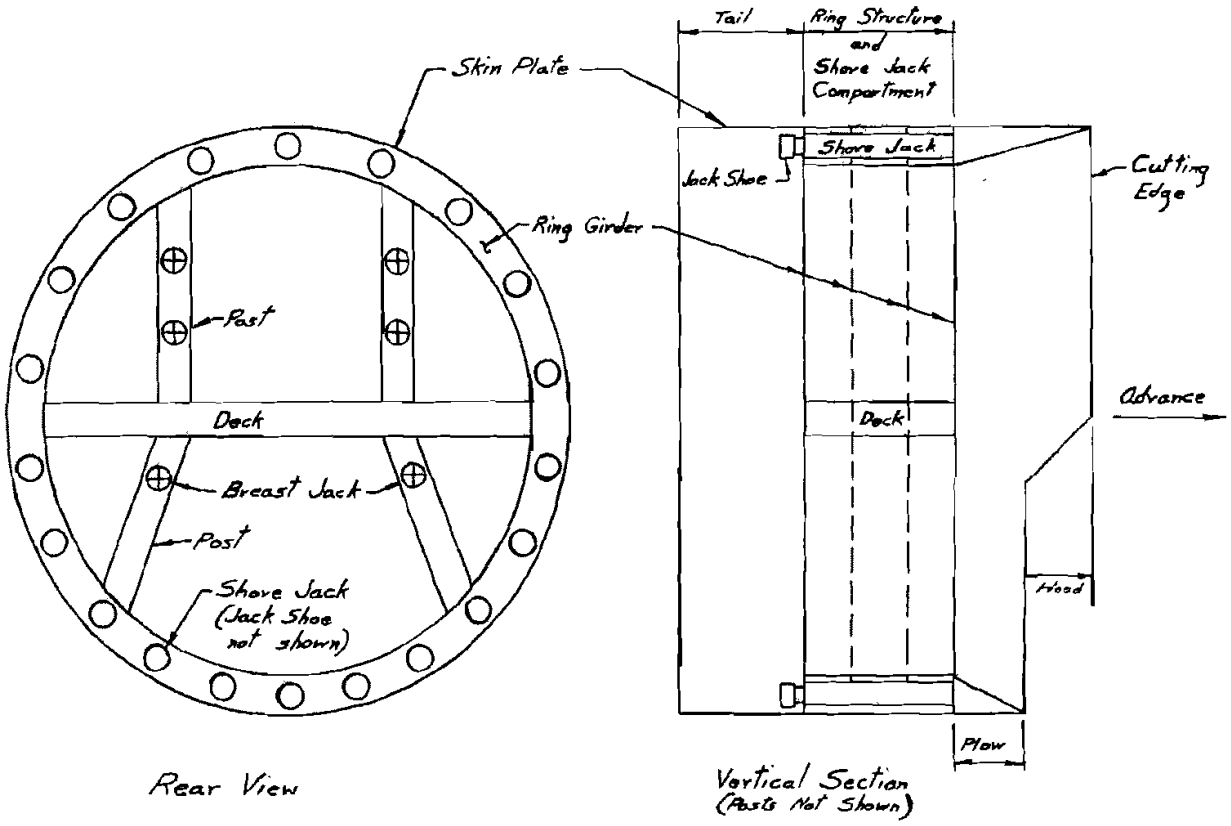


Figure 1: Typical Shield Section

The tunnel shield, to be used to the fullest advantage, must offer the support required in the most timely of manners and must be arranged so that there is sufficient access to the face in front of the shield, where the excavation takes place to allow support of the face while at the same time allowing forward progress. The tunnel shield consists essentially of a horizontal cylinder called the skin, the forward part of the shield is called the cutting edge, and an internal structure or structural frame with hydraulic jacks as a propulsion system and a rear section or hood.

The tunneling procedure or advance in shield tunneling is first to excavate a length ahead of the shield, the length of which is the width of one ring or initial support element length, then push the shield forward the distance excavated by means of the jacks reacting against the previously erected lining. Once extended the jacks are withdrawn within the shield to make room for the erection of the lining and finally to erect the lining in the clear space left in the tail of the shield. This process is repeated again and again in sequence.

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In order for the shield to be satisfactory it must:

(1) Support all soil loads either radially applied or at the face.
(2) Minimize soil disturbance and vibration.
(3) Be of the correct geometry *and* size for the tunnel section.
(4) Facilitate lining.
(5) Allow construction to line and grade and required tolerances.

In order for the shield to be effective and perform the functions intended it is imperative that the full range of tunneling conditions to be encountered are known and that the shield and tunneling technique chosen allows for means of coping with this range of conditions.

Detail of Shield Structure

i) *Annular Void* - The diameter of the skin plate is determined by the type of tunnel lining and required clearance to maintain steerage and control of the line and grade. This requires an annular space between the outside or the lining and the inside of the tail. This space facilitates the erection of the lining and allows deviation of the diameter of the axis of the shield from that of the lining. These erection clearances are generally a function of judgement and experience, however the allowance for deviation is a function of shield size, shield lead, and the lining element length. Generally the average value of the total clearance is about 0.8 percent of the outside diameter or in other words, it is the usual practice to make the inside diameter of the tunnel tail of the shield equal to approximately 1.008 times the outside diameter of the lining. The annular space then formed between the lining and excavation perimeter generally ranges from 2-5 percent of the mean excavation. The smaller the clearance, the smaller the subsequent over excavation and the size of the annular space. The use of an expanded lining system, which allows increase of tunnel lining size upon emergence from the shield, is used to advantage to minimize the size of the annular void.

ii) *Tail Length* - Length of the tail depends on the length of the initial support element width. The tail should be long enough to allow erection of one complete ring or unit and still provide overlap with the previously erecter unit. While this would generally favor the use of a long tail some instance 2 segments or units long) the tail is the weakest element of the shield and the longer its length, the greater the clearances required to provide steerage

iii) *Hoods* - The hood of the shield is a forward extension of the circumferential structure of the shield. The hood may be detachable, and in some cases extendable, composed of a series of poling plates, allowing a change in the slope of the shield hood. The slope of the hood should be adjusted to conform to the natural angle of repose of the ground to be encountered to minimize the need for supporting the face of the excavation. For many soil types, this angle of repose becomes large

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especially in granular soils and is impractical. Support of the face is then required.

iv) Shield Length - Length of the shield is the sum of the length of the hood and cutting edge, the tail and the internal structure length required to stow the jacks and hydraulics. The smaller the ratio of the length to the diameter the easier the shield is to steer. It is preferable to keep this ratio below 80 percent. In recent years, the advent of tunnel boring machines with mechanical excavators has made it almost impossible to keep machines within this acceptable range because of the size of the excavation equipment itself. In such cases the ratio of length to diameter may become larger; however, when this ratio approaches one, an articulated shield should be used. In such shields, the shield is provided with an articulated joint which enables the forward section of shield to be aligned independently from the rear section. This allows the shield to maintain horizontal and vertical alignment without extremely larger clearances.

v) *Face Control Methods* - The tunnel shield should be equipped to control the exposed face of the tunnel, especially if forward advance is to be accomplished with speed and with minimum losses of ground. This may be accomplished by a number of means including:

(1) Face jacks - which are jacks placed horizontally to the axis of the shield which can be extended forward to contact the face, usually used in conjunction with breasting boards. During a forward shove of the shield, the hydraulic pressure on the face jack is retained but the greater forces of the shove jacks forces them to close up. The shield operator then maintains a positive face pressure or support allowing enough force to prevent overstressing of the breasting members. Application of too little pressure will allow breasting members to fall.

(2) Breasting or sliding platforms usually consist of steel sliding platforms attached to forward extending face jacks. These platforms divide the face into a number of small vertical increments which for unstable soils with large angles of repose then allows a modest hood overhang to prevent inundation of the entire shield as well as providing protection for the workers on lower level.

(3) Mechanical doors have been used on modern tunnel boring machines, many consist of orange peel or segmental doors which rotate downward and outward from within the shield from positions around the inside of the cutting edge. While these doors do provide some support of the face, when fully deployed often allow forward movement of the shield only by relief of the doors which may allow soil movement into the center of the shield as the doors rotate. To be effective radial doors must be attached to move within the machine and along the axis of the shield as breast boards do.

vi) *Mechanical Excavators* - Mechanical excavators have been successfully installed on a number of tunneling machines. In some instances these excavations are large enough to be used to control

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localized raveling but in many instances these devices are too large and bulky to allow easy access to the face to control large scale travelling or running ground. Excavator size should be kept in proportion with the face size to allow access to the face. Provision should be made on any such machines to be able to mobilize additional face support including face jacks or tables.

vii) Shield Shove Jacks - Shield jacks are required to propel the shield forward once the erection of a completed lining element or ring has been completed. In order to move the shield forward a number of forces must be overcome, including: the friction of the ground on the shield's exterior surface, the friction of the lining in the tail of the shield and the resistance to the displacement of the ground in front of the shield. The required jacking resistance may vary depending on the amount of face control used and the nature of the ground. Generally the hydraulic system is designed to provide hydraulic capacity of varying degrees and to individual jacks as well as to ensure steerage and control of alignment. Working pressure generally of most hydraulic systems is about 24 MPa (3,500 psi) with a maximum hydraulic pressure of 34 MPa (5,000 psi). The jacks are placed in the shield in such a manner that their cylinders move forward with the shield while the piston rods or plungers remain stationary.

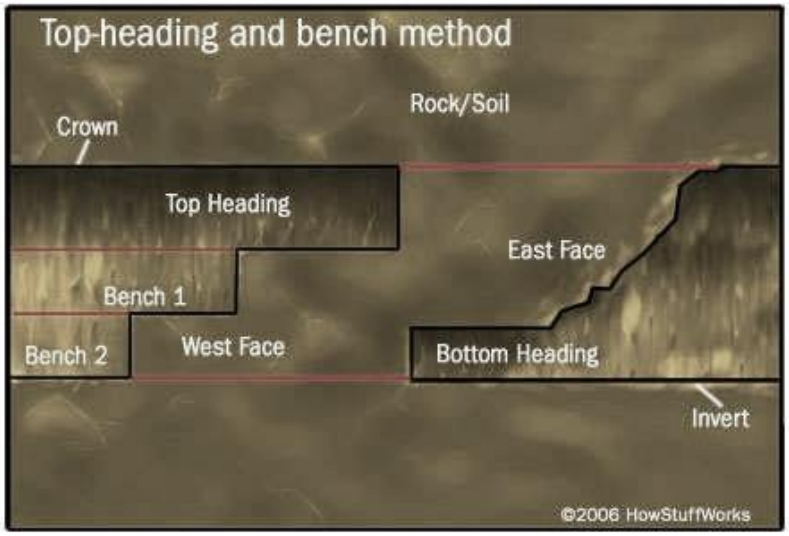
viii) Jacking Ring - In order to bear against the lining and minimize eccentric loadings the jacks are placed circumferentially as close to the skin as possible. Jacks may be uniformly spaced around the circumference but more commonly jacks are placed with more jacks placed below the horizontal diameter than above because of the natural tendency of the shield to dive and the need for greater force at the bottom than the top to forestall such rotation. Jack rams and heads should be equipped so that the induced jacking load is equally distributed over as large an area as possible of the tunnel lining. Special jacking shoes and jacking rings are often used to ensure equal pressure distribution. Often the induced jacking stresses are more detrimental and of larger magnitudes than the ground loadings.

Tunnel Construction: Soft Ground and Hard Rock

Workers generally use two basic techniques to advance a tunnel (see diagram on page 14 under Tunnel Terminology). In the full-face method, they excavate the entire diameter of the tunnel at the same time. This is most suitable for tunnels passing through strong ground or for building smaller tunnels. The second technique is the top-heading-and-bench method. In this technique, workers dig a smaller tunnel known as a heading. Once the top heading has advanced some distance into the rock, workers begin excavating immediately below the floor of the top heading; this is a bench. One advantage of the top-heading-and-bench method is that engineers can use the heading tunnel to gauge the stability of the rock before moving forward with the project.

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Notice that the diagram below shows tunneling taking place from both sides. Tunnels through mountains or underwater are usually worked from the two opposite ends, or faces, of the passage. In long tunnels, vertical shafts may be dug at intervals to excavate from more than two points.



Soft Ground (Earth)

Soft ground tunnels generally are defined as those in which the ground may be excavated or dug by conventional means. These means may include picks, shovels, spades, digger, or backhoes and similar earth excavating equipment. Generally, as opposed to rock tunnels, soft ground tunnels require more or less immediate support to maintain the opening. Soft ground tunneling in civil construction is performed at relatively shallow depths and is most often found in our most urban areas.

Hard Rock

Tunneling through hard rock almost always involves blasting. Workers use a scaffold, called a jumbo, to place explosives quickly and safely. The jumbo moves to the face of the tunnel, and drills mounted to the jumbo make several holes in the rock. The depth of the holes can vary depending on the type of rock, but a typical hole is about 10 feet deep and only a few inches in diameter. Next, workers pack explosives into the holes, evacuate the tunnel and detonate the charges. After vacuuming out the noxious fumes created during the explosion, workers can enter and begin carrying out the debris, known as muck, using carts. Then they repeat the process, which advances the tunnel slowly through the rock.

Fire-setting is an alternative to blasting. In this technique, the tunnel wall is heated with fire, and then cooled with water. The rapid expansion and contraction caused by the sudden temperature change causes large chunks of rock to break off. The Cloaca Maxima, one of Rome's oldest sewer tunnels, was built using this technique.

The stand-up time for solid, very hard rock may measure in centuries. In this environment, extra support for the tunnel roof and walls may not be required. However, most tunnels pass through rock that contains breaks or pockets of fractured rock, so engineers must add additional support in the form of bolts, sprayed concrete or rings of steel beams. In most cases, they add a permanent concrete lining.

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The objectives of any tunneling project, whether in soil or rock are fundamentally the same:

(1) Maintain a safe and stable opening to protect tunnel workers and minimize ground movements until an initial, initial/final lining is installed.
(2) Minimize tunneling effect on the surrounding or overlaying utilities and structures.
(3) Meet the user requirements.
(4) Remain relatively maintenance-free and operational for the useful service life of the structure - which for tunnels generally means 50 - 100's of years.
(5) Provide for economical/practical construction.

One or several of these often times dictate the final configuration, construction method, and/or techniques.

To maintain a safe and stable opening generally requires support both during the excavation process and thereafter. The support of the excavation immediately after excavation is termed initial support. This initial support system is generally erected within a temporary movable support system, a shield.

Tunnel Construction: Soft Rock and Underwater Tunnel Excavation by Mechanical Means

Much underground excavation today is performed by mechanical means. Tools for excavation range from excavators equipped with ripper teeth, hydraulic rams, and roadheaders to TBMs (Tunnel Boring Machine) of various designs. By far, TBMs are the most popular method of excavation. Roadheaders are versatile machines, useful in many instances where a TBM is not cost-effective. This section describes roadheader and TBM excavation methods and the factors that affect the selection of mechanical excavation methods.

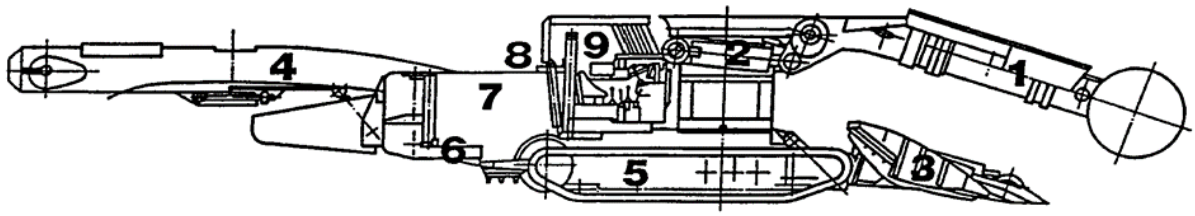
a. Roadheader excavation. Roadheaders come in many sizes and shapes, equipped for a variety of different purposes. They are used to excavate tunnels by the fullface or the partial-face method, and for excavation of small and large underground chambers. They may also be used for TBM starter tunnels, ancillary adits, shafts, and other underground openings of virtually any shape and size, depending on rock hardness limitations. Most roadheaders include the following components:

- Rotary cutterhead equipped with picks.
- Hydraulically operated boom that can place the cutterhead at a range of vertical locations.
- Turret permitting a range of horizontal motion of the cutterhead.
- Loading device, usually an apron equipped with gathering arms.
- Chain or belt conveyor to carry muck from the loading device to the rear of the machine for off-loading onto a muck car or other device.

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- Base frame, sometimes with outriggers or jacks for stabilization, furnished with electric and hydraulic controls of the devices and an operator's cab.
- Propelling device, usually a crawler track assembly.

A typical, large roadheader is shown on Figure 2.



1. Cutter Boom
2. Turret
3. Loading Device
4. Chain Conveyor
5. Crawler Track Assembly
6. Base Frame
7. Electrics
8. Hydraulics
9. Operator's Cab

Figure 2: Alpine Miner 100

(1) Several types and sizes of cutterheads exist. Some rotate in an axial direction, much like a dentists drill, and cut the rock by milling as the boom forces the cutterhead, first into the face of the tunnel, then slewing horizontally or in an arch across the face. Others rotate on an axis perpendicular to the boom. The cutterhead is symmetrical about the boom axis and cuts the rock as the boom moves up and down or sideways. The cutterhead is equipped with carbide-tipped picks. Large radial drag picks or forward attack picks are used, but the most common are the point attack picks that rotate in their housings. The spacing and arrangement of the picks on the cutterhead can be varied to suit the rock conditions and may be equipped with high-pressure water jets in front of or behind each pick, to cool the pick, improve cutting, remove cuttings, and suppress dust generation. Depending on the length of the boom and the limits of the slewing and elevating gear, the cutterhead can reach a face area of roughly rectangular or oval shape. The largest roadheaders can cut a face larger than 60 m^2 from one position. Booms can be extended to reach further, or can be articulated to excavate below the floor level, or may be mounted on different bodies for special purposes, such as for shaft excavation, where space is limited.

(2) Most roadheaders can cut rock with an unconfined compressive strength of 60 to 100 MPa (10,000-15,000 psi). The most powerful can cut rock with a strength of 150 MPa (22,000 psi) to 200 MPa (30,000 psi) for a limited duration. Generally, roadheaders cut most effectively into rocks of a strength less than 30 MPa, unless the rock mass is fractured and bedded. The cutting ability depends to a large measure on the pick force, which again depends on the torque

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available to turn the cutterhead, the cutterhead thrust, slewing, and elevating forces. The advance rate depends on the penetration per cut and the rotary speed of the cutterhead. The torque and speed of the cutterhead determines the power of the head. Cutting hard rock can be dynamic and cause vibrations and bouncing of the equipment, contributing to component wear; therefore, a heavy, sturdy machine is required for cutting hard rock. Typical small- to-medium roadheaders weigh about 20 to 80 tons and have available cutterhead power of 30 to 100 KW, total power about 80 to 650 KW. The larger machines weigh in excess of 90 tons, with cutterhead power of up to 225 KW. With a well- stabilized roadheader body, a cutterhead thrust of more than 50 tons can be obtained.

b. Excavation by tunnel boring machine. A TBM is a complex set of equipment assembled to excavate a tunnel. The TBM includes the cutterhead, with cutting tools and muck buckets; systems to supply power, cutterhead rotation, and thrust; a bracing system for the TBM during mining; equipment for ground support installation; shielding to protect workers; and a steering system. Back-up equipment systems provide muck transport, personnel and material conveyance, ventilation, and utilities.

(1) The advantages of using a TBM include the following:

- Higher advance rates.
- Continuous operations.
- Less rock damage.
- Less support requirements.
- Uniform muck characteristics.
- Less ground disturbance
- Greater worker safety.
- Potential for remote, automated operation.

(2) Disadvantages of a TMB are the fixed circular geometry, limited flexibility in response to extremes of geologic conditions, longer mobilization time, and higher capital costs.

c. TBM system design and operation. A TBM is a system that provides thrust, torque, rotational stability, muck transport, steering, ventilation, and ground support. In most cases, these functions can be accomplished continuously during each mining cycle. The TBM cutterhead is rotated and thrust into the rock surface, causing the cutting disc tools to penetrate and break the rock at the tunnel face. Reaction to applied thrust and torque forces may be developed by anchoring with braces (grippers) extended to the tunnel wall, friction between the cutterhead/shield and the tunnel walls, or bracing against support installed behind the TBM.

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d. TBM performance parameters.

(1) TBM system performance is evaluated using several parameters that must be defined clearly and used consistently for comparative applications.

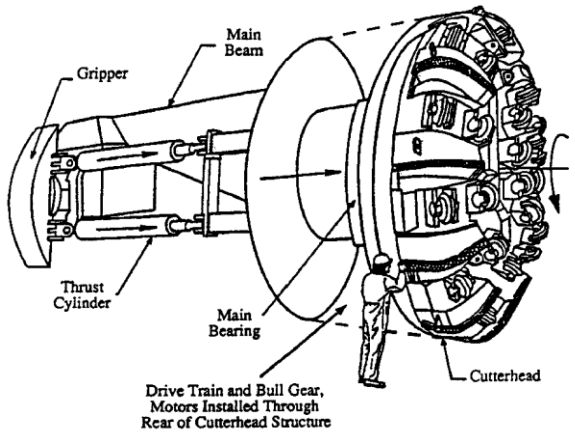


Figure 3: Unshielded TBM (schematic drawing)

(a) Shift time. Some contractors will use 24-hr shifting and maintain equipment as needed " on the fly." As used here, the shift time on a project is all working hours, including time set aside solely for maintenance purposes. All shift time on a project is therefore either mining time when the TBM operates or downtime when repairs and maintenance occur. Therefore,

Shift time = TBM mining time + Downtime

(b) Penetration rate. When the TBM is operating, a clock on the TBM will record all operating time. The TBM clock is activated by some minimum level of propel pressure and/or by a minimum torque and the start of cutterhead rotation. This operating time is used to calculate the penetration rate (PR), as a measure of the cutterhead advance per unit mining time. Therefore,

PR = distance mined/TBM mining time

PR is often calculated as an average hourly value over a specified basis of time (i.e., instantaneous, hour, shift, day, month, year, or the entire project), and the basis for calculation should be clearly defined. When averaged over an hour or a shift, PR values can be on the order of 2 to 10 m per hour. The PR can also be calculated on the basis of distance mined per cutterhead revolution and expressed as an instantaneous penetration or as averaged over each thrust cylinder cycle or other time period listed above. The particular case of penetration per cutterhead revolution is useful for the study of the mechanics of rock cutting and is here given the notation PRev (penetration per revolution). Typical values of PRev can be 2 to 15 mm per revolution.

(c) Utilization. The percentage of shift time during which mining occurs is the Utilization, U.

U (%) = TBM mining time/Shift Time x 100

and is usually evaluated as an average over a specified time period. It is particularly important that U is reported together with the basis for calculation— whole project (including start-up),

after start-up "production" average, or U over some other subset of the job. On a shift basis, U varies from nearly 100 percent to zero. When evaluated on a whole project basis, values of 35 to 50 percent are typical. There is no clear evidence that projects using a reconditioned machine have a lower U than projects completed with a new machine. Utilization depends more on rock quality, equipment condition, commitment to maintenance, contractor capabilities, project conditions (entry/access, alignment curves, surface space constraints on operations), and human factors (remoteness, underground temperature, and environment).

(d) *Advance rate (AR).* AR is defined on the basis of shift time as:

AR = Distance mined/Shift time

If U and PR are expressed on a common time basis, then the AR can be equated to:

AR = PR U (%)/100

Advance rate can be varied by changes in either PR (such as encountering very hard rock or reduced torque capacity when TBM drive motors fail) or in U changes (such as encountering very poor rock, unstable invert causing train derailments, or highly abrasive rock that results in fast cutter wear).

(e) *Cutting rate (CR).* CR is defined as the volume of intact rock excavated per unit TBM mining time. Again, the averaging time unit must be defined clearly, and typical values of CR range from 20 to 200 m^3 per TBM mining hour.

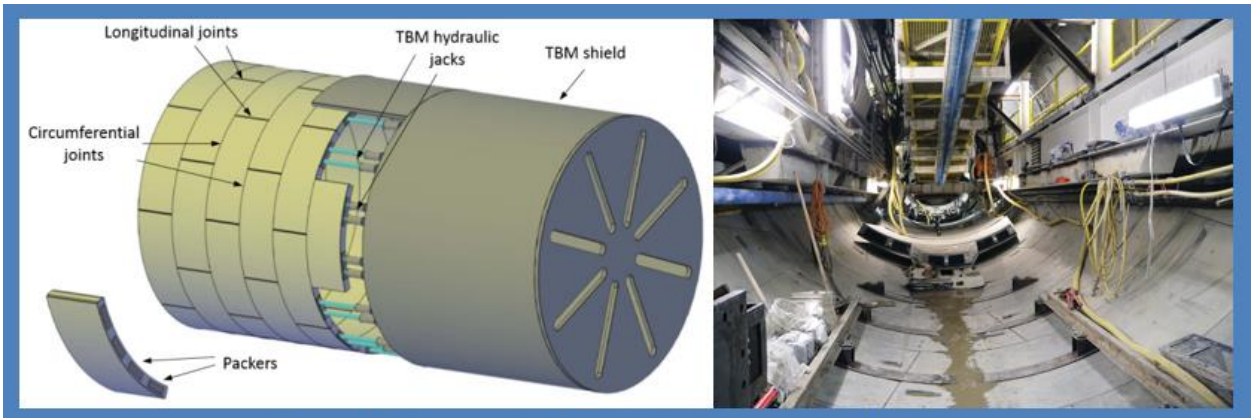
Experience indicates that tunnel depth has little impact on advance rates in civil projects, providing that the contractor has installed adequate mucking capacity for no-delay operation. Therefore, tunnel depth should be chosen primarily by location of good rock. Portal access, as opposed to shafts, will facilitate mucking and material supply, but more important is that the staging area for either shaft or portal be adequate for contractor staging. Confined surface space can have a severe impact on project schedule and costs. For long tunnels, intermediate access points can be considered for ventilation and mucking exits. However, assuming the contractor has made appropriate plans for the project, a lack of intermediate access may not have a significant impact on project schedule.

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Initial Ground Support

a. General

Initial ground support is usually installed concurrently with the excavation. For drill and blast excavations, initial ground support is usually installed after the round is shot and mucked out and before drilling, loading, and blasting of the next round. For TBM-driven tunnels, excavation is carried out more or less continuously, with the support installed as the TBM moves forward. Because of the close relationship between excavation and initial support activities, they must be well coordinated and should be devised such that the process is cyclic and routine. Initial ground support may consist of steel ribs, lattice girders, shotcrete, rock dowels, steel mesh, and mine straps. The main purposes served by these support elements include stabilizing and preserving the tunnel after excavation and providing work safety. As the quality of the rock increases, the amount of required initial ground support decreases. After installation of initial ground support, no other additional support may be required. In this case, the initial support will also fulfil the role of final support. In other cases, additional support, such as a cast-in-place concrete lining, may be installed. The initial and the final ground support then comprise a composite support system. An example of tunnel support fulfilling the initial and final support functions is when precast concrete segmental linings are used to support a tunnel in weak rock behind a TBM. One issue that must be considered when contemplating the use of initial support for final support is the longevity of the initial support components. While these components may behave satisfactorily in the short term, phenomena such as corrosion and deformation must be considered for permanent applications.



Tunnel Support - Concrete Segmental Lining

b. Initial ground reinforcement. Initial ground reinforcement consists of untensioned rock dowels and, occasionally, tensioned rock bolts. These are referred to as ground reinforcement, because their function is to help the rock mass support itself and mobilize the inherent strength of the rock as opposed to supporting the full load of the rock. It is much more economical to

reinforce the rock mass than to support it. The reinforcement elements are installed inside the rock mass and become part of the rock mass. Rock support such as concrete linings and steel sets restrict the movement of the rock mass and offer external support to the rock mass. There are three types of rock bolts:

- Mechanically anchored (rock bolt)
- Grouted bars (dowels)
- Friction dowels

Friction dowels are usually considered temporary reinforcement because their long-term corrosion resistance is uncertain.

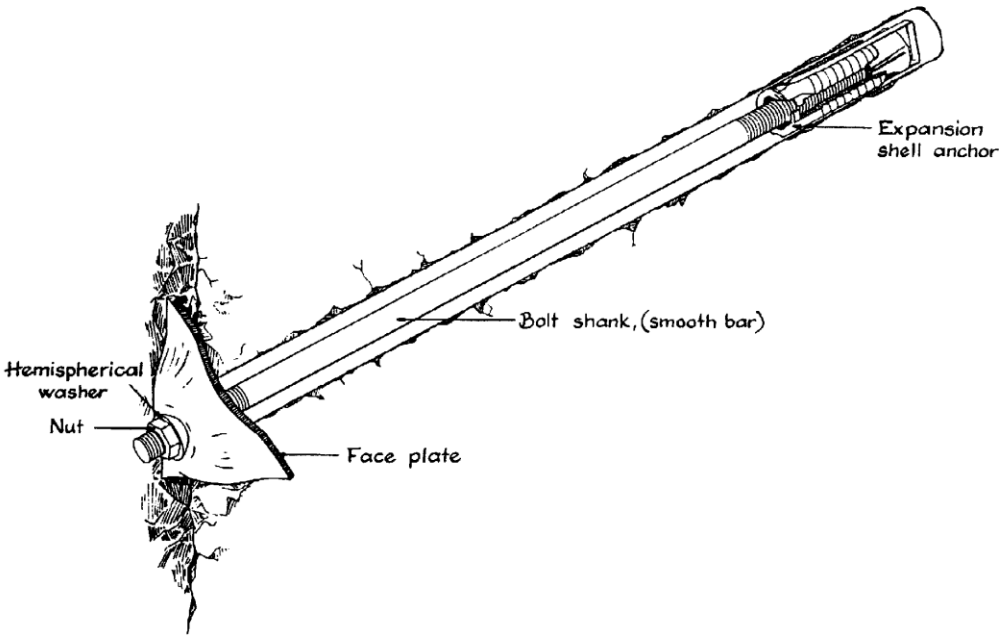


Figure 4: Mechanically anchored rock bolt – expansion shell anchor

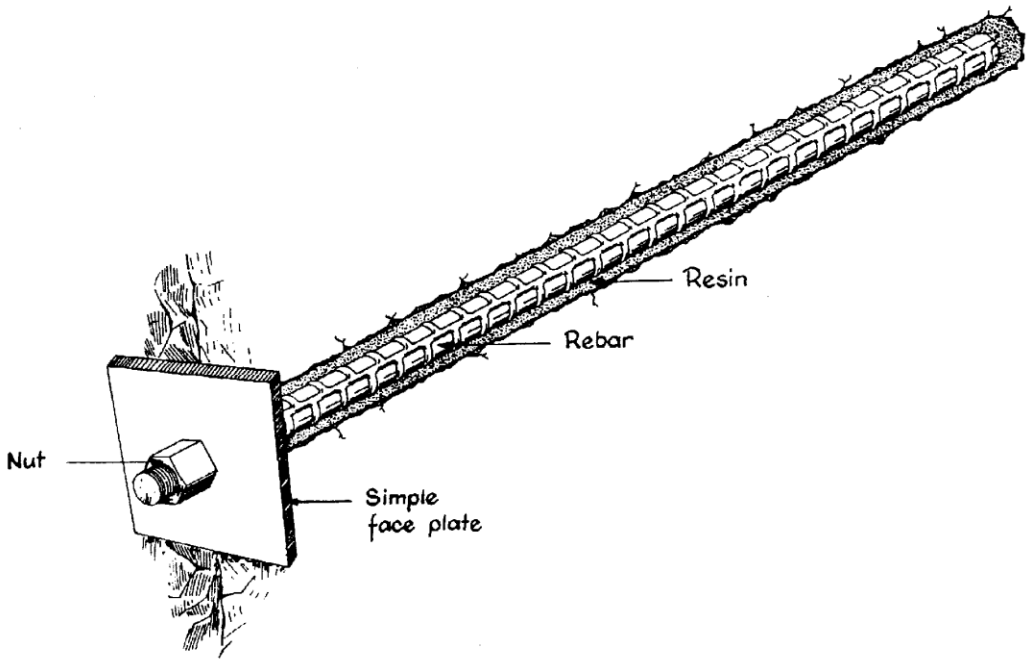


Figure 5: Grouted dowel - rebar

(1) Installation. To install a rock bolt or dowel, a borehole must be drilled into the rock of a specific diameter and length no matter what type of bolt or dowel is being used. This can be accomplished with a jack leg for small installations or a drill jumbo when high productivity is required. Special rock dowel installation gear is often used. In a blasted tunnel, the drill jumbo used for drilling the blast holes is frequently used to drill the rock bolt holes. Except for split sets, the diameter of the rock bolt hole can vary somewhat. It is common to have up to 10 or 20 percent variation in the hole diameter because of movement and vibration of the drill steel during drilling and variations in the rock. For expansion anchors and grouted and Swellex bolts, this is not a serious problem. Split sets are designed for a specific diameter hole, however; if the hole is larger, it will not have the required frictional resistance. Therefore, drilling of the hole for split sets must be closely controlled. After the hole is drilled, it should be cleaned out (usually with an air jet) and the bolt or dowel installed promptly. For mechanically anchored rock bolts, the bolt is preassembled, slid into the hole, and tightened with a torque wrench. The final tension in the bolt should be created by a direct-pull jack, not by a torque wrench. For resin-grouted rock dowels, the grout is placed in the hole using premade two-component cartridges; the bar is installed using a drill that turns the bar, breaks open the cartridges, and mixes the two components of the resin. The time and method of mixing recommended by the manufacturer should be used. Cement-grouted dowels can be installed the same way except that the grout is pumped into the hole through a tube in the center of the bar.

(2) Tensioning. Grouted bolts can be left untensioned after installation or can be tensioned using a torque wrench or a hydraulic jack after the grout has reached adequate strength. Fast-set

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resin grout can be used to hasten the process for resin- grouted bolts. Cement grout takes longer to cure even if an accelerator is used. Rock bolts in tunnels are usually left untensioned after installation and become tensioned as the rock mass adjusts to the changes in stress brought on by the process of excavation.

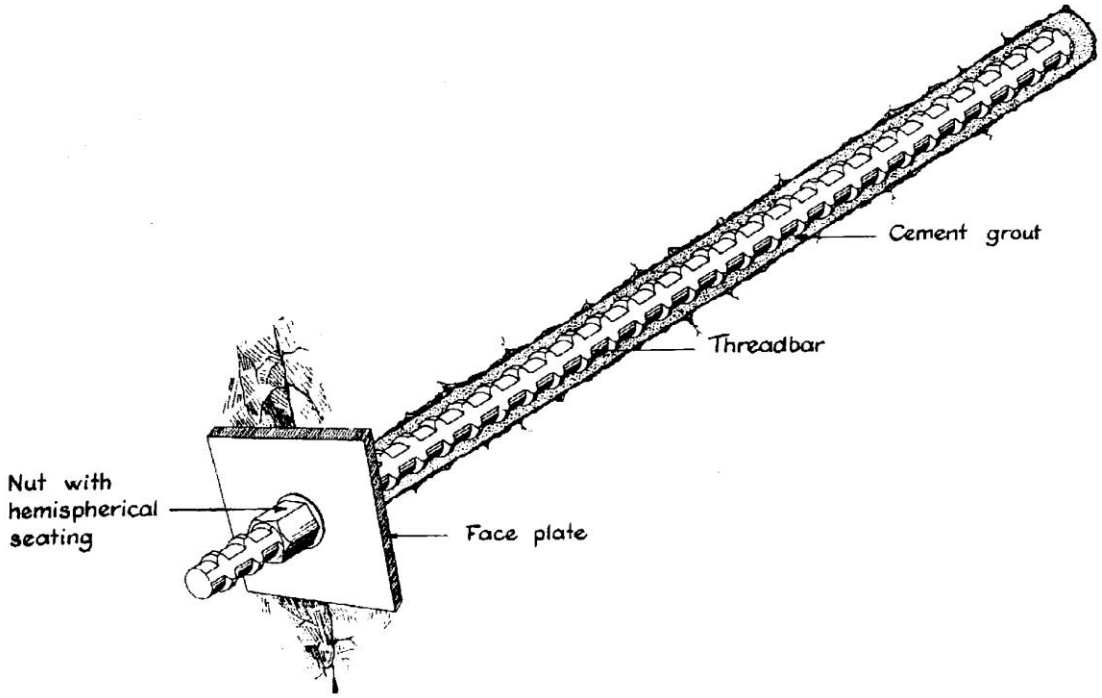


Figure 6 :Grouted dowel - Dywidage ® Steel

Split sets and Swellex bolts work this way since they cannot be pretensioned. There are cases when pretensioning the bolt is necessary, such as to increase the normal force across a joint along which a wedge or block can slip.

(3) Hardware. Rock bolts usually have end plates held in place with nuts and washers on the ends of bars or by enlargement of the head of split sets and Swellex bolts. End plates provide the reaction against the rock for tensioned bolts. End plates also are used to hold in place steel mesh and mine straps. They can also be embedded in shotcrete to provide an integral system of rock reinforcement and surface protection. End plates are generally square, round, or triangular shaped. Steel mesh, mine straps, and shotcrete are used to hold small pieces of rock in place between the rock bolts.

(4) Testing. Testing rock bolts is an important part of the construction process. If the rock bolts are not adequately installed, they will not perform the intended function. Possible reasons for faulty installations include the following:

- Incorrect selection of the rock bolt system.

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- Incorrect placement of borehole.
- Incorrect length of borehole.
- Incorrect diameter of borehole.
- Inadequate cleaning of borehole.
- Inadequate placement of grout.
- Inadequate bond length of grout.
- Corrosion or foreign material on steel.
- Misalignment of rock bolt nut and bearing plate assembly.
- Out-of-date grouting agents.
- Inappropriate grout mixture.
- Damage to breather tube.
- Inadequate borehole sealing.
- Inadequate lubrication of end hardware.
- Incorrect anchor installation procedure.
- Inadequate test program.
- No monitoring of rock bolt system performance.

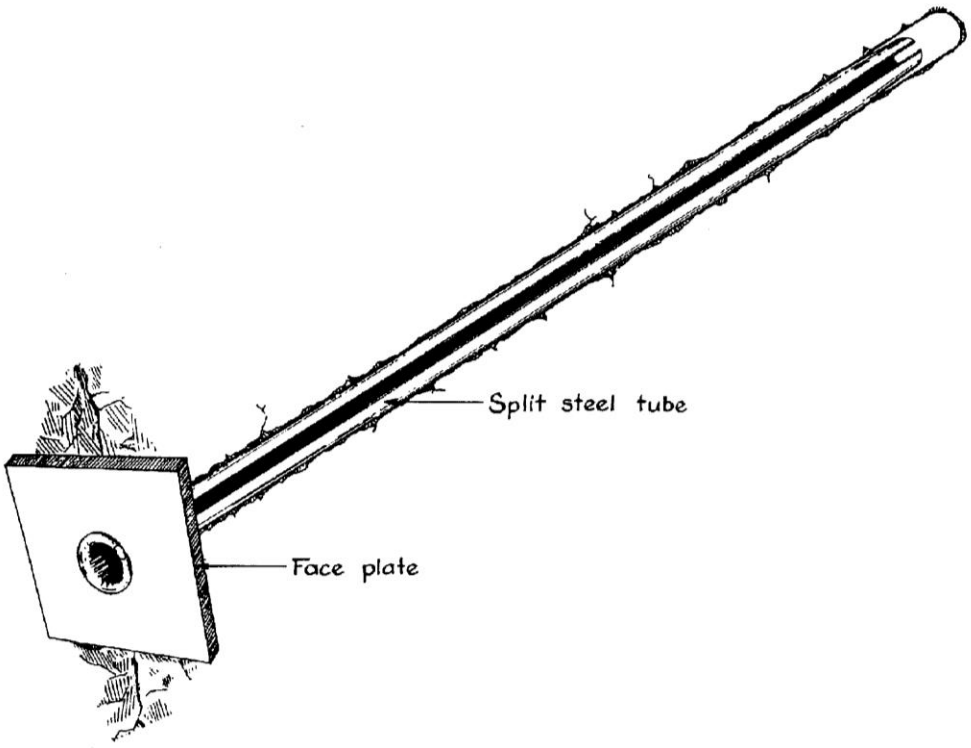


Figure 7 :Friction dowel - Split Set ®

Many of these problems can be avoided by adherence to manufacturer installation recommendations, and manufacturer representatives may be required to be onsite at the beginning of rock bolting operations to ensure conformance and trouble-shoot problems. The most common method of testing rock bolts or dowels is the pull-out test. A hydraulic jack is attached to the end of the rock bolt and is used to load the rock bolt to a predetermined tensile load and displacement.

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Rock bolts may be tested to failure or to a lesser value so that they can be left in place to perform their intended function. If the test load or displacement is exceeded, that rock bolt or dowel has failed and others in the area are tested to see if the failure is an isolated problem or indicative of a systematic problem related to all of the bolts or dowels. Usually, many units are tested at the beginning of tunneling, and once installation procedures, methods, and personnel skills are adequately confirmed, then a more moderate testing rate is adopted. If problems occur, changes are made, and a more rigorous testing scheme is reinstated until confidence is restored. Pull-out tests do not test the entire dowel. Only that length of the dowel that is required to resist the pull-out force is tested. For example, a dowel may be only partially grouted and still resist the pull-out force. These uncertainties are generally accepted in tunnel construction, and credence is placed on tunnel performance and pull-out test results. To further test the installation, the dowel can be overcored and exhumed from the rock for direct inspection. However, this requires costly special equipment and is only done under unusual circumstances. Other methods of testing include checking the tightness of a mechanically anchored rock bolt with a torque wrench, installing load cells on the end of tensioned rock bolts, and non-destructive testing by transmitting stress waves down through the bolt from the outer end and monitoring the stress wave return. The less stress wave reflection that is observed, the better the installation is. Swellex bolts can be tested using non-destructive techniques by reattaching the installation pump to the end of the bolt and testing to see that the tube still holds the same amount of pressure as when it was installed.

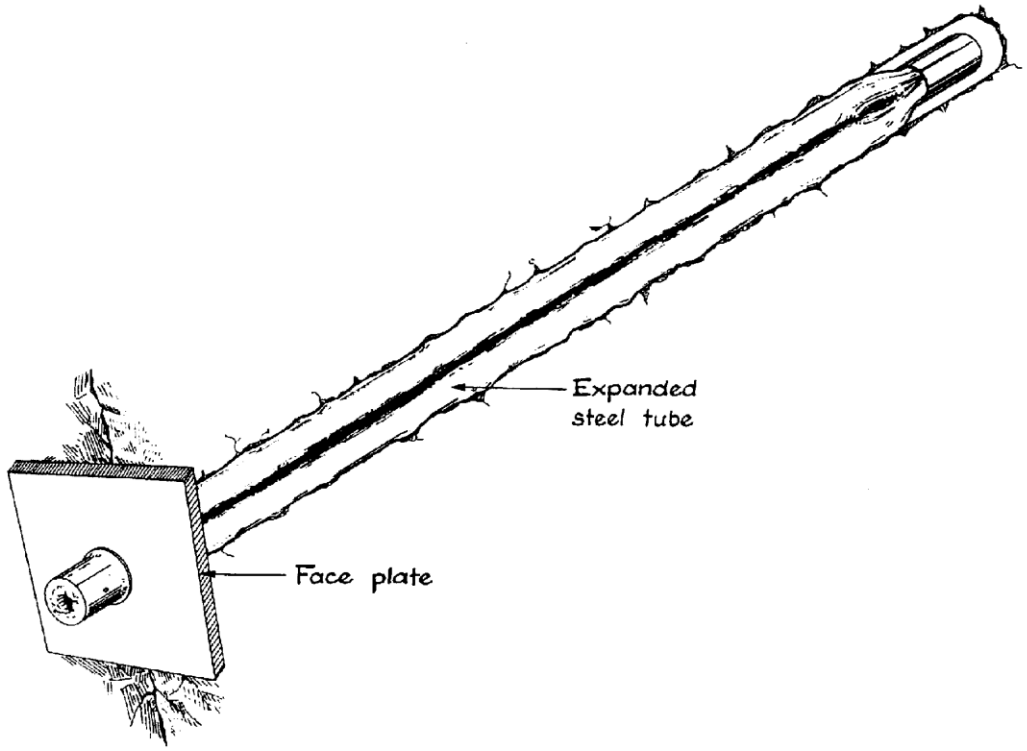


Figure 8 : Friction dowel - Swellex ®

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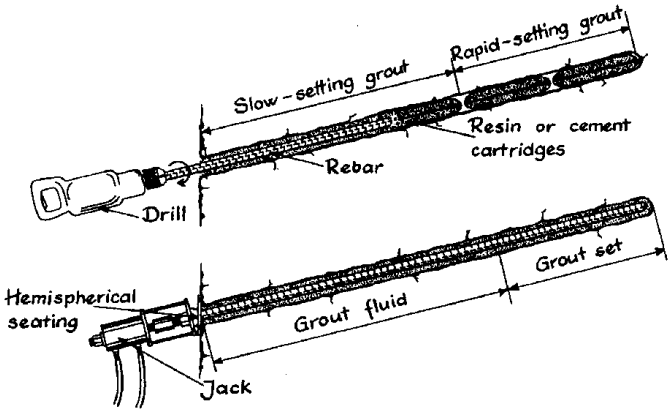


Figure 9: Tension resin dowel installation

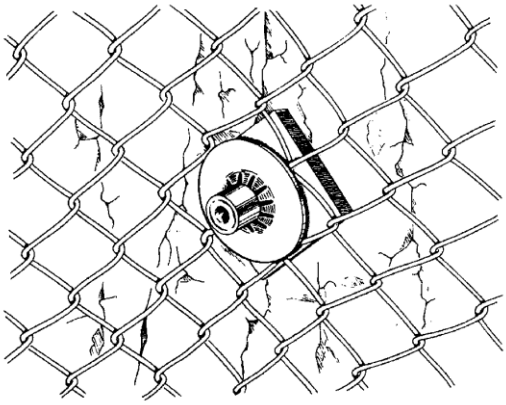


Figure 10: Mesh washer end hardware

c. Shotcrete application. Shotcrete today plays a vital role in most tunnel and shaft construction in rock because of its versatility, adaptability, and economy. Desirable characteristics of shotcrete include its ability to be applied immediately to freshly excavated rock surfaces and to complex shapes such as shaft and tunnel intersections, enlargements, crossovers, and bifurcations and the ability to have the applied thickness and mix formulation varied to suit variations in ground behavior. A brittle material by nature, shotcrete used for ground support often requires reinforcement to give it strain capacity in tension (i.e., ductility) and to give it toughness. Chain link mesh or welded wire fabric has long served as the method to reinforce shotcrete, but has now been largely supplanted by steel fibers mixed with the cement and the aggregate. Steel fiber reinforced shotcrete (SFRS) was first used in tunnels in North America by the USACE in 1972 in an adit at Ririe Dam (Idaho) (Morgan 1991). In addition to improving toughness and flexural strength, steel fibers improve the fatigue and impact resistance of the shotcrete layer. Other relatively recent improvements to shotcrete applications include admixtures for a variety of purposes, notable among which is the use of micro silica, which greatly reduces rebound and increases density, strength, and water tightness. EM 1110-2-2005 provides guidance in the design and application of shotcrete.

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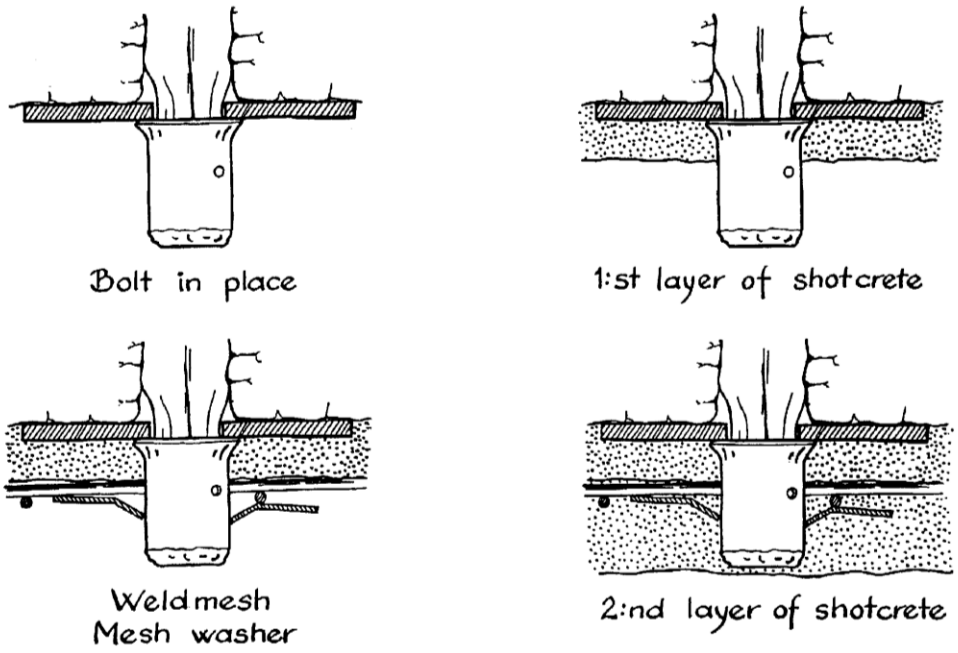


Figure 11: Dowels with end hardware embedded in shotcrete

(1) *Range of applications.* For most tunnels and shafts, shotcrete is used as an initial ground support component. It is sprayed on freshly exposed rock in layers 2 to 4 in. thick where it sets in a matter of minutes or hours, depending on the amount of accelerator applied, and helps support the rock. In blasted rock with irregular surfaces, shotcrete accumulates to greater thicknesses in the over breaks. This helps prevent block motion and fallout due to shear, by adhering to the irregular surface. On more uniform surfaces, the shotcrete supports blocks by a combination of shear, adhesion, and moment resistance and supports uniform and nonuniform radial loads by shell action and adhesion. By helping prevent the initiation of rock falls, shotcrete also prevents loosening of the rock mass and the potential for raveling failure. Shotcrete also protects surfaces of rock types that are sensitive to changes of moisture content, such as swelling or slaking rock. The application of shotcrete is an essential ingredient in the construction method of sequential excavation and support, where it is used in combination with rock bolts or dowels and, sometimes, steel ribs or lattice girders in poor ground. For TBM tunnels, initial ground support usually consists of dowels, mesh, mine straps, channels, or steel ribs; shotcrete can be applied some distance behind the advancing face. Only in a few instances have TBMs been built with the possibility to apply shotcrete a short distance behind the face.

(2) *Reinforced shotcrete.* In poor or squeezing ground, additional ductility of the shotcrete is desirable. Until recently this ductility was generally achieved by welded wire fabric usually applied between the first and the second coat of shotcrete. While wire fabric does add to the ductility of the shotcrete, it has several disadvantages. It is laborious and costly to place; it is difficult to obtain good shotcrete quality around and behind wires; and it often results in greater required shotcrete volumes, because the fabric cannot be draped close to the rock surface on

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irregularly shot surfaces. Modern reinforced shotcrete is almost always steel fiber-reinforced shotcrete. The steel fibers are generally 25- to 38-mm-long deformed steel strips or pins, with an aspect ratio, length to width or thickness, between 50 and 70. These steel fibers are added to the shotcrete mix at a rate of 50-80 kg/m^3 without any other change to the mix. The steel fibers increase the flexural and tensile strength but more importantly greatly enhance the post failure ductility of the shotcrete.

d. Steel ribs and lattice girders. Installing steel and wooden supports in a tunnel is one of the oldest methods in use. Many years ago, wooden supports were used exclusively for tunnel support. In later years, steel ribs (Figure 5-12) took the place of wood, and, most recently, steel lattice girders (Figure 5-13) are being used in conjunction with shotcrete. Figure 5-14 shows an application of shotcrete, lattice girders, and dowels for a rapid transit tunnel through a fault zone. It is usually faster and more economical to reinforce the rock with rock bolts, steel mesh or straps, and shotcrete so the rock will support itself. However, if the anticipated rock loads are too great, such as in faulted or weathered ground, steel supports may be required. Steel ribs and lattice girders usually are installed in the tunnel in sections within one rib spacing of the tunnel face. The ribs are generally assembled from the bottom up making certain that the rib has adequate footing and lateral rigidity. Lateral spacer rods (collar braces) are usually placed between ribs to assist in the installation and provide continuity between ribs. During and after the rib is erected, it is blocked into place with grout-inflated sacks as lagging, or shotcrete. In modern tunnel practice, the use of wood blocking is discouraged because it is deformable and can deteriorate with time. The rib functions as an arch, and it must be confined properly around the perimeter. The manufacturer of steel ribs provides recommendations concerning the spacing of blocking points that should be followed closely (see Proctor and White 1946). When shotcrete is used as lagging, it is important to make sure that no voids or laminations are occurring as the shotcrete spray hits the steel elements. Steel ribs should be fully embedded in the shotcrete. The lattice girders are filled in by shotcrete in addition to being embedded in shotcrete. Steel ribs and lattice girders are often not the sole method of tunnel support but are only provided in the event that bad tunneling conditions are encountered. In this case, it is necessary to have all the required pieces at the site and have adequately trained personnel ready when they are required in order to reduce delays in switching to a different type of tunnel support.

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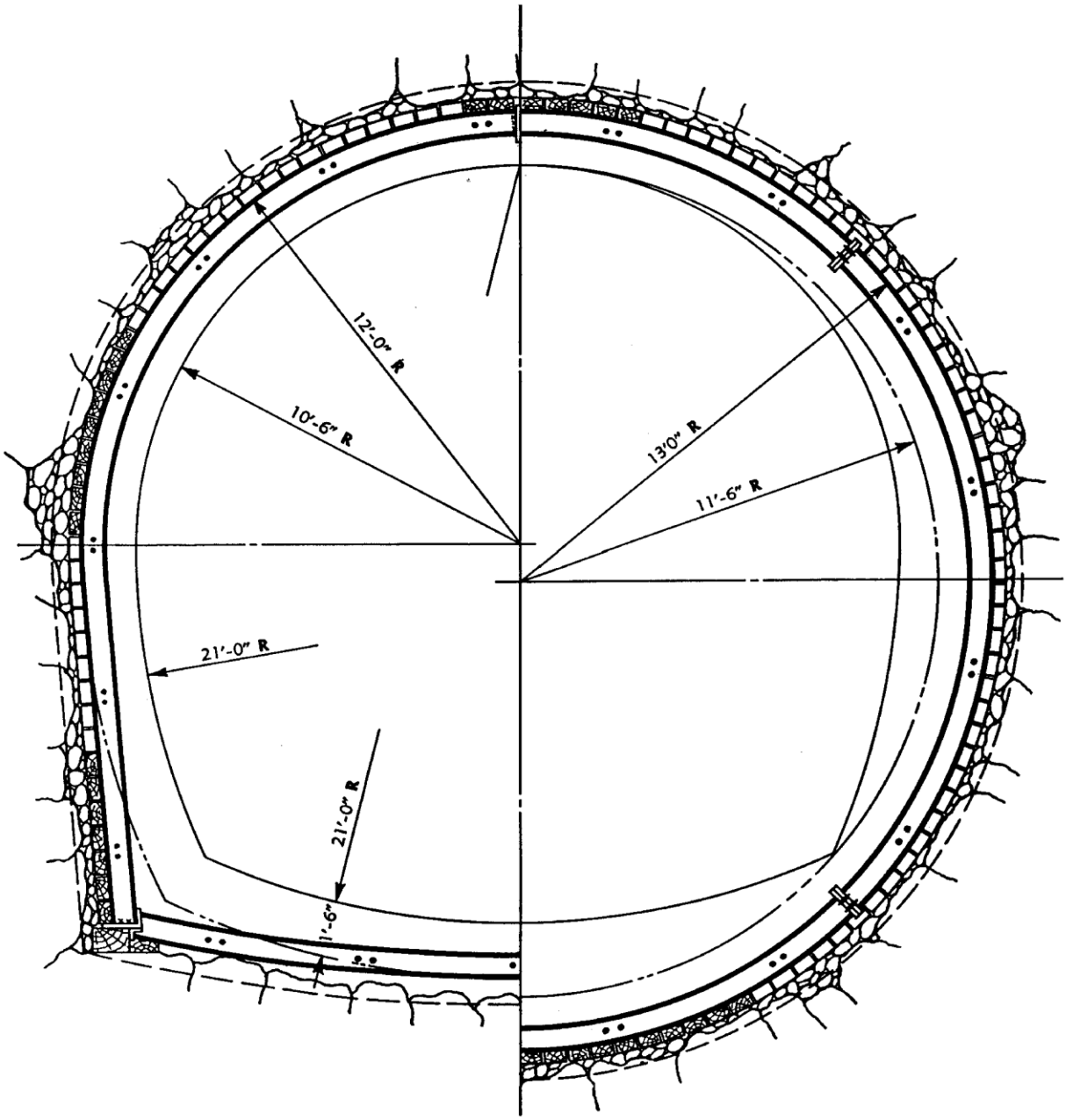


Figure 12: Steel rib examples, conversion of a horseshoe-shaped flow tunnel to a circular shape in squeezing ground

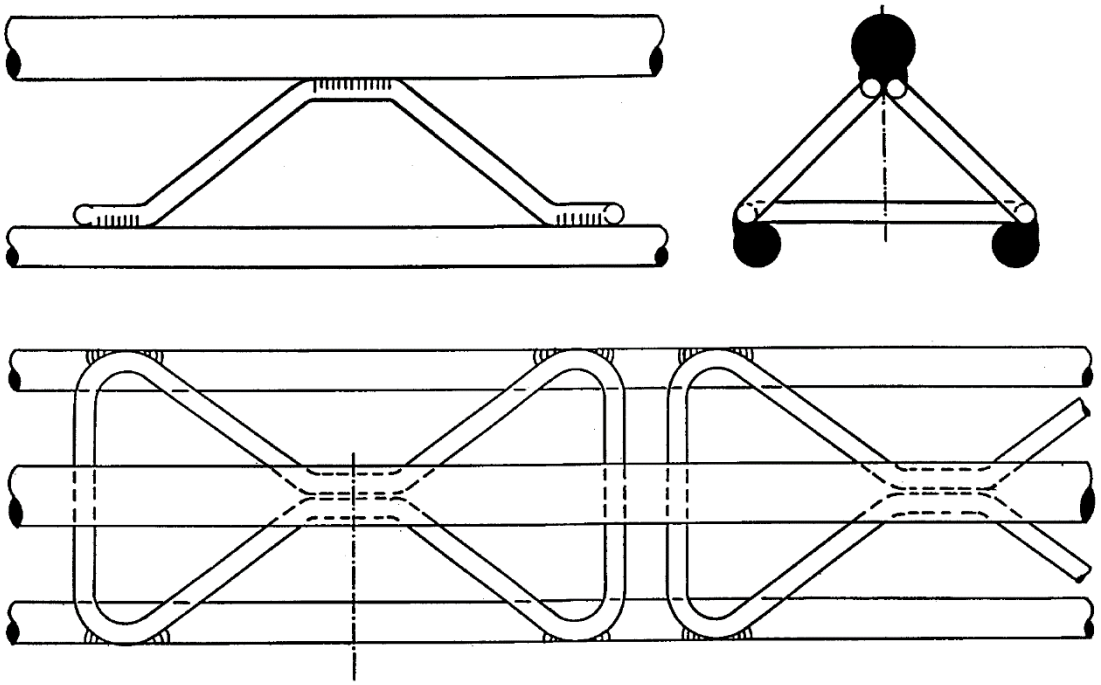


Figure 13: Lattice girders

e. Precast concrete segments used with TBM. Soft ground tunnels are most often constructed using shields or shielded TBMs with precast concrete segments. Below the groundwater table, the segments are bolted with gaskets for water tightness. Above the groundwater table, unbolted, expanded segmental linings are often used, followed by a cast-in-place concrete lining (two-pass lining). If necessary, a water- or gas-proofing membrane is placed before the cast-in-place concrete is placed. The shield or TBM is usually moved forward using jacks pushing on the erected segmental concrete lining. Hard rock tunnels driven with a TBM may also be driven with some form of segmental lining, either a one-pass or two-pass lining. There are several reasons for this choice.

(1) For the completion of a long tunnel, the schedule may not permit the length of time required to cast a lining in place. The option of casting lining concrete while advancing the TBM is feasible, at least for a large-diameter tunnel, but often not practical. Interference between concrete transportation and placement and tunnel excavation and mucking is likely to slow tunnel driving. Transporting fresh concrete for a long distance can also be difficult. In this instance, placing a one-pass segmental lining is a practical solution, provided that lining erection does not significantly slow the advance of the TBM.

f. Bolted or unbolted segments. A gasketed and bolted segmental lining must be fabricated with great precision, and bolting extends the time required for erection. Hence, such a lining is usually expensive to manufacture and to erect. For most water tunnels, and for many other tunnels, a fully gasketed and bolted, watertight lining is not required, and an unbolted segmental lining is adequate.

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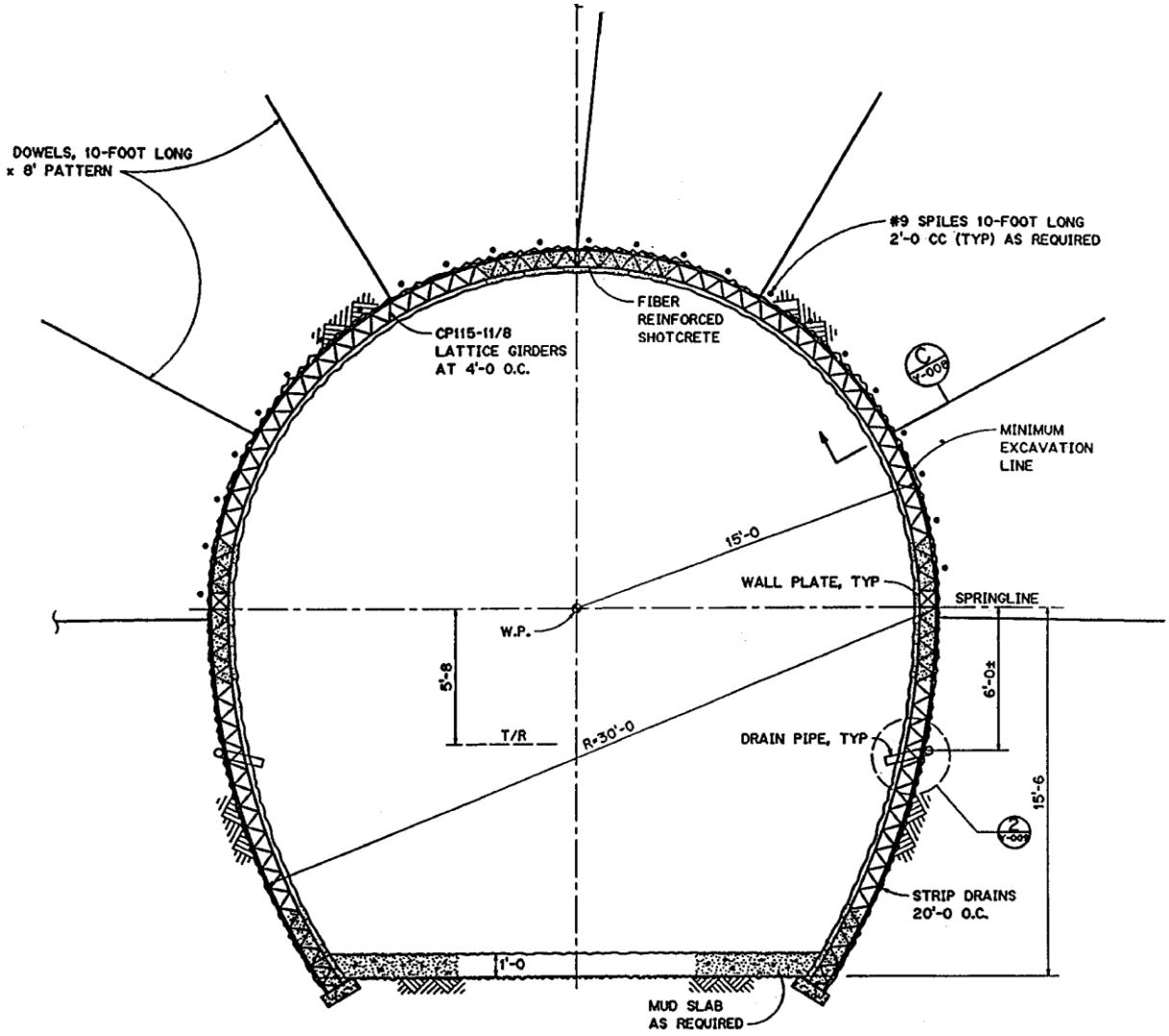


Figure 14: Lattice girders used as final support with steel-reinforced shotcrete, dowels, and spiles

g. Segment details. Once a segmental lining has been determined to be feasible or desirable, the designer has a number of choices to make. In the end, the contractor may propose a different lining system of equal quality that better fits his/her proposed methods of installation. A selections of lining and joint details are shown on Figures 5-15 to 5-17. Details are selected to meet functional requirements, and for practicality and economy of construction. For the most part, details can be mixed liberally to match given requirements and personal preferences.

h. Matching construction methods and equipment. When a tunnel lining system has been selected, construction methods and equipment must be designed to match the specific needs of this system. With a full shield tail, the invert segment is placed on the shield surface at the bottom. When the shield passes, the invert segment falls to the bottom, unless it is bolted to the previous

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segment. The erector equipment must match the pick-up holes in the segments, be able to rotate the segment into its proper place, and must have all of the motions (radial, tangential, axial, tilt, etc.) to place the segment with the tolerances required. Relatively high speed motion is required to bring a segment to its approximate location, but inching speed is often required for precise positioning. Unless each segment is stable as placed, holding devices are required to prevent them from falling out until the last segment is in place. Such holding devices are not required for a bolted and for most dowelled linings.

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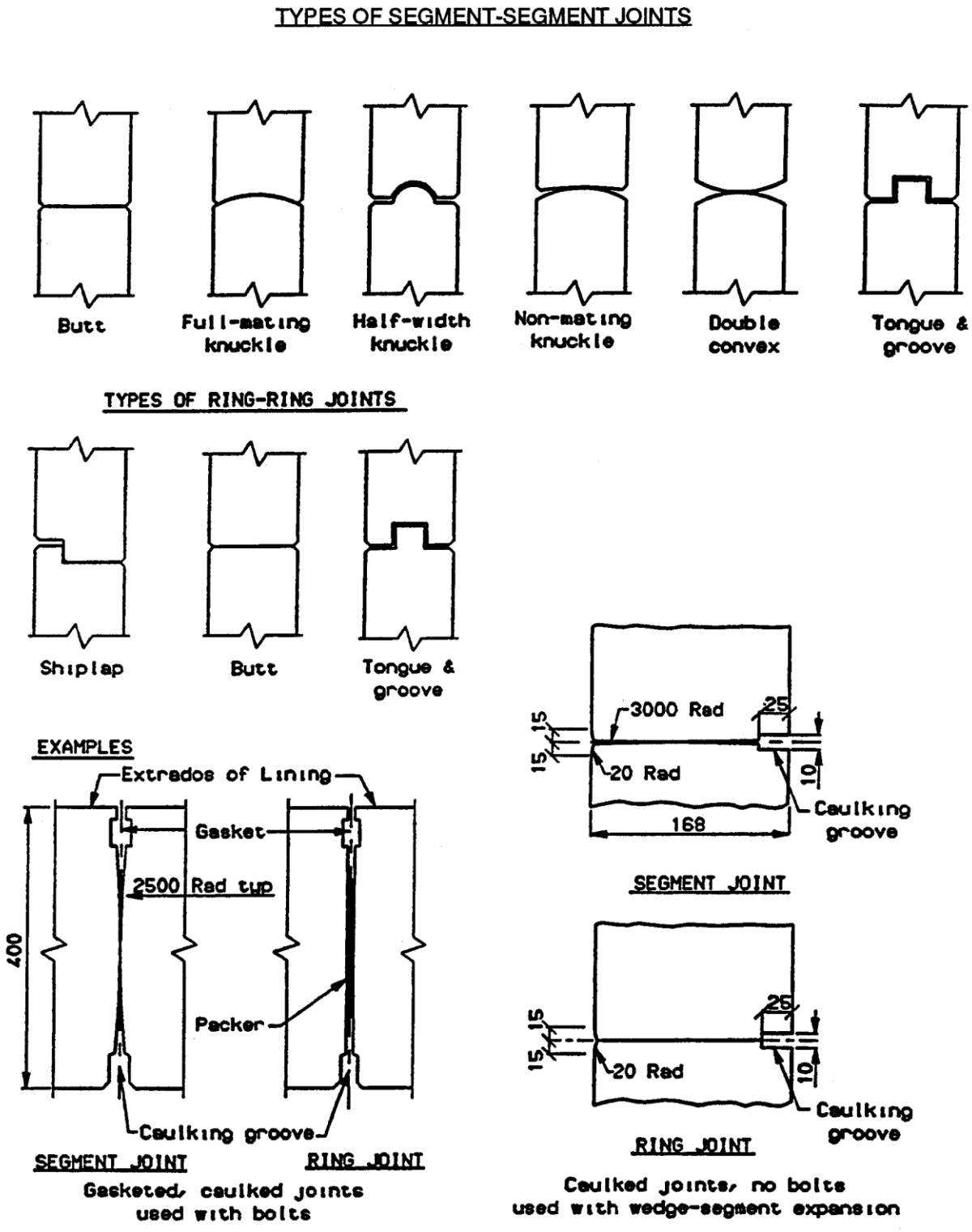


Figure 15: Types of joints in segmental concrete lining

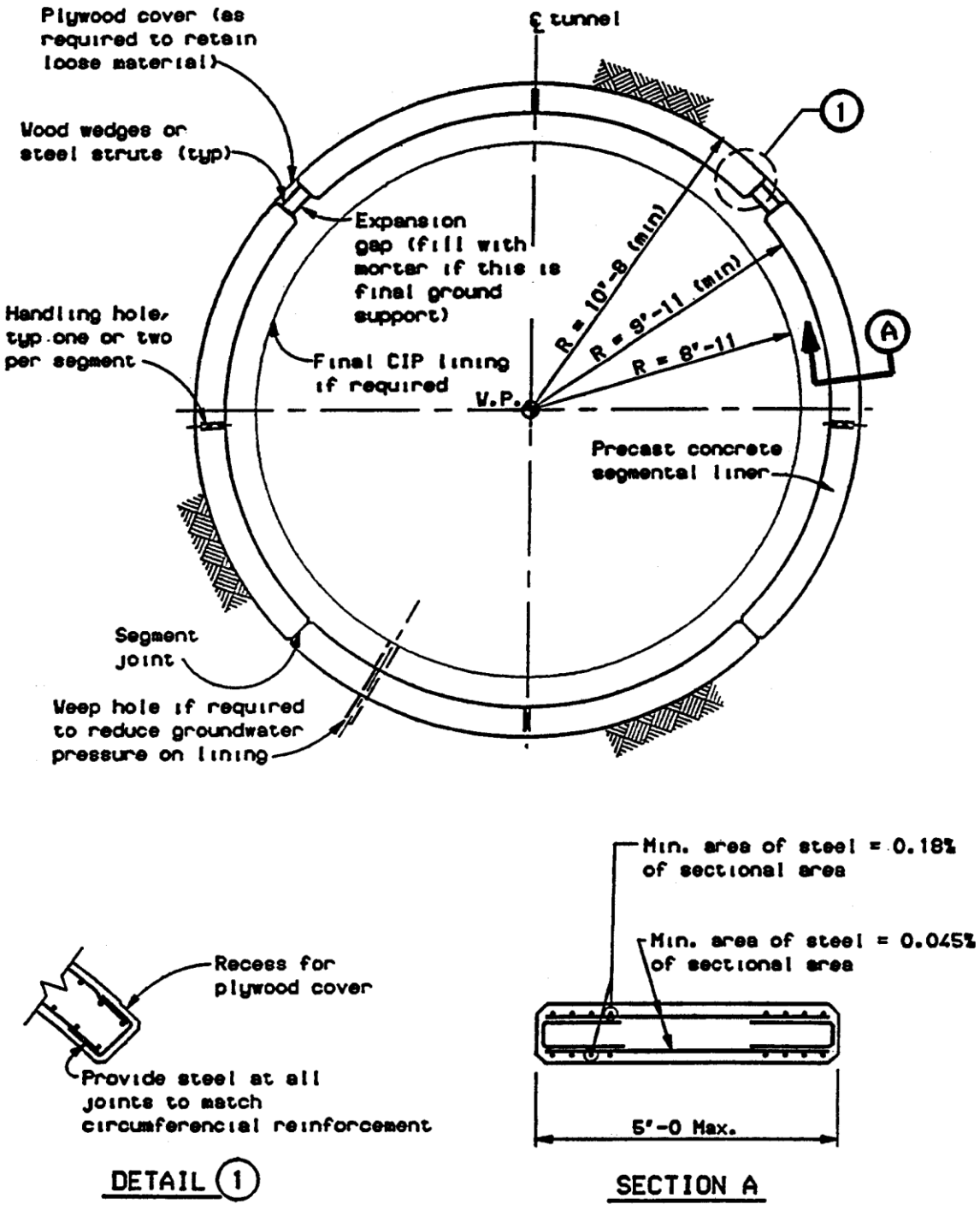


Figure 16: Simple expanded precast concrete lining used as initial ground support or as final ground support

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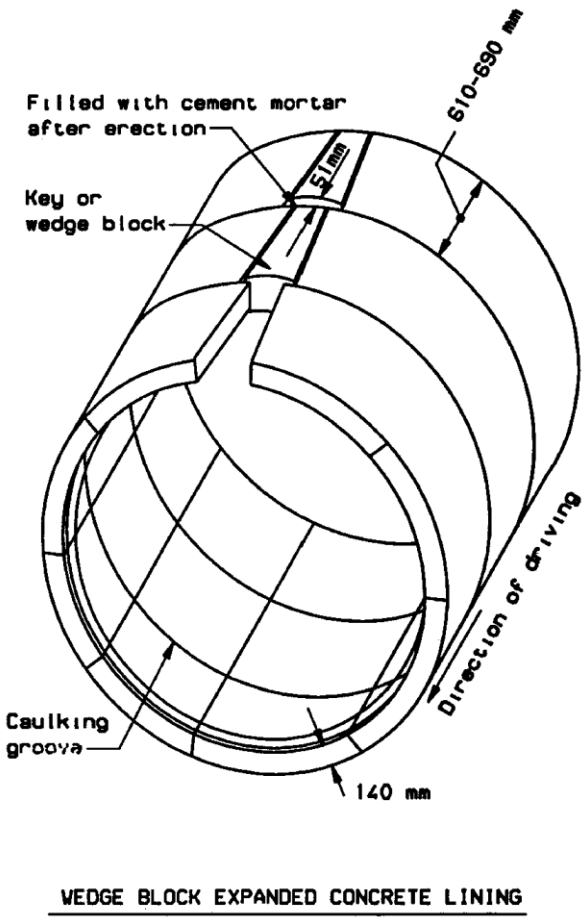


Figure 17: Wedge block expanded concrete lining

i. Functional criteria for one pass segmental linings.

(1) Selection of a segmental lining system is based on considerations of cost and constructability, and many details depend on the construction procedure. Functional criteria, however, must also be met.

(2) Water flow and velocity criteria often require a smooth lining to achieve a reasonably low Manning's number. This may require limitations on the offset permitted between adjacent segment rings. With an expanded lining, it is often not possible to obtain full expansion of all rings, and offsets between rings can be several centimeters. If this is not acceptable, an unexpanded dowelled or bolted ring may be required.

(3) In the event that some segments are, in fact, erected with unacceptable offsets, the hydraulic effect can be minimized by grinding down the protrusions or filling the shadows.

(4) A watertight lining is difficult to obtain using segments without gaskets. In some lining systems, sealing strips or caulking are employed to retain grout filling, but cannot sustain high groundwater pressures. In wet ground, it may be necessary to perform formation grouting to reduce water flows. Alternatively, fully gasketed and bolted linings may be used through the wet zones. This choice depends on the acceptability of water into or out from the tunnel during operations and the differential water pressure between the formation and the tunnel. The choice also depends on the practicality and economy of grouting during construction.

(5) The lining segments must be designed to withstand transport and construction loads. During storage and transport, segments are usually stacked with strips of timber as separation. Invert segments must withstand uneven loads from muck trains and other loads. The design of invert segments must consider that the segments may not be perfectly bedded. Lining rings used as reaction for shield propulsion must be able to withstand the distributed loads from the jacks, including eccentricities resulting from mismatching adjacent rings.

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(6) Joint details must be reinforced to resist chipping and spalling due to erection impact and the effect of uneven jacking on imprecisely placed segments. Tongue and-groove joints are particularly susceptible to spalling, and the edges of the groove may require reinforcement.

(7) Permanence of the finished structure requires consideration of long-term corrosion and abrasion effects. For a one-pass segmental lining, a high-strength concrete with a high pozzolan replacement is usually desirable for strength, density, tightness, and durability. Precast concrete of 41.4 MPa (6,000-psi) (28-day cylinder) strength or more is routinely used for this purpose. Reinforcement should be as simple as possible, preferably using prefabricated wire mesh.

(8) Once construction and long-term performance requirements have been met, postulated or actual exterior ground or water loads are usually of minor consequence. In rare instances, squeezing ground conditions at great depth may require a thicker lining or higher concrete strength. Water pressures may be reduced by deliberately permitting seepage into the tunnel, and moments in the lining are reduced by using unbolted joints.

Sequential Excavation and Support

Recognizing the inherent variability of geologic conditions, several construction methods have been developed so that methods of excavation and support can be varied to suit encountered conditions. The most famous of these methods is the New Austrian Tunneling Method (NATM), developed and commonly used in Central Europe. Much older, and applied throughout the world, is the observational method. Both of these methods are discussed in the following sections.

a. NATM.

(1) The so-called NATM is employed for large, non-circular tunnels in poor ground where ground support must be applied rapidly. NATM usually involves the following components:

- Heading-and-bench or multidrift excavation (no shield or TBM).

  Excavation by blasting or, more commonly, by roadheader or other mechanical means.

- Initial ground support usually consisting of a combination of shotcrete, dowels, steel sets, or (now more commonly) lattice girders, installed quickly after exposure by excavation.

- Forepoling or spiling where the ground requires it.

- Stabilizing the face temporarily, using shotcrete and possibly glass-fiber dowels.

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- Ground improvement (grouting, freezing, dewatering).

  Extensive use of monitoring to ascertain the stability and rate of convergence of the opening.

(2) The final lining usually consists of reinforced, cast-in-place concrete, often with a waterproofing membrane between the cast-in-place concrete and the initial ground support.

(3) It would appear that the NATM employs virtually all of the means and methods available for tunneling through poor ground. What distinguishes the method is the extensive use of instrumentation and monitoring as an essential part of the construction method. Traditionally, monitoring involves the use of the following devices.

- Convergence measurements, wall to wall and wall to crown.
- Surveying techniques, floor heave, crown sag.
- Multiposition borehole extensometers.
- Strain gages or load cells in the shotcrete, at the rock-shotcrete interface, or on dowels or steel sets, or lattice girders.

(4) The instrumentation is used to assess the stability and state of deformation of the rock mass and the initial ground support and the build-up of loads in or on support components. In the event that displacements maintain their rate or accelerate, that loads build to greater values than support components can sustain, or if instability is visually observed (cracks, distortion), then additional initial ground support is applied. Final lining is placed only after ground movements have virtually stopped.

(5) Initial ground support intensity (number of dowels, thickness of shotcrete, and spacing of steel sets or lattice girders) is applied according to conditions observed and supplemented as determined based on monitoring data. The overall cross section can also be varied according to conditions, changing from straight to curved side walls. The invert can be over excavated to install a straight or downward curved strut when large lateral forces occur. In addition, sequences of excavation can be changed, for example from heading-and-bench excavation to multiple drifting.

(6) The NATM has been used successfully for the construction of large tunnel cross sections in very poor ground. On a number of occasions, the method has been used even for soft-ground tunnel construction, sometimes supplemented with compressed air in the tunnel for groundwater control and to improve the stand-up time of the ground. Using the NATM in poor rock requires careful execution by contractor personnel well experienced in this type of work. In spite of careful execution, the NATM is not immune to failure. A number of failures, mostly at or near the tunnel

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face, have been recorded. These have occurred mostly under shallow cover with unexpected geologic or groundwater conditions or due to faulty application insufficient shotcrete strength or thickness, belated placement of ground support, or advancing the excavation before the shotcrete has achieved adequate strength.

(7) It is common to model the complete sequence of excavation and construction using a finite element or finite differences model so as to ascertain that adequate safety factors are obtained for stresses in the final lining. Elastic or inelastic representations of the rock mass properties are used, and tension cracks in unreinforced concrete or shotcrete that propagate to the middle of the cross section are acceptable.

(8) The NATM method of construction requires a special contract format to permit payment for work actually required and carried out and a special working relationship between the contractor and the owner's representative onsite to agree on the ground support required and paid for. Writing detailed and accurate specifications for this type of work is difficult.

(9) While commonly used in Central Europe, the NATM has not been popular in the United States for a number of reasons:

(a) Ground conditions are, for the most part, better in the United States than in those areas of Europe where NATM is popular. In recent years, there have been few opportunities to employ the NATM in the United States.

(b) Typical contracting practices in the United States make this method difficult to administer.

(c) Emphasis in the United States has been on high-speed, highly mechanized tunneling, using conservative ground support design that is relatively insensitive to geologic variations. NATM is not a high-speed tunneling method.

(d) Most contractors and owners in the United States are not experienced in the use of NATM.

This is not meant to imply that the method should not be considered for use in the United States. Short tunnels or chambers (example: underground subway station) located in poor ground that requires rapid support may well be suited for this method. More often, however, the instrumentation and monitoring component of the NATM is dispensed with or relegated to a minor part of the construction method, perhaps applicable only to limited areas of known difficulty. This type of construction is more properly termed "sequential excavation and support.

b. The observational method and sequential excavation and support.

(1) Sequential excavation and support can incorporate some or most of the NATM

components, but instrumentation and monitoring are omitted or play a minor role. Instead, a uniform, safe, and rapid excavation and support procedure is adopted for the project for the full length of the tunnel. Or several excavation and support schemes are adopted, each applicable to a portion of the tunnel. The typical application employs a version of the observational method, as follows:

(a) Based on geologic and geotechnical data, the tunnel profile is divided into three to five segments of similar rock quality, where similar ground support can be applied.

(b) Excavation and initial ground support schemes are designed for each of the segments. Excavation options may include full-face advance, heading and- bench, or multiple drifting. The initial support specification should include designation of maximum time or length of exposure permitted before support is installed.

(c) A method is devised to permit classification of the rock conditions as exposed, in accordance with the excavation and ground support schemes worked out. Sometimes a simplified version of the Q-method of rock mass classification is devised.

(d) Each ground support scheme is priced separately in the bid schedule, using lengths of tunnel to which the schemes are estimated to apply.

(e) During construction the ground is classified as specified, and the contractor is paid in accordance with the unit price bid schedule. The final price may vary from the bid, depending on the actual lengths of different ground classes observed.

(2) The term "sequential excavation and support is usually employed for excavations that may involve multiple drifting and rapid application of initial support. The observational method works well with this type of construction. However, the observational method also works well with tunneling using TBM. Here, the opening is typically circular, and the initial ground support options do not usually include rapid application of shotcrete, which is considered incompatible with most TBMs. The following is an example of the observational method specified for a TBM driven tunnel.

(3) With a TBM-driven tunnel, shotcrete was considered inappropriate, particularly since the types of rock expected would not suffer slaking or other deterioration upon exposure. Maximum use was made of rock dowels, wire mesh, and straps in the form of curved channels. Class A rock might in most instances require no support for the temporary condition; nonetheless, initial ground support was specified to add safety and to minimize the effort required for continuous classification of the rock mass.

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(4) The contract also provided for having a number of steel sets on hand for use in the event that bolts or dowels are ineffective in a particular reach. Estimates were made for bidding purposes as to the total aggregate length of tunnel for which each rock class was expected, without specifying where.

Portal Construction

a. Tunnels usually require a minimum of one to two tunnel diameters of cover before tunneling can safely commence. An open excavation is made to start, which when finished will provide the necessary cover to begin tunneling. Rock reinforcement systems are often used to stabilize the rock cut above the tunnel and are usually combined with a prereinforcement system of dowels installed around the tunnel perimeter to facilitate the initial rounds of excavation (Figure 18). If a canopy is to be installed outside of the tunnel portal for protection from rock falls, it should be installed soon after the portal excavation has been completed. If multiple stage tunnel excavation is to be used on the project, the contractor may excavate the portal only down to the top heading level and commence tunneling before taking the portal excavation down to the final grade.

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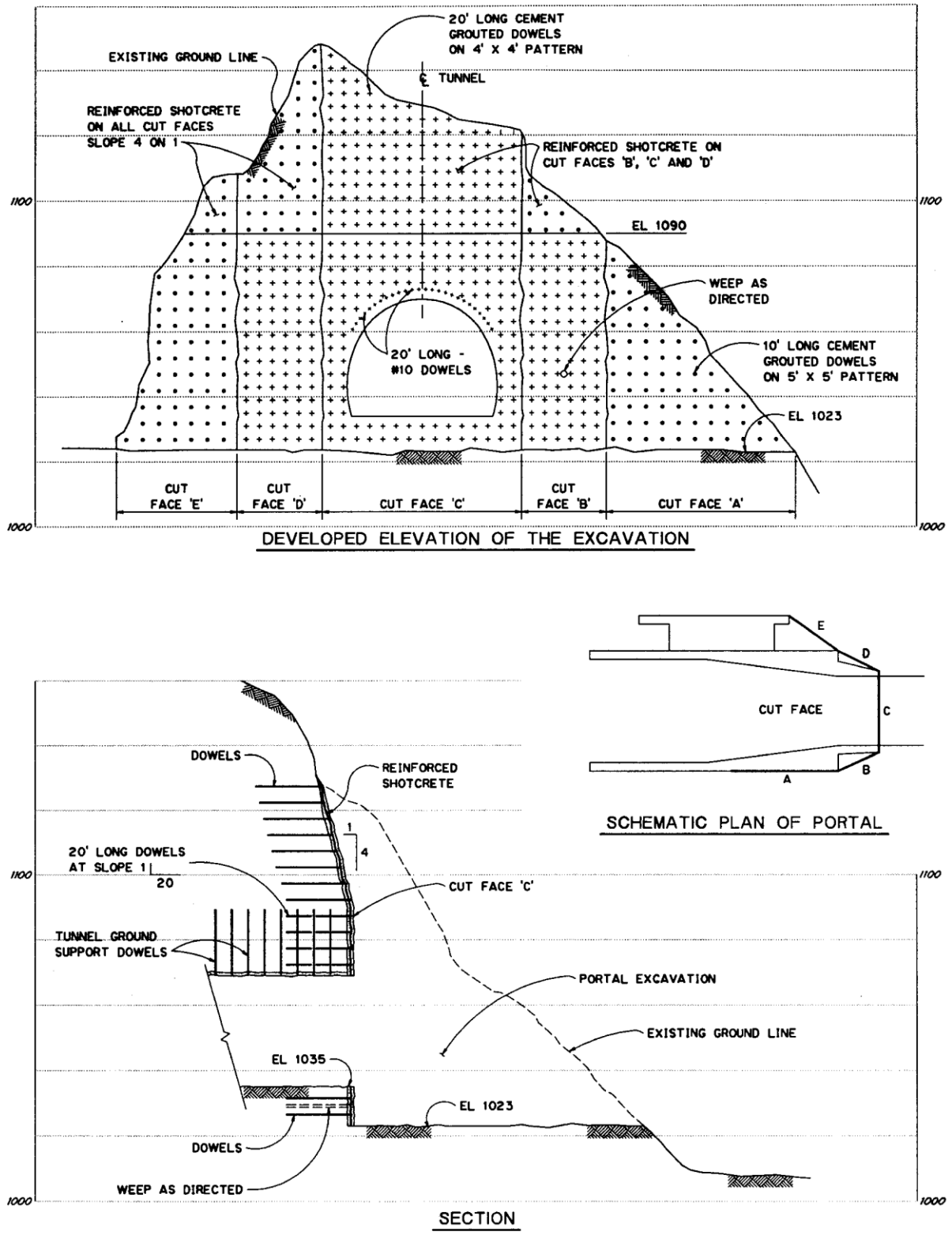


Figure 18: Portal excavation and support (H-3 tunnel, Oahu)

b. Tunnel excavation from the portal should be done carefully and judiciously. Controlled blasting techniques should be used and short rounds of about 1 m in depth are adequate to start. After the tunnel has been excavated to two or three diameters from the portal face, or as geology dictates, the blasting rounds can be increased progressively to standard length rounds used for normal tunneling.

c. When constructing portals, the following special issues should be accounted for:

(1) The rock in the portal is likely to be more weathered and fractured than the rock of the main part of the tunnel.

(2) The portal must be designed with proper regard for slope stability considerations, since the portal excavation will unload the toe of the slope.

(3) The portal will be excavated at the beginning of mining before the crew has developed a good working relationship and experience.

(4) The slope must be adequately designed to adjust to unloading and stress relaxation deformations.

(5) The portal will be a heavily used area, and a conservative design approach should be taken because of the potential negative effects instability would have on the tunneling operations.

d. The design of portal reinforcement will depend on geologic conditions. Rock slope stability methods should be used unless the slope is weathered or under a deep layer of overburden soil. In this case, soil slope stability analyses must be performed for the soil materials. Often, both types of materials are present, which will require a combined analysis.

e. The types of portal treatments that may be considered include the following:

- No support at the portal when excellent geologic conditions prevail.
- Portal canopy only for rock fall protection.
- Rock reinforcement consisting of a combination of rock bolts, steel mesh, shotcrete, and weeps.
- Rock reinforcement and a canopy for very poor conditions.

Tunnel reinforcement is usually more intense in the vicinity of the portal until the effects of the portal excavation are no longer felt.

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Shaft Construction

Most underground works include at least one deep excavation or shaft for temporary access or as part of the permanent facility. Shafts typically go through a variety of ground conditions, beginning with overburden excavation, weathered rock, and unweathered rock of various types, with increasing groundwater pressure. Shaft construction options are so numerous that it is not possible to cover all of them in this manual. The reader is referred to standard foundation engineering texts for shaft construction, temporary and permanent walls through soil and weathered rock, and to the mining literature for deep shafts through rock. The most common methods of shaft excavation and ground support are summarized in this section.

a. *Sizes and shapes of shafts.* Shafts serving permanent functions (personnel access, ventilation or utilities, drop shaft, de-airing, surge chamber, etc.) are sized for their ultimate purpose. If the shafts are used for construction purposes, size may depend on the type of equipment that must use the shaft. Shallow shafts through overburden are often large and rectangular in shape. If space is available, a ramp with a 10-percent grade is often cost-effective. Deeper shafts servicing tunnel construction are most often circular in shape with a diameter as small as possible, considering the services required for the tunnel work (hoisting, mucking, utilities, etc.). Typical diameters are between 5 and 10 m (16-33 ft). If a TBM is used, the shaft must be able to accommodate the largest single com- ponent of the TBM, usually the main bearing, which is usually of a size about two-thirds of the TBM diameter.

b. Shaft excavation and support through soil overburden.

(1) Large excavations are accomplished using conventional soil excavation methods such as backhoes and dozers, supported by cranes for muck removal. In hard soils and weathered rock, dozers may require rippers to loosen the ground. The excavation size will pose limits to the maneuverability of the excavation equipment.

(2) Smaller shafts in good ground, where groundwater is not a problem, can be excavated using dry drilling methods. Augers and bucket excavators mounted on a Kelly, operated by a crane-mounted torque table attachment, can drill holes up to some 75m (250-ft) depth and 8m (25-ft) diam.

(3) Many options are available for initial ground support, including at least the following:

- Soldier piles and lagging, in soils where groundwater is not a problem or is controlled by dewatering.
- Ring beams and lagging or liner plate.
- Precast concrete segmental shaft lining.
- Steel sheet pile walls, often used in wet ground that is not too hard for driving the sheet piles.

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- Diaphragm walls cast in slurry trenches; generally more expensive but used where they can have a permanent function or where ground settlements and dewatering must be controlled.
- Secant pile walls or soil-mixing walls as substitutes for diaphragm walls, but generally less expensive where they can be used.

(4) Circular shafts made with diaphragm or secant pile walls usually do not require internal bracing or anchor support, provided circularity and continuity of the wall is well controlled. Other walls, whether circular or rectangular, usually require horizontal support, such as ring beams for circular shafts, wales and struts for rectangular shafts, or soil or rock anchors or tiebacks that provide more open space to work conveniently within the shaft.

(5) In good ground above the groundwater table, soil nailing with shotcrete is often a viable ground support alternative.

c. Shaft excavation through rock.

(1) Dry shaft drilling using a crane attachment or a derrick, as briefly described in the previous subsection, has been proven viable also in rock of strength up to 15 MPa (2,200 psi), provided that the ground is initially stable without support. Use of a bucket with extendable reamer arms permits installation of initial ground support, which would consist of shotcrete and dowels as the shaft is deepened.

(2) Deep shafts can be drilled using wet, reverse circulation drilling. Drilling mud is used to maintain stability of the borehole and counterbalance the formation water pressure, as well as to remove drill cuttings. The drilling is done with a cutterhead, furnished with carbide button cutters and weighted with large donut weights to provide a load on the cutterhead. The drill string is kept in tension, so that the pendulum effect can assist in maintaining verticality of the borehole. Mud is circulated by injecting compressed air inside the drill string; this reduces the density of the drilling mud inside the string and forces mud and drill cuttings up the string, through a swivel, and into a mud pond. From there the mud is reconditioned and led back into the borehole. This type of shaft construction usually requires the installation of a steel lining or casing with external stiffeners, grouted in place. If the steel casing is too heavy to be lowered with the available hoisting gear, it is often floated in with a bottom closure and filled partly with water. This method permits shafts of 2m (7-ft) diam to be constructed to depths of about 1,000 m (3,300 ft). Larger diameters can be achieved at shallower depths.

(3) If underground access is available, shafts can be drilled using the raise drilling method. A pilot bore is drilled down to the existing underground opening. Then a drill string is lowered, and a drill head is attached from below. The string is turned under tension using a raise drill at

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the ground surface, and the shaft is created by backreaming, while cuttings drop into the shaft to the bottom, where they are removed. This method requires stable ground. Raise boring can also be used for nonvertical shafts or inclines. A raised bore can be enlarged using the slashing method of blasting. The bore acts as a large burn cut, permitting blasting with great efficiency and low powder factors.

(4) Conventional shaft sinking using blasting techniques can be used to construct a shaft of virtually any depth, size, and shape. A circular shape is usually preferred, because the circular shape is most favorable for opening stability and lining design. Shaft blasting tends to be more difficult and more confined than tunnel blasting. Typically, shorter rounds are pulled, and the powder factor is greater than for a tunnel in the same material. Variations of the wedge cut are used rather than the burn cut typically used for tunnels. Shallow shaft construction can be serviced with cranes, but deeper shaft construction requires more elaborate equipment. The typical arrangement includes a head frame at the top suspending a two- or three-story stage with working platforms for drilling and blasting, equipment for mucking, initial ground support installation, and shaft lining placement. The typical shaft lining is a cast-in-place concrete lining, placed 10 to 15 m (33 – 50 ft) above the advancing face.

(5) If the shaft is large enough to accommodate a roadheader, and the rock is not too hard, shaft excavation can be accomplished without explosives using crane service or head frame and stage equipment.

(6) Most shaft construction requires the initial construction of a shaft collar structure that supports overburden and weathered rock near the surface and construction loads adjacent to the top of the shaft. It also serves as a foundation for the temporary head frame used for construction as well as for permanent installations at the top of the shaft.

(7) Inclines of slopes up to about 25 deg can be bored using a TBM specially equipped to maintain its position in the sloping hole. Inclines at any angle can be excavated using blasting methods, with the help of climbing gear such as the Alimak climber.

Options for Ground Improvement

When difficult tunnel or shaft construction conditions are foreseen, ground improvements are often advisable and sometimes necessary. There are, generally speaking, three types of ground improvement that can be feasibly employed for underground works in rock formation:

- Dewatering.
- Grouting.
- Freezing.

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a. Ground improvement for shaft sinking.

(1) Ground improvement must be considered when shaft sinking involves unstable ground associated with significant groundwater inflow. At a shallow depth, groundwater is often found in potentially unstable, granular materials, frequently just above the top of rock. If sufficiently shallow, the best solution is to extend the shaft collar, consisting of a nominally tight wall, into the top of rock. Shallow groundwater can also often be controlled by dewatering.

(2) An exploratory borehole should be drilled at or close to the center of all shafts. Borehole permeability (packer) tests can be used to estimate the potential groundwater inflow during construction that could occur if the groundwater were not controlled. If the estimated inflow is excessive, ground improvement is called for. At the same time, core samples will give an indication of ground stability as affected by groundwater inflow. Poorly cemented granular sediments and shatter zones are signs of potential instability.

(3) Deep groundwater usually cannot be controlled by dewatering; however, grouting or freezing can be tried. This is usually done from the ground surface before shaft sinking commences, because it is very costly to work down the shaft. Both methods require the drilling of boreholes for the installation of freeze pipes or for grouting. When the shaft is very deep, high-precision drilling is required to reduce the deviation of boreholes to acceptable magnitudes. Considering that borehole spacing is of the order of 1.5 to 2 m (6-7 ft) and that both grouting and freezing rely on accurate placement of the holes, it is readily appreciated that even a deviation of 1 m (3.3 ft) can be critical. Nonetheless, freezing and grouting have been successfully carried out to depths greater than 500 m (1,700 ft). It is also readily appreciated that both grouting and freezing are very costly; however, they are often the only solutions to a serious potential problem.

(4) Freezing is often more expensive than grouting, and it takes some time to establish a reliable freeze wall, while grouting can be performed more quickly. Professionals in the shaft sinking business generally consider freezing to be substantially more reliable and effective than grouting. It is not possible to obtain a perfect grout job - a substantial reduction of permeability (say, 80-90 percent) is the best that can be hoped for - and grouting may leave some areas ungrouted. On the other hand, a freeze job can more readily be verified and is more likely to create a continuous frozen structure, thus is potentially more reliable.

(a) Grouting. The detailed grouting design for deep shafts is often left to a specialist contractor to perform and implement. While chemical grouting is often used in loose sediments and overburden materials, grouting in rock is usually with cement. Grout penetration into fractures is limited by aperture of the fractures relative to the cement particle sizes. As a rule, if the rock formation is too tight to grout, it is also usually tight enough that groundwater flow is

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not a problem. Shaft grouting typically starts with the drilling of two or three rows of grout holes around the shaft perimeter, spaced 1.5 to 2.0 m (5-7 ft) apart. Grout injection is performed in the required zones usually from the bottom up, using packers. The effectiveness of the grout job can be verified by judicious sequencing of drilling and grouting. If secondary grout holes drilled after the first series of grouted holes display little or no grouts take, this is a sign of the effectiveness of grouting. Additional grout holes can be drilled and grouted as required, until results are satisfactory. If it becomes necessary to grout from the bottom of the shaft, indicated, for example, by probeholes drilled ahead of the advancing shaft, then grout holes are drilled in a fan pattern covering the stratum to be grouted. It is important to perform the grouting before a condition has arisen with large inflows, because grouting of fissures with rapidly flowing water is very difficult. When drilling from the bottom of a deep shaft, it is often necessary to drill through packers or stuffing boxes to prevent high-pressure water from entering the shaft through the drill holes.

(b) Freezing. Brine is usually used as the agent to withdraw caloric energy from the ground and freeze the water in the ground. The brine is circulated from the refrigeration plant in tubes placed in holes drilled through the ground to be frozen. The tubes can be insulated through ground that is not intended to be frozen. The detailed design and execution of a freezing program requires specialist knowledge and experience that is only available from firms that specialize in this type of work. The designer of the underground work should prepare a performance specification and leave the rest to the contractor and his specialist subcontractor. The detailed design of a freeze job includes the complete layout of plant and all freeze pipes so as to achieve a freeze wall of adequate required energy consumption and the time required to achieve the required results, with appropriate safety factors. The strength of frozen ground is dependent on the character and water content of the ground and increases with decreasing temperature of the frozen ground. Some rock types, notably weak, fine-grained rocks, suffer a substantial strength loss upon thawing. The effects of thawing must be considered in the design of the final shaft lining. Saline groundwater is more difficult to freeze because of its lower freezing temperature. If the formation water is not stagnant but moves at an appreciable rate, it will supply new caloric energy and delay the completion of the freeze job. The velocity of formation water movement should be estimated ahead of time, based on available head and gradient data. At the ground surface, brine distribution pipes are often laid in a covered trench or gallery around the shaft, keeping them out of the way from shaft construction activities. Since freezing involves expansion of the formation water, a relief borehole is usually provided at the center of the shaft so that displaced water can escape. The freezing process is controlled by installing temperature gages at appropriate locations between freeze pipes, as well as through monitoring of the temperature of return brine and the overall energy consumption. On rare occasions it becomes necessary to implement a freezing installation from the bottom of the shaft. This usually requires the construction of a freezing gallery encircling the shaft. Shaft excavation cannot proceed during the implementation of an underground freeze job, including the time required to achieve the

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necessary reduction in ground temperature. Down-the-shaft freezing, therefore, is very costly. Quicker implementation of a freezing application can be accomplished using liquid nitrogen as coolant rather than brine.

b. Ground improvement for tunneling. Rock tunnels generally do not require ground improvement as frequently as shafts. Examples of ground improvements using grout applications are briefly described in the following.

(1) Preconstruction application. Where it is known that the tunnel will traverse weak ground, such as unconsolidated or poorly consolidated ground or a wide shatter zone, with high water pressure, the ground can be grouted ahead of time. It is preferable to grout from the ground surface, if possible, to avoid delaying tunneling operations. Such grout applications are particularly helpful if the water is contaminated with pollutants or if the groundwater is hot. The primary purpose of applying grout is to reduce sometimes a side benefit.

(2) Application during construction. When grouting cannot be applied from the ground surface, it can be carried out from the face of the tunnel before the tunnel reaches the region with the adverse condition. An arrangement of grout holes is drilled in fan shape some 20 to 40 m (60-130 ft) ahead of the face. Quality control is achieved by drilling probe holes and testing the reduction of permeability. Grouting is continued until a satisfactory permeability reduction is achieved.

(3) Application after probe hole drilling. Where adverse conditions are expected but their location is unknown, probe hole drilling will help determine their location and characteristics. Such probe holes can be simple percussion holes with a record of water inflow, or packer tests can be performed in these probe holes. The grout application can be designed based on the results of one or more probe holes.

(4) Post excavation grouting. If it is found that water inflow into the excavated tunnel is too large for convenient placement of the final lining, radial grouting can be performed to reduce the inflow. Generally, the grout is first injected some distance from the tunnel, where water flow velocities are likely to be smaller than at closer distances. It is sometimes necessary to perform radial grouting after the completion of the tunnel lining. Here, the finished lining helps to confine the grout, but the lining must be designed to resist the grout pressures.

(5) Freezing in tunnels. Freezing is sometimes a suitable alternative to grouting for temporary ground strengthening and inflow control. Freezing is particularly effective if the ground is weak, yet too impervious for effective grout penetration.

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Drainage and Control of Groundwater

a. General. The design of a permanent drainage system and the control systems required for groundwater begins during the geotechnical exploration phases with an assessment of the potential sources and volumes of water expected during construction. The type of permanent drainage system required will depend upon the type of tunnel and site groundwater conditions.

b. Assessment of water control requirements. Prior to construction, estimates of the expected sources of groundwater and the expected inflow rates and volumes must be identified in order for the contractor to provide adequate facility for handling inflow volumes.

c. Care of groundwater during construction.

(1) Care of groundwater generally is the responsibility of the contractor; however, the specifications for a tunnel contract may require that certain procedures be followed. For example, if it is expected that water-bearing joints will be present that contain sufficient head and volume to endanger the safety of the tunnel, the drilling of a probe-hole ahead of the working face should be required. The following discussion is for guidance.

(2) Water occurring in a tunnel during construction must be disposed of because it is a nuisance to workers and may make the placement of linings difficult or cause early weakening of the linings. It also makes the rock more susceptible to fallout by reducing the natural cohesion of fine-grained constituents.

(3) The excavation sequence should be such that drainage of the sections to be excavated is accomplished before excavation. Thus, a pilot drift near the invert in a wet environment is more effective than a top heading although enlargement to full size is more difficult. It is an excellent practice to carry a drill hole three tunnel diameters in advance of the working face. The drill hole has an additional advantage of revealing rock conditions more clearly than defined by the initial investigation.

(4) When encountered, water should be channeled to minimize its effect on the remaining work. To accomplish this, the surface of a fissure may be packed with quick-setting mortar around a tube leading to a channel in the invert. Ingenuity on the part of workers and supervisors can produce quick, effective action and should be encouraged so long as objectionable materials do not intrude within the concrete design line.

(5) If groundwater inflow is extremely heavy and drainage cannot be accomplished effectively, it will be necessary to install a "grout umbrella" from the face before

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each tunnel advance is made. This consists of a series of holes angled forward and outward around the perimeter of the face that are pumped with grout to fill fractures and form a tunnel barrier against high inflows.

(6) For permanent protection from the flow of water along the outside of the concrete lining, no better method exists than filling with grout any void that remains after the concrete is set.

d. Permanent drainage systems.

(1) Drainage system. The drainage system required in a tunnel will depend on the type of tunnel, its depth, and groundwater conditions. Some tunnels may not require special drainage. Others may require drainage to limit the pressure behind the lining or to remove water due to condensation and leakage through the tunnel joints. A detailed design procedure for drains will not be attempted here; however, a brief description will be included to indicate what is involved in providing drainage for the various types of tunnels.

(a) Pressure tunnels. Drainage for pressure tunnels may be required if normal outlets through gates or power units do not accomplish complete unwatering of the tunnel. The drains are then located at the low point of the tunnel and are provided with a shutoff valve. In some cases, it is desirable to provide drainage around a pressure tunnel. This may be done to limit the external head on the lining or to limit pressures in a slope in the event leakage developed through the lining. Drainage may be provided by drilling holes from the downstream portal or by a separate drainage tunnel.

(b) Outlet tunnels. Drainage for outlet tunnels may be required to completely unwater the tunnel if some point along the tunnel is lower than the outlet end. To limit the external head, drains can be provided that lead directly into the tunnel. In this manner, the outlet tunnel also serves as a drain tunnel.

(c) Vehicular tunnels. Drainage for vehicular tunnels will usually consist of weep holes to limit the pressure behind the lining and an interior drain system to collect water from condensation and leakage through the joints in the lining. Interior drainage can be either located in the center of the tunnel between vehicular wheel tracks or along the curbs. If the tunnel is located in areas where freezing temperatures occur during part of the year, precautions should be taken to prevent freezing of the drains. If the tunnel is long, protection against freezing need not be installed along the entire length of tunnel, depending on the climate and depth at which the tunnel is located.

(d) Drain and access tunnels. Drainage from these tunnels may require a sump and pump,

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depending on the location of the outlet end. Drain tunnels usually have drain holes that extend from the tunnel through the strata to be drained.

(e) Waterstop. To prevent uncontrolled water seepage into a concrete-lined tunnel, the construction joints are water stopped.

(2) Grouting. Recommendations are made below regarding special grouting treatment typically required to prevent drainage problems in various types of tunnels or shafts. Ring grouting (i.e., grouting through radial holes drilled into the rock at intervals around the tunnel periphery) is used to reduce the possibility of water percolating from the reservoir along the tunnel bore and for consolidation grouting along pressure tunnels. Contact grouting refers to the filling of voids between concrete and rock surface with grout.

(a) *Outlet works tunnels.* As a minimum, the crown of outlet works tunnels should be contact grouted for their entire length. Grouting to prevent water from percolating along the tunnel bore should consist of a minimum of one ring, interlocked with the embankment grout curtain. If the impervious core of the embankment extends upstream from the grout curtain and sufficient impervious material is available between the tunnel and the base of the embankment, the location near the upstream edge of the impervious core also should extend into the rock approximately one tunnel diameter.

(b) *Pressure tunnels.* Pressure tunnel linings are designed in two ways. Either the concrete and steel linings act together to resist the entire internal pressure or concrete and steel linings or the surrounding rock act together to resist the internal pressure. Contact and ring grouting for pressure tunnels is done the same as for outlet tunnels except one additional ring should be grouted at the upstream end of the steel liner. Consolidation grouting of the rock around the lining of a pressure tunnel and the filling of all voids is a necessity if the rock is to take part of the radial load. Consolidation grouting of the rock behind the steel liner is good practice and should be done whether or not the rock is assumed to resist a portion of the internal pressure.

(c) *Shafts*. Shafts are normally grouted the same as tunnels except that grouting is done completely around the shaft in all cases.

Construction of Final, Permanent Tunnel Linings

When the initial ground support components described in the previous sections do not fulfil the long-term functional requirements for the tunnel, a final lining is installed. On occasion, an initial ground support consisting of precast segments will also serve as the final lining. More typically, the final lining will be constructed of cast-in-place concrete, reinforced or unreinforced, or a steel lining surrounded by concrete or grout. The following subsections describe cast-in place concrete

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lining and steel lining construction.

a. *Cast-in-place concrete lining.* When a concrete lining is required, the type most commonly used is the cast-in-place lining. This lining provides a hydraulically smooth inside surface, is relatively watertight, and is usually cost competitive. Concrete linings can be of the following types:

- Unreinforced concrete.
- Concrete reinforced with one layer of steel, largely for crack control.
- Concrete reinforced with two layers of steel, for crack control and bending stresses.
- Unreinforced or reinforced concrete over full waterproofing membrane.

(1) *Placement sequence.* Depending on tunnel size and other factors, the entire cross section is placed at one time, or the invert is placed first, or the invert is placed last. Sometimes precast segments are placed in the invert to protect a sensitive rock from the effects of tunnel traffic, followed by placement of the crown concrete. This method will leave joints between the invert segments, but these joints can be designed for sealing or caulking. Barring construction logistics constraints, the most efficient method of placement is the full-circle concreting operation. When schedule or other constraints require that concrete be placed simultaneously with tunnel excavation and muck removal through the tunnel segment being concreted, then either the precast-invert segment method or the arch- first method is appropriate. Depending on the tunnel size, the upper 270 deg of a circular tunnel are placed first to permit construction traffic to flow uninterrupted and concurrently with lining placement. With the precast-invert segment method, the segment is made wide enough to permit all traffic operations. The invert-first placement method is not now frequently used for circular tunnels, in part because the invert takes time to cure and is subject to damage during placement of the crown. This method is sometimes advantageous when a waterproofing membrane is used. When the final lining is horseshoe-shaped, the invert is usually placed first, furnished with curbs to guide the placement of sidewall forms. Sometimes, especially in tunnels with ribs as initial ground support, L-shaped wall foundations are placed first; these will then guide the placement of the invert and the side walls.

(2) *Formwork.* Except for special shapes at turns and intersections, steel forms are used exclusively for tunnels of all sizes. The forms often come in widths of 1.5 to 1.8 m (5-6 ft), with provisions to add curve filler pieces to accommodate alignment radii. The segments are hinged and collapsible to permit stripping, transporting, and reerection, using special form carriers that ride on rails or rubber tires. The forms are usually equipped with external vibrators along with provisions to use internal vibrators through the inspection ports if necessary. Telescoping forms permit leapfrogging of forms for virtually continuous concrete placement.

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(3) *Concrete placement.* Placement is accomplished using either of two methods: the conventional slick line method and the multiport injection method.

(a) The slick line is a concrete placement pipe, 150 to 200 mm (6-8 in.) in size, placed in the crown from the open end of the form up to the previously placed concrete. Concrete is pumped into the form space until a sloping face of the fresh concrete is created in the form space. The slick line is gradually withdrawn, keeping the end of the pipe within the advancing fresh concrete. Minimum depth of pipe burial varies between 1 and 3 m (3-10 ft), depending on size of tunnel and thickness of lining. The advancement of the concrete is monitored through inspection ports and vibrated using form vibrators and internal vibrators.

(b) With the injection method, special injection ports are built into the form, through which concrete is placed using portable pumping equipment. Again, placement occurs in the direction from the previously placed concrete. Depending on the diameter of the tunnel, one to five injection ports may be located at any given cross section, with one port always at the crown. For large-diameter tunnels, and for reinforced linings, it is inadvisable to let the fresh concrete fall from the crown to the invert. Here, concrete must be placed through ports. Concrete forms are usually stripped within 12 hr of placement so as to permit placement of a full form length every day. Concrete must have achieved enough strength at this time to be self-supporting. Usually strength of about 8.3 MPa (1,200 psi) is sufficient.

(4) *Groundwater control during concreting.* Water seepage into the tunnel may damage fresh concrete before it sets. Side wall flow guides, piping, and invert drains may be used to control water temporarily. After completion of the lining, such drain facilities should be grouted tight. High-water flows may require damming or pumping, or both, to remove water before placing concrete. On occasion, formation grouting may be required.

(5) *Concrete conveyance.* The concrete is brought from the surface to the tunnel level either by pumping or through a drop pipe. If conveyed through a drop pipe, the concrete is remixed to eliminate separation. If the concrete is pumped, the pumping may continue through the tunnel all the way to the point of placement. Depending on the distance, booster pumps may be used. If possible, additional shafts are placed along the tunnel to reduce the distance of concrete conveyance in the tunnel. Other conveyance methods in the tunnel include conveyors, agitator cars, or nonagitated cars, trammed by locomotives to the point of placement. Remixing may be required, depending on the system used, to maintain the proper consistency of the fresh concrete. It is also possible at this location to add an accelerator if necessary. When conveying concrete for long distances, it is possible to add a retarder to maintain fluidity, then supplemented with an accelerator prior to placement.

(6) *Construction joints.* Transverse joints are located between pours, often 30 m (100 ft) apart

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or up to nearly 60 m (200 ft), depending on the form length used by the contractor. Either a sloping joint or a vertical joint can be used. Either type will result in a structurally acceptable joint. When a sloping joint is used, a low bulkhead is usually used to limit the feathering out of the concrete at the invert. The advantage of the sloping joint is that only a low bulkhead is required; this method is least likely to result in voids when using a slick line method. Disadvantages of the sloping joint include the following:

- Difficulty in proper preparation of joints before the next pour.
- Water stop placement not feasible.
- Underutilization of total length of the form.
- Formation of much longer construction joint, compared with the vertical joint.

The sloping joint is often more convenient when an unreinforced lining is constructed. The advantages of the vertical joint are accessibility of the joints for proper preparation, formation of the shortest possible length of joint, and full utilization of formwork. The vertical joint is most often used with reinforced concrete linings. Some of the disadvantages include the additional time required for bulkhead installation, provisions for maintaining reinforcing steel continuity across the joint, and the probability of forming voids when using the slick line method. From the perspective of water tightness, longitudinal joints resulting from the two-pour methods are not desirable. In particular, the arch-first method poses the greatest difficulty in joint surface treatment to achieve desired water tightness. Water-stops are not used for construction joints in unreinforced concrete linings. Water stops and expansion joints are of doubtful value in reinforced concrete linings but are sometimes used at special locations, such as at changes in shape of opening, intersections, and transitions to steel-lined tunnels.

(7) *Contact grouting.* When a tunnel lining has to withstand appreciable loads, external or internal, it is essential that the lining acts uniformly with the surrounding rock mass, providing uniformity of loading and ground reaction. Hence, significant voids cannot be tolerated. Voids are often the result of imperfect concrete placement in the crown. Voids are virtually unavoidable in blasted tunnels with irregular over break. It is therefore standard practice to perform contact grouting in the crown, using grout holes that have been either preplaced or drilled through the finished lining, so as to fill any crown voids that remain. Grouting is usually made to cover the upper 120 to 180 deg of circumference, depending on tunnel size and amount of over break.

(8) *Supplementary grouting and repair.* In the event that groundwater leaks excessively into the finished tunnel, formation grouting can be used to tighten the ground. This is done through radial grout holes through the lining. Leaking joints can also be repaired by grouting or epoxy treatment.

b. *Steel lining.* A steel lining is required when leakage through a cracked concrete lining

can result in hydro-fracturing of the surrounding rock mass or deleterious leakage or water loss. In most respects, the steel lining is similar to open-air penstocks, except that the tunnel steel lining is usually designed for an exterior water pressure and is furnished with external stiffeners for high external pressure conditions. Fabrication and assembly of a steel lining generally follow the same standards and practices as penstocks described in American Society of Civil Engineers (ASCE) (1993). Some construction aspects of steel-lined tunnels, however, deserve special attention, particularly as they affect the preparation of contract documents.

(1) Constructability. Individual pipes and joints are usually made as large as can be practically transported on the highway to the site and into the tunnel for placement and joining, leaving field welding to a minimum. Each motion through shafts, adits, and tunnel must be considered in the evaluation of the maximum size of the individual pieces.

(2) Handling and support. Pipes without external stiffeners should be internally supported during transport and installation if their diameter/thickness ratio, D/t, is less than 120. The internal bracing can be timber or steel stulling or spiders with adjustable rods. The minimum thickness of the steel shell is usually taken as $t_{min} = (D + 20)/400$, with dimensions in inches, or more simply $t_{min} = D/350$ (in millimeters). Externally coated pipes must be protected from damage to coating, using appropriate support and handling, e.g., fabric slings.

(3) Support during concrete placement. The pipe must be centrally aligned in the excavated tunnel and prevented from distortion and motion during concrete placement. This may require the pipe to be placed on cradles, usually of concrete, with tiedowns to hold the pipe in place against flotation and internal stulling. Steel or concrete blocking (not timber) is often used to resist flotation.

(4) Jointing. Welding procedures, including testing of welds, are similar to those of surface penstocks. It is often impractical to access the exterior of the pipe for welding and testing. An external backup ring, though less satisfactory, may be required. All welds should be tested using non-destructive testing methods using standards of acceptance similar to surface penstocks.

(5) Concrete placement. The tunnel must be properly prepared for concrete placement. Because the concrete must provide a firm contact between steel and ground, all loose rock and deleterious materials, including wood blocking, must be removed and groundwater inflow controlled as discussed in the previous subsection. Adequate clearances must be provided around the pipe. The concrete is usually placed using the slick line method. The concrete mix should be selected to minimize the build-up of heat due to hydration; subsequent cooling will result in the creation of a thin void around the pipe. Usually a relatively low strength (14 MPa, 2,000 psi, at 28 days) is adequate. Sloping cold joints are usually permissible.

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(6) Contact grouting. Grouting applications include the filling of all voids between concrete backfill and rock, which is termed contact grouting, and skin grouting of the thin void between steel lining and concrete. Contact grouting is often carried out through grout plugs provided in the pipe, located at the top and down 15 and 60 deg on each side to cover the upper 180 deg of installation. The grout plugs are spaced longitudinally every 3 m (10 ft), staggered, or between stiffeners if the pipe has external stiffeners. Grout holes are drilled through the predrilled holes in the steel plate, the concrete, and up to about 600 mm (2 ft) into the surrounding rock. The grout is a sand-cement mix, applied at pressures up to 0.7 MPa (100 psi).

(7) Skin grouting. The purpose of skin grouting is to fill the thin void that may exist between concrete and steel after the concrete cures. Theoretically, skin grouting is not required if a conservative value of the void thickness has been assumed in design, and a safe and economical structure can be achieved without skin grouting. If skin grouting is to be performed, it is usually according to the following procedure:

(a) After curing of the concrete (days or weeks), sound the steel for apparent voids and mark the voids on the steel surface.
(b) Drill 12- to 18-mm (0.5- to 0.75-in.) holes at the lower and the upper part of the voids.
(c) Grout with a flowable nonshrink grout, using the upper hole as a vent.
(d) After grout has set, plug holes with threaded plugs and cap with a welded stainless steel plate.

Ventilation of Tunnels and Shafts

Shaft and tunnel construction generally occurs in closed, dead-end spaces, and forced ventilation is essential to the safety of the works. Specifically, the Occupational Health and Safety Act (OSHA), applies to underground construction. Circumstances that may call for ventilation specification requirements include the following:

- An unusually long tunnel without intermediate ventilation shaft options.
- Certain potentially hazardous conditions, such as noxious or explosive gas occurrences, hot water inflow.
- Particularly extreme environmental conditions, such as very hot or very cold climatic conditions, where heating or cooling of air may be required.
- Circumstances where the ventilation system is left in place for use by a subsequent contractor or the owner; in these cases, the ventilation system should be designed almost as a part of the permanent system, rather than a temporary installation.

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a. *Purposes of underground ventilation.* Underground ventilation serves at least the following purposes:

- Supply of adequate quality air for workers.
- Dilution or removal of construction-generated fumes from equipment and blasting or of gases entering the tunnel.
- Cooling of air - heat sources include equipment, high temperature of in situ rock or groundwater, high ambient temperature.
- Heating of air - sometimes required to prevent creation of ice from seepage water or from saturated exhaust air.
- Smoke exhaust in the event of underground fire-dust control.

Thus, designers of an underground ventilation system must consider the ambient and in situ temperatures, projected water inflow, potential for adverse conditions (gases), maximum number of personnel in the underground, types and number of equipment working underground, and methods of equipment cooling employed. In the permanent structure, ventilation provisions may be required for at least the following purposes:

- To bleed off air at high points of the alignment.
- To purge air entrained in the water, resulting, for example, from aeration in a drop shaft.
- For odor control and dilution of sulfide fumes in a sewer tunnel.
- To provide ventilation for personnel during inspection of empty tunnels.

These ventilation requirements often result in the use of separate permanent ventilation shafts with appropriate covers and valves.

b. *Components of ventilation system.* The principal components of a ventilation system are briefly listed below:

(1) Fans. Usually in-line axial or centrifugal fans are used. Fans can be very noisy, and silencers are usually installed. In a sensitive neighborhood, silencers are particularly important; alternatively, fans can be installed a sufficient distance away from the tunnel or shaft portals to reduce noise levels. Fans are designed to deliver a calculated airflow volume at a calculated pressure. With long vent lines, the required pressure may be too high for effective fan operation at one location (air leakage from vent lines also increase with increased differential pressure), and booster fans along the line are used. In the working areas, auxiliary fan installations are often required for dust control, ventilation of ancillary spaces, local air cooling, removal of gases or fumes, or other special services. When auxiliary fan systems are used, such systems shall minimize recirculation and provide ventilation that effectively sweeps the working places. Reversibility of

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fans is required to permit ventilation control for exhaust of smoke in case of fire.

(2) Fan lines. Rigid-wall fan lines made of steel ducting or fiberglass are sometimes used, mostly for exhaust; however, flexible ducting, made of flame retardant material, is more commonly used. Flexible ducting must retain an internal overpressure in order not to collapse. This requires reliable fan start control of all main and booster fans.

(3) Scrubbers. Excessive dust is generated from roadheader or TBM operation and is usually exhausted through scrubbers or dust collectors.

(4) Ancillary ventilation structures. These may include stoppings and brattices to isolate areas with different ventilation requirements or where no ventilation is required. In hot environments, cooling can be applied to the entire ventilation system, or spot coolers can be applied to working areas. Heaters can be required to prevent ice from forming at exhausts.

(5) Monitors and controls. These include air pressure and air flow monitors within the ducting or outside, monitoring of gases (methane, oxygen, carbon monoxide, radon, and others), temperature, humidity, and fan operation status. Stationary gas detectors located at strategic points in the ventilation system and at the face (e.g., mounted on the TBM) are often supplemented with hand-held detectors or sampling bottles. Signals would be monitored at the ventilation control center, usually at the ground surface, where all ventilation controls would be operated. Secondary monitors are often installed at the working area underground.

Fire Hazards in Tunnels

There are particular hazards with tunnels, especially from vehicle fires when combustion gases can asphyxiate users. The following are some of the notable tunnel fires in the last few decades:

- Isola della Femmine, Italy 1996, 5 dead, 20 injured.
- Mont Blanc Tunnel, France/Italy, March 1999, 39 dead.
- Tauern Tunnel, Austria, May 1999, 12 dead.
- Kaprun Funicular Tunnel, Austria, in November 2000, 155 dead.
- Gotthard, Switzerland, October 2001, 11 dead.
- Frejus, France/Italy, June 2005, 2 dead.
- Channel Tunnel, France/UK, November 1996 (no fatalities but a freight train along with all 27 trucks on board were destroyed. In addition, portions of the tunnel ceiling's concrete became disintegrated under intense heat.)

These tunnel fires have caused public concern regarding the safety of our road network. This public concern is no doubt in part due to the scale of the losses, both in human life, property and

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consequential losses to business and transport systems. It may also in part be due to the nature of tunnels; the risks associated with transportation are put into one of the worst environments for a fire – underground and potentially many miles from safety.

Tunnel fires involve all aspects of fire safety science, engineering and management. In general, the two main fire related areas of concern for tunnel engineers have been structural safety and ventilation, in particular smoke ventilation.

The structural fire protection in a tunnel serves a number of purposes, which includes protection of the means of escape and/or safe havens (refuges), assisting (or protecting) rescue efforts, avoiding major business interruption, and seeking to avoid complete or partial loss of the tunnel. The provision of refuges and/or escape routes can be costly and difficult. They may increase the section of a tunnel and introduce new engineering challenges. The combination of fire, risk assessment and ventilation must in particular consider and develop solutions which are cost effective and fully justifiable on a rational basis and not just from a perception of horizontal danger.

Similarly, the provision of tunnel equipment is an essential part of the fire safety system, and will include protecting or assisting the means of escape, with lighting, emergency lighting and communications, power for essential safety systems, and provision of fire-fighting facilities.

Careful planning, in advance of any formal design, is becoming increasingly important. In selecting the fire safety philosophy and designing the fire safety systems for a tunnel there are a range of aspects to consider.

The following are some of the strategies which have been used in other tunnel operations:

- Constructing a service tunnel for evacuation purpose. The Channel Tunnel has three tubes, one of which is the service tunnel for escape and rescue.
- Prohibiting trucks carrying highly flammable goods from entering the tunnel. Most long tunnels prohibit trucks carrying hazardous materials (HAZMAT) such as toxic chemicals, fuels, nuclear waste products, and biological, chemical, and radiological agents. Examples include Lincoln Tunnel connecting New Jersey to New York City and Fort McHenry Tunnel in Baltimore.
- Monitoring the heat emitted by vehicles through 3-D thermal imaging. The safety technology in the Gotthard Base Tunnel in Switzerland includes an integrated fire detection system with thermal imaging cameras, smoke detectors and a heat detection system.
- Controlling the number of trucks entering the tunnel within a specified time period.
- Replacing road tunnels with rail tunnels to reduce the fire hazards.

There are a number of management issues which can have a significant impact on the outcome of

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an incident, and need to be carefully considered at the design stage, at the initial opening of a tunnel, and maintained during its operational lifetime. These include:

- Arranging effective communications between relevant agencies and emergency services.
- Arranging for detection and alarm provisions to be continually checked and reassessed once the tunnel is operation.
- Providing for communication and control systems to be routinely tested under 'realistic' 'emergency' conditions, and ensuring that key personnel are not overloaded.
- Ensuring that all safety equipment is subject to routine maintenance, testing and repair.

Fire safety engineers need to ensure that they identify all of the management implications, implicit and explicit, in their designs. They should consult fire safety managers, or representatives of eventual fire safety managers, from the earliest stage of the design process, and ensure that the concerns of these managers are taken into account.

The Future of Tunneling

As their tools improve, engineers continue to build longer and bigger tunnels. Recently, advanced imaging technology has been available to scan the inside of the earth by computing how sound waves travel through the ground. This new tool provides an accurate snapshot of a tunnel's potential environment, showing rock and soil types, as well as geologic anomalies such as faults and fissures.

While such technology promises to improve tunnel planning, other advances will expedite excavation and ground support. The next generation of tunnel-boring machines will be able to cut 1,600 tons of muck per hour. Engineers are also experimenting with other rock-cutting methods that take advantage of high-pressure water jets, lasers or ultrasonics. And chemical engineers are working on new types of concrete that harden faster because they use resins and other polymers instead of cement.

With new technologies and techniques, tunnels that seemed impossible even 10 years ago suddenly seem doable. One such tunnel is a proposed Transatlantic Tunnel connecting New York with London. The 3,100-mile-long tunnel would house a magnetically-levitated train traveling 5,000 miles per hour. The estimated trip time is 54 minutes - almost seven hours shorter than an average transatlantic flight.

List of Longest Tunnels

The table on next page lists the top 20 longest tunnels in the world as of June 30, 2020. The longest tunnels have been constructed for water distribution, followed by tunnels for railways.

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World's Longest Tunnels (in use) (Source: Wikipedia.org)

Type	Name	Location	Length	Year	Comment
Water supply	Delaware Aqueduct	New York State, United States	137,000 m (85.1 mi)	1945	4.1 m in diameter (13.2 m^2). New York City's main water supply tunnel.
Water supply	Päijänne Water Tunnel	Southern Finland, Finland	120,000 m (74.6 mi)	1982	16 m^2 cross section. Main water supply tunnel for southern Finland, including Helsinki, drilled through solid rock.
Water supply	Dahuofang Water Tunnel	Liaoning Province, China	85,320 m (53.0 mi)	2009	8 m in diameter (50m^2 cross section)
Water supply	Orange–Fish River Tunnel	South Africa	82,800 m (51.4 mi)	1972	Longest continuous enclosed aqueduct in the Southern Hemisphere (22.5 m^2 cross section). Built to divert water from the Orange River to the Great Fish River.
Water supply	Bolmen Water Tunnel	Kronoberg/ Scania, Sweden	82,000 m (51.0 mi)	1987	8 m^2 cross section
Hydro-electric	Neelum Jhelum HydroPower Tunnel	Muzaffarabad, Azad Kashmir Pakistan	68,000 m (42.3 mi)	2017	Part of the 969 MW Neelum Jhelum Hydropower Plant
Waste-water	Tunel Emisor Oriente	Mexico City, Mexico	62,500 m (38.8 mi)	2008-2014	Water management in Greater Mexico City. Longest wastewater tunnel.
Metro	Guangzhou Metro Line 3	Guangzhou, China	60,400 m (37.5 mi) Excl. branch	2005-2010	Guangzhou Metro. Longest metro/rapid transit tunnel

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Type	Name	Location	Length	Year	Comment
Railway Twin Tube	Gotthard Base Tunnel	Central Swiss Alps, Switzerland	57,104 m (35.5 mi) and 57,017 m (35.4 mi)	2016	Longest railway tunnel; by geodetic distance (of 55.782 km (34.661 mi)) between the two portals, it is also the world's longest transit tunnel. Total 151.84 km (94.35 mi) of broken out tunnels through solid rocks. Part of the NRLA.
Metro	Beijing Subway Line 10	Beijing, China	57,100 m (35.5 mi)	2008-2012	Beijing Subway
Railway Single Tube	Seikan Tunnel	Tsugaru Strait, Japan	53,850 m (33.5 mi)	1988	74 m^2; longest railway tunnel until 2016. Longest tunnel with an undersea section, running between Honshu and Hokkaido. Undersea section measures 23.3 kilometers (14.5 mi).
Water supply	Želivka Water Tunnel	Central Bohemian Region, Czech Republic	51,075 m (31.7 mi)	1972	5 m^2
Railway Twin Tube	Channel Tunnel	English Channel, United Kingdom/France	50,450 m (31.3 mi)	1994	Second longest railway tunnel until 2016. Longest underwater section, longest international tunnel (2×45 m^2 + 1×18 m^2), running between Folkestone, Kent,and Coquelles, Pas-de-Calais.
Railway Single Tube	Yulhyeon Tunnel	Seoul Capital Area, South Korea	50,300 m (31.3 mi)	2016	107 m^2, part of the Suseo high-speed railway.

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Type	Name	Location	Length	Year	Comment
Water supply	River Arpa – Lake Sevan tunnel	Armenia (at the time of construction Soviet Union)	48,314 m (30.0 mi)	1981	Longest tunnel built to restore ecological balance. Transfers water from Arpa River to the Lake Sevan to recover its level.
Metro	Seoul Subway: Line 5	Seoul, South Korea	47,600 m (29.6 mi)	1995	Longest metro/rapid transit tunnel until Guangzhou Metro Line 3 extension opened in 2010, crosses west to east under the Seoul Capital Area.
Water supply	Pahang-Selangor Raw Water Transfer Project	Pahang & Selangor, Malaysia	44,600 m (27.7 mi)	2014	5.2 meters in diameter
Metro	Mass Rapid Transit: Downtown Line	Singapore, Singapore	43,700 m (27.2 mi)	2017	Longest rapid transit line in South East Asia
Water supply	#1 Tunnel, Yellow River Diversion to Shanxi North Line	Shanxi, China	43,670 m (27.1 mi)	2011	See South–North Water Transfer Project
Water supply	#7 Tunnel, Yellow River Diversion to Shanxi South Line	Shanxi, China	43,500 m (27.0 mi)	2002	See South–North Water Transfer Project



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