Design and Construction of Road Tunnels: Part 3 Design, and Detailing

Five (5) Continuing Education Hours
Course #CV7053

Approved Continuing Education for Licensed Professional Engineers

EZ-pdh.com
Ezekiel Enterprises, LLC
301 Mission Dr. Unit 571
New Smyrna Beach, FL 32170
800-433-1487
helpdesk@ezpdh.com

Course Description:

The Design and Construction of Road Tunnels: Part 3 History, Design, and Detailing course satisfies five (5) hours of professional development.

The course is designed as a distance learning course that enables the practicing professional engineer to further understand road tunnels, tunnel types, and excavation methods.

Objectives:

The primary objective of this course is enable the student to understand the concepts of the sequential excavation method as well as the design, purpose and construction of tunnel linings, immersed tunnels, and jacked box tunnels.

Grading:

Students must achieve a minimum score of 70% on the online quiz to pass this course. The quiz may be taken as many times as necessary to successful pass and complete the course.

A copy of the quiz questions are attached to last pages of this document.

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CHAPTER 9
SEQUENTIAL EXCAVATION METHOD (SEM)

9.1 INTRODUCTION

The Sequential Excavation Method (SEM), also commonly referred to as the New Austrian Tunneling Method (NATM), is a concept that is based on the understanding of the behavior of the ground as it reacts to the creation of an underground opening. In its classic form the SEM/NATM attempts to mobilize the self-supporting capability of the ground to an optimum thus achieving economy in ground support. Building on this idea practical risk management and safety requirements add to and dictate the required tunnel support. Initially formulated for application in rock tunneling in the early 1960's, NATM has found application in soft ground in urban tunneling in the late 60's and has since then enjoyed a broad, international utilization in both rural and urban settings.

A large number of tunnels have been built around the world using a construction approach which was loosely termed NATM. During the years of discussions and the application of NATM a variety of terms have been used for the same construction approach. These terms were primarily aimed at describing the construction approach rather than the region of its reported origin. While in the 70's and early 80's the term "Shotcrete Method" was frequently used in Germany and Switzerland, besides NATM, developments in the UK in the late 90's led to the use of the term "Sprayed Concrete Lining" or SCL. Alternatively, "Conventional Tunneling Method" was used in Austria and Germany. As the NATM is largely based on an observational approach, the term "Observational Method" was introduced and used in many countries. The term "Conventional" as opposed to TBM driven tunnels has recently found its way into publications by the International Tunneling Association's (ITA) Working Group 19. In the German speaking countries in Europe namely in Austria and Germany very recent standards and codes use the term "Cyclic Tunneling Method."

In the US, where NATM was systematically applied for the first time in the late 70's and early 80's for the construction of the Mount Lebanon tunnel in Pittsburgh and the Redline tunnels and Wheaton Station of the Washington, DC metro the term adopted was NATM. Gradually, however, the term has been and is being abandoned in the US and replaced by Sequential Excavation Method or SEM. Today, SEM is becoming increasingly popular in the US for the construction of tunnels, cross passages, stations, shafts and other underground structures (Gildner et. al., 2004).

The SEM offers flexibility in geometry such that it can accommodate almost any size of opening. The regular cross section involves generally an ovoid shape to promote smooth stress redistribution in the ground around the newly created opening. By adjusting the construction sequence expressed mainly in round length, timing of support installation and type of support it allows for tunneling through rock (Chapter 6), soft ground (Chapter 7) and a variety of difficult ground conditions (Chapter 8). Depending on the size of the opening and quality of the ground a tunnel cross section may be subdivided into multiple drifts.

Application of the SEM involves practical experience, earth and engineering sciences and skilled execution. The SEM tunneling process addresses:

- Ground and excavation and support classification based on a thorough ground investigation
- Definition of excavation and support classes by:
    - Round length (maximum unsupported excavation length)
    - Support measures (shotcrete lining and its reinforcement, ground reinforcement by bolts or dowels in rock)
    - Subdivision of the tunnel cross section into multiple drifts or headings as needed (top heading, bench, invert, side wall drifts)
    - Ring closure requirements
    - Timing of support installation (typically every round)
    - Pre-support by spiling, fore poling, and pipe arch canopy
    - Local, additional initial support by dowels, bolts, spiles, face support wedge, and shotcrete
- Instrumentation and monitoring
- Ground improvement measures prior to tunneling

A key support element is shotcrete mainly due to its capability to provide an interlocking, continuous support to the ground. Implementation of ground improvement measures in the form of dewatering, grouting, ground freezing and others and of pre-support measures in the various forms of spiling have further widened the range of SEM applications mainly in urban tunneling. These are specified to enhance the quality of the ground prior to and during the tunneling process. The SEM features typically a dual lining cross section by which a waterproofing membrane is inserted between an initial shotcrete and a final, typically cast-in-place concrete lining as addressed in Chapter 10 Tunnel Lining. An instrumental element of SEM tunneling is the monitoring of deformations of tunnel and surrounding ground (Chapter 15). Evaluation of monitoring allows for the verification of design assumptions or adjustment of the tunneling process.

Lastly, because SEM tunneling allows for an adjustment to ground conditions as encountered in the field it benefits from a unit-price contract form. Geotechnical baseline reports (GBR) as discussed in Chapter 4 and Geotechnical Design Summary Reports (GDSR) facilitate the adjustment process and aid in risk sharing between owner and contractor. Geotechnical investigations are discussed in Chapter 3.

9.2 BACKGROUND AND CONCEPTS

The origins of the NATM lie in the alpine tunnel engineering in the early 1960s. In 1948, Ladislaus von Rabcewicz applied for a patent for the use of a dual lining system with the initial lining being allowed to deform. The NATM is based on the philosophy that the ground surrounding the tunnel is used as an integrated part of the tunnel support system. The deformable shotcrete initial lining allows a controlled ground deflection to mobilize the inherent shear strength in the ground and to initiate load redistribution. The key for the successful use of a relatively thin lining layer applied to the excavation surface lies in the smooth tunnel shape to avoid stress concentrations and the tight contact between the shotcrete lining and the surrounding ground to provide an intense interaction between the support and the ground. In order to augment the support provided by the initial lining, rock reinforcement is used in response to the rock mass conditions. The rock reinforcement avoids the development of wedge failure (keystone), and it generates a rock mass ring with significantly improved strength characteristics around the opening.

Smooth, concavely rounded excavation surfaces initiate confinement forces and limit bending and tension forces in the lining and the ground in the vicinity of the tunnel opening. This is of particular importance for tunneling in ground with limited stand-up time, where fracturing and weathering reduce the ground's natural shear strength.

NATM was the first concept, where the ground and its strength were used as a building material and became an integrated part of the tunnel support system. Rather than implementing stiff support members that attract high loads to fight the ground deformation, the flexible, yet strong shotcrete lining shares with and re-distributes loads into the ground by its deflection.

Rabcewicz summarizes the philosophy of NATM in his patent of 1948 (Rabcewicz, 1948) as follows: "NATM is based on the principle that it is desirable to take utmost advantage of the capacity of the rock to support itself, by carefully and deliberately controlling the forces in the readjustment process which takes place in the surrounding rock after a cavity has been made, and to adapt the chosen support accordingly." By briefly reviewing the stress conditions around a newly created cavity and the interaction between ground and its support needs the following lays out the principle approach taken in NATM tunnel design (Rabcewicz et al., 1973).

The stress conditions around a cavity after Kastner are schematically provided in Figure 9-1. The primary stress σ_0 in the surrounding ground before any cavity is created depends mainly on the overburden, the unit weight and any tectonic stresses σ_s. Following tunnel excavation the tangential stresses will increase next to the tunnel circumference (solid line σ_t^0). If the induced tangential and radial stresses (σ_t and σ_r) around the tunnel opening exceed the strength of the surrounding ground yielding will occur. Such yielding will create a plastified zone that reaches to a certain distance R into the ground beyond the tunnel circumference (dashed line R).

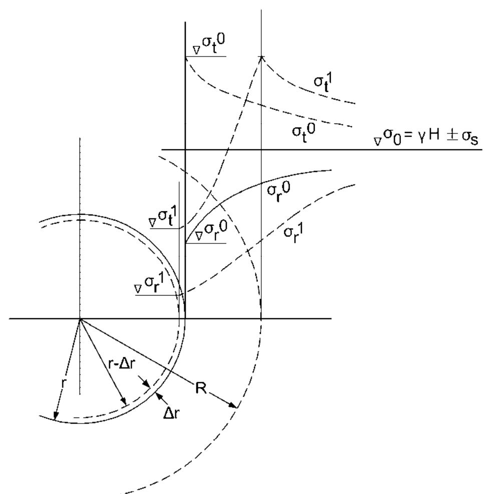


Figure 9-1 Schematic Representation of Stresses Around Tunnel Opening (Rabcewicz et al., 1973)

A schematic illustration of the relationships between the radial stresses σ_r , deformation of the tunnel opening Δr, the required outer and inner supports $p_i{}^a$ and $p_i{}^I$ respectively, and the time of support application T is provided in Figure 9-2. According to Rabcewicz the outer support or outer arch ($p_i{}^a$) involves the ground itself, its reinforcement by rock bolts and any support applied to the opening itself ranging from sealing shotcrete (flashcrete) to a structural initial lining involving reinforced shotcrete or concrete and steel ribs. The inner support involves a secondary lining that is applied after the tunnel opening with the help of the outer arch has reached equilibrium. The σ_r /Δr curve, often referred to as ground reaction curve schematically describes the relationship between deformation of the tunnel opening and tunnel support provided by the outer arch. At any intersection point between the support p_i and the σ_r

curve equilibrium is reached for the respective support. It is characteristic for the NATM that the intersection between the support and the σ_r curve takes place at the descending side of the curve. Undesirable loosening of the ground starts at point B of the σ_r curve if the minimum support p_i min is not provided. Within the ascending side of the σ_r curve the ground has lost strength and consequently its supporting capacity and thus requires enhanced tunnel support to passively support the overburden.

Examination of curves Figure 9-1 and Figure 9-2 exemplifies the relationship between timing of support installation, yielding of the ground and the amount of support needed. The minimum support is required at point B to prevent loosening and loss of strength in the surrounding ground. It will result in the largest deformations but the most economical tunnel support. Curve 1 which intersect the σ_r curve in point A will require enhanced support $p_i^{\ a}$ but result in less deformation Δr and a higher factor of safety. Selection of a stiffer outer arch in curve 2 will result in more support loads because the ground has not been allowed to deform and mobilize its strength and consequently led to a decrease of the safety factor.

The capacity of the inner arch is chosen to satisfy a desired safety factor s. This will depend on specific needs and assuming that the initial tunnel support (outer arch) will deteriorate over time then $p_i^{\ a}$ may be used as guidance to arrive at a desired safety factor. C and C' denote a loaded and unloaded condition of the inner arch respectively.

The σ_r /Δr curve may be approximated by means of numerical modeling using the deformation and strength characteristics of the ground along with the specific geometry of the opening and the envisioned excavation sequencing (Rabcewicz, 1973).

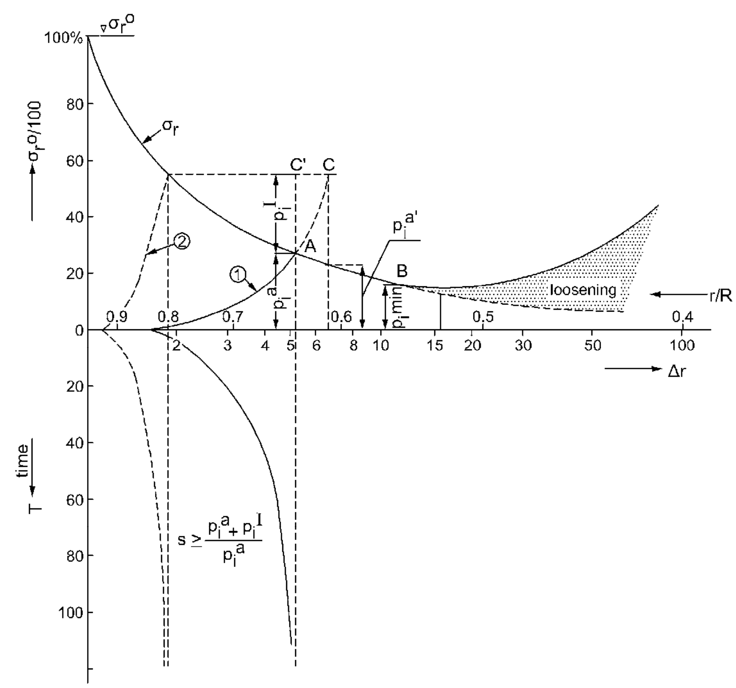


Figure 9-2 Schematic Representation of Relationships Between Radial Stress σ_r, Deformation of the Tunnel Opening Δr, Supports p_i, and Time of Support Installation T (Rabcewicz et al., 1973)

While the NATM had its origins in alpine tunneling in fractured or squeezing rock, its field of application expanded dramatically in the 70's and the following decades. The superb flexibility of the construction concept to adapt to a wide range of ground conditions and tunnel shapes in combination with significant developments in construction materials, installation techniques as well as ground treatment methods formed the basis for a radical expansion from the alpine rock tunneling into soft ground tunneling. The use of NATM thus expanded from rural tunneling in rock into urban tunneling in predominantly soft ground and highly built-up environments with sensitive structures above the tunnel alignment.

The major focus in rock tunneling in rural settings is to find equilibrium in the surrounding ground with the best possible economy in the amount of initial support installed. In urban settings however, in particular when tunneling at shallow overburden depths in soft ground the main goal is to minimize the impact on the surface and adjacent structures thus to minimize settlements. As shown in Figure 9-2 less and delayed support installation will be associated with larger deformations of the tunnel Δr and consequently with larger surface settlements when tunneling at shallow depth. Curve No. 1 describes a relatively "soft" support that is applied later than the support represented by curve No. 2, which is applied earlier and is "stiffer." The curves point out that in order to reduce settlements generally an early and stiffer support should be used. Reduction of the round length and subdivision of the tunnel cross section will aid in applying support to the ground early thus reducing deformations. The stiffness of the support can be increased by increasing the initial shotcrete lining thickness and using shotcrete with early and high strength development.

Today's tunnel construction economies require tunneling approaches that are competitive to fully mechanized tunneling methods by TBMs with their high initial capital cost while being adjustable to project specific space demands. The main field of SEM application is, apart from rural railway and highway tunnels, in the construction of tunnel schemes with complex geometries, short tunnels, large size tunnels and caverns in urban areas at shallow depths. Shallow tunnel depths frequently involve the challenge of soft ground tunneling. With the help of modern equipment for rapid excavation, modern high quality construction materials (mainly shotcrete), and modern ground support installation techniques as well as the overarching SEM concept, complex and challenging underground structures can be built in practically all types of ground. A major advantage of the SEM is its flexibility.

9.3 SEM REGULAR CROSS SECTION

9.3.1 Geometry

The shape of the tunnel cross section is designed to comply with SEM principles, which are to (as effectively as possible) activate the self-supporting arch in the surrounding ground. To accommodate this principle cross section geometries shall be curvilinear, consisting of compound curves in both arch and invert (if constructed in soft ground like conditions). Any straight walls and sharp edges in the excavation cross section shall be avoided. Thus the geometry of the excavation cross section will enable a smooth flow of stresses in the ground around the opening, minimizing loads acting on the tunnel linings. While adhering to these principles the excavation cross section shall be optimized in size to achieve economy. The layout of the invert will depend on the ground conditions in which the tunnel is constructed. In competent rock formations the tunnel invert will be flat, whereas in weak rock and soft ground tunnels the invert will be rounded to facilitate ring closure and stability.

9.3.2 Dual Lining

The SEM regular cross section is of dual lining character and consists of an initial shotcrete lining and a final, cast-in-place concrete or shotcrete lining. A waterproofing system is sandwiched between the initial and final linings. The waterproofing system consists of a flexible, continuous membrane (typically PVC). A regular cross section is developed for each tunnel geometry: the main tunnel, widenings, niches, cross passages, and other miscellaneous structures. A typical regular SEM cross-section for a two-lane highway tunnel is shown in Figure 9-3 distinguishing between a rounded (right side) and flat (left side) invert. A rounded invert is typically associated with tunneling in soft ground whereas a flat invert is used in competent ground conditions, typically rock. As discussed in Chapter 2, the tunnel cross section is designed around the project clearance envelope including tolerances. Figure 9-4 displays a completed SEM tunnel section for a three-lane road tunnel showing rounded cast-in-place concrete tunnel walls. The alignment of the tunnel is curved to accommodate alignment needs of an urban environment. In the front the SEM tunnel abuts a straight tunnel wall of an adjoining cut-and-cover box tunnel. Figure 9-4 also displays tunnel installations including lighting and jet fans for tunnel ventilation.

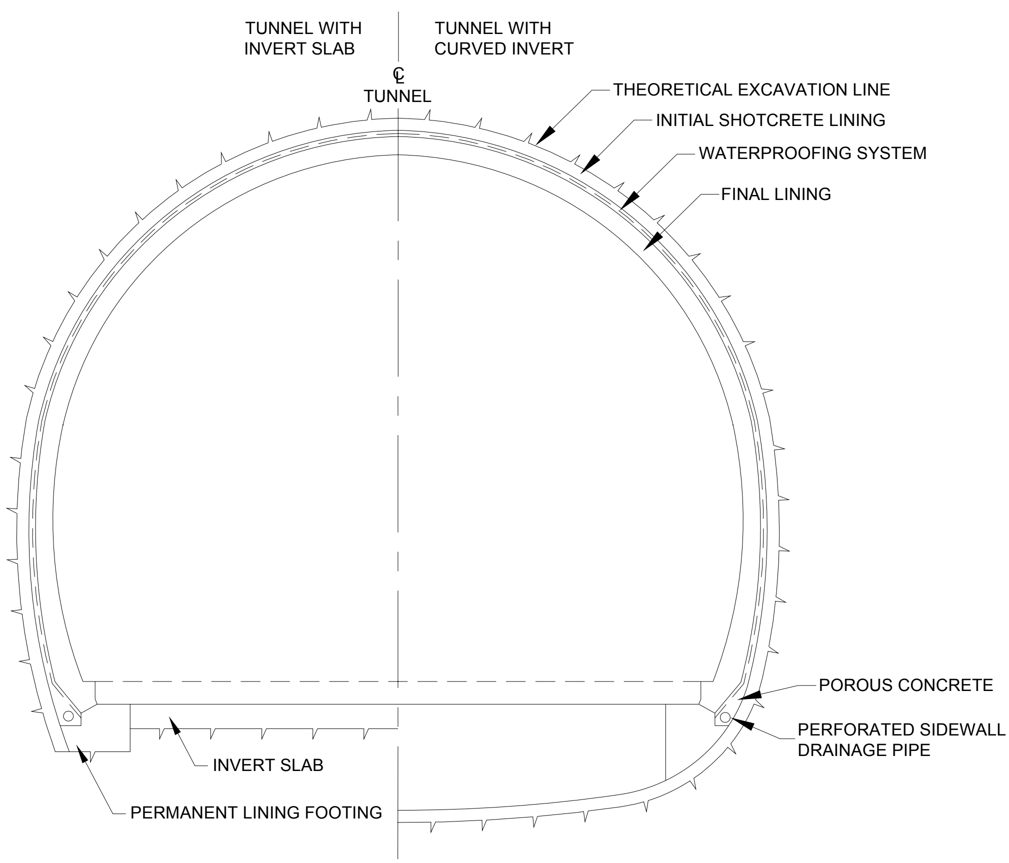


Figure 9-3 Regular SEM Cross Section

Figure 9-4 Three-Lane SEM Road Tunnel Interior Configuration (Fort Canning Tunnel, Singapore)

9.3.3 Initial Shotcrete Lining

The initial shotcrete lining is the layer of shotcrete applied to support the ground following excavation. It has a thickness ranging generally from 4 to 16 inches (100 to 400 mm) mainly depending on the ground conditions and size of the tunnel opening. It is reinforced by either welded wire fabric or steel fibers; the latter have generally replaced the traditional welded wire fabric over the last ten to fifteen years.

Occasionally structural plastic fibers are used in lieu of steel fibers. This is the case where the shotcrete lining is expected to undergo high deformations and ductility post cracking is of importance. Where the shotcrete lining is greater than about 6 inches (150 mm) it further includes lattice girders. Depending on loading conditions and purpose rolled steel sets may replace lattice girders or act in combination.

9.3.4 Waterproofing

The SEM uses flexible, continuous membranes for tunnel waterproofing. Most frequently PVC membranes are used at thicknesses of 80 to 120 mil (2.0 to 3.0 mm) depending on the size of the tunnel. Only in special circumstances, for example when contaminated ground water is present, special membranes are applied using hydrocarbon resistant polyolefin or very light density polyethylene (VLDPE) membranes.

The impermeable membrane is backed by a geotextile that also acts as a protection layer, and in drained systems as a drainage layer behind the membrane. This waterproofing system is placed against the initial lining and prior to installation of the final lining. Prior to waterproofing system installation all tunnel deformations must have ceased.

In drained system applications water is collected behind the membrane and conducted to perforated sidewall drainage pipes located at tunnel invert elevation on each side of the tunnel. From there collected water is conveyed via transverse, non-perforated pipes to the tunnel's main roadway drain. In undrained systems the membrane and geotextile wrap around the entire tunnel envelope and prevent water seepage into the tunnel thereby subjecting it to hydrostatic pressures. If this is the case the tunnel invert geometry and structural design must be adapted to accommodate for the hydrostatic head. Utilization of drained vs. undrained systems is discussed in Chapter 1.

Over the past decades a so called "compartmentalization system" has been developed and nowadays supplements the installation of flexible membrane based waterproofing systems. The purpose of this compartmentalization is to provide repair capability in case of leakage. In particular, when the tunnel is not drained and the waterproofing has to withstand long-term hydrostatic pressures, installation of such systems provides a cost effective back up and assures a dry tunnel interior. Compartmentalization refers to the concept of subdividing the waterproofing membrane into individual areas of self-contained grids (compartments) by means of base seal water barriers. These water barriers are specifically formulated for the purpose of creating these compartments. They feature ribs of 1.3-inch (30 mm) minimum height to properly key into the final lining, which is cast (or sprayed) against the waterproofing. In case of water leakage the water infiltration is limited to the individual compartment thus preventing uncontrolled water migration over long distances behind the final lining. Within each compartment control and grouting pipes are installed. These pipes penetrate through the final lining and are in contact with the membrane. Figure 9-5 displays an installed PVC waterproofing system with compartments, control and grouting pipes, and hoses prior to final lining installation. Control and grouting pipes serve a twofold purpose; should leakage occur then water would find its path to these pipes and exit there thus signaling a breach within the compartment. Once detected, the same pipes may be used for injection of low viscosity, typically hydro-active grouts into the compartments. The injection of grout is limited to leaking compartment(s) and once cured provides a secondary waterproofing layer in the form of a membrane that acts as a remedial waterproofing layer.

Figure 9-5 Waterproofing System and Compartmentalization (Automated People Mover System at Dulles International Airport, Virginia)

9.3.4.1 Smoothness Criteria

To provide a suitable surface for the installation of the waterproofing system, all shotcrete surfaces to which the membrane is to be applied must meet certain smoothness criteria. These are expressed in the waviness of the shotcrete surface to which the waterproofing system will be applied. The waviness is measured with a straight edge laid on the surface in the longitudinal direction. The maximum depth to wavelength ratio should be generally 1:5 or smoother. The surface has to be inspected prior to installation of the waterproofing system and all projections should be removed or covered by an additional plain shotcrete layer, which meets the smoothness criteria. The SEM design documents will address required smoothness criteria and set those in relation to the waterproofing system to be used.

9.3.5 Final Tunnel Lining

The final permanent lining for a SEM tunnel may consist of cast-in-place concrete or shotcrete. Cast-in-place concrete can be un-reinforced or reinforced. Shotcrete is generally fiber reinforced. Chapter 10 provides general discussions about permanent tunnel lining. The following addresses design and construction considerations specifically for SEM application.

9.3.5.1 Cast-in-Place Concrete Final Lining

The traditional final lining consists of cast-in-place concrete at a thickness of generally 12 inches for two-lane road tunnels. While the lining may generally remain unreinforced, structural design considerations and project design criteria will dictate the need for and amount of reinforcement. The Lehigh Tunnel (Pennsylvania) and Cumberland Gap Tunnels (Kentucky / Tennessee) are the first road tunnels built in the US in the late 80’s and early 90’s using SEM construction methods. Both feature unreinforced, 12-inch thick cast-in-place concrete final linings. The flexible membrane based waterproofing is in particular beneficial in unreinforced cast-in-place concrete lining applications in that it acts as a de-bonding layer between the initial and final linings and therefore reduces shrinkage cracking in the final lining.

To ensure a contact between the initial and final linings, contact grouting is performed as early as the final lining has achieved its 28-day design strength. With this grouting the contact is established between the initial lining and final tunnel support. Any deterioration or weakening of the initial support will lead to an increased loading of the final support by the increment not being supported by the initial lining. The loads can be directly transferred radially due to the direct contact between initial and final linings.

Cast-in-place final concrete linings (concrete arch placed on sidewall footings) are frequently installed in pour lengths not exceeding 30 feet (10 meters). This restriction is important to limit surface cracking in general and becomes mandatory if unreinforced concrete linings are used. A 30 feet (10 meter) long section in a typical two-lane highway tunnel is also practical in terms of formwork installation and sequencing and duration of concrete placement.

Adjacent concrete pours feature construction joints that are true lining separators designed as contraction joints. The inside face at joint location shall be laid out with a trapezoidally shaped joint. A continuous reinforcement is not desired in construction joints to allow their relative movement in particular for thermal deformation effects.

9.3.5.2 Water Impermeable Concrete Final Lining

Use of water impermeable cast-in-place concrete linings as an alternative to membranes is generally not considered due to the high demands on construction quality and exposure to freeze thaw conditions in cold climates. Elaborate measures are needed to prevent cracking. Detailed arrangement of construction joints is needed as well as complex concrete mix designs to suppress excessive hydration heat. The curing requires elaborate procedures. These aspects generally do not render water impermeable concrete practical in road tunnels. If selected these construction aspects have to be addressed in detail in specifications and working procedures and they have to be rigidly enforced.

9.3.5.3 Shotcrete Final Lining

Shotcrete represents a structurally and qualitatively equal alternative to cast-in-place concrete linings. When shotcrete is utilized as a final lining in dual lining applications it will be applied against a waterproofing membrane. The lining thickness will be generally 12 inches (300 mm) or more and its application must be carried out in layers with a time lag between layer applications to allow for shotcrete setting and hardening. Its surface appearance can be tailored to the desired project goals. It may remain of a rough, sprayer type shotcrete finish, but may have a quality comparable to cast concrete when trowel finish is specified. Shotcrete as a final lining is typically utilized when the following conditions are encountered:

- The tunnels are relatively short in length and the cross section is relatively large and therefore investment in formwork is not warranted, i.e. tunnels of less than 100-250 m (300-800 feet) in length and larger than about 8-12 m (25-40 feet) in springline diameter.
- The access is difficult and staging of formwork installation and concrete delivery is problematic.
- The tunnel geometry is complex and customized formwork would be required. Tunnel intersections, as well as bifurcations qualify in this area. Bifurcations are associated with tunnel widenings and would otherwise be constructed in the form of a stepped lining configuration and increase cost of excavated material.

Figure 9-6 displays a typical shotcrete final lining section with waterproofing system, welded wire fabric (WWF), lattice girder, grouting hoses for contact grouting and a final shotcrete layer with PP fiber addition.

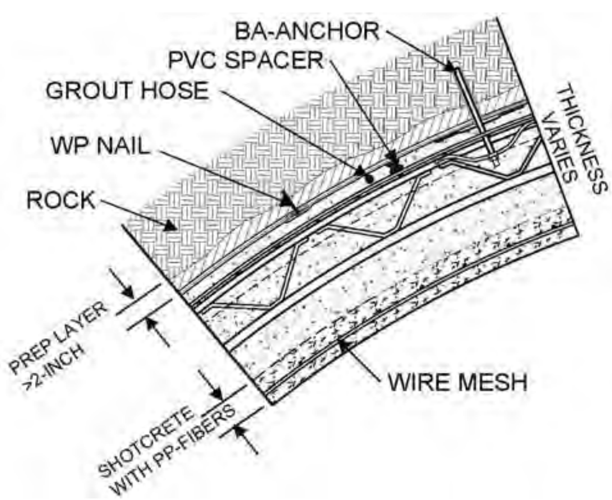


Figure 9-6 Typical Shotcrete Final Lining Detail

Readers are referred to Chapter 10 for detailed discussion about utilizing shotcrete as a final lining.

9.3.5.4 Single Pass Linings

Under special circumstances the initial shotcrete lining alone or with the addition of an additional shotcrete layer designed to withstand long-term loads may be used as a single support lining for the long term. Although labeled “single pass” this final shotcrete lining may be applied in multiple shotcrete application cycles. Use of a single pass lining will generally be limited to conditions where the ground water inflow is not of concern and deterioration of the shotcrete product over the life time of the tunnel lining can be excluded or partially tolerated. In multiple layer applications the shotcrete surface to which additional layers will be applied must be sufficiently clean and free of any layer that may cause de-bonding over the long term (Kupfer, et al., 1990 and Hahn, 1999). Specially detailed construction joints and high quality shotcrete must be required to assure water tightness and long-term integrity.

9.4 GROUND CLASSIFICATION AND SEM EXCAVATION AND SUPPORT CLASSES

9.4.1 Rock Mass Classification Systems

A series of qualitative and quantitative rock mass classification systems have been developed over the years and are implemented on tunneling projects worldwide. Section 6.3 provides an overview of the most commonly used rock mass classification systems including Terzaghi’s qualitative classification (Table 6-1), and quantitative systems such as the Q system and the Rock Mass Rating (RMR) system.

Rock mass classification systems aid in the assessment of the ground behavior and ultimately lead to the definition of the support required to stabilize the tunnel opening. While the above quantitative classification systems lead to a numerical rating system that results in suggestions for tunnel support requirements (Section 6.5), these systems cannot replace a thorough design of the excavation and support system by experienced tunnel engineers.

9.4.2 Ground Support Systems

In the early years of the use of NATM (SEM) in Austria, Switzerland and Germany, standards and codes used descriptive (qualitative) categories to define ground support classes. Recent standards, codes and guidelines implemented in Austria and Germany utilize a process-oriented approach (OGG, 2007). This approach defines the process of using relevant parameters from ground investigation to derive a ground response classification and subsequently assess tunnel support needs. This forms a more objective basis for all parties involved and promotes the understanding of the rationale in retrospect by persons that have not been involved in the design process. It also provides a common platform for contractors, owners and engineers to negotiate the project specific challenges in the field during actual construction.

All classification systems have in common that they should be based on thorough ground investigation and observation. The process from the ground investigation to the final definition of the ground support system can be summarized in three models:

- Geological Model
- Geotechnical Model
- Tunnel Support Model

9.4.2.1 Geological Model

A desk study of the geological information available for a project area forms the starting point of the ground investigation program. Literature, maps and reports (e.g. from the US Geological Survey) form the basis for a desk study. Subsequently and in coordination with initial field mapping results, a ground investigation program is developed and carried out. The geological information from the ground investigation, field mapping, and the desk study are compiled in the geological model.

9.4.2.2 Geotechnical Model

With the data from the geological model in combination with the test results from the ground investigation program and laboratory testing the ground response to tunneling is assessed. This assessment takes into account the method of excavation, tunnel size and shape as well as other parameters such as overburden height, environmental issues and groundwater conditions. The geotechnical model assists in deriving zones of similar ground response to tunneling along the alignment and Ground Response Classes (GRC) are defined. These GRCs form the baseline for the anticipated ground conditions. Typically, the ground response to an unsupported tunnel excavation is analyzed in order to assess the support requirements for the stabilization of the opening (OGG, 2007).

9.4.2.3 Tunnel Support Model

After assessing the ground support needs, excavation and support sequences, subdivision into multiple drifts, as well as the support measures are defined. These are combined in Excavation and Support Classes (ESCs) that form the basis for the Contractor to develop a financial and schedule bid as well as to execute SEM tunnel work.

9.4.3 Excavation and Support Classes (ESC) and Initial Support

Excavation and Support Classes (ESCs) contain clear specifications for excavation round length, subdivision into multiple drifts, initial support and pre-support measures to be installed and the sequence of excavation and support installation. They also define means of additional initial support or local support or pre-support measures that augment the ESC to deal with local ground conditions that may require additional support.

In SEM tunneling initial support is provided early on. In soft ground and weak rock it directly follows the excavation of a round length and is installed prior to proceeding to the excavation of the next round in sequence. In hard rock tunneling initial support is installed close to the face. The intent is to provide structural support to the newly created opening and ensure safe tunneling conditions. Initial support layout is dictated by engineering principles, economic considerations, and risk management needs.

The amount and design of the initial support was historically motivated mainly by the desire to mobilize a high degree of ground self support and therefore economy. This was possible at the outset of SEM applications in “green field” conditions where deformation control was of a secondary importance and tolerable as long as equilibrium was reached. Nowadays, however, safety considerations, risk management, conservatism and design life, and the need for minimizing settlements in urban settings add construction realities that ultimately decide on the layout of the initial support.

Initial support is provided by application of a layer of shotcrete to achieve an interlocking support with the ground. Shotcrete is typically reinforced by steel fibers or welded wire fabric. Plastic fibers are used for reinforcement only occasionally. With higher support demands of the ground and with shotcrete thicknesses of generally 6 inches or greater lattice girders are embedded within the shotcrete. Occasionally and if needed by special support needs rolled steel sets are used in lieu of, or in combination with lattice girders. Initial support also includes all measures of rock reinforcement in rock tunneling. Types of rock reinforcement are provided in Section 9.7.1.

Figure 9-7 and Figure 9-8 show a prototypical ESC cross section and longitudinal section respectively. Figure 9-7 displays a cross section without a closed invert on the left side and ring closure on its right side. Invert closure is typically required in weak rock conditions and squeezing ground. Figure 9-7 includes elements of typical initial support including rock bolts/dowels, initial shotcrete lining and tunnel pre-support. The arrangement of rock bolts/dowels is typical and varies depending on the excavation and support. The table in Figure 9-8 provides details of initial support measures for a prototypical ESC Class IV. In that sense, the SEM is a prescriptive method which defines clearly and in detail tunnel excavation and initial support means.

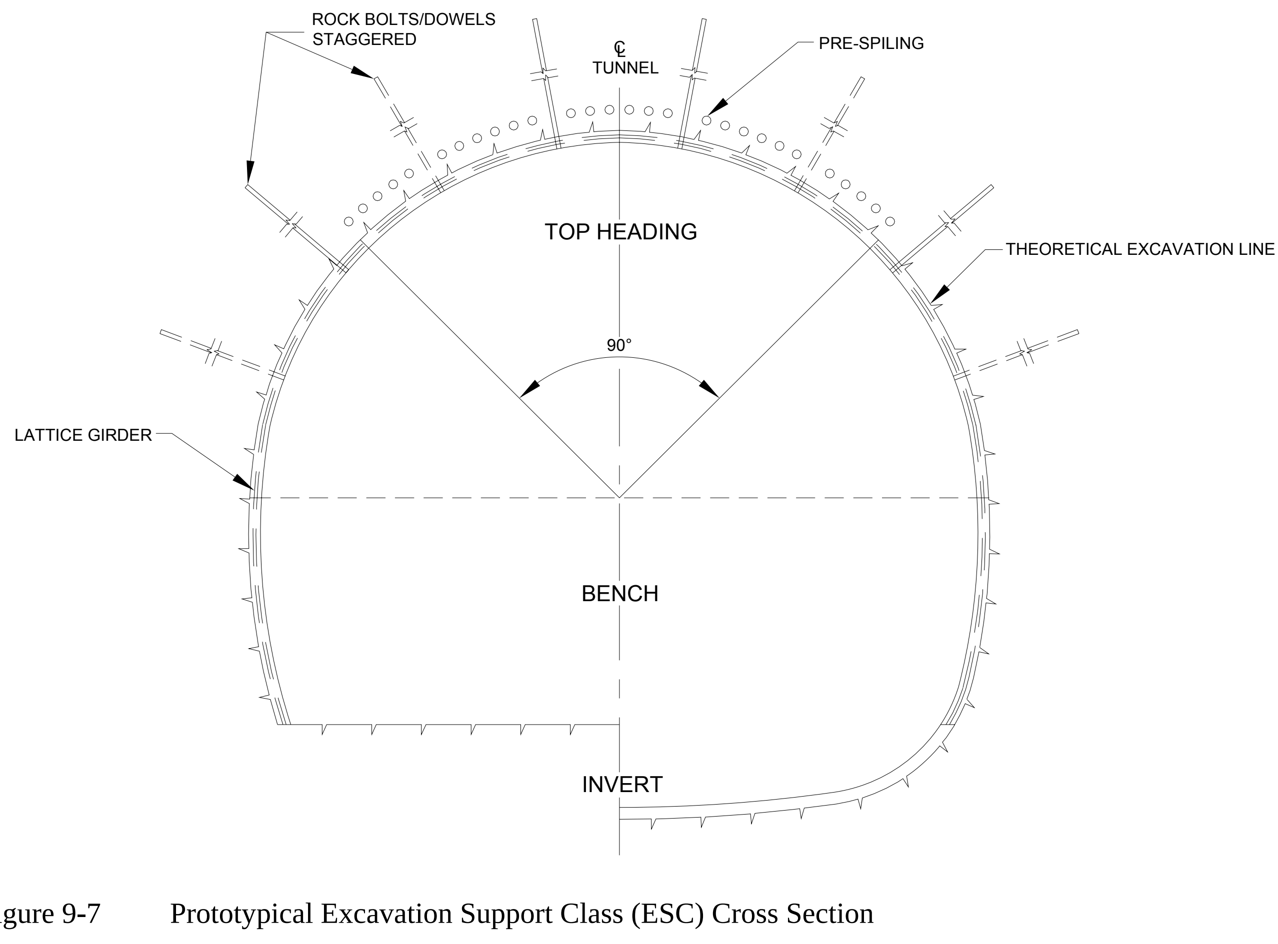
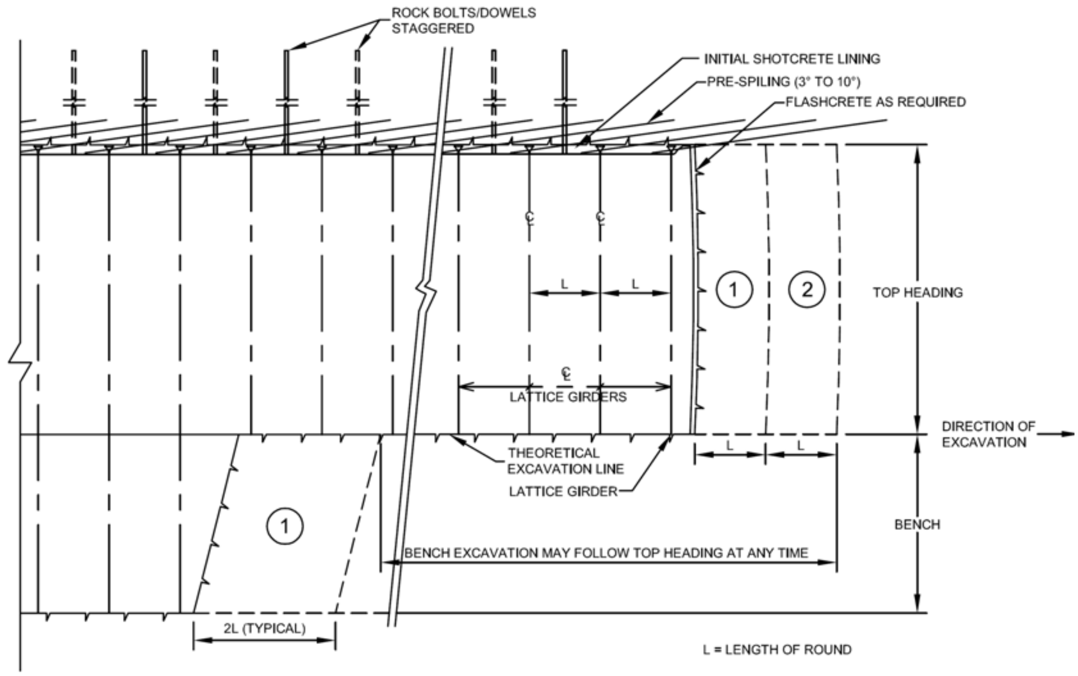
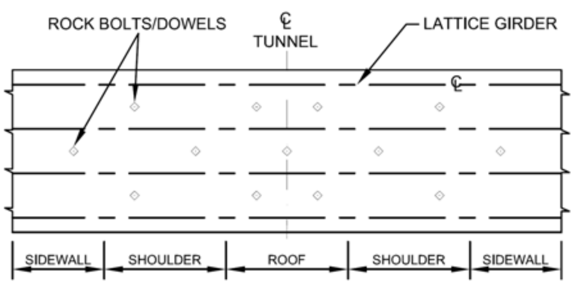


Figure 9-7 Prototypical Excavation Support Class (ESC) Cross Section





EXCAVATION AND SUPPORT: CLASS IV	
LOCATION:	STA. 200+00 to STA. 200+54
LATTICE GIRDER:	6' (1.8m) MAX. O.C.
MAX. LENGTH OF ROUND (L) TOP HEADING:	6' - 0" (1.8m)
MAX. LENGTH OF ROUND BENCH:	12' - 0" (3.6m)
PRE-SPILING:	#8 REBAR, 10' TO 20' (3-6m) LONG @ 1' - 2" (25-50mm) O.C. (TYP.)
INITIAL SHOTCRETE LINING:	10" (250mm) MIN. STEEL FIBER REINFORCED SHOTCRETE (SFRS)
FINISH SHOTCRETE:	1" (25mm) MIN. REINFORCED SHOTCRETE
ROCK BOLTS/DOWELS:	#8 REBAR, LENGTH = 12' - 0" (3.6m) BOLT PATTERN AS SHOWN

Figure 9-8 Prototypical Longitudinal Excavation and Support Class (ESC)

9.4.4 Longitudinal Tunnel Profile and Distribution of Excavation and Support Classes (ESCs)

SEM contract documents contain all Excavation and Support Classes (ESCs) assigned along the tunnel alignment in accordance with the Ground Response Classes (GRCs) and serve as a basis to estimate quantities. A summary longitudinal section along the tunnel alignment shows the anticipated geological conditions, the GRCs with the relevant description of the anticipated ground response, hydrological conditions and the distribution of the ESCs. Figure 9-9 displays a prototypical longitudinal profile with an overlay of GRCs and corresponding ESCs, which form a baseline for the contract documents.

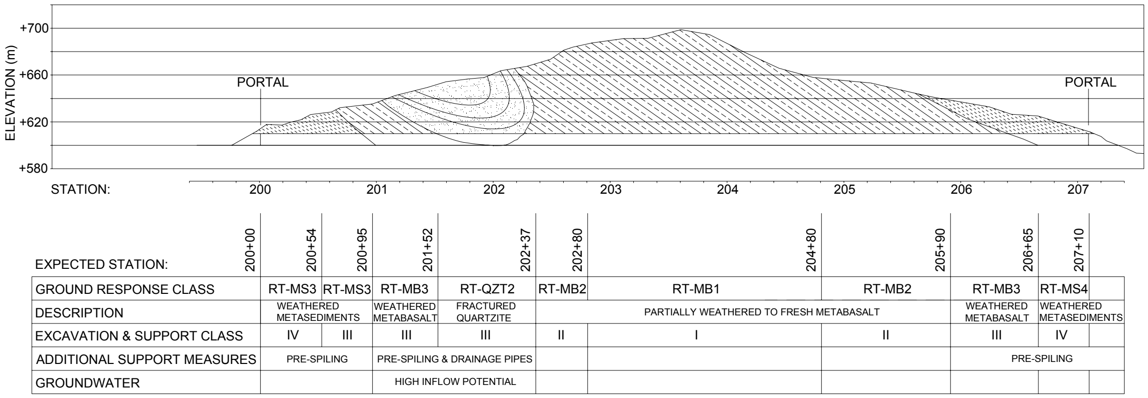


Figure 9-9 Prototypical Longitudinal Profile

Geological data, Ground Response Classes, Excavation and Support Classes, the Longitudinal Tunnel Profile as well as design assumptions and methods shall be described and displayed in reports that become part of the contract documents. When defining the reaches and respective lengths of GRCs and corresponding ESCs it is understood that these are a prognosis and may be different in the field. Therefore contract documents establish the reaches as a basis and call for observation of the ground response in the field and the need for their adjustment as required by actual conditions encountered. Actual conditions must be accurately mapped in the field to allow for a comparison with the baseline assumptions portrayed in the GRCs. For that purpose standard form sheets are developed as portrayed for a typical SEM rock tunnel mapping in Section 9.9.

9.4.5 Tunnel Excavation, Support, and Pre-Support Measures

Table 9-1 and Table 9-2 exemplify the use of most common initial support measures, along with excavation and support installation sequencing frequently associated with SEM road tunnels depending on the basic types of ground encountered, i.e. rock and soft ground respectively. These tables indicate basic concepts to derive Excavation and Support Classes (ESCs) for typical ground conditions portrayed. The support and pre-support means addressed in the tables are further detailed in Section 9.7 Ground Support Elements.

Table 9-1 builds on the use of Terzaghi's Rock Mass Classification. According to this Classification, it can be distinguished between the following rock mass qualities:

- Intact Rock
- Stratified Rock
- Moderately Jointed Rock
- Blocky and Seamy Rock
- Crushed, but Chemically Intact Rock
- Squeezing Rock
- Swelling Rock

The column labeled "Excavation Sequence" in Table 9-1 lists typical heading sequences used for road tunnels in ground conditions portrayed. Further subdivison of the headings into multiple drifts either for the purpose of construction logistics or to handle extraordinary ground conditions is not addressed. Table 9-2 characterizes the typical soils characteristics in column 1 directly.

Table 9-1 Elements of Commonly Used Excavation and Support Classes (ESC) in Rock

Ground Mass Quality - Rock	**Excavation Sequence**	**Rock Reinforcement**	**Initial Shotcrete Lining**	**Installation Location**	**Pre-Support**	**Support Installation influences progress**	**Remarks**
Intact Rock	Full face or large top heading & bench	Spot bolting (fully grouted dowels, Swellex ®)	Patches to seal surface in localized fractured areas	Typically Several rounds behind face or directly near face to secure isolated blocks/slabs/ wedges	None	No	
Stratified Rock	Top heading & bench	Systematic doweling or bolting in crown considering strata orientation (fully grouted dowels, Swellex®, rock bolts)	Thin shell (fiber reinforced) typically 4 in (100 mm) to bridge between rock reinforcement in top heading; alternatively chain link mesh; installed with the rock reinforcement.	Two to three rounds behind face	None	No or eventually	
Moderately Jointed Rock	Top heading & bench	Systematic doweling or bolting in top heading considering joint spacing (fully grouted dowels, Swellex®, rock bolts)	Systematic shell with reinforcement (welded wire fabric or fibers) in top heading and potentially bench; dependent on tunnel size thickness of 6 in (150 mm) to 8 in (200mm); installed with the rock reinforcement.	One to two rounds behind face	Locally to limit over break	Yes	
Blocky and Seamy Rock	Top heading &bench	Systematic doweling or bolting in top heading & bench considering joint spacing	Systematic shell with reinforcement (welded wire fabric or fibers) in top heading & bench; depending on tunnel size thickness 8 in (200 mm) to 12 in (300 mm)	At the face or maximum one round behind face	Systematic spiling in tunnel roof or parts of it	Yes	

Ground Mass Quality - Rock	Excavation Sequence	Rock Reinforcement	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation influences progress	Remarks
Crushed, but Chemically Intact Rock	Top heading, bench, invert	N/A	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 12 in (300 mm) and more; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required	After each round	Systematic grouted pipe spiling or pipe arch canopy	Support installation dictates progress	If water is present, groundwater draw down or ground improvement is required
Squeezing Rock	Top heading, bench, invert	Systematic doweling or bolting in top heading & bench considering joint spacing; extended length	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 12 in (300 mm) and more; potential use for yield elements; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required	After each round	Systematic grouted pipe spiling or pipe arch canopy	Support installation dictates progress	
Swelling Rock	Top heading, bench, invert	Systematic doweling or bolting in top heading & bench considering joint spacing; extended length	Systematic shell with reinforcement (welded wire fabric or fibers) and ring closure in invert; dependent on tunnel size thickness 12 in (300 mm) and more; potential use for yield elements	After each round	Systematic grouted pipe spiling or pipe arch canopy may be required depending on degree of fracturing	Support installation dictates progress	Deepened invert for additional curvature

Table 9-2 Elements of Commonly Used Soft Ground Excavation and Support Classes (ESC) in Soft Ground

Ground Mass Quality – Soil	Excavation Sequence	Initial Shotcrete Lining	Installation Location	Pre-Support	Support Installation	Remarks
Stiff/hard cohesive soil - above groundwater table	Top heading, bench & invert; dependent on tunnel size, further sub-divisions into drifts may be required	Systematic reinforced (welded wire fabric or fibers) shell with full ring closure in invert; dependent on tunnel size 6 in (150 mm) to 16 in (400 mm) typical; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required	Installation of shotcrete support immediately after excavation in each round. Early support ring closure required. Either temporary ring closure (e.g. temporary top heading invert) or final ring closure to be installed within one tunnel diameter behind excavation face.	Typically none; local spiling to limit over-break	Support installation dictates progress	Overall sufficient stand-up time to install support without pre-support or ground modification
Stiff/hard cohesive soil - below groundwater table	Top heading, bench and invert; dependent on ground strength, smaller drifts required than above	Systematic reinforced (welded wire fabric or fibers) shell with full ring closure in invert; dependent on tunnel size 6 in (150 mm) to 16 in (400 mm) typical; for initial stabilization and to prevent desiccation, a layer of flashcrete may be required; frequently more invert curvature than above	Installation of shotcrete support immediately after excavation in each round. Early support ring closure required. Either temporary ring closure (e.g. temporary top heading invert) or final ring closure to be installed within less than one tunnel diameter behind excavation face; typically earlier ring closure required than above	Typically none; locally pre-spiling to limit over-break	Support installation dictates progress	Sufficient stand-up time to install support without pre-support or ground improvement; dependent on water saturation, swelling or squeezing can occur
Well consolidated non-cohesive soil - above groundwater table	Top heading, bench & invert; dependent on tunnel size, further sub-divisions into drifts may be required	Systematic reinforced (welded wire fabric or fibers) shell with full ring closure in invert; dependent on tunnel size 6 in (150 mm) to 16 in (400 mm) typical; for initial stabilization and to prevent desiccation, a layer of flashcrete is required	Installation of shotcrete support immediately after excavation in each round. Early support ring closure required. Either temporary ring closure (e.g. temporary top heading invert) or final ring closure to be installed within less than one tunnel diameter behind excavation face	Frequently systematic pre-support required by grouted pipe spiling or grouted pipe arch canopy; alternatively ground improvement	Support installation dictates progress	Stand-up time insufficient to safely install support without pre-support or ground improvement
Well consolidated	Top heading, bench &	Systematic reinforced	Installation of shotcrete support	Frequently	Support	Stand-up time

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

9.4.6 Example SEM Excavation Sequence and Support Classes

While Section 9.5.3. introduced excavation and support classes in a prototypical context the following tables show examples on how, based on a ground classification, excavation and support classes were realized on selected projects. Grouped into two main types of ground, rock and soft ground, the examples are shown in tables Table 9-3 and Table 9-4 for rock and soft ground respectively.

The three examples in Table 9-3 outline tunnel constructions in three different characteristic rock mass types ranging from intact to fractured rock. The examples have rock mass reinforcement as a common element of initial support while systematic shotcrete support is used in stratified and fractured rock. Tunnel cross sections typically have horse-shoe-like shapes and no structural tunnel invert closure.
For the tunnel construction in intact rock, drill-and-blast excavation with round lengths of up to 12 feet (3.7 m) was utilized at the Bergen Tunnel in New Jersey. The initial tunnel support consisted of spot bolting to support loose rock blocks and slabs. Shotcrete was not systematically used as initial shotcrete lining but for local sealing of the rock face and for smoothening of the rock surface prior to waterproofing installation. Support was generally installed as required by field conditions.

The construction for the Zederhaus tunnel in Austria in stratified rock required systematic rock doweling and initial shotcrete lining installation. Excavation was carried out using drill-and-blast techniques with round lengths of typically 6 feet and 6 inches (2 m). The initial shotcrete lining was installed after each excavation round, whereas the installation of the rock dowels lagged 1 to 2 rounds behind the excavation. The bench excavation followed in a distance to the top heading excavation to suit the tunnel construction logistics.

A dense, systematic rock doweling pattern and an initial shotcrete lining were installed after each excavation round when tunneling through fractured rock at the Devil’s Slide tunnel project in California. Drill-and-blast techniques and road headers were employed for excavation depending on ground quality. The maximum length of round in the top heading was limited to 7 feet and 2 inches (2.2 m), while the bench excavation was limited to twice that length. There was no restriction on the distance between the top heading and bench construction.

The three examples in Table 9-4 are taken from typical soft ground tunneling projects where different sizes of tunnels were constructed at different overburden depths.

The three examples show the typical, rounded tunnel geometry with a systematic initial shotcrete lining that is closed in the curved invert. The support is installed after each excavation round prior to commencement of the next round in sequence.

The shallow cover of maximum about 16 feet (5 m) combined with soft ground conditions required the systematic installation of a grouted steel pipe arch pre-support canopy over the entire tunnel length at the Fort Canning Tunnel in Singapore. The tunnel cross section was split into top heading, bench and invert excavation with a shotcrete invert closure. To enable longer advances of the top heading ahead of the final invert closure, a temporary shotcrete invert was provided in the top heading. Excavators were used for the excavation of residual soils with round lengths limited to 3 feet and 4 inches (1 m) in the top heading and 6 feet and 8 inches (2 m) in the bench and invert.

Table 9-3 Example SEM Excavation and Support Classes in Rock

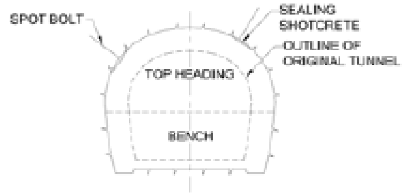
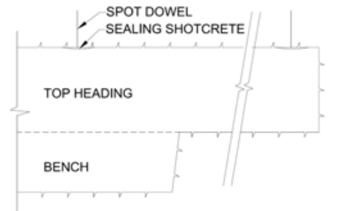
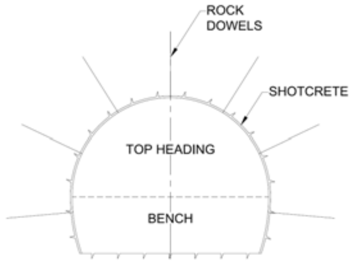
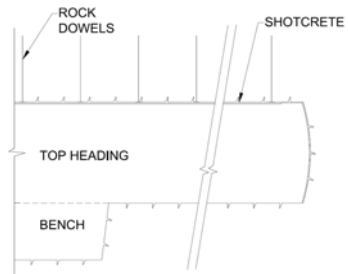
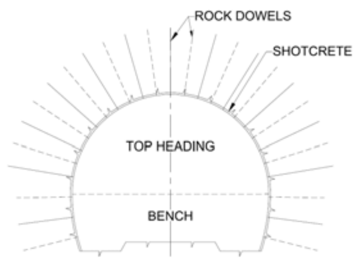
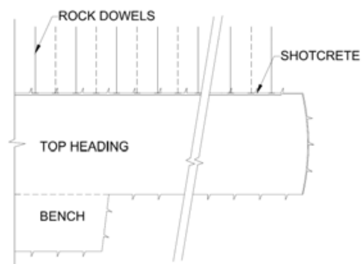
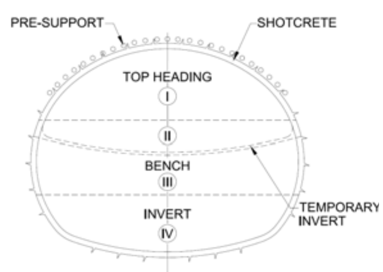
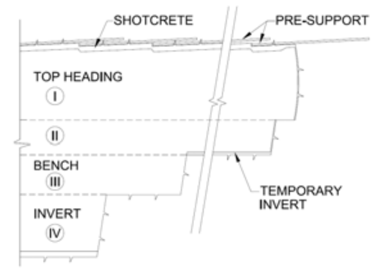
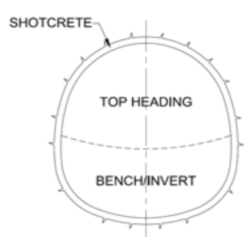
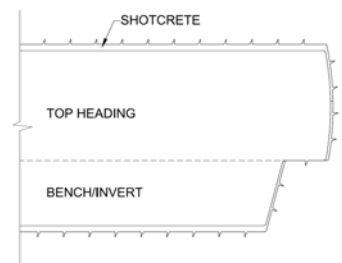
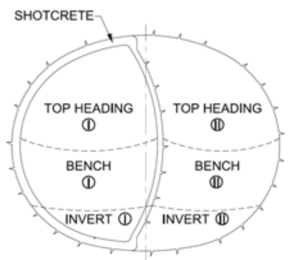
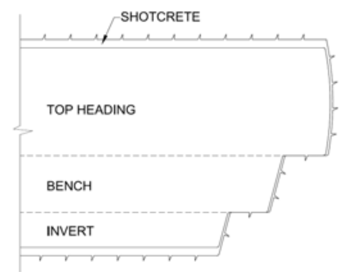
<table>
<tr><th>Description</th><th>Cross Section</th><th>Longitudinal Section</th><th>Photo</th></tr>
<tr><td>Intact Rock:
▪ Spot bolting
▪ Occasional sealing shotcrete
▪ Full face or top heading/bench excavation
▪ Round Length
Top Heading: 8’-12” (2.5-3.7 m)
Bench: Up to 16’-0” (4. 9 m)
▪ Dimensions
Height: 20’-0” (6 m)
Width: 29’-0” (8.8 m)
Example: Bergen Tunnels, NJ</td>
<td>
</td>
<td>
</td>
<td></td></tr>
<tr><td>Stratified Rock:
▪ Systematic rock doweling
▪ Systematic shotcrete initial lining
▪ Top heading excavation
▪ Bench excavation follows distant
▪ Round Length
Top Heading: 6’-6” (2 m)
Bench: 6’-6” (2 m)
▪ Dimensions
Height: 29’-6” (9 m)
Width: 36’-0” (11 m)
Example: Zederhaus, Austria</td>
<td>
</td>
<td>
</td>
<td></td></tr>
<tr><td>Fractured Rock:
▪ Systematic rock doweling
▪ Systematic shotcrete initial lining
▪ Top heading excavation
▪ Bench excavation follows any time
▪ Round Length
Top Heading: 7’-2” (2.2 m)
Bench: 13’-0” (4.0 m)
▪ Dimensions
Height: 28’-0” (8.5 m)
Width: 36’-5” (11.1 m)
Example: Devil’s Slide Tunnels, CA</td>
<td>
</td>
<td>
</td>
<td></td></tr>
</table>

Table 9-4 Example SEM Excavation and Support Classes in Soft Ground

Description	Cross Section	Longitudinal Section	Photo
Soft Ground – shallow cover: ▪ Systematic pre-support ▪ Systematic shotcrete initial lining support with early ring closure ▪ Top heading excavation (with temporary invert), bench and invert excavation ▪ Round Length Top Heading: I – 3'-3" (1 m) Top Heading: II – 6'-6" (2 m) Bench III/Invert IV – 6'-6" (2 m) ▪ Dimensions Height: 38'-0" (11.6 m) Width: 48'-0" (14.7 m) **Example**: Fort Canning Tunnel, Singapore			
Soft Ground – deep level: ▪ Systematic shotcrete support with early ring closure ▪ Top heading excavation closely followed by bench/invert excavation ▪ Round Length Top Heading: 3'-3" (1 m) Bench: 6'-6" (2 m) ▪ Dimensions Height: 20'-3" (6.3 m) Width: 20'-3" (6.3 m) **Example:** London Bridge Station, London, UK			

Description	Cross Section	Longitudinal Section	Photo
Soft Ground – deep level: ▪ Systematic shotcrete support with early ring closure ▪ Sub-division into sidewall drifts ▪ Top heading excavation closely followed by bench and invert excavation ▪ Round Length Top Heading: 3’-3” (1 m) Bench: 6’-6” (2 m) Invert: 6’-6” (2 m) ▪ Dimensions Height: 30’-2” (9.2 m) Width: 37’-0” (11.3 m) **Example:** London Bridge Station, London, UK			

The tunnels built for London Bridge subway station in London, UK, located at approximately 80 feet (25 m) depth below ground surface, were excavated in over-consolidated clays using excavators and road headers with maximum round lengths of 3 feet and 4 inches (1 m) and 6 feet and 8 inches (2 m) in the top heading and bench/invert respectively. While the smaller running tunnels were excavated and supported in a staggered full face sequence in a top heading and bench/invert arrangement, the 37 feet (11.3 m) wide turn-out was constructed using a single-sidewall drift with a top heading, bench and invert excavation in each partial drift. The temporary middle wall provided temporary sidewall support for the first tunnel half during construction. During the enlargement to the full tunnel size the temporary middle wall was removed.

9.4.7 Excavation Methods

During the history of application of the SEM/NATM, tunneling methods for a wide variety of ground conditions have been developed. With the further development and refinement of support means, the application field of the SEM has ever been expanded. From its original implementation in alpine, “green field” rock and soft rock tunnels the focus moved into urban areas and soft ground tunneling. SEM tunneling is typically accomplished in hard rock using drill-and-blast excavation techniques (Section 6.4.1), medium hard and soft rock using a road header (6.4.3) and in soft ground using backhoe excavation.

Figure 9-10 through Figure 9-13 display such SEM excavations from hard rock through soft ground. Figure 9-10 displays drilling of a face in a rock tunnel for a drill-and-blast excavation. A close up of the drilling at the face is shown in Figure 9-11 that also displays the shotcrete initial lining installed close to the face. The rock face has been sealed by a layer of flashcrete. Figure 9-12 shows a close up of a road header boom excavating a medium hard, jointed rock mass. Figure 9-13 displays tunnel construction of a soft ground tunnel in a top-heading, bench, and invert excavation using backhoes. The backhoe is in the background at the tunnel face.

Figure 9-10 Face Drilling for Drill-and-Blast SEM Excavation (Andrea Tunnel, Austria)

Figure 9-11 Shotcrete Lining Installed at the Face in a SEM Tunnel Excavated by Drill-and-Blast (Andrea Tunnel, Austria)

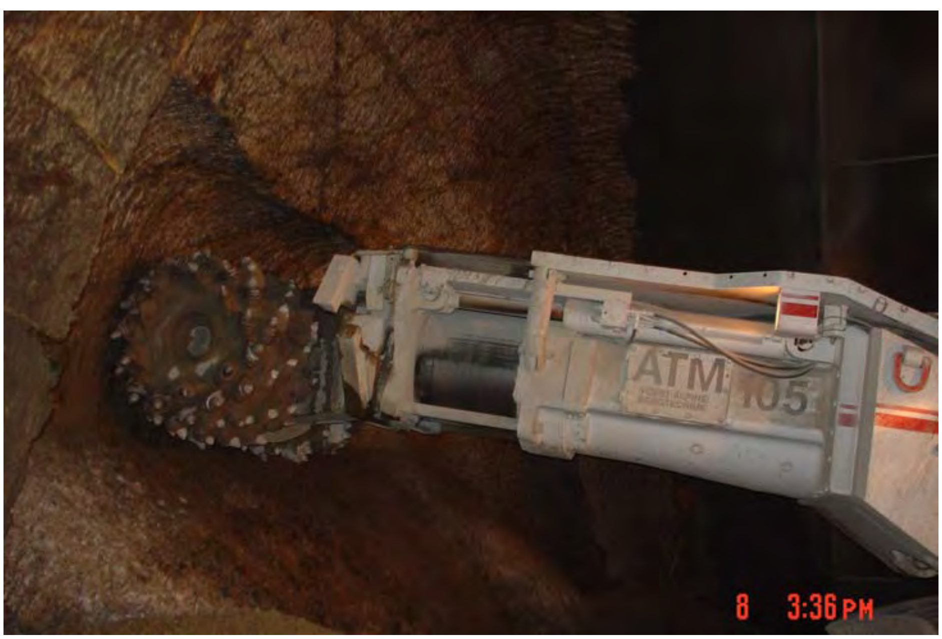


Figure 9-12 Road Header SEM Excavation in Medium Hard, Jointed Rock (Devil's Slide Tunnels, California)

Figure 9-13 Soft Ground SEM Excavation Tunnel Using Backhoes (Fort Canning Tunnel, Singapore)

9.5 GROUND SUPPORT ELEMENTS

This section addresses special ground support and material considerations that have evolved with the application of shotcrete supported SEM excavations. Chapter 6 also provides detailed discussions about rock reinforcement elements.

9.5.1 Shotcrete

The original name for shotcrete was "Gunite" when it was used for the purpose of taxidermy by spraying mortar on wire frames in the US in the early 1900’s. In its early applications, sprayed dry mix material has also been used for the improvement of the fire resistance of timber supports in mines. During the course of the early 1930's, the term "Shotcrete" was introduced and has been widely used since. Development of equipment technology for the application of shotcrete progressed rapidly and the use of shotcrete for ground support purposes spread worldwide. In particular, the use of NATM / SEM, and the associated extensive use of shotcrete contributed to development of shotcrete which nowadays can be viewed as sprayed concrete, the major distinction between concrete and shotcrete being merely the method of placement (Vandewalle, 2005).

9.5.1.1 Effect of Shotcrete

When concrete is sprayed on a rough ground surface, it fills small openings, cracks and fissures and as initial support provides immediate support after excavation. It reduces the potential for relative movement of rock bodies or soil particles and, therefore, limits loosening of the exposed ground surrounding the tunnel. The adhesion depends on the condition of the ground surface, the dampness and presence of water and the composition of the shotcrete. Generally, the rougher the ground surface the better the adhesion. Dry rock surfaces have to be sufficiently dampened prior to application of shotcrete. Dusty or flaky surfaces, water inflow or a water film on the rock surface or other contaminant reduce the adhesion of shotcrete.

Modern admixtures improve the "stickiness" of shotcrete significantly such that rebound is reduced considerably. Fibers increase the adhesion and cohesion of the freshly applied ("green") shotcrete and therefore improve the build-up quality of the shotcrete. In turn, excessive stickiness of the shotcrete mix (as frequently observed when sodium silicate accelerators are used) can have an adverse effect. Too sticky shotcrete tends to accumulate around reinforcement bars, resulting in insufficiently compacted, low quality concrete or even voids or "shadows" behind the reinforcement bars.

In order to stabilize small wedges and slabs, shotcrete is applied locally. This application type does not form a continuous layer of shotcrete over an extended area to form a supporting member in the sense of a lining or structural shell. Rather, edges and corners generated by the intersection of discontinuities are filled with shotcrete bonding the bodies together thus forming local support.

Flashcrete: also referred to as sealing shotcrete, is applied immediately after excavation by spraying a thin layer of shotcrete if required to seal off the exposed ground surface. Flashcrete is often used in poor rock or soft ground (soil) in combination with (steel) fibers for reinforcement. This application limits desiccation, effects of humidity on sensitive ground material, softening due to contact with water, and loosening of the ground due to differential movement of ground particles. Flashcrete may be applied locally (and in areas where required) or over the entire exposed ground surface after excavation. Flashcrete is not considered to be an active support and, therefore is normally followed by a systematically applied initial shotcrete lining.

Shotcrete Face Support*:* In poor ground conditions a temporary face support may be required to restrict the ground from moving into the excavation. Dependent on the length of period through which the support is required and the ground conditions, the thickness and reinforcement of the face support varies. For tunnel stubs, permanent head walls are constructed with shotcrete. A domed face shape is of great importance in poor ground for successful face stabilization.

Experience gained from tunnel projects in soft ground demonstrates that ground deflections and hence surface settlements continue until a final, fully domed head wall with sufficient connection to the tunnel shotcrete initial lining is established.

Temporary Shotcrete Support: In poor ground conditions or where large tunnel cross sections are constructed, the excavation area must often be split into several drifts. To provide immediate support and, if required, ring closure for each sub-drift, temporary shotcrete support shells or linings are used. The thickness of the temporary lining is designed based on the cross sectional area of the drift to be supported and the period for which the support is required. The temporary shell is removed during subsequent construction steps that complete the excavation to the full tunnel opening. Figure 9-14 shows a typical SEM tunnel excavation with a temporary middle wall.

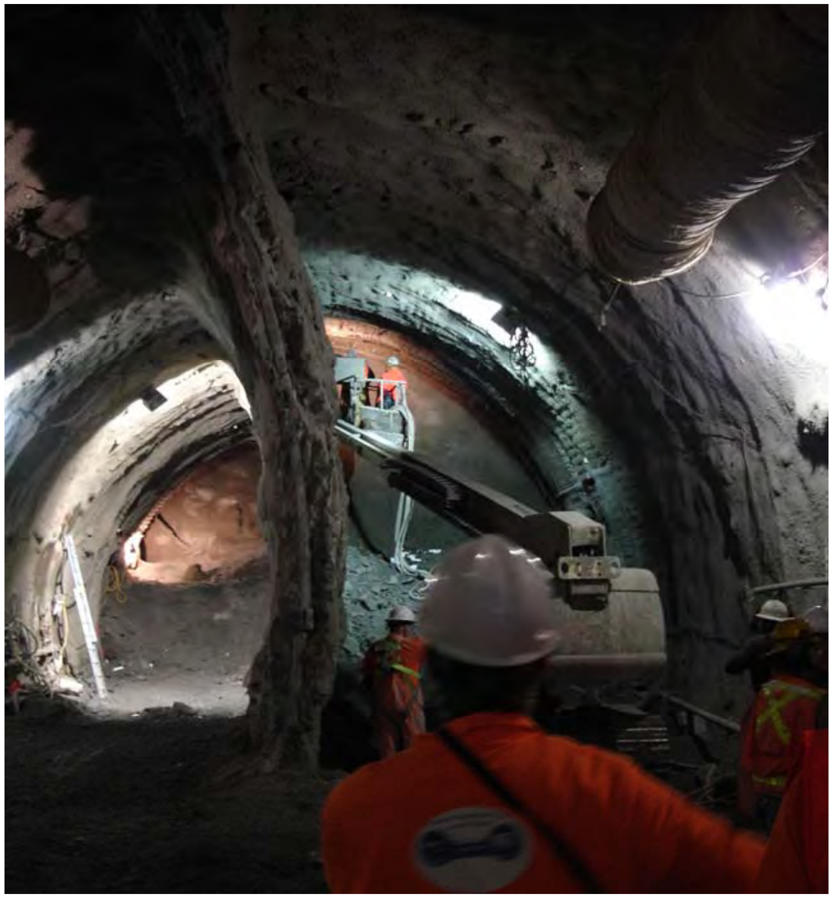


Figure 9-14 Typical Tunnel Excavation with Temporary Middle Wall (Beacon Hill Station, Washington)

Initial Shotcrete Lining: Initial shotcrete lining typically consists of 4 to 16 inch (100 to 400 mm) thick shotcrete layer mainly depending on the ground conditions and size of the tunnel opening, and provides support pressure to the ground. It is also referred to as shotcrete lining. A shotcrete ring can carry significant ground loads although the shotcrete lining forms a rather flexible support system. This is the case where the shotcrete lining is expected to undergo high deformations and hence ductility post cracking is of importance. By deforming, it enables the inherent strength and self-supporting properties of the ground to be mobilized as well to share and re-distribute stresses between the lining and ground. During deformation, stresses acting within the shotcrete lining are transferred into the surrounding ground. This process generates subgrade reaction of the ground that provides support for the lining. From the ground support point of view the design of the shotcrete lining is governed by the support requirements, i.e., the amount of ground deformations allowed and ground loads expected as well as economical aspects. The earlier the sprayed concrete gains strength the more the support restrains ground deformation. However, by increasing stiffness the support system increasingly attracts loads. It depends on the ground conditions and local requirements how stiff or flexible the support system has to be and thus what early strength requirements, thickness and reinforcement should be specified.

In shallow tunnel applications and beneath surface structures that are sensitive to deformations such as buildings, ground deformations and consequently surface settlements have to be kept within acceptable limits. The advantage of the mobilization of the self-supporting capacity of the ground can therefore be only taken into account to a very limited extent. Here, early strength of the shotcrete is required to gain early stiffness of the support to limit ground deflections. Under these conditions the shotcrete lining takes on significant ground loads at an early stage however in a generally low stress environment due to the shallow overburden. Early strength can be achieved with admixtures and modern cement types.

In contrast for tunnels under high overburden the prevention of ground deformation and surface settlement plays a secondary role. Excessive ground loads in squeezing ground and active tectonic pressures applied on the tunnel perimeter may be the design criteria for deep tunnels. By allowing the ground to deflect (without over-straining it) the ground's self-supporting capability, mainly shear strength, is mobilized. Consequently, the ground loads acting upon the shotcrete lining can be limited significantly because the ground assumes a part of the support function and a portion of the ground loads is dissipated before the initial support is loaded. For rock tunnels under high cover, early strength is not a necessity but final strength of the entire system is of importance. In special cases it may be required to construct "deformation joints" by implementing special yield elements to allow substantial deformation of the ground without generating uncontrolled cracks in the shotcrete lining while maintaining a defined support pressure during the process of deformation. Yield elements are designed such as to allow pre-scribed maximum deformation under defined lining loads (Section 8.3.3).

Rock reinforcement installed in rock tunnels augments the strength of the surrounding ground, controls deformation and limits the ground loads acting upon the shotcrete initial lining. Shotcrete support and rock reinforcement are designed to form an integrated support system in view of the excavation and support sequence. The design engineer must define the requirements for the support system based on thorough review of the ground response anticipated.

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

Friction between the ground and the sprayed concrete lining (tangential subgrade reaction) is paramount for the support system. This friction reduces differential movement of ground particles at the ground surface and contributes to the ground-structure interaction. Even the shotcrete arch not forming a closed ring provides substantial support to the ground, given that tight contact between the sprayed concrete and the ground is maintained.

The requirement for a ring closure, be it temporary or permanent, is governed by the size of the underground opening and the prevailing ground conditions. In a good quality rock mass, no ring closure is required. In low quality ground (weak rock and soil), it has been proven in numerous case histories that the time of support application after excavation, length of excavation round and time lag between the excavation of the top heading and the invert closure rules the ground and lining deflection. To reduce ground deflection and the potential for ground/lining failure, the excavation and support sequence must be designed such that an early ring closure of the shotcrete support in soft ground is achieved. Also the timely (immediately after excavation) installation of the shotcrete support members is of utmost importance. To achieve an early, temporary ring closure and to reduce excavation face size, partial drifts such as sidewall drifts, middle drifts and top heading, bench, and invert drifts can be used. These partial drifts are supported by temporary shotcrete support, such as temporary middle walls, invert supports, etc.

An important aspect of shotcrete linings is the design and construction of construction joints. These joints are located at the contact between shotcrete applications in longitudinal and circumferential directions between the initial lining shells of the individual excavation rounds and drifts. An appropriate location and shape as well as connection of the reinforcement through the longitudinal joints is of utmost importance to the integrity and capacity of the support system. Longitudinal joints have to be oriented radially, whereas circumferential joints should be kept as rough as possible. Splice bars/clips and sufficient lapping of reinforcement welded wire fabric maintain the continuity of the reinforcement across the joints. Rebound, excess water, dust or other foreign material must be removed from any shotcrete surface against which fresh concrete will be sprayed. The number of construction joints should be kept to a minimum.

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

9.5.2 Rock Reinforcement

As discussed in Chapter 6, rock reinforcement and rock mass act as a complex interactive system, where the individual elements always have to be seen in view of their interaction and interdependence. The overall strength of a reinforced rock mass with a joint system is governed by the characteristics of the joints (roughness, fill, rock material, orientation) and the contribution provided by the reinforcement elements. For the design of rock mass reinforcement systems, sufficient appreciation of the expected ground conditions and experience are of fundamental importance. Readers are referred to Section 6.5.2 for more detailed discussion for each type of rock reinforcement. The following focuses on the SEM applications and issues.

9.5.2.1 Types of Rock Reinforcement

<u>Rock Dowels:</u> (Figure 6-17), are passive reinforcement elements that require some ground displacement to be activated. In deep tunnels or under tunneling conditions where ground deflection is permitted or even desired, passive rock reinforcement is frequently installed. This applies for example to tunnel construction sequences where the excavation and support installation is carried out in sequences (e.g. top heading, bench, invert). In order to best use the support effect of the rock dowels, an early installation is required. The majority of ground deflections develop during excavation and closely behind the progressing tunnel face. In sequential rock tunneling using multiple drifts, ground deflection typically ceases after top heading excavation and support but commences again after a period of relative stability during excavation for bench and invert construction. Therefore, rock dowels should be installed right after excavation or close to the progressing excavation face.

<u>Rock Bolts:</u> (Figure 6-18) actively introduce a compressive force into the surrounding ground. This axial force acts upon the rock mass discontinuities thus increasing their shear capacity and is generated by pre-tensioning of the bolt. The system requires a ‘bond length’ to enable the bolt to be tensioned. Rock bolts frequently are fully bonded to the surrounding ground after tensioning, for long-term load transfer considerations.

Rock bolts are not only installed during construction. Rock bolts may also be used for existing under-ground openings, where further deformation of the ground and/or the support is to be inhibited or for additional support of existing structures that will undergo subsequent enlargement or be influenced by adjacent tunnel construction.

For tunnels constructed in an environment where ground deflection and surface settlement hast to be limited (e.g. shallow tunnels in urban areas), rock bolt aid in limiting the ground displacement caused by the SEM tunneling. Furthermore, during construction of large openings ground deflection limitation may be desired to avoid loosening (and hence weakening) of the rock mass. In high stress environments, special compressible elements have been developed, that are installed between the ground/support surface and the face plate of the bolt allowing a certain amount of displacement while the tension force at the bolt is kept constant.

Rock Anchors: Rock anchors are used under conditions where high anchor forces have to be accommodated often significantly higher than for example rock bolt forces. For instance in very large span tunnels, where high support forces have to be generated to stabilize the ground, anchors are frequently used.

Generally, it can be stated that pre-tensioning of bolts establishes a stiffer system of the reinforced rock mass after installation and minimizes the magnitude of shear displacement. The design and application of a pre-tensioned rock reinforcement system requires excellent knowledge of the ground conditions and ground behavior to avoid over-tensioning during ground displacement. In comparison, an initially untensioned rock dowel reinforcement may ultimately lead to the same strength and reinforced rock mass capacity, however, only along with larger deformations.

Table 9-5 summarizes commonly used rock reinforcement elements and application considerations for the installation as part of initial support in SEM tunneling in rock.

9.5.2.2 Practical Aspects

Several practical aspects related to rock dowel/bolt installation in the field have been summarized below based on experience in SEM tunneling. Each individual project has its own particularities and, therefore, this list is not exhaustive.

Layout of Rock Mass Reinforcement Pattern: While it also has to observe theoretical considerations, the design must take practical issues of installation into account. As a consequence of a design lacking practical considerations, rock mass reinforcement systems are frequently ‘adjusted’ on site to suit practical aspects without considering the ground conditions and the design intent. Such installed rock reinforcement systems may be of limited benefit or even have an adverse effect.

Grouting: Rock dowel/bolt grouting systems aim for the full embedment of a rock dowel/bolt in grout. Full embedment not only ensures bond over the entire length of the dowel/bolt but also provides corrosion protection. Regardless of the method used, the appropriate consistency of the grout material is the most important factor in achieving the required bond between the ground and the reinforcement element. This particularly applies for cementitious grout materials. While the available diameter of the grouting hose dictates the consistency of the grout material to some extent, too high or too low viscosity can lead to insufficient bond. It can frequently be observed that installation crews adjust grout mixing plants and pumps and do not visually check the consistency of the grout mix produced. Even with the use of the most sophisticated mixing and pumping devices, it is required to visually check the grout mix produced before commencing each installation operation. All foreign material must be removed prior to installation to ensure proper bond.

Table 9-5 Commonly used Rock Reinforcement Elements and Application Considerations for SEM Tunneling in Rock

No.	Name	Material*	Anchorage	Tensioned	Installation	Ground **	Advantages	Limitations
1	Steel rebar dowel	Deformed (solid) steel rebar	Fully bonded using cement grout or resin	No	Rebar inserted into pre-drilled and grout filled hole; Rebar inserted in pre-drilled hole together with grouting hose and grouted subsequently	Massive to highly jointed rock mass	Low cost; Availability; If properly installed, high performance and heavy duty support	Requires skilled and experienced installation personnel; collapsing boreholes hamper installation
2	Glass Fiber Dowel	Deformed fiber glass bar	Fully bonded using cement grout, more frequently with resin	No	Rebar inserted into pre-drilled and grout filled hole; Rebar inserted in pre-drilled hole together with grouting hose and grouted subsequently	Massive to highly jointed rock mass; frequently used in areas to be excavated subsequently (e.g. face bolting, break-out areas)	High performance heavy duty support; can be easily removed during subsequent excavations within reinforced rock mass	Requires skilled and experienced installation personnel; limited shear resistance; collapsing boreholes hamper installation
3	Split Set	Longitudinally split steel pipe	Friction over entire length generated by spring action of pipe	No	Forced into pre-drilled borehole of slightly smaller diameter than outer diameter of split set	Massive to jointed rock mass	Immediate support action; simple installation; no grouting required	Very limited shear resistance; Light support only; very corrosion sensitive; cannot be used in collapsing borehole
4	Swellex	Folded, inflatable steel pipe	Friction over entire length generated by inflation of	No	Inserted into pre-drilled borehole and inflated with highly	Massive to jointed rock mass	Immediate support action; Can achieve	Limited shear resistance and durability; cannot be re-tightened; requires special equipment for inflation;

No.	Name	Material*	Anchorage	Tensioned	Installation	Ground **	Advantages	Limitations
			tube		pressurized water		significant support capacity	higher material cost; collapsing boreholes hamper installation
5	Grouted Pipes	Perforated steel pipe	Fully bonded with cement or resin grout	No	Inserted into pre-drilled borehole (or rammed into soft ground with thick walled pipes) and grouted through pipe and perforation holes	Jointed to heavily fractured ground (soil like)	Simple installation; Availability; More controllable embedment results	Limited shear resistance (depending on wall thickness); collapsing boreholes hamper installation
6	Self-drilling dowels	Thick walled steel pipes with disposable drill bit	Fully bonded with cement. or resin grout	No	Reinforcement element functions as drill rod, drill bit and dowel remains in ground after drilling and is grouted through flushing openings	Jointed to heavily fractured rock mass	Installation steps limited to two steps (fast installation); High performance heavy duty support;	More expensive than bar reinforcement; May become trapped in collapsing boreholes as it does not have reverse cutting tools;
7	Rammed Dowels	Steel rebar or thick walled steel tube	Shear resistance generated between ground and element (friction, adhesion)	No	Rammed into ground	Decomposed rock, soil	Least ground disturbance during installation; Immediate support action	Relies on shear resistance generated between ground and element; requires ramming equipment; limited to soft ground conditions
8	Steel rebar bolt	Deformed steel rebar	a. End anchored: cement grout or resin;	Yes	a. Grouting behind grout seal through grouting hose (aeration	Massive to highly jointed rock mass	Low cost; Availability; if properly installed, high	Requires skilled and experienced installation personnel; collapsing boreholes hamper

No.	Name	Material*	Anchorage	Tensioned	Installation	Ground **	Advantages	Limitations
					hose);		performance heavy duty support	installation a. Requires grout seal;
			b. Fully bonded: two phase resin		b. resin grout with two different setting times			b. Resin is more expensive than grout; Requires different types of resin
9	Glass fiber bolt	Deformed glass fiber bar	a. End anchored: cement grout, resin;	Yes	a. Grouting behind grout seal through grouting hose (aeration hose);	Massive to highly jointed rock mass	High performance heavy duty support; can be easily removed due to limited shear resistance	Requires skilled and experienced installation personnel; collapsing boreholes hamper installation a. Requires grout seal;
			b. Fully bonded: two phase resin		b. resin grout with two different setting times			b. Resin is more expensive than grout; Requires different types of resin
10	Expansion Shell Bolt	Steel rebar	Mechanically end anchored	Yes	Inserted in pre-drilled borehole, shell at end expanded by tightening the bolt	Massive to jointed rock mass; requires competent rock material	Immediate support effect; can provide high support capacity;	Relatively expensive; Slip or rock crushing may occur; tends to lose tension due to vibration (blasting) and ground deformation

* Reinforcement material

** Ground conditions described are typical application examples; reinforcement elements may also be used in other ground conditions.

Contact: Frequently rock dowels and face plates as well as nuts are installed in time, but the nuts are not tightened or are tightened only after a long period of time and far behind the progressing excavation face. While tensioning of a fully bonded rock dowel does not have any effect on the strength of the integrated rock - reinforcement system (rock mass and reinforcement), it is important to tighten the nuts to ensure a tight fit of the face plate and, if used in combination with a shotcrete support, to aid an appropriate contact between the ground surface and the shotcrete support lining/face plate. If used without a shotcrete lining, tightening of nuts assists in limiting early deformation and loosening of the rock mass close to the opening.

Testing and monitoring: Pull-out-tests are an important tool to ensure adequate anchorage of rock bolts. While useful to check the bond strength and therefore, the support capacity of a tendon with a defined bonded anchorage section and a free section, pull-out tests are irrelevant when used for testing fully bonded rock dowels, because they do not provide any information on the overall performance of a fully bonded rock reinforcement. The conventional pull-out test, when used for fully bonded reinforcement, provides information on the shear capacity between the bolt and grout and the ground adjacent to the head of the tested element, but it does not yield any information of the overall bond along the reinforcement element or whether the element is fully embedded in grout.

Similar to above, monitoring the anchor forces between the ground surface and the face plate of a fully bonded rock dowel/bolt does not provide any information on the forces acting within the fully bonded reinforcement element over its length. Therefore, only monitoring devices (e.g. strain gages) mounted directly onto the shank along the reinforcement element can supply information on the performance and stresses acting within the reinforcement during ground deformation.

9.5.3 Lattice Girders and Rolled Steel Sets

As discussed in Chapter 6, lattice girders (Figure 6-20) are lightweight, three-dimensional steel frames typically fabricated of three primary bars connected by stiffening elements. Lattice girders are used in conjunction with shotcrete and once installed locally act as shotcrete lining reinforcement. The girder design is defined in the contract documents by specifying the girder section and size and moment properties of the primary bars. To address stiffness of the overall girder arrangement the stiffening elements must provide a minimum of five percent of the total moments of inertia. This percentage is calculated as an average value along repeatable lengths of the lattice girder. The arrangement of primary bars and stiffening elements is such as to facilitate shotcrete penetration into and behind the girder, thereby minimizing shadows. Lattice girders are installed to provide:

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

In particular cases where, for example, immediate support is necessary for placing heavy spilling for pre-support, the use of rolled steel sets may be appropriate. In such instances steel sets are used for implementation of contingency measures. Steel sets of bell shaped profile (Heintzmann profile) are also used as structural members in temporary shotcrete sidewalls in multiple drift tunneling. Their primary purpose apart from increased capacity over lattice girders is their ease of removal when demolishing temporary shotcrete walls in multiple drift tunneling applications.

9.5.4 Pre-support Measures and Ground Improvement

When tunneling in competent ground, the ground surrounding the tunnel opening provides sufficient strength to ensure stand-up time needed for the installation of the initial SEM support elements without any pre-support or improvement of ground strength prior to tunneling.

With the significantly increased use of the SEM in particular in soft ground and urban areas over the past decades, traditional measures to increase stand-up time were adopted and further developed to cope with poor ground conditions and to allow an efficient initial support installation and safe excavation.

These measures are installed ahead of the tunnel face. They include ground modification measures to improve the strength characteristics of the ground matrix including various forms of grouting, soil mixing and ground freezing, the latter for more adverse conditions. Most commonly they include mechanical pre-support measures consisting of spiling methods installed ahead of the tunnel face often with distances of up to 60 to 100 feet (18 to 30 m) referred to as pipe arch canopies or at shorter distances, as short as 12 ft (3.6 m) utilizing traditional spiling measures such as grouted solid bars or grouted, perforated steel pipes. Ground improvement and pre-support measures can be used in a systematic manner over long tunnel stretches or only locally as required by ground conditions.

9.5.4.1 Pre-support Measures

Pre-support measures involve spiling or grouted pipe arch canopies that bridge over the unsupported excavation round. These longitudinal ground reinforcement elements are supported by the previously installed initial shotcrete lining behind the active tunnel face and the unexcavated ground ahead of the face. These mechanical pre-support measures are generally used to:

- Increase stand-up time by preventing ground material from raveling into the tunnel opening causing potentially major over-break or tunnel instabilities
- Limit over-break
- Reduce the ground loads acting on the immediate tunnel face
- Reduce ground deflection and, consequently surface settlements.

Mechanical pre-support measures are generally less intrusive than systematic ground modifications. They rely on the ground reinforcing action of passive reinforcement elements such as steel or fiberglass pipes/bars. Similar to passive concrete reinforcement the elements must directly interact with the surrounding ground to be efficient as reinforcement. This interaction can only be established by a tight contact between the reinforcement element and the ground. This interaction can be achieved by either fully grouting the pre-support elements to lock the reinforcement in with the ground or by ramming the reinforcement elements into ground if susceptible to this action in soft ground conditions. Loosely installed elements installed in soft rock or soil do not achieve their intended function and such installations must be avoided. In fractured, but competent rock, steel rebars loosely installed in boreholes may be acceptable but merely to limit over-break. Figure 9-15 displays closely spaced No. 8 rebar spiles bridging across an excavation round and keeping soft, cohesive fine soil materials in place. Spiles rest on the initial shotcrete lining (front) and on the unexcavated ground beyond the tunnel face. The narrow spacing allows even very soft and soils with little cohesion to bridge between individual spiles.

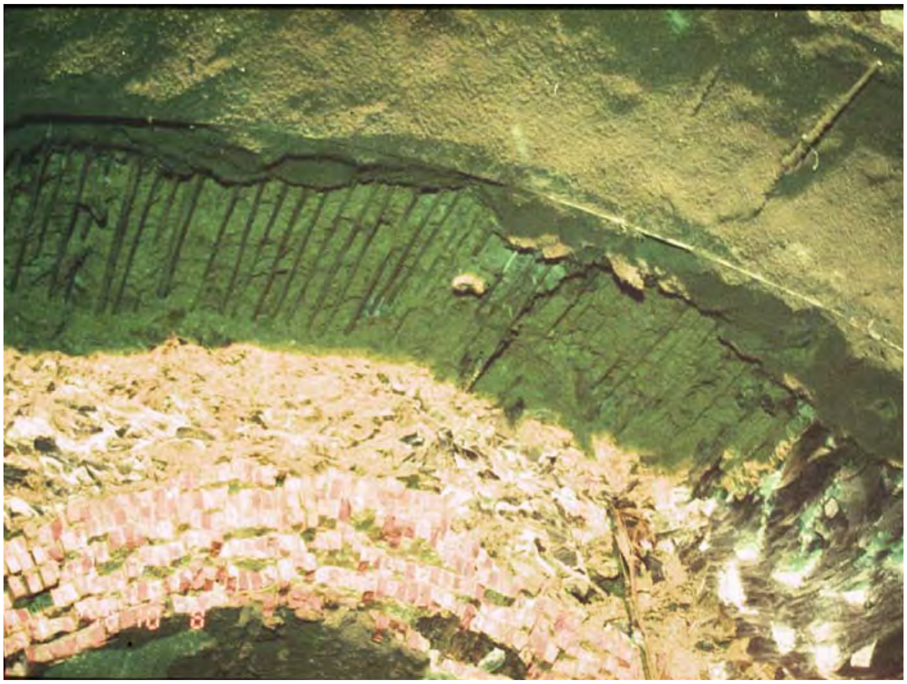


Figure 9-15 Spiling Pre-support by No. 8 Solid Rebars (Berry Street Tunnel, Pennsylvania)

The effect of mechanical pre-support has frequently been misjudged. On one hand, the stiffness of the steel elements used for pre-support is often taken as basis for assessing an increase of the overall stiffness of the ground surrounding the pre-support. This can easily lead to an over-estimation of the pre-support function, as the longitudinal stiffness of the entire system must be taken into account in those considerations. On the other hand, the radial action of a systematic pre-support arch is often underestimated or not considered at all.

The longitudinal effect of a pre-support element is less governed by the stiffness of the reinforcement element than by the improvement of the tensile and shear capacity of its surrounding ground.

When grout is used to establish the bond between the reinforcing element and the ground, grouting pressure used for installation, type of grout and grouted length have a paramount influence on the effect and efficiency of the pre-support in particular in soft ground conditions.

Though it has been proven in countless applications that mechanical pre-support has the effects mentioned above, quantification of the effect by numerical analyses methods proved to be difficult involving efforts that go beyond the usual design efforts. Hence, the effect of pre-support is often assessed using simple approaches that result in very conservative assessments, thus underestimating the actual effect of pre-support. In many cases, the effect of pre-support is even ignored in a design and pre-support is viewed merely as an increase of the safety margin rather than a settlement-limiting element of the tunnel support.

<u>Pre-Support in Rock Tunneling:</u> Pre-support installation in fractured, yet competent rock mass types is typically aimed at limiting the over-break during and after excavation. Pre-spiling with steel rebars is a frequent method to keep rock fragments in place (Section 6.5.6). Dependent on the degree of fracturing, the rebars are installed in empty boreholes arranged around the perimeter of the roof, or the boreholes are filled with cement grout prior to insertion of the rebars. Alternatively, perforated steel pipes are used that

are inserted into the boreholes and subsequently grouted. In a severely fractured rock mass where boreholes tend to collapse, self-drilling rock reinforcement pipes are used. With the grouted applications, grout may intrude into cracks and fractures introducing a limited cementing effect of the surrounding material.

In soft rock mass types, where fracturing and limited material strength result in conditions with low overall strength, grouted pipe spiling or grouted pipe arches are used for pre-support. If required, these pre-support measures are combined with groundwater drawdown measures to reduce the joint water pressure and to increase the frictional capacity along the joints.

Permeation grouting of the discontinuities is used to reduce the mass permeability and to increase the overall shear strength by cementing the rock fragments together.

<u>Pre-Support in Soft Ground (Soil) Tunneling:</u> Similar to soft rock, grouted mechanical pre-support measures are used to pre-stabilize soil or soil like ground. Dependent on the susceptibility of the soil to grout, these mechanical pre-support methods are combined with grouting systems that allow penetration of grout into the ground leading to cementation of the ground surrounding the pre-support. Penetrability of the ground and the intended purpose of the pre-support govern the selection of the grouting materials. While grout with standard cements has a limited capability for penetrating ground containing sand or smaller fractions, penetration results can be improved by the use of micro or ultra fine cement products or chemical grouting (resin grouting). The current market offers resin grouting materials with viscosity values close to water.

In many cases, particularly under shallow cover with the groundwater table in the lower part or below the tunnel invert, mechanical pre-support measures are sufficient as long as the support elements are sufficiently locked into the ground over their entire length by an appropriate grouting material. Any additional effect by grout material penetrating voids in vicinity of the installed pre-support is considered an additional benefit.

In very loose, generally non-cohesive ground, ground improvement measures may be required to cement the ground and to decrease the permeability of the soil.

<u>Pre-Support Elements:</u> Most commonly used mechanical pre-support elements include grouted pipe spiling of typically 2-inch (50 mm) diameter perforated steel pipes and rebar spiling using solid No. 8 (25 mm diameter) steel rebars as shown in Figure 9-15. These are primarily installed in the area of the tunnel roof and shoulders, but may also be installed in the sidewall and invert if suitable and required. Grouting of these spiling elements establishes a tight contact between the reinforcement element and the surrounding ground. So-called self-drilling and grouted rebars (type IBO, ISCHEBECK or similar) provide for a very efficient installation of grouted, solid steel bars.

<u>Grouted Pipe Arch Canopy:</u> Pipe arch canopy methods involve a systematic installation of grouted pipes at a spacing of typically 12 inch (300 mm) around the tunnel crown. This installation typically involves one single row of pipes but under critical ground conditions and / or when surface settlements must be restricted may involve a double row of pipes. The pipes are installed at lengths typically not to exceed 15 to 24 meters (50 to 80 feet) using conventional drilling techniques at a shallow lookout angle from the tunnel and ahead of the tunnel excavation. Specialized drill bit and casing systems are utilized that aim at limiting and strictly controlling the over cut, i.e., annular void space between inserted pipe and the surrounding ground. They also provide for direction control and high installation accuracy. Drilling techniques include ODEX®, CENTREX®, ALWAG and similar methods.

The steel pipes are typically perforated and have a diameter of between 4.5 inch and 6 inch (114 mm to 150 mm). The steel pipes are grouted to facilitate contact between steel pipe and the surrounding ground and to create the desired arching effect around the tunnel opening during excavation. Depending on purpose and susceptibility of the ground to grouting, the perforated steel pipes may be grouted either from the single entry point at pipe end within the tunnel or using packers or double packers. Grouting with double packers will allow for targeted grouting with respect to location, grout mix, injected volumes, and pressures. These pipe arch systems have furthered the use of SEM applications in particular in urban settings under shallow overburdens and also in difficult ground conditions.

Figure 9-16 displays the installation of a steel pipe for an arch application for a 3-lane road tunnel in soft ground. The figure displays the steel pipe on a drill jumbo boom and a 4.5 inch (114 mm) steel pipe being drilled near the circumference of the shotcrete initial lining. Figure 9-17 displays previously installed pipe arch steel pipes exposed in the ground when opening a new excavation round.

Figure 9-16 Steel Pipe Installation for Pipe Arch Canopy (Fort Canning Tunnel, Singapore)

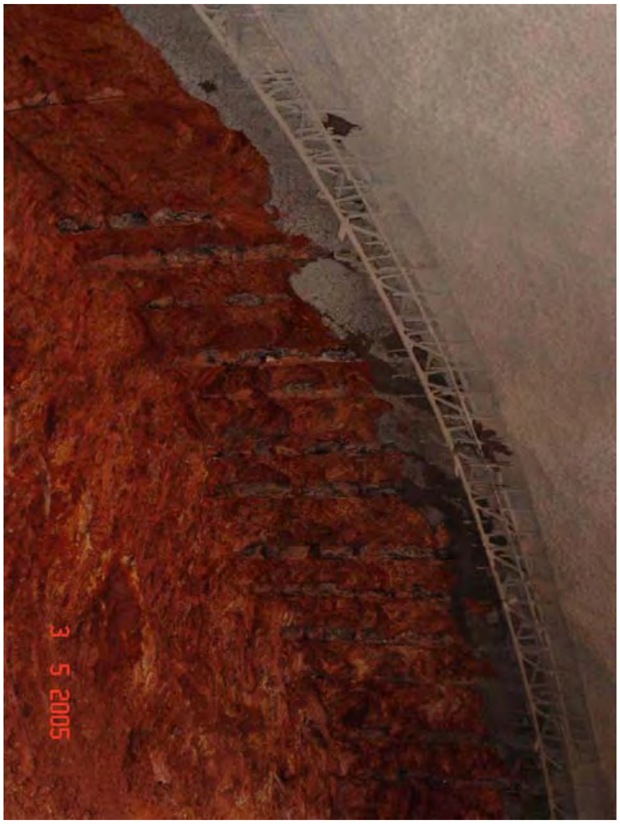


Figure 9-17 Pre-support by Pipe Arch Canopy, Exposed Steel Pipes Upon Excavation of a New Round (Fort Caning Tunnel, Singapore)

Face Doweling: Face doweling forms a specific form of pre-support. Other than the mechanical pre-support installed in the tunnel roof and shoulder area, the face pre-support is installed within the excavation face to stabilize squeezing or raveling ground at the face prior to excavation. Passive elements are installed in the ground and usually grouted in place to increase the tensile and shear strength of the ground material. Since the reinforcement elements have to be excavated during subsequent excavation rounds, fiberglass reinforced resin dowels or pipes are frequently used. Steel elements for face doweling hamper the excavation progress and during excavation their removal transfers tension forces into the ground, promoting ground disturbance ahead of the progressing tunnel face. Face doweling can be combined with application of grouting methods to locally improve the overall strength of the ground within the tunnel cross-section and act with the face dowels.

Face support dowels are usually made of GFRP (glass fiber reinforced polyester resin) and provide significant tensile strength while allowing for easy removal during excavation due to the material composition and low shear resistance.

9.5.4.2 Ground Improvement

Ground improvement measures are primarily aimed at modifying the ground matrix to increase its shear (cohesion) and compressive strengths. An increase of the stiffness (deformation modulus) is coincidental to this improvement. These measures are frequently installed from the surface and well in advance of the tunnel excavation or are applied from within the tunnel ahead of the face. Ground improvement measures range from lowering of the groundwater table or reduction of the pore/joint water pressure to intrusive changes of the ground composition such as jet grouting, soil mixing or ground freezing.

Groundwater Draw Down: Draw down of the groundwater table reduces or eliminates the groundwater inflow into tunnels during construction and increases the effective shear strength of the ground. Groundwater flowing into the tunnel opening during construction not only causes unsafe conditions and increases equipment wear and tear; it also can promote ground instabilities. The reduction of the hydrostatic head reduces the water pressure acting within discontinuities and soil pores. Groundwater draw down can be carried out from the surface or from within the tunnel.

In fine-grained soils (fine sands, silts, clays) the reduction of the pore pressure results in a significant increase of the overall strength of the ground. Where gravity drainage is insufficient, vacuum wells or other means such as drainage by osmosis can be applied.

Permeation Grouting: Permeation grouting is frequently used to cement the ground matrix if it is sufficiently coarse and uniform to achieve reliable grout penetration. Microfine cement or chemical (resin) grouts are used for finer grained soils.

Where soils are not sufficiently uniform or groutable, other measures such as jet grouting or soil mixing are used. These methods actively modify the ground's fabric by mixing the ground with a cementing agent such as cement grout or lime. Jet grouting uses a high-pressure water-grout mix jet to cut the ground and mix it with the stabilizing agent generating improved soil columns of significant diameter. Readers are referred to Ground Improvement Methods Reference Manual (FHWA 2004) for more details.

Ground Freezing: Ground freezing is often considered as 'last resource' due to its high cost when compared to other ground improvement measures. However, ground freezing achieves a high degree of reliability of ground modification. This particularly applies for non-uniform soils. The frozen ground provides groundwater cut-off while its mechanical properties are sufficiently increased to allow an efficient and safe tunnel excavation and support installation under the protection of the frozen soil body. Ground freezing has provided solutions for tunneling under very complex conditions in urban settings.

Readers are also referred to Chapter 7 for discussions about the above ground improvement techniques. Chapter 15 presents a ground freezing application for jacked box tunnels.

9.5.5 Portals

9.5.5.1 General

This section describes the layout of temporary tunnel portal structures for highway tunnels that are frequently built with SEM tunneling. These structures provide a protection against rock fall, and stabilize the portal face from which SEM tunneling commences thus provide start-up condition for safe tunnel excavation.

Shotcrete canopies are also frequently used as an extension of the tunnel and are integrated into the final tunnel portal architecture. The tunnel final lining is cast against these shotcrete canopies and therefore the tunnel internal geometry is uniform from the cut-and-cover (shotcrete canopy) section into the mined tunnel. The shotcrete canopies are backfilled for the final condition.

9.5.5.2 Pre-Support and Portal Collar

The level of weathering and loosening of rock close to the surface must be addressed when starting tunnel construction. Even in generally good rock mass, surface near weathering and loosening requires pre-support at the portal.

After clearing the surface and installing required rock support at the portal face, a row of horizontal pre-spiling or grouted steel pipes should be installed to provide pre-support for the initial excavation rounds for the tunnel construction. Dependent on the degree and depth of weathering, this pre-support is typically 10 ft (3 m) to 60 ft (18 m) long and the reinforcement elements are grouted in place. The pre-support elements are typically spaced at 12-inch (0.30 m) centers around the future tunnel opening. Such tunnel pre-support at the portal is shown in Figure 9-18.

Figure 9-18 Pre-Support at Portal Wall and Application of Shotcrete for Portal Face Protection (Devil's Slide Tunnels, California)

Following the pre-support installation, a reinforced shotcrete collar should be installed that is tied in with the protruding pre-support elements. The collar shall follow the tunnel perimeter extending from one sidewall to the other. In soft ground, the collar may extend over the entire tunnel perimeter. The collar provides stability to the ground in the immediate vicinity of the future tunnel opening and is structurally connected to the initial shotcrete lining for the first round of tunnel excavation.

9.5.5.3 Shotcrete Canopy

The shotcrete canopy comprises reinforced shotcrete and lattice girders. The canopy is founded on a strip foundation that extends over the entire length of the canopy. The length of the canopy is dependent on the rock fall protection required and on local conditions such as wind loads, temporary ventilation requirements, and needs of the final tunnel structure.

Portal canopies have to be designed for rock fall and snow loads, construction loads, dead loads, and any wind loads, as dictated by local site conditions. The canopy also serves as a counter form for final lining installation in the portal area. Figure 9-19 displays construction of a shotcrete canopy whereas the first three lattice girders and reinforcement have been placed and shotcrete is being sprayed against an expanded metal sheet placed on the outside of the lattice girders.

Figure 9-19 Shotcrete Canopy Construction after Completion of Portal Collar and Pre-support (Schürzeberg Tunnel, Germany)

9.6 STRUCTURAL DESIGN ISSUES

9.6.1 Ground-Structure Interaction

The SEM realizes excavation and support in distinct stages with limitations imposed on size of excavation and length of round followed by the application of initial support measures. In particular the shotcrete lining has an interlocking function and provides an early, smooth support. To adequately address this sequenced excavation and support approach the structural design shall be based on the use of numerical, i.e. finite element, finite difference, or distinct element methods (see also Chapter 6). These numerical methods are capable of accounting for ground structure interaction. They allow for representation of the ground, the structural elements used for initial and final ground support, and enable an approximation of the construction sequence.

Embedded frame analyses have limitations in adequately describing the ground structure interaction. Due to this and the fact that these methods can not simulate excavation sequencing their use shall be limited to applications where the ground structure interaction phenomenon, in particular development of a ground-supporting arch, is of secondary importance. This is for example the case for shotcrete canopies that are frequently erected at tunnel portals as freestanding or backfilled reinforced shell structures and tunnel final linings.

9.6.2 Numerical Modeling

9.6.2.1 Two (2)-Dimensional and Three (3)-Dimensional Calculations

In general, use of two-dimensional models is sufficient for line structures. Where three-dimensional stress regimes are expected, such as at intersections between main tunnel and cross passages, or where detailed investigations at the tunnel face are undertaken such as for the behavior of pipe arch pre-supports, three-dimensional models should be used.

9.6.2.2 Material Models

In representing the ground, the structural models shall account for the characteristics of the tunneling medium. Material models used to describe the behavior of the ground shall apply suitable constitutive laws to account for the elastic, as well as inelastic ranges of the respective materials. For example when tunneling in rock, intact rock as well as the rock structure, i.e., the presence of discontinuities shall be taken into account. It is customary to apply Mohr-Coulomb or Drucker-Prager failure criteria for the representation of both rock and soil materials. Finite element programs that were developed initially for the simulation of underground excavations in rock such as Phase 2 by Rockscience, Inc. also allow use of rock mass material behavior using Hoek and Brown rock mass parameters (Hoek and Brown, 1980, 2002).

9.6.2.3 Ground Loads - Representation of the SEM Construction Sequence

Tunnel excavation causes a disturbance of the initial state of stress in the ground and creates a three-dimensional stress regime in the form of a bulb around the advancing tunnel face. Such a stress regime is indicatively displayed in Figure 9-20.

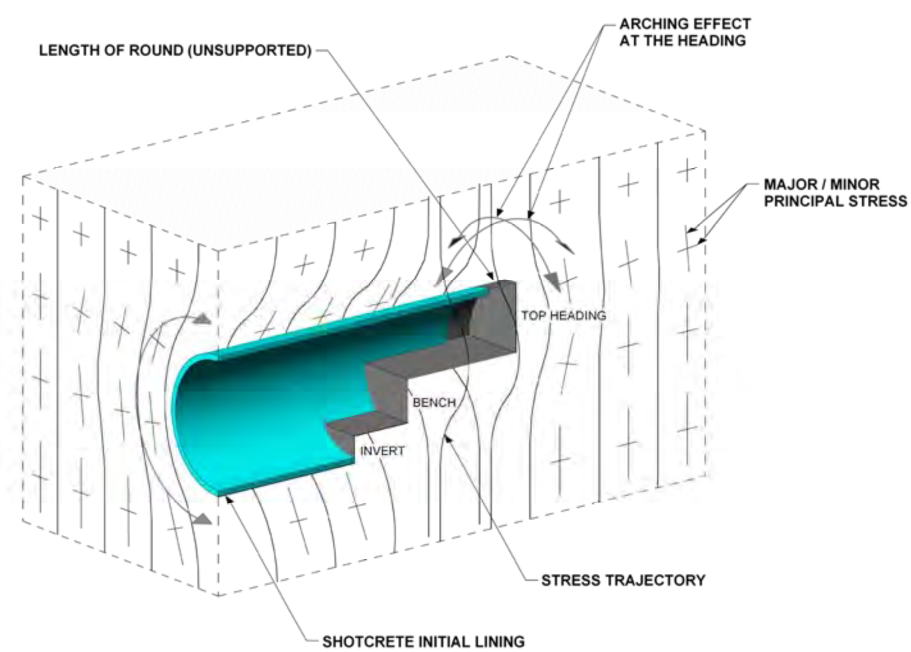


Figure 9-20 Stress Flow Around Tunnel Opening (after Wittke, 1984 and Kuhlmann)

Far ahead of the advancing tunnel face the initial state of stress is represented by vertical and horizontal stress trajectories denoting major and minor principal stresses respectively (assuming that vertical stresses are higher than horizontal stresses in a geostatic stress field). At the tunnel face the stresses flow around the tunnel opening arching ahead of the tunnel excavation and behind it onto the newly constructed initial lining in longitudinal direction and to the sides of the opening perpendicular to the tunneling direction. At a distance where the tunnel is no longer affected by the three-dimensional stress conditions around the active tunnel face two-dimensional arching conditions are established.

The extent of the stress disturbance around an active heading depends mainly on ground conditions, size of the excavation and length of round. This disturbance begins up to two excavation diameters ahead of the active tunnel face as shown indicatively in Figure 9-21. The SEM design dictates limits on excavation size and length of round and prescribes installation of initial support elements often following each individual excavation round directly or shortly thereafter. These requirements are portrayed in the Excavation and Support Class (ESC, see Chapter 9.5.3). Initial support elements are therefore installed within the shelter of a load-carrying arch around the newly created opening in an area where some pre-deformation has occurred. As the excavation of the tunnel advances the shotcrete hardens from an initially "green" shotcrete and becomes fully loaded at a distance of about one to two tunnel diameters from the face. Such sequencing combined with early support installation contributes to the development of the self-supporting capability of the ground. It further aids in minimizing deformations and ground loosening.

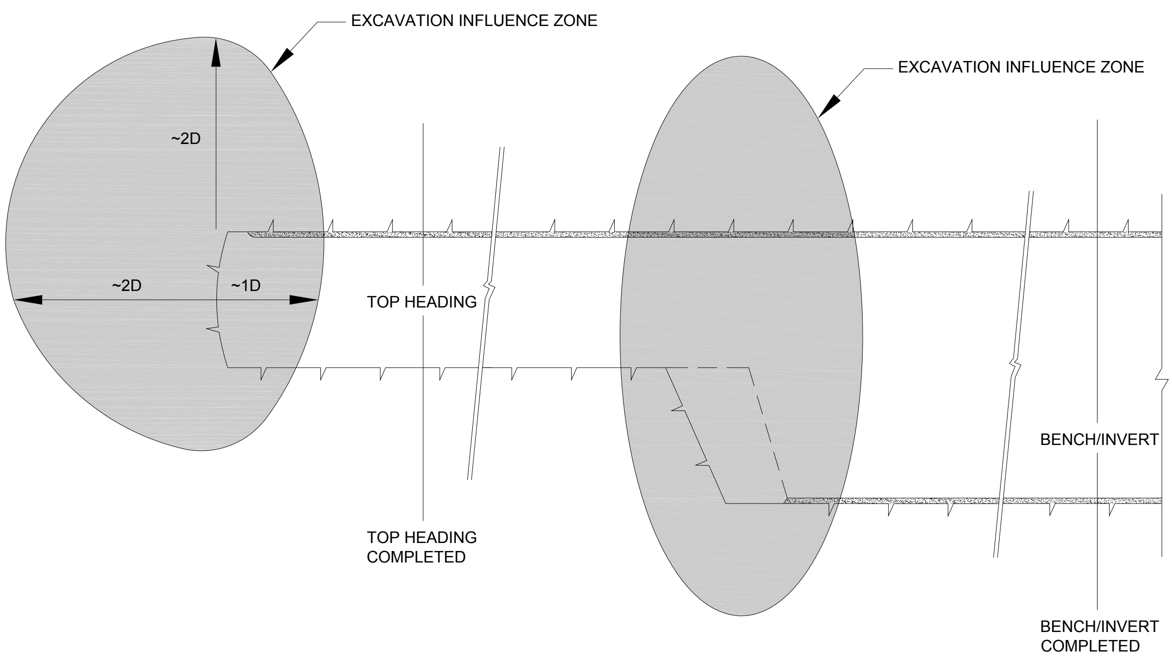


Figure 9-21 SEM Tunneling and Ground Disturbance (after OGG, 2007)

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

Excavation and support installation sequencing can be readily realized in three-dimensional models. In two-dimensional modeling, however, auxiliary techniques must be utilized. A frequently utilized approach relies on the use of ground modulus reduction within the excavation perimeter prior to the insertion of the initial lining elements into the model. Other techniques involve the use of supporting forces applied to the circumference of the tunnel opening. Because of its frequent use the ground modulus reduction approach is used in describing a typical two-dimensional modeling sequence of SEM tunnel excavation and support of a line structure below. A calculation example is provided in Paragraph 9.7.3.7.

- Represent the in-situ stresses including the geostatic stress field and surface loads as applicable.
- Represent the excavation of the respective round by reducing the elastic modulus of the ground located within the geometric boundaries of the round to about 40% - 60% of its original value. The purpose of the modulus reduction is to achieve a pre-deformation of the ground prior to installation of the initial support measures. The extent of modulus reduction is only within the region where excavation takes places, i.e. a drift (top heading, bench, invert). It is an arbitrary measure applied to simulate an otherwise three-dimensional stress distribution at the face (see Figure 9-20) in two-dimensional computations. The value of 40-60% is a frequently used reduction amount and represents a typical range (Mohr and Pierau, 2004 and Coulter and Martin, 2004). A higher reduction will yield larger, a lower reduction will yield smaller deformations of the surrounding ground. A sensitivity analysis related to the actual reduction value is typically part of the computations.
- Activate the initial support elements per design assumptions to represent the installation of initial support. Because the shotcrete will not have developed its design strength at this stage reduced shotcrete elastic properties (modulus) are initially taken into account and amount to about 1/3 of the hardened shotcrete. During subsequent simulation stages the shotcrete modulus is then increased to its 28-day design strength to represent a fully hardened shotcrete lining.
- Remove the ground elements within the respective drift thereby completing excavation and support within the round.
- Repeat this sequence until all drifts of the final tunnel cross section geometry have been excavated and supported.

Once accomplished, this completes modeling of the tunnel excavation and installation of initial support. The installation of the final tunnel lining generally occurs once all deformations of the tunnel opening have ceased. To account for this fact the calculations perform installation of the final lining into a stress-free state. The final lining becomes loaded only in the long-term resulting from a (partial) deterioration of the initial support (shotcrete initial lining and rock bolts if any), rheological long-term effects and ground water if applicable. Although modeling of the final lining is often undertaken by embedded frame analyses (see Chapter 10) its analysis within a ground-structure interaction numerical model will be most appropriate and can follow directly after the initial support is installed as follows:

- Activate the structural elements representing the final tunnel lining.
- If the modeling was carried out with temporary rock reinforcing elements without corrosion protection then all such supporting elements are deactivated.
- If the ground water is generally aggressive and it may be assumed that the shotcrete initial lining will deteriorate long-term then it is assumed that no contributing support function may be derived from it for long-term considerations. This has been traditionally assumed on projects such as the Lehigh Tunnel, Cumberland Gap Tunnels and on NATM tunnels of the Washington, DC Metro. Washington Metropolitan Area Transit Authority (WMATA) has substantial experience with the design and construction of NATM tunnels in both soft ground and rock (Rudolf et al., 2007). To date it is

customary on WMTA projects to assume that the shotcrete initial lining will deteriorate over time. Such computational approach will yield a conservative final lining design. Due to the nowadays high quality shotcrete fabrication however, and in particular in non-aggressive ground and ground water conditions it is admissible to assume that when the shotcrete initial lining is more than approximately 6-inch (150 mm) thick then 50% of its structural capacity may be taken into account in the final lining computations. The combined removal of initial support elements (rock reinforcement and shotcrete initial lining) will result in ground loads imposed onto the final lining in the long-term.

- In addition to the ground loads, the concrete lining will be loaded with hydrostatic loads in un-drained or partially drained waterproofing systems. This load case generally occurs well before the final lining is loaded with any ground loads and shall be considered separately in the calculations.
- Final lining calculations consider the existence of the waterproofing system, which is embedded between the initial shotcrete lining and the final lining. A plastic membrane will act as a de-bonding layer in terms of the transfer of shear stresses. Therefore simulation techniques should be used to simulate this "slip" layer. This is accomplished by only allowing the transfer of radial forces from the initial lining onto the final lining.

9.6.2.4 Ground Stresses and Deformations

Each step involving the simulation of excavation and installation of initial support allows for analysis of the ground response expressed in deformations, strains, and stresses. Both, elastic and, if yielded, inelastic portions of strains can be obtained and used to evaluate the state of stress in the ground and its capacity reserves. Stresses, strains and section forces are available in ground support elements such as dowels and rock bolts. The computational programs (for a selected list see Chapter 6) often provide such information in a user friendly display using numeric and graphic formats.

9.6.2.5 Lining Forces

Section forces and stresses are available for beam (2-D) or shell (3-D) elements. Section force and moment combinations are used to evaluate the capacity of the initial shotcrete and final concrete linings using ACI 318 or other accepted concrete design codes. Acceptance of codes is generally an owner driven process. For example, Washington Metropolitan Area Transit Authority (WMATA) allowed the use of the German Industry Standard DIN 1045 for the design of plain (unreinforced) cast-in-place concrete final linings (Rudolf et al., 2007 and Gnilsen, 1986).

Based on this evaluation the adequacy of lining thickness and its reinforcement (if any) is assessed. If the selected dimensions are found not to be adequate then the model must be re-run with increased dimensions and/or reinforcement. The process is an iterative approach until the design codes are satisfied.

These calculations do not distinguish between the type of lining application and therefore shotcrete and cast-in-place final linings are treated in the same manner within the program using the material properties and characteristics of concrete.

9.6.2.6 Ground Reinforcing Elements

Ground reinforcing elements are rock bolts and dowels. These are activated in the computations in accordance with the design of the SEM excavation and support installation. Once implemented and loaded during the simulation of excavation and support, section forces and stresses are available to evaluate their adequacy. Stresses and forces are compared with the capacity of the individual dowel or bolt.

9.6.2.7 Calculation Example

A calculation example (Appendix F) demonstrates the SEM tunneling analysis and lining design of a typical two-lane highway tunnel using the finite element code Phase2 by Rocscience, Inc. The calculation is carried out in stages and follows the approach laid out in 9.6.2.3 above and evaluates ground reaction as indicated in 9.6.2.4 and evaluates support elements as described in 9.6.2.5 and 9.6.2.6.

9.6.3 Considerations for Future Loads

Mainly due to its flexibility and ability to minimize surface settlements often in combination with ground improvement methods the SEM is frequently utilized for the construction of roadway tunnels in urban settings. In particular under such circumstances it is important to consider any future loads that may be imposed onto the tunnel for which the final linings must be designed. Such loads include among others buildings, foundations, and miscellaneous underground structures fulfilling future infrastructure needs. These can be readily implemented in the computation approach presented above in the form of external or internal modeling loads.

9.7 INSTRUMENTATION AND MONITORING

9.7.1 General

An integral part of SEM tunneling is the verification by means of in-situ monitoring of design assumptions made regarding the interaction between the ground and initial support as a response to the excavation process.

For this purpose, a specific instrumentation and monitoring program is laid out in addition to general instrumentation programs connected with the overall tunneling work, i.e. surface and subsurface instrumentation. The SEM tunnel instrumentation aims at a detailed and systematic measurement of deflection of the initial lining. While monitoring of deformation is the main focus of instrumentation historically stresses in the initial shotcrete lining and stresses between the shotcrete lining and the ground were monitored to capture the stress regime within the lining and between lining and ground. Reliability of stress cells, installation complexity and difficulty in obtaining accurate readings have nowadays led to the reliance on deformation monitoring only in standard tunneling applications. Use of stress cells is typically reserved for applications where knowledge of the stress conditions is important, for example where high and unusual in-situ ground stresses exist or high surface loads are present in urban settings.

Monitoring data are collected, processed and interpreted to provide early evaluations of:

- Adequate selection of the type of initial support and the timing of support installation in conjunction with the prescribed excavation sequence
- Stabilization of the surrounding ground by means of the self-supporting ground arch phenomenon,
- Performance of the work in excavation technique and support installation
- Safety measures for the workforce and the public
- Long-term stress/settlement behavior for final safety assessment
- Assumed design parameters, such as strength properties of the ground and in-situ stresses used in the structural design computations (see Chapter 9.7).

Based on this information, immediate decisions can be made in the field concerning proper excavation sequences and initial support in the range of the given ground response classes (GRC) and with respect to the designed excavation and support classes (ESC).

9.7.2 Surface and Subsurface Instrumentation

The general instrumentation should include surface settlement markers, cased deep benchmarks, sub-surface shallow and deep settlement indicators, inclinometers, multiple point borehole extensometers, and piezometers (see Chapter 15).

The locations, types and number of these instruments should be determined by consultations between the civil, structural, geotechnical and SEM design teams to provide information on surface and subsurface structure settlements and to complement the SEM tunnel instrumentation readings.

9.7.3 Tunnel Instrumentation

Deformation Measurements Instruments are installed in the tunnel roof and at selected points along the tunnel walls to monitor vertical, horizontal, and longitudinal (in tunnel direction) deformation components. The number of points and their detailed location depends on the size of the tunnel and the excavation sequencing in multiple drift applications. As a minimum, the wall of each drift (including temporary) should be equipped with a device capable of measuring deformations. It is customary to install optical targets for this purpose. Figure 9-22 shows a series of deformation monitoring cross sections using optical targets in a SEM tunnel. Optical targets are the white reflecting points arranged in the tunnel roof and tunnel sidewalls.

Figure 9-22 Deformation Monitoring Cross Section Points (Light Rail Bochum, Germany)

Stress Measurements If stress information is sought then measurements should be taken with a direct measuring tool that does not rely on any further conversions from say strains to stresses. For example, instruments based on strain gage principles require the knowledge of the elastic modulus of the material to covert strains to stresses. This introduces an additional parameter that must be estimated thus introducing a secondary uncertainty.

Stress measurements within shotcrete linings are frequently carried out using hydraulic pressure cells filled with mercury whereas ground stress measurements are carried out with cells filled with oil. If stress measurements are to be monitored then ground load cells and concrete pressure cells should be grouped in pairs.

9.7.4 SEM Monitoring Cross Sections

Monitoring devices are grouped into monitoring cross sections (MCS). These MCS are depicted with their respective instrument layout indicating location and number of instruments within that MCS. Typical MCSs are shown on the design drawings for each individual tunnel cross-section geometry and excavation sequence. Locations of the respective monitoring cross-sections are shown on dedicated instrumentation drawings by station references. An example deformation MCS is shown in Figure 9-23.

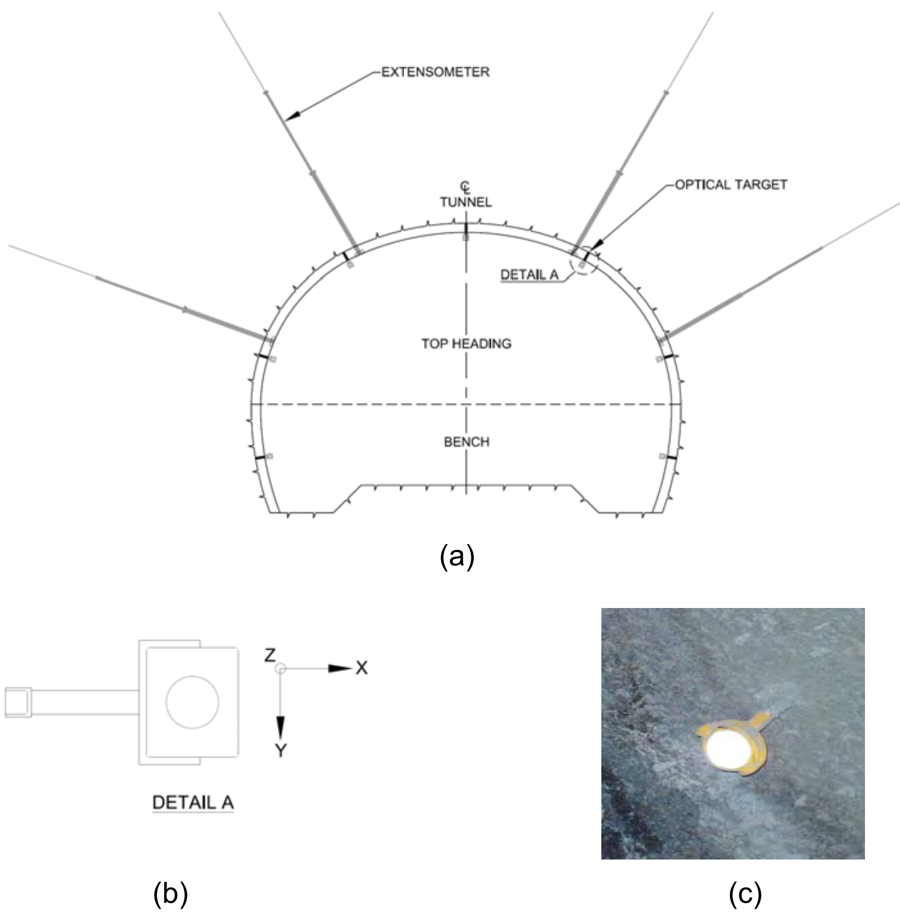


Figure 9-23 Typical SEM Deformation Monitoring Cross Section - a) Typical Tunnel Monitoring Cross Section displaying Extensometers and Optical Targets, b) Detail A, view of Optical target displaying axes of measurement: Y=Vertical Displacement, X=Lateral Displacement, Z=Longitudinal Displacement, c) Image of Optical Target in place.

During execution the installation of all MCSs is documented by a detailed description of the geological and tunneling conditions in the field using sketches showing the exact location of the instruments and the actual thickness of the shotcrete lining.

9.7.5 Interpretation of Monitoring Results

All readings must be thoroughly and systematically collected and recorded. An experienced SEM tunnel engineer, often the SEM tunnel designer, must evaluate the data, occasionally complemented by visual observations of the initial shotcrete lining for any distress, for example as indicated by cracking. To establish a direct relationship between the behavior of the tunnel and the ground as these react to tunnel excavation it is recommended to portray the development of monitoring values as a function of the tunnel progress. This involves a combined graph showing the monitoring value (i.e. deformation, stress or other) vs. time and the tunnel progress vs. time. An example is shown in Figure 9-24. In this example a prototypical deformation of a surface settlement point located above the tunnel centerline has been graphed on the ordinate (left vertical axis) vs. time on the horizontal axis. The same time horizontal axis is used to portray the tunnel excavation progress by station location on the right vertical axis. As can be seen from this graph the surface settlement increases as the top heading and later bench/invert faces move towards and then directly under that point and gradually decrease as both faces again move away from the station location of the surface settlement point. The settlement curve shows an asymptotic behavior and becomes near horizontal as the faces are sufficiently far away from the monitoring point indicating that no further deformations associated with tunnel excavation and support occur in the ground indicating equilibrium and therefore ground stability.

The evaluation of monitoring results along with the knowledge of local ground conditions portrayed on systematic face mapping sheets forms the basis for the verification of the selected excavation and support class (ESC) or the need to make any adjustments to it.

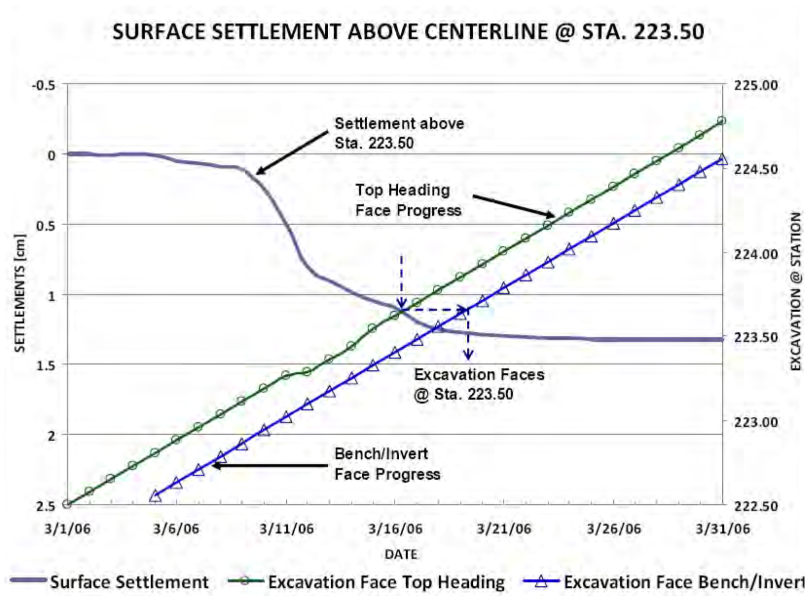


Figure 9-24 Prototypical Monitoring of a Surface Settlement Point Located Above the Tunnel Centerline in a Deformation vs. Time and Tunnel Advance vs. Time Combined Graph

9.8 CONTRACTUAL ASPECTS

SEM construction requires solid past experience and personnel skill. This skill relates to the use of construction equipment and handling of materials for installation of the initial support including shotcrete, lattice girders, pre-support measures, and rock reinforcing elements and even more importantly observation and evaluation of the ground as it responds to tunneling. It is therefore important to invoke a bidding process that addresses this need formally by addressing contractor qualifications and skills and payment on a unit price basis described below. For general contractual aspects refer to Chapter 14.

9.8.1 Contractor Pre-Qualification

It is recommended that the bidding contractors be pre-qualified to assure a skilled SEM tunnel execution. This pre-qualification can occur very early on during the design development but at a minimum should be performed as a separate step prior to soliciting tunnel bids. On critical SEM projects such as the NATM tunneling at Russia Wharf in Boston in the late 90's the project owner solicited qualifications from contractors at the preliminary design stage. This pre-qualification resulted in a set of pre-qualified contractors that were invited to comment on the design at the preliminary and intermediate design stages. This early process ensured that contractors were aware of the upcoming work and could plan ahead in assembling a qualified work force. Pre-qualification documents shall identify the scope of work and call for a similar experience gained on past projects by the tunneling company and key tunneling staff including project manager, tunnel engineers, and tunnel superintendents. As a minimum the documents lay out description of ground conditions, tunnel size and length, excavation and support cycles, and any special methods intended for ground improvement.

9.8.2 Unit Prices

It is recommended that SEM tunneling be procured within a unit price based contract. Unit prices suit the observational character of SEM tunneling and the need to install initial support in accordance with a classification system and amount of any additional initial or local support as required by field conditions actually encountered. The following shall be bid on a unit price basis:

- Excavation and Support on a linear foot basis for all excavation and installation of initial support per Excavation and Support Class (ESC). This shall include any auxiliary measures needed for dewatering and ground water control at the face.
- Local support measures including:
    - Shotcrete per cubic yard installed.
    - Pre-support measures such as spiling, canopy pipes and any other support means such as rock bolts and dowels, lattice girders, and face dowels shall be paid per each (EA) installed.
    - Instrumentation and monitoring shall be paid for either typical instrument section (including all instruments) or per each instrument installed. Payment will be inclusive of submitted monitoring results and their interpretation.
    - Ground improvement measures per unit implemented, for example amount of grout injected including all labor and equipment utilized.

Waterproofing and final lining installed to complete the typical SEM dual lining structure may be procured on either lump sum basis or on a per tunnel foot basis.

The quantity of local support (additional initial support) measures shall be part of the contract to establish a basis for bid.

9.9 EXPERIENCED PERSONNEL IN DESIGN, CONSTRUCTION, AND CONSTRUCTION MANAGEMENT

Because the SEM relies on tunneling experience it is imperative that experienced personnel be assigned from the start of the project, i.e., in its planning and design phase. The SEM design must be executed by an experienced designer. At this level of project development it is incumbent upon the owner to select a team that includes a tunnel designer with previous, proven, and relevant SEM tunneling design and construction experience.

The SEM tunnel contract documents have to identify minimum contractor qualifications. In this case it is secondary whether the project is executed in a design-bid-build, design-build or any other contractual framework. For example, if the project uses the design-build framework then it is imperative that the builder take on an experienced SEM tunnel designer.

The construction contract documents must spell out minimum qualifications for the contractor’s personnel that will initially prepare and then execute the SEM tunnel work. This is the case for field engineering, field supervisory roles and the labor force who must be skilled. SEM contract documents call for a minimum experience of key tunneling staff by number of years spent in the field on SEM projects of similar type. Experienced personnel will include Senior SEM Tunnel Engineers, Tunnel Superintendents and Tunnel Foremen. All of such personnel should have a minimum of ten (10) years SEM tunneling experience. These personnel are charged with guiding excavation and support installation meeting the key requirements of SEM tunneling:

- Observation of the ground
- Evaluation of ground behavior as it responds to the excavation process
- Implementation of the “right” initial support.

Knowledgeable face mapping, execution of the instrumentation and monitoring program, and interpretation of the monitoring results aid in the correct application of excavation sequencing and support installation. Figure 9-24 displays a typical face mapping form sheet that is used to document geologic conditions encountered in the field. While this form sheet portrays mapping for rock tunneling, mapping of soft ground conditions is similar and lays out the characteristics of anticipated soil conditions. Face mapping should occur for every excavation round and be formally documented and signed off by both the contractor and the owner’s representative.

The Senior SEM Tunnel Engineer is generally the contractor’s highest SEM authority and supervises the excavation and installation of the initial support, installation of any local or additional initial support measures and pre-support measures in line with the contract requirements and as adjusted to the ground conditions encountered in the field. As a result the ground encountered is categorized in accordance with the contract documents into ground response classes (GRCs) and the appropriate excavation and support classes (ESC) per contract baseline. Any need for additional initial support and/or pre-support measures is assessed and implemented. This task is carried out on a daily basis directly at the active tunnel face and is discussed with the Owner’s representative for each round. The outcome of this process is subsequently documented on form sheets that are then signed by the Contractor’s and Owner’s representatives for concurrence.

This frequent assessment of ground conditions provides for a continuous awareness of tunneling conditions, for an early evaluation of adequacy of support measures and as needed for implementation of contingency measures that may involve more than additional initial support means. Such contingency measures may include heavy pre-support and face stabilization measures or even systematic ground improvement measures.

To be able to support this on-going evaluation process on the owner's behalf the construction management (CM) and inspection team must also include SEM experience. These CM supervisory personnel are independent of the executing party and it is recommended that it include a designer's representative. Represented in the field the designer is able to verify design assumptions and will aid in the implementation of the design intent.

Engineering Geological Tunnel Face Mapping

DATE:	TIME:	STATION:	ROUND NO:	ROCK SUPPORT CLASS:	ROUND LENGTH:	LATTICE GIRDER NO:

SAMPLE NO: PHOTO NO:

WEATHERING					
unweathered	slightly	moderately	highly	completely	residual soil

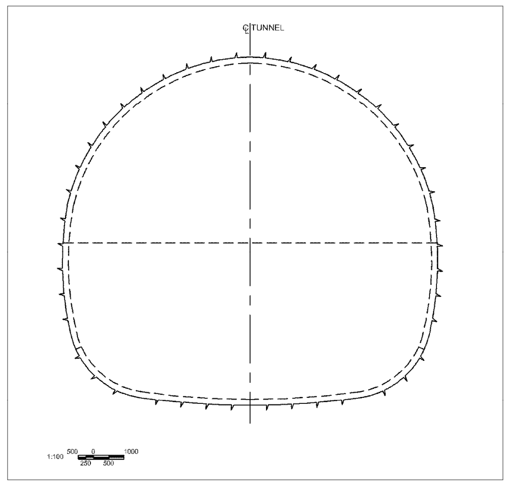
ROCK STRENGTH						
extremely strong	very strong	strong	medium strong	weak	very weak	extr. weak

COHESIVE SOIL					
very soft	soft	firm	stiff	very stiff	hard

WATER CONDITION						
dry	damp	wet	dripping	seepage	flowing	(l/sec)

CHARACTERISTICS OF DISCONTINUITIES

TYPE	ORIENTATION	ROUGHNESS	PERSISTANCE (ft)					SPACING (in)						FILLING			NO. OF SETS
	(Dir./Dip Angle)	S U P r s sl	< 3'	3' – 10'	10' – 30'	30' – 100'	> 100'	< 1"	3" – 1"	8" – 3"	25" – 8"	80" – 25"	> 80"				



Type		Roughness				Spacing	
Fault	F	STEPPED	S	rough	r	Extremely close	< 1 in
Joint	J			smooth	s	Very Close	3 -1 in
Slickensided	Sl			slickensided	sl	Close	8 – 3 in
Fractured	Fr	UNDULATING	U	rough	r	Moderate	25 – 8 in
Schistosity	Sc			smooth	s	Wide	80 – 25 in
Bedding Plane	B			slickensided	sl	Very wide	> 80 in
Foliation	Fi	PLANAR	P	rough	r		
Shear	Sn			smooth	s		
Fold	Fo			slickensided	sl		

Persistance		Type of Filling		Fracture Frequency per yard	
Very low	< 3 ft	clay	: cl	Very poor	> 15
Low	3 – 10 ft	Mn/Fe - mineralisations	: mfm	Poor	15 – 8
Medium	10 – 30 ft	mylonitic fault gouge	: g	Fair	8 – 5
High	30 – 100 ft	mylonitic fault breccia	: fb	Good	5 – 1
Very High	> 100 ft	cemented, other	: co	Excellent	< 1

Lithological Properties	Rock Mass Behavior

Client's Representative	Contractor	SEM Tunnel Subconsultant

Figure 9-25 Engineering Geological Tunnel Face Mapping

However, it is often the case that the CM role is filled by a construction management entity that has been assigned an overall role for a project of which the tunneling may only be a subset of the work. If this is the case it is important that the CM be thoroughly familiar with the SEM tunnel design and its design basis. For this purpose it is recommended that the CM participate in the design review process during design development from an early stage through the bidding of the tunnel work. If it is not possible to integrate the tunnel designer within the CM staff then the CM should be augmented by third party SEM experienced personnel who then oversee the tunnel execution in the field.

The key of safe and successful SEM tunneling is a solid knowledge of SEM principles and thorough experience with its execution.

CHAPTER 10
TUNNEL LINING

10.1 INTRODUCTION

This chapter covers considerations for the structural design, detailing and construction of tunnel linings for highway tunnels focusing on mined or bored tunnels. Tunnel linings are structural systems installed after excavation to provide ground support, to maintain the tunnel opening, to limit the inflow of ground water, to support appurtenances and to provide a base for the final finished exposed surface of the tunnel. Tunnel linings can be used for initial stabilization of the excavation, permanent ground support or a combination of both. The materials for tunnel linings covered in this chapter are cast-in-place concrete lining (Figure 10-1), precast segmental concrete lining (Figure 10-2), steel plate linings (Figure 10-3), and shotcrete lining (Figure 10-4). Uses, design procedures, detailing and installation are covered in subsequent sections of this chapter. The final finishes are not specifically addressed.

Figure 10-1 Cumberland Gap Tunnel

Cast-in-place concrete linings are generally installed some time after the initial ground support. Cast-in-place concrete linings are used in both soft ground and hard rock tunnels and can be constructed of either reinforced or plain concrete. Cast-in-place concrete linings can take on any geometric shape, with the shape being determined by the use, mining method and ground conditions.

Precast concrete linings are used as both initial and final ground support (Figure 10-2). Segments in the shape of circular arcs are precast and assembled inside the shield of a tunnel boring machine to form a ring. If necessary they can be used in a two pass system as only the initial ground support. Initial Support Segments for a two pass system are often lightly reinforced and rough cast. The second pass or final lining typically is cast-in-place concrete. Precast concrete linings can also be used in a one pass system where the segments provide both the initial and final ground support. One pass precast segmental concrete linings are cast to strict tolerances and are provided with gaskets and bolted together to reduce the inflow of water into the tunnel.

Figure 10-2 Precast Segmental Lining

Steel plate linings (liner plates) are a type of segmental construction where steel plates are fabricated into arcs that typically are assembled inside the shield of a tunnel boring machine to form a ring. The steel plate lining may form the initial and final ground support. The segments are provided with gaskets to limit the inflow of ground water into the tunnel. Steel plates are also used in lieu of lagging where steel ribs are used as the initial ground support. With the advent of precast concrete segments, liner plates are not used as much as previously.

Figure 10-3 Baltimore Metro Steel Plate Lining

As discussed in Chapter 9, shotcrete is a pneumatically applied concrete that is used frequently as an initial support but now with the advances in shotcrete technology permanent shotcrete lining is designed and constructed in conjunction with sequential excavation method (SEM) tunneling (Chapter 9). One of the first applications of final shotcrete lining in the United States is at Lehigh Tunnel No. 2 of Pennsylvania Turnpike. Shotcrete can take on a variety of compositions as discussed in Chapters 9 and 16. It can be applied over the exposed ground, reinforcing steel, welded wire fabric or lattice girders. It can be used in conjunction with rock bolts and dowels, it can contain steel or plastic fibers and it can be composed of a variety of mixes. It is applied in layers to achieve the desired thickness. Chapter 16 addresses using shotcrete for concrete lining repairs.

Figure 10-4 New Lehigh Tunnel on Pennsylvania Turnpike Constructed with Final Shotcrete Lining

Cross passages and refuge areas are usually mined by hand after the main tunnel is excavated. These areas, due to their unique shape and small areas are typically lined with cast-in-place concrete. There is insufficient quantity involved in the lining of these features to make prefabricated linings economic.

10.1.1 Load and Resistance factor Design (LRFD)

The design of tunnel linings, with the exception of steel tunnel lining plates, is not addressed in standard design codes. This chapter is intended to establish procedures for the design of tunnel linings utilizing the American Association of State Highways and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, current edition.

LRFD is a design philosophy that takes into account the variability in the prediction of loads and the variability in the behavior of structural elements. It is an extension of the load factor design methodology that has been in use for a number of years. This chapter is intended to assist the designer in the application of the LRFD specifications to tunnel lining design and to provide for a uniform interpretation of the AASHTO LRFD specification as it applies to tunnel linings.

10.2 DESIGN CONSIDERATIONS

10.2.1 Lining Stiffness and Deformation

Tunnel linings are structural systems, but differ from other structural systems in that their interaction with the surrounding ground is an integral aspect of their behavior, stability and overall load carrying capacity. The loss or lack of the support provided by the surrounding ground can lead to failure of the lining. The ability of the lining to deform under load is a function of the relative stiffnesses of the lining and the surrounding ground. Frequently, a tunnel lining is more flexible than the surrounding ground. This flexibility allows the lining to deform as the surrounding ground deforms during and after tunnel excavation. This deformation allows the surrounding ground to mobilize strength and stabilize. The tunnel lining deformation allows the moments in the tunnel lining to redistribute such that the main load inside the lining is thrust or axial load. The most efficient tunnel lining is one that has high flexibility and ductility.

A tunnel lining maintains its stability and load carrying capacity through contact with the surrounding ground. As load is applied to one portion of the lining, the lining begins to deform and in so doing, develops passive pressure along other portions of the lining. This passive pressure prevents the lining from buckling or collapsing. Ductility in the lining allows for the creation of “hinges” at points of high moment that relieve the moments so that the primary load action is axial force. This ductility is provided for in concrete by the formation of cracks in the concrete. Under reinforcing or no reinforcing help promote the initiation of the cracks. The joints in segmental concrete linings also provide ductility. In steel plate linings, the negligible bending stiffness of the steel plates and the inherent ductility of steel allow for the creation of similar hinges.

10.2.2 Constructability Issues

Each tunnel is unique. Ground conditions, tunneling means and methods, loading conditions, tunnel dimensions and construction materials all vary from tunnel to tunnel. Each tunnel must be assessed on its own merits to identify issues that should be considered during design such that construction is feasible. Some common elements that should be considered are as follows:

Materials: Selection of tunnel lining materials should be made to facilitate transportation and handling of the materials in the limited space inside a tunnel. Pieces should be small and easily handled. Piece lengths should be checked to ensure that they can negotiate the horizontal and vertical geometry of the tunnel. Materials should be nontoxic and nonflammable.

Details: Detailing should be performed to facilitate ease of construction. For example, sloping construction joints in cast-in-place concrete linings can eliminate the difficulty associated with building a bulkhead against an irregular excavated surface.

Procedures: Construction procedures should be specified that are appropriate for conditions encountered in the tunnel; conditions that are often moist or wet, sometimes even with flowing water. Allow means and methods that do not block off portions of the tunnel for significant periods of time. The entire length of the tunnel should be available as much as practical.

10.2.3 Durability

Tunnels are expensive and are constructed for long term use. Many existing tunnels in the United States have been in use for well over one hundred years with no end in sight to their service lives. Having a tunnel out of service for an extended period of time can result in great economic loss. As such, details and materials should be selected that can withstand the conditions encountered in underground structures. All structures, including tunnels require inspection, periodic maintenance and repair. Chapter 17 discusses tunnel inspection, maintenance, and rehabilitation. Nonetheless, detailing should be such that anticipated maintenance is simplified and long term durability is maximized.

Highway tunnels can also be exposed to extreme events such as fires resulting from incidents inside the tunnel. Tunnel lining design should consider the effects of a fire on the lining. The lining should be able to withstand the heat of the fire for some period of time without loss of structural integrity. The length of time required will be a function of the intensity of the anticipated fire and the response time for emergency personnel capable of fighting the fire. The tunnel lining should also sustain as little damage as possible so that the tunnel can go back into service as soon as possible. Protection from fire can be gained from concrete cover, tunnel finishes and the inclusion of plastic fibers in concrete mixes.

10.2.4 High Density Concrete

High density concrete is produced by using very finely ground cement and/or substituting various materials such as fly ash or blast furnace slag for cement. The cementitious content of high density concrete is very high. The high cement content makes handling difficult under ideal conditions. Complicated mixes with multiple admixtures and careful water monitoring are required to keep the concrete in a plastic state long enough to be placed in forms. High cement content will result in high heat of hydration. Proper curing of these materials is essential to produce a quality end product. Improper or incomplete curing can be the cause of severe cracking due to shrinkage. Shrinkage cracks can reduce the effectiveness of the product, affect its durability and potentially make it unusable.

High density concrete, however, can be beneficial in many tunnel applications. It can limit the inflow of water and provide significant protection against chemical attack. High density concrete has low heat conductivity which is beneficial in a fire. High density concrete should be used in conjunction with careful inspection and strict enforcement of specifications during construction.

10.2.5 Corrosion Protection

Corrosion is associated with steel products embedded in the concrete and otherwise used in tunnel applications. Ground water, ground chemicals, leaks, vehicular exhaust, dissimilar metals, deicing chemicals, wash water, detergents, iron eating bacteria and stray currents are all sources of corrosion in metals. Each of these and any other aspect that is unique to the tunnel under consideration must be evaluated during the design phase. Corrosion protection methods designed to combat the source of corrosion should be incorporated into the design.

Corrosion protection can take the form of coatings such as epoxies, powder coatings, paint or galvanizing. Insulation can be installed between dissimilar metals and sources of stray currents. High density concrete can provide protection for reinforcing steel. Coatings on concrete can minimize the infiltration of water, a component of almost all corrosion processes. Tunnel finishes can also protect the tunnel structural elements from attack by the various sources of corrosion.

Cathodic protection uses sacrificial material to protect the primary material from corrosion. In highly corrosive environments, an electrical current is induced in the materials to force corrosion to occur in the sacrificial material. These systems are highly effective when properly designed, installed and maintained. Sacrificial elements must be replaced and electrical supply equipment serviced regularly. Cathodic protection also requires a reliable long term source of electricity and adds to the maintenance and operation costs of the tunnel.

Increased concrete cover over reinforcing steel is an effective means of protecting reinforcing steel from corrosion. Increasing the concrete cover, however will also increase the thickness of the lining. The increased thickness will result in a larger excavation which will increase the overall cost of the tunnel. The use of increased concrete cover should be evaluated in terms of the overall cost of the tunnel compared to the benefit derived.

10.2.6 Lining Joints

Joints in linings are required to facilitate construction. Cast-in-place concrete requires construction joints. Construction joints can be sloped or formed. Segmental linings constructed from concrete or steel can have either bolted or unbolted joints. Unbolted joints are used in both gasketed and ungasketed concrete segments. Steel liner plates are bolted. More detailed information on the advantages and disadvantages of joints is provided in subsequent sections of this chapter.

Joints in linings also provide relief from stresses induced by movements due to temperature changes. Cast-in-place linings should have contraction joints every 30 feet and expansion joints every 120 feet. Expansion joints should also be used where cut and cover portions of the tunnel transition to the mined portion. Segmental concrete linings do not require contraction joints and require expansion joints only at the cut and cover interface.

10.3 STRUCTURAL DESIGN

Structural design will be governed by the latest AASHTO LRFD Bridge Design Specifications. The AASHTO specifications do not cover structural plain concrete which is frequently used in tunnel lining construction. This chapter will provide design procedures based on the AASHTO specifications for structural plain concrete. These procedures can be found in section 10.4 Cast-in-Place Concrete.

10.3.1 Loads

The loads to be considered in the design of structures along with how to combine the loads are given in Section 3 of the LRFD specifications. Section 3 of the LRFD specification divides loads into two categories: Permanent Loads and Transient Loads. Paragraph 3.3.2 “Load and Load Designation” of the LRFD specifications defines the following permanent loads that are applicable to the design of mined tunnel linings:

DC = Dead Load: This load comprises the self weight of the structural components as well as the loads associated with nonstructural attachments. Nonstructural attachments can be signs, lighting fixtures, signals, architectural finishes, waterproofing, etc. Typical unit weights for common building materials are given in Table 3.5.1-1 of the AASHTO LRFD specifications. Actual weights for other items should be calculated based on their composition and configuration.

DW = Dead Load: This load comprises the self weight of wearing surfaces and utilities. Utilities in tunnels can include power lines, drainage pipes, communication lines, water supply lines, etc.

Wearing surfaces can be asphalt or concrete. Dead loads of wearing surfaces and utilities should be calculated based on the actual size and configuration of these items.

EH = Horizontal Earth Pressure Load. The information required to calculate this load are derived by the geotechnical data developed during the subsurface investigation program. The methods used in determining earth loads on mined tunnel linings are described in Chapters 6 and 7 of this manual.

ES = Earth surcharge load. This is the vertical earth load due to fill over the structure that was placed above the original ground line. It is recommended that a minimum surcharge load of 400 psf be used in the design of tunnels. If there is a potential for future development adjacent to the tunnel structure, the surcharge from the actual development should be used in the design of the structure. In lieu of a well defined loading, it is recommended that a minimum value of 1000 psf be used when future development is a possibility.

EV = Vertical earth pressure. The methods used in determining earth loads on mined tunnel linings are described in Chapters 6 and 7 of this manual.

Paragraph 3.3.2 "Load and Load Designation" of the LRFD specifications defines the following transient loads that are applicable to the design of mined tunnel linings:

CR = Creep.

CT = Vehicular Collision Force: This load would be applied to individual components of the tunnel structure that could be damaged by vehicular collision. Typically, tunnel linings are protected by redirecting barriers so that this load need be considered only under usual circumstances. It is preferable to detail tunnel structural components and appurtenances so that they are not subject to damage from vehicular impact.

EQ = Earthquake. This load should be applied to the tunnel lining as appropriate for the seismic zone for the tunnel. Other extreme event loadings such as explosive blast should be considered. The scope of this manual does not include the calculation of or design for seismic and blast loads, however, the designer must be aware that extereme event loads should be accounted for in the design of the tunnel lining.

IM = Vehicle dynamic load allowance: This load is applied to the roadway slabs of mined tunnels. This load can also be transmitted to a tunnel lining through the ground surface when the tunnel is under a highway, railroad or runway. Usually a mined tunnel is too far below the surface to have this transmitted to the structure. However, this load may be a consideration near the interface between the cut and cover approaches and the mined tunnel section. An equation for the calculation of this load is given in paragraph 3.6.2.2 of the AASHTO LRFD specifications.

LL = Vehicular Live Load: This load is applied to the roadway slabs of mined tunnels. This load can also be transmitted to a tunnel lining through the ground surface when the tunnel is under a highway, railroad or runway. Usually a mined tunnel is too far below the surface to have this loads from the surface transmitted to the structure however, this load may be a consideration near the interface between the cut and cover approaches and the mined tunnel section.. Guidance for the distribution of live loads to buried structures can be found in paragraphs 3.6.1 of the AASHTO LRFD specifications.

LS = Live Load Surcharge: This load is applied to the lining of tunnels that are constructed under other roadways, rail lines, runways or other facilities that carry moving vehicles. This is a uniformly distributed load that simulates the distribution of wheel loads through the earth fill. Usually a mined tunnel is too far below the surface to have this loads from the surface transmitted to the structure, however, this load may be a consideration near the interface between the cut and cover approaches and the mined tunnel section.

PL = Pedestrian Live load. Pedestrian are typically not permitted in highway tunnels, however, there are areas where maintenance and inspection personnel will need access. Areas such as ventilation ducts when transverse ventilation is used, plenums above false ceilings, and safety walks. These loads are transmitted to the lining through the supporting members for the described features.

SH = Shrinkage. Cut and cover tunnel structural elements usually are relatively massive. As such, shrinkage can be a problem. This load should be accounted for in the design or the structure should be detailed to minimize or eliminate it.

TU = Uniform Temperature. This load is used primarily to size expansion joints in the structure. If movement is permitted at the expansion joints, no additional loading need be applied to the structure. Since the structure is very stiff in the primary direction of thermal movement, the effects of the friction force resulting from thermal movement can be neglected in the design.

WA = Water load. This load represents the hydrostatic pressure expected outside the tunnel structure. Mined tunnels are usually detailed to be watertight without provisions for relieving the hydrostatic pressure. As such, the tunnel lining is subject to hydrostatic pressure. Hydrostatic pressure acts normal to the surface of the tunnel. It should be assumed that water will develop full hydrostatic pressure on the tunnel when no relief mechanism is used. The calculation of this load should take into account the specific gravity of the groundwater which can be saline near salt water. Both maximum and minimum hydrostatic loads should be used for structural calculations. For the purpose of design, the hydrostatic pressures assumed to be applied to underground structures should ignore pore pressure relief obtained by any seepage into the structures unless an appropriately designed pressure relief system is installed and maintained. Two groundwater levels should be considered: normal (observed maximum groundwater level) and extreme, 3 ft (1 m) above the 200-year flood level. The buoyancy force should be carefully evaluated to ensure that the applied dead load effect is larger than the applied buoyancy effect. Calculations for buoyancy should be based on minimum characteristic material densities and maximum water density. The total uplift force is equal to the weight of water displaced. Friction effects (the theoretical force required to dislodge the wedge of material over the tunnel) of overlying natural materials and backfill should not be taken into account, however the weight of soil and water over the tunnel should be used to calculate the resisting forces. When a relief system is included, the functioning of the relief system is evaluated to determine the hydrostatic pressure to be applied to the tunnel.

Some of the loads shown in paragraph 3.3.2 of the LRFD specifications are not shown above because they are not applicable to the design of mined highway tunnels as described below.

DD = Downdrag: This load comprises the vertical force applied to the exterior of the lining that can result from the subsidence of the surrounding soil due to the subsidence of the in-situ soil below the bottom of the tunnel. This load would not apply to mined tunnels since it requires subsidence or settlement of the material below the bottom of the structure to engage the downdrag force of the lining. For the typical highway tunnel, the overall weight of the structure is usually less than the soil it is replacing. As such, unless backfill in excess of the original ground elevation is paced over the tunnel or a structure is constructed over the tunnel, settlement will not be an issue for mined tunnels.

BR = Vehicular Breaking Force: This load would be applied only under special conditions where the detailing of the structure requires consideration of this load. Under typical designs, this force is resisted by the mass of the roadway slab and need not be considered in design.

CE = Vehicular centrifugal force: This load would be applied only under special conditions where the detailing of the structure requires consideration of this load. Under typical designs, this force is resisted by the mass of the roadway slab and need not be considered in design.

CV = Vessel Collision Force is not applicable since it would only be applied to immersed tube tunnels. Immersed tube tunnels are a specialized form of cut and cover tunnel and are covered separately in Chapter 12 of this manual.

EL = Accumulated locked-in force effects resulting from the construction process including secondary forces from post tensioning.

FR = Friction. As stated above, the structure is very stiff in the direction of thermal movement. Thermal movement is the source of the friction force. In a typical tunnel, the effects of friction can be neglected.

IC = Ice load. Since the tunnel is not subjected to stream flow nor exposed to the weather in a manner that could result in an accumulation of ice, this load is not used in cut and cover tunnel design.

SE = Settlement. For the typical highway tunnel, the overall weight of the structure is usually less than the soil it is replacing. As such, unless backfill in excess of the original ground elevation is paced over the tunnel or a structure is constructed over the tunnel, settlement will not be an issue for cut and cover tunnels. If settlement is anticipated due to poor subsurface conditions or due to the addition of load onto the structure or changing ground conditions along the length of the tunnel, it is recommended that a deep foundation (piles or drilled shafts) be used to support the structure. Ground settlements are difficult to predict and are best eliminated by the use of deep foundations.

TG = Temperature Gradient. This load should be examined on case by case basis depending on the local climate and seasonal variations in average temperatures. Typically due to the relative thin members used in tunnel linings, this load is not used. Paragraph 4.6.6 of the AASHTO LRFD specifications provides guidance on calculating this load. Note that paragraph C3.12.3 of the AASHTO LRFD Specifications allows the use of engineering judgment to determine if this load need be considered in the design of the structure.

WL = Wind on live load. The tunnel structure is not exposed to the environment, so it will not be subjected to wind loads.

WS = Wind load on structure. The tunnel structure is not exposed to the environment, so it will not be subjected to wind loads.

Section 3 of the LRFD specifications provides guidance on the methods to be used in the computations of these loads. The design example (Appendix G) shows the calculations involved in computing these loads.

10.3.2 Load Combinations

The AASHTO Specification defines four limit states; service, fatigue and fracture, strength, and extreme event). Each of these limits states contain several load combinations. These limit states and load combinations were developed for loadings that are typically encountered by highway bridges. Many of the loadings that bridges are subjected to are not applicable to tunnel linings. Loads such as wind, stream flow, vessel impact and fatigue do not occur in mined tunnels. The unique conditions under which tunnels operate allow for eliminating many of the loading conditions used for bridges. Tunnels should be designed for the following load combinations.

The loads described above should be factored and combined in accordance with the LRFD specification and applied to the tunnel lining. These load combinations are given in Table 3.4.1-1 of the AASHTO specifications. The recommended load cases for the design of linings for mined highway tunnels are given in Table 10-1.

Table 10-1 Load Factor (γ_i) and Load Combination Table

Load Comb. Limit State	DC		DW		EH* EV#		ES		LL, IM, LS, CT, PL	WA	TU, CR, SH		TG
	Max	Min	Max	Min	Max	Min	Max	Min			Max	Min	
Strength I	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.75	1.00	1.20	0.50	0.00
Strength II	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.35	1.00	1.20	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	0.00	1.00	1.20	0.50	0.00
Service I	1.00		1.00		1.00		1.00		1.00	1.00	1.20	1.00	0.50
Service IV	1.00		1.00		1.00		1.00		0.00	1.00	1.20	1.00	1.00
Extreme Event I	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	$\gamma_i EQ^+$	1.00	N/A	N/A	N/A

* The load factors shown are for at-rest earth pressure. At-rest earth pressure should be used for all conditions of design of cut and cover tunnel structures.

\# The load factors shown are for rigid frames. All cut and cover tunnel structures are considered rigid frames.

\+ This load factor is determined on a project specific basis (refer to Chapter 13 Seismic Considerations).

When developing the loads to be applied to the structure, each possible combination of load factors should be developed.

10.3.3 Design Criteria

Historically there have been three basic methods used in the design of structures:

- Service load or allowable stress design which treats each load on the structure equally in terms of its probability of occurrence at the stated value. The factor of safety for this method is built into the material's ability to withstand the loading.

- Load factor design accounts for the potential variability of loads by applying varying load factors to each load type. The resistance of the maximum capacity of the structural member is reduced by a strength reduction factor and the calculated resistance of the structural member must equal or exceed the applied load.
- Load and resistance factor design takes into account the statistical variation of both the strength of the structural member and of the magnitude of the applied loads.

The fundamental LRFD equation can be found in paragraph 1.3.2.1 of the AASHTO specification. This equation is:

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \qquad \text{10-1}$$

(AASHTO Equation 1.3.2.1-1)

In this equation, η is a load modifier relating to the ductility, redundancy and operation importance of the feature being designed. The load modifier η_i is comprised of three components;

η_D = a factor relating to ductility = 1.0 for tunnel linings constructed with conventional details and designed in accordance with the AASHTO LRFD specification.
η_R = a factor relating to redundancy = 1.0 for mined tunnel linings.
η_I = a factor relating to the importance of the structure = 1.05 for tunnel design. Tunnels usually are important major links in regional transportation systems. The loss of a tunnel will usually cause major disruption to the flow of traffic, hence the high importance factor.

γ_i is a load factor applied to the force effects (Q_i) acting on the member being designed. Values for γ_i can be found in Table 10.1 above.

R_r is the calculated factored resistance of the member or connection.

ϕ is a resistance factor applied to the nominal resistance of the member (R_n) being designed. The resistance factors are given in the AASHTO LRFD specifications for each material in the section that covers the specific material. Specifically, Section 5 of the AASHTO LRFD specifications covers Concrete Structures and in general, the resistance factors to be used in concrete design can be found there. These values are as follow.

<u>For Reinforced Concrete Linings:</u>

ϕ = 0.90 for flexure
ϕ = 0.90 for shear
ϕ = 0.70 for bearing on concrete

Since tunnel linings will experience axial loads, the resistance factor for compression must be defined. The value of ϕ for compression can be found in Section 5.5.4.2.1 of the AASHTO LRFD specification as:

ϕ = 0.75 for axial compression

Structural steel is covered in Section 6 of the AASHTO LRFD specification. Paragraph 6.5.4.2 gives the following values for steel resistance factors:

For Structural Steel Members:

ϕ_f = 1.00 for flexure
ϕ_v = 1.00 for shear
ϕ_c = 0.90 for axial compression for plain steel and composite members

Chapter 12 of the AASHTO specifications addresses the design of tunnel linings constructed from steel lining plate. Table 12.5.5-1 provides the following additional resistance factors to be used in the design of steel lining plate:

ϕ = 1.00 for minimum wall area and buckling
ϕ = 1.00 for minimum longitudinal seam strength

For Plain Concrete Members: Un-reinforced concrete is also referred to as plain concrete. The AASHTO provisions do not address plain concrete. The following design procedures should be followed for structural plain concrete.

Calculate the moment capacity on the compression face of the lining as follows:

$$\phi M_{nC} = \phi 0.85\, f_c'S \qquad \text{10-2}$$

Where:
M_{nC} = The nominal resistance of the compression face of the concrete
ϕ = 0.55 for plain concrete
f_c' = 28 day compressive strength of the concrete
S = The section modulus of the lining section based on the gross uncracked section

Calculate the moment capacity on the tension face of the lining as follows:

$$\phi M_{nT} = \phi 5(f_c')^{1/2}S \qquad \text{10-3}$$

Where:

M_{nT} = The nominal resistance of the tension face of the concrete
ϕ = 0.55 for plain concrete
f_c' = 28 day compressive strength of the concrete
S = The section modulus of the lining section

Calculate the compressive strength of the lining as follows:

$$\phi P_C = \phi 0.6 f_c'A \qquad \text{10-4}$$

Where:

P_C = The nominal resistance of lining in compression
ϕ = 0.55 for plain concrete
f_c' = 28 day compressive strength of the concrete
A = The cross sectional area of the lining section

Check the compression face as follows:

$$Q_A/\phi P_C + Q_M/\phi M_{nC} \leq 1 \qquad \text{10-5}$$

Where:

Q_A = The axial load force effect modified by the appropriate factors
Q_M = The moment force effect modified by the appropriate factors

Calculate the tension strength of the lining as follows:

$$\phi P_T = 5\phi\ (f_c')^{1/2} \qquad \text{10-6}$$

Where:

P_T = The nominal resistance of lining in tension
ϕ = 0.55 for plain concrete
f_c' = 28 day compressive strength of the concrete

Check the tension face as follows:

$$Q_M/S - Q_A/A \leq \phi P_T \qquad \text{10-7}$$

Where the values of the variables are described above.

The shear strength of the lining is calculated as follows:

$$\phi V_n = \phi 1.33(f_c')^{1/2} b_w h \qquad \text{10-8}$$

Where:

V_n = The nominal resistance of lining in shear
ϕ = 0.55 for plain concrete
f_c' = 28 day compressive strength of the concrete
b_w = the length if tunnel lining under design
h = the design thickness of the tunnel lining

This design method is adapted for LRFD from the provisions for structural plain concrete from the American Concrete Institute's Building Requirements for Structural Concrete (ACI 318).

10.3.4 Structural Analysis

Structural analysis of tunnel linings has been a subject of numerous papers and theories. Great disparity of opinion exists on the accuracy and usefulness of these analyses. However, some rational method must be adopted to determine a lining's ability to maintain the excavated opening of a tunnel. Some widely accepted methods are described in this section.

Beam Spring Models A general purpose structural analysis program can be used to model the soil structure interaction. This method is known as the beam spring model. The computer model is constructed by placing a joint or node at points along the centroid of the lining. These nodes are joined by straight beam members that approximate the lining shape by a series of chords. When constructing this type of model, the chord lengths should be approximately the same as the lining thickness for the radii that can be expected in highway tunnels. Chord members that are too long can produce fictitious moments and chord members that are too short can result in computational difficulties because of the very small angles subtended by short members. A subtended angle dimension of approximately 60/R, where R is the radius of the tunnel in feet, will generally produce acceptable results. Properties such as cross sectional area and moment of inertia should be entered to accurately depict the real behavior of the lining. Since the compressive forces are generally large enough to have compression over the entire thickness of the lining, the area and moment of inertia are calculated using the gross, uncracked dimensions of the lining. In rock tunnels, overbreak will result in a lining thickness larger than the design thickness. The design thickness is used in the analysis. This type of model is useful in analyzing all geometric shapes.

The surrounding ground is modeled by placing a spring support at each joint. Springs can be placed in the radial and tangential directions. The tangential springs offer little value in the analysis and an unnecessary complication to the model. The numerical value of the spring constant at each support is calculated from the modulus of subgrade reaction of the surrounding ground multiplied by the tributary length of lining on each side of the spring. Many ground conditions can be encountered within the length of a single tunnel. Parametric studies that vary the ground conditions and the spring constants should be performed to determine the worst case scenario for the lining.

Loads are applied to the model and the displacement at each joint is checked. For joints that move away from the center of the tunnel into the ground, the spring is left active. When the joint displacement is toward the center of the tunnel, the spring is removed or made inactive. This process in repeated until all displacements match the spring condition (active or inactive) at that joint. Once the model converges, the moments, thrusts and shears are used to design the lining.

If the model reveals that the lining is beyond its capacity, making the lining thicker or stiffer will not alleviate the problem. In fact, stiffening the lining will cause it to attract more moment and it will likely continue to fail. The lining must be made to be more flexible. This can be accomplished by making the lining thinner, which may not work. The primary load action on the lining is axial load or thrust. If the lining is close to its capacity under this load action, then thinning will not work. Modeling lining flexibility such that the moments are relieved may show the lining to be adequate. This is what happens in reality. One way to model this phenomenon is to install full or partial hinges in the lining at points of theoretical high moment. The hinge can be modeled to accept as much moment as the lining can support or it can be modeled as a full hinge with no moment capacity. In reality, the lining is performing somewhere in between these two extremes. Analyzing both conditions will bracket the lining behavior and provide a reasonable assurance that the lining can support the loads.

Three Dimensional Models The model described above is usually a two dimensional model that represents a single foot along the length of the tunnel. More sophisticated models are required when large penetrations of the lining or intersecting tunnels are being analyzed. To model these conditions, a three dimensional finite element model is used. The model is constructed in a similar manner to the two dimensional model, with finite elements used to connect the nodes and create the three dimensional model. The modeling parameters described above hold true for this type of model also. The model should extend a minimum of one tunnel diameter beyond the feature being investigated on each side of the feature.

It has been argued that this model does not account for the nonlinearity of the surrounding ground, particularly in soft ground, nor does it account for the variation of ground movement with time. Careful development of loading diagrams and spring constants for this model can bracket the actual behavior of the surrounding ground. This will provide results that are comparable to more sophisticated analysis methods. It should be noted that this method of analysis typically over estimates the bending moment in the lining.

Empirical Method for Soft Ground For circular tunnels in soft ground, the validity of the beam spring model has been highly criticized. The beam spring model described above assumes the soil to be a homogenous elastic material when in fact it is often non-homogenous and the behavior is plastic rather than elastic. Plastic deformations of the soil take place and the lining "goes along for the ride", that is, the stiffness of the lining is incapable of resisting the soil deformations. Since the lining is typically more flexible than the surrounding soil, it distorts as the soil displaces and the lining's flexibility allows it to shed moments to the point where it is acting almost entirely in compression. Since the lining is not completely flexible, some residual moment remains in the lining. This moment is accounted for by assigning an arbitrary change in radius and calculating the theoretical moment resulting from this change in radius. Using this method, the thrust in the tunnel lining is calculated by the formula:

$$T = wR \tag{10-9}$$

Where:

T = the thrust in the tunnel lining
w = the earth pressure at the spring line of the tunnel due to all load sources
R = the radius of the tunnel

The percentage of radius change to be used is a function of the type of soil. Values for this percentage estimated by Birger Schmidt are shown in Table 10-2.

Table 10-2 Percentage of Lining Radius Change in Soil

Soil Type	ΔR/R – Range
Stiff to Hard Clays	0.15 – 0.40%
Soft Clays or Silts	0.25 – 0.75%
Dense or Cohesive Soils, Most Residual Soils	0.05 – 0.25%
Loose Sands	0.10 – 0.35%

Notes:
1. Add 0.1 to 0.3 percent for tunnels in compressed air, depending on air pressure.
2. Add appropriate distortion for effects such as passing neighbor tunnel.
3. Values assume reasonable care in construction, and standard excavation and lining methods.

The resulting bending moment in the lining is calculated using the following formula:

$$M = 3EI/R \times \Delta R/R \tag{10-10}$$

Where:

M = the calculated bending moment
R = radius to the centroid of the lining
ΔR = tunnel radius change

E = modulus of elasticity of the lining material
I = effective moment of inertia of the lining section

The effective moment of inertia can be calculated for precast segmental linings using the following formula:

$$I_e = I_j + I(4/n)^2 \qquad 10\text{-}11$$

Where:

I_e = The effective moment of inertia
I_j = The joint moment of inertia (conservative taken as zero)
I = The moment of inertia of the gross lining section
n = The number of joints in the lining ring

This formula was developed by Muir Wood

The moment of inertia for the uncracked section should be used for cast-in-place concrete linings.
This method should be used in conjunction with any other analysis for round tunnels in soft ground as verification. The method described above can be used for both concrete and steel segmental linings. It is recommended that steel lining plate also be checked using the provisions of Section 12.7 of the AASHTO specifications for wall resistance and resistance to buckling.

Numerical Methods Commercial software is also available to model both the lining and the surrounding ground as a continuum utilizing a three dimensional finite element or finite difference approach. FLAC3D is a finite difference based continuum analysis program, where the domain (ground) is assumed to be a homogeneous media. The structural elements (beam or shell elements) can be used to model the tunnel lining. Using interface elements between the lining elements and the surrounding ground, rock-lining interaction including slip can be simulated.

If the ground contains predominant weak planes and those are continuous and oriented unfavorably to the excavation, then the analysis should consider incorporating specific characteristics of these weak planes. In this case, mechanical stiffness (force/displacement characteristics) of the discontinuities may be much different from those of intact rock. Then, a discrete element method (DEM) can be considered to solve this type of problem. 3DEC is a commercially available program for this type of analysis. Unlike continuum analysis, the DEM permits a large deformation and finite strain analysis of an ensemble of deformable (or rigid) bodies (intact rock blocks) which interact through deformable, frictional contacts (rock joints).

It is greatly task dependent whether a continuum (FLAC3D) or discrete analysis (3DEC) is adequate. If the ground is soil, the FLAC3D is adequate. If the ground is jointed rockmass and the joints are predominant in rock-lining interaction, 3DEC should be utilized. These programs can be used to calibrate and verify beam spring models, and vice versa.

10.4 CAST-IN-PLACE CONCRETE

10.4.1 Description

Cast-in-place concrete linings are used as final linings in two pass lining systems. Initial ground support is installed in the tunnel as the tunnel is excavated and can take any form from steel ribs and lagging to precast concrete segments. A water proofing system or drainage blanket is typically placed between the initial ground support and the cast-in-place concrete lining.

Figure 10-5 shows the typical section for the cast-in-place lining used for the Cumberland Gap Tunnel. The Cumberland Gap Tunnel is a highway tunnel excavated in rock by the drill and blast method. Initial ground support is untreated rock, shotcrete and rock bolts. The initial ground support varied along the length of the tunnel due to varying ground conditions.

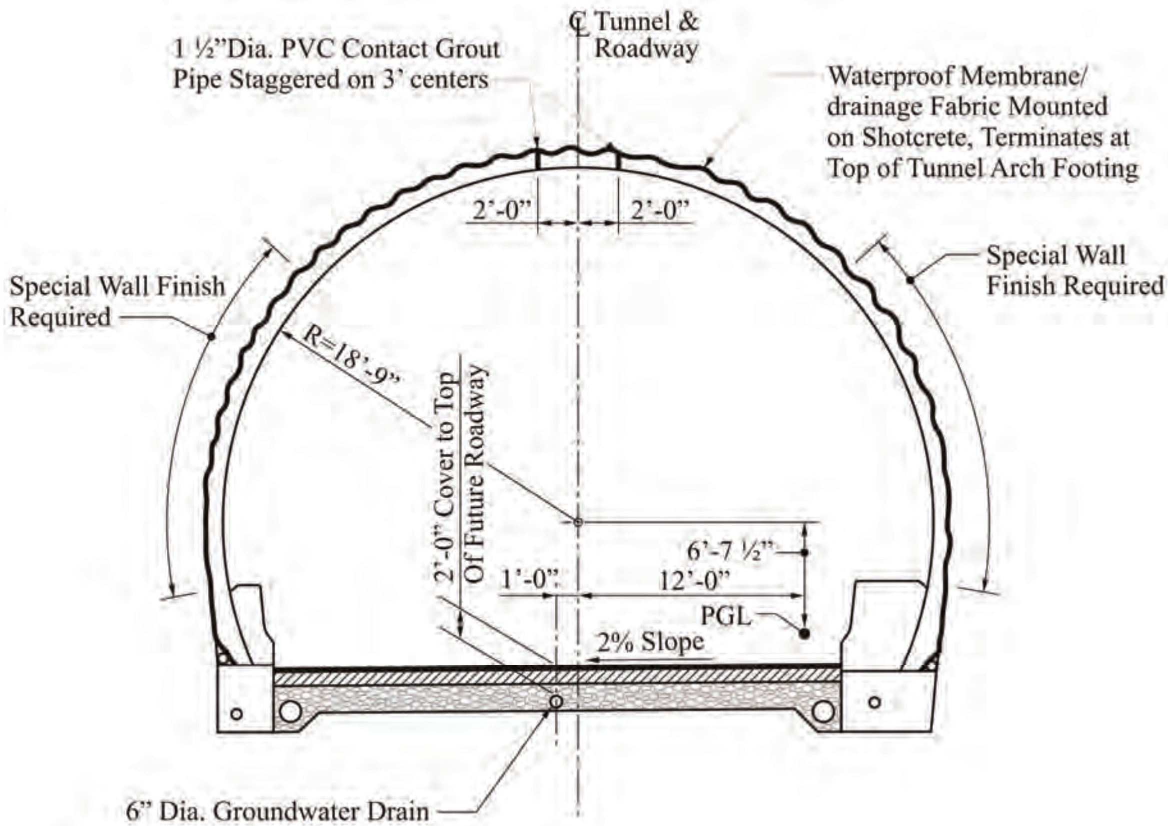


Figure 10-5 Cumberland Gap Tunnel Lining (Unfinished)

Figure 10-6 is a photograph of a heavy rail transit tunnel in Washington, DC. This tunnel was excavated in soft ground by a tunnel boring machine. This tunnel utilized a two pass system consisting of rough cast precast concrete segments as the initial ground support and a final lining of cast-in-place concrete. A high density polyethlylene waterproofing membrane was placed between the precast segments and the cast-in-place concrete final lining.

Advantages of a cast-in-place concrete lining are as follows:

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

Figure 10-6 Cast-In-Place Concrete Lining, Washington DC

Disadvantages of a cast-in-place concrete lining are as follows:

- Concrete placement, especially around reinforcement can be difficult. The nature of the construction of the lining restricts the ability to vibrate the concrete. This can result in incomplete consolidation of the concrete around the reinforcing steel.
- Reinforcement when used is subject to corrosion and resulting deterioration of the concrete. This is a problem common to all concrete structures, however underground structures can be also be subject to corrosive chemicals in the groundwater that could potentially accelerate the deterioration of reinforcing steel.
- Cracking that allows water infiltration can reduce the life of the lining.
- Chemical attack in certain soils can reduce lining life.
- Construction requires a second operation after excavation to complete the lining.

10.4.2 Design Considerations

In order to maximize flexibility and ductility, a cast-in-place concrete lining should be as thin as possible. There are, however, practical limits on how thin a section can be placed and still obtain proper consolidation and completely fill the forms. 10 inches (25 cm) is considered the practical minimum thickness for a cast-in-place concrete lining.

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

Cast-in-place concrete is used as the final lining. In many cases a waterproofing system is placed over the initial ground support prior to placing the final concrete lining. Placing reinforcing steel over the waterproofing system increases the potential for damaging the waterproofing. In all cases that are practical to do so, cast-in-place concrete linings should be designed and constructed as plain concrete, that is with no reinforcing steel. The presence of the waterproofing systems precludes load sharing between the final lining and the initial ground support. A basic design assumption is that the final lining carries long term earth loads with no contribution from the initial ground support.

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

Concrete behavior in a fire event must also be considered. When heated to a high enough temperature, concrete will spall explosively. This produces a hazardous condition for motorists attempting to exit the tunnel and for emergency response personnel responding to the incident. This spalling is caused by the vaporization of water trapped in the concrete pores being unable to escape. Spalling is also caused by fracture of aggregate and loss of strength of the concrete matrix at the surface of the concrete after prolonged exposure to high temperatures. Reinforcing steel that is heated will loose strength. Spalling and loss of reinforcing strength can cause changes in the shape of the lining, redistribution of stresses in the lining and possibly structural failure.

The lining should be protected against fire. Both external and internal protection can be provided. External protection in the form of coatings or boarding is available commercially. These are specialty products that can provide a measure of protection against relatively low temperature fires. Manufacturers should be consulted to ascertain the exact level of protection that they can provide. Including polypropylene fibers in the concrete mix can reduce vaporization of entrapped water. The fibers melt during a fire and provide a pathway for water to escape.

10.4.3 Materials

Mixes for cast-in-place concrete should be specified to have a high enough slump to make placement practical. A slump of 5” (12.7 cm) is recommended. Air entrainment should be used. The moist environment in many tunnels combined with exposure to cold weather makes air entrainment important to durable concrete; 3 to 5 percent air entrainment is recommended.

Compressive strength should be kept to a minimum. High strength concretes require complex mixes with multiple admixtures and special placing and curing procedures. Since concrete lining acts primarily in compression, 28 day compressive strengths in the range of 3,500 to 4,500 psi (24 to 31 MPa) are generally adequate.

Reinforcing steel bars should conform to the requirements of ASTM A615 grade 60 and welded wire fabric when used should conform to ASTM A185.

10.4.4 Construction Considerations

Cast-in-place concrete must attain a minimum strength prior to stripping forms. The concrete must also be cured. Leaving the forms in place can accomplish both these goals, but can inhibit the rate of construction. The concrete should reach some minimum strength prior to stripping the forms. This should be computed by the designer assuming that the tunnel is supported by the initial support and thus the final lining at the time of stripping will be carrying only its own weight. The strength of the concrete in the forms can be verified by breaking field cured cylinders. This will allow the forms to be stripped as soon as possible. Curing can continue after stripping by keeping the concrete moist or by applying a curing compound. Curing compounds should only be used if the concrete is the finished exposed surface. The curing compound will act as a bond breaker if finishes such as ceramic tile are applied to the concrete. Sealants and coating will not adhere to concrete surfaces that have had curing compound applied unless the curing compound is removed via sand blasting or other technique.

The length of pour along the centerline of tunnel should be limited to minimize shrinkage in the concrete. Lining forms are usually designed to be re-used so limiting the length of pour does not impose a hardship on the contractor. Construction joints can be bulkheaded or sloping. Bulkheaded joints provide a uniform appearance, however, depending on how uneven the face of excavation is, construction of the bulkhead may be difficult. Sloped construction joints do not affect the performance of the lining, but can be unattractive and should be rubbed out after the forms are stripped.

Placing concrete in a curved shape overhead will leave a void at the crown. This void is filled after the concrete is cured by pumping grout into the void. Grout pipes are installed in the forms prior to placing the concrete to facilitate this operation. Spacing of the grout pipes along the tunnel should be limited to 10 feet and the pipes should be offset from the crown by 15 degrees on both sides.

When appurtenances are attached to the finished concrete lining, using adhesive type anchors their use and inspection should follow FHWA’s Technical Advisory TA5140.26: Use and Inspection of Adhesive Anchors in Federal-aid Projects (see Appendix I).

10.5 PRECAST SEGMENTAL LINING

10.5.1 Description

Precast segmental linings are used in circular tunnels that are mined using a tunnel boring machine. They can be used in both soft and hard ground. Several curved precast elements or segments are assembled inside the tail of the tunnel boring machine to form a complete circle. The number of segments used to form the ring is a function of the ring diameter and to a certain respect, contractor’s preferences. The segments are relatively thin, 8 to 12 inches (20 to 30 cm) and typically 40 to 60 (1 to 1.5 m)inches (cm) wide measured along the length of the tunnel.

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

Precast segmental linings used as both initial support and final lining are built to high tolerances and quality. They are typically heavily reinforced, fitted with gaskets on all faces for waterproofing and bolted together to compress the gaskets after the ring is completed but prior to advancing the TBM. As the completed ring leaves the tail of the shield of the TBM, contact grouting is performed to fill the annular space that was occupied by the shield. This provides continuous contact between the ring and the surrounding ground and prevents the ring from dropping into the annular space. Bolting is often performed only in the circumferential direction. The shove of the TBM is usually sufficient to compress the gaskets in the longitudinal direction. Friction between the ground and the segments hold the segment in place, maintaining compression on the gasket. When first introduced into the United States in the mid-1970's, segmental linings were fabricated in a honeycomb shape that allowed for bolting in both the longitudinal and circumferential directions. Figure 10-2 shows the lining used for Section A of the Baltimore Metro. After 30 years of service, this lining is still providing a stable dry opening for over a hundred trains per day. Recent lining designs have eliminated the longitudinal bolting and the complex forming and reinforcing patterns that were required to accommodate the longitudinal bolts. Segments now have a flat inside surface as shown in Figure 10-7 and Figure 10-8. Figure 10-8 shows the segments in the casting bay after being stripped of the forms. Once adequate strength is achieved, the segments are inverted to the position they must be in for erection inside the side the tunnel. Segments are generally stored in a stacked arrangement, with one stack containing the segments required to construct a single ring inside the tunnel. As with segments used for initial lining, horizontal and vertical tunnel alignment is achieved through the use of tapered segments. Figure 10-8 shows the segments stacked in the storage yard awaiting transport into the tunnel.

Advantages of a precast segmental lining are as follows:

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

Disadvantages of a precast segmental lining are as follows:

- Segments must be fabricated to very tight tolerances
- Reinforcing steel must be fabricated and placed to very tight tolerances.
- Storage space for segments is required at the job site.
- Segments can be damaged if mishandled.
- Spalls, cracked and damaged edges can result from mishandling and over jacking.
- Gasketed segments must be installed to high tolerances to assure that gaskets perform as designed.
- Reinforcement when used is subject to corrosion and resulting deterioration of the concrete.
- Cracking that allows water infiltration can reduce the life of the lining.
- Chemical attack in certain soils can reduce lining life.

Figure 10-7 Precast Segments for One-Pass Lining, Forms Stripped

Figure 10-8 Stacked Precast Segments for One-Pass Lining

10.5.2 Design Considerations

Initial Lining Segments Segments used as an initial support lining are frequently designed as structural plain concrete. Reinforcing steel is placed in the segments to assist in resisting the handling and storage loads imposed on the segments. Reinforcement is often welded wire fabric or small reinforcing steel bars. The segments are usually cast by a precaster or in a yard set up specifically for manufacturing the segments.

Figure 10-9 Stacked Precast Segments for Two-Pass Lining

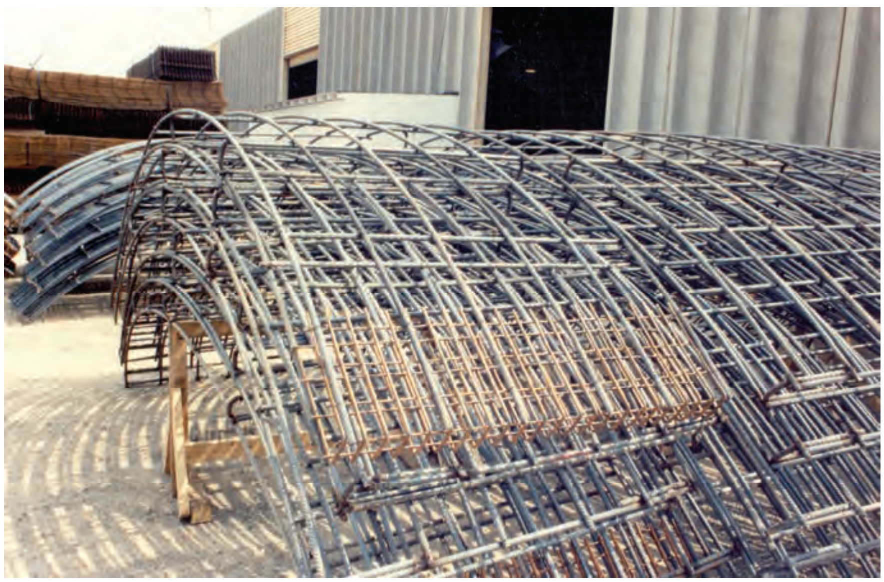


Figure 10-10 Steel Cage for Precast Segments for Two-Pass Lining

Figure 10-9 shows stacked segments for a two-pass liner system. These segments are used as the initial lining and are not required to be waterproof. Therefore no gaskets are used. No keyway for a gasket in cast into the segment. Note however, the keyway cast into the sides of the segments used to help with placement of the segment and maintaining alignment of the segments in the radial direction. Figure 10-10 shows the reinforcing steel cages for the segments.

Structural analysis is performed by one of the methods described in section 10.3.4. When using a structural analysis program for analysis, the structural model should include hinges (points where no bending moment can develop) at the locations of the joints in the ring. Using hinges at the joint locations provides the ring with the flexibility required to adjust to the loads, resulting in the predominant loading being axial load or thrust. This is an approximation of the behavior of the lining since joints will transfer some moment. The actual behavior of a segmental lining can be bounded by models that have zero fixity at the joints and full fixity at the joints.

Radial joints in between segments can be flat or concave/convex as shown on Figure 10-11 Convex/concave joints facilitate rotation at the joint, allowing the segment to deform and dissipate moments. Flat joints are more efficient at transferring axial load between segments and may result in less end reinforcement. In either case, the ends of the segments that form the joints should be reinforced to facilitate the transfer of load from one segment to another without cracking and spalling. The amount of reinforcement used should consider the type of joint and the resulting load transfer mechanism. Handling and erecting the segments are also sources of damage at the joints. Reinforcing can mitigate this damage.

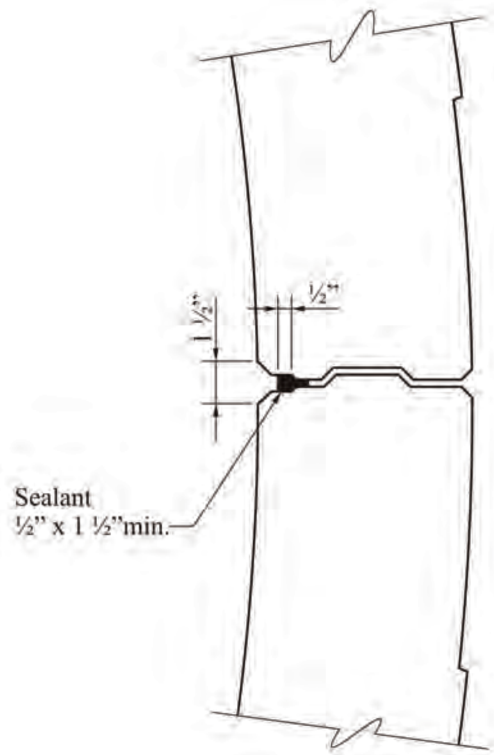


Figure 10-11 Radial Joints, Baltimore, MD

The primary load carried by the precast segments is axial load induced by ground forces acting on the circumference of the ring. However, loads imposed during construction must also be accounted for in the design. Loads from the jacking forces of the TBM are significant and can cause segments to be damaged and require replacement. These forces are unique to each tunnel and are a function of the ground type and the operational characteristics of the TBM. Reinforcement along the jacking edges of the segments is usually required to resist this force. The segments should be checked for bearing, compression and buckling from TBM thrust loads.

Handling, storage, lifting and erecting the segments also impose loads. The segments should be designed and reinforced to resist these loads. The dead weight of the segment with a dynamic factor of 2.0 applied to that dead weight is recommended for design to resist these loads. When designing reinforcement for these loads, the provisions of Chapter 5 of the LRFD specifications should be used. Grouting pressure can also impose loads on the lining. Grouting pressures should be limited to reduce the possibility of damage to the ring by these loads. A value of 10 psi (69 kPa) is recommended as the maximum permissible grouting pressure. The anticipated grouting pressure should be added to the load effects of the ground loads applied to the lining.

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

Final Lining Segments Segments used as a final lining are designed as reinforced concrete. The reinforcement assists in resisting the loads and limits cracking in the segment. Limiting cracking helps make the segments waterproof. The provisions of chapter 5 of the AASHTO LRFD specifications should be used to design the segments. The segments are manufactured by a precaster or in a yard set up specifically for manufacturing the segments. Since the segments are cast and cured in a controlled environment, higher tolerances can be attained than in cast-in-place concrete construction.
Structural analysis is performed by one of the methods described in section 10.3.4. When using a structural analysis program for analysis, an effective moment of inertia should be used to account for the flexibility induced in the ring at the bolted joints. The effective moment of inertia can be calculated using formula 10-10. When using this effective moment of inertia, no hinges are installed in the beam spring model.

Final lining segments can be fabricated with straight or skewed joints. Figure 10-12 shows a schematic of a lining system with straight joints. The orientation of the joint should be considered in the design of the lining to account for the mechanism of load transfer across the joint between segments. Skewed joints will induce strong axis bending in the ring and this should be accounted for in the design of the ring. Whether using straight or skewed joints, segments are rotated from ring to ring so that the joints do not line up along the longitudinal axis of the tunnel. Figure 10-13 is a picture of a mock-up of a ring of segmental lining.

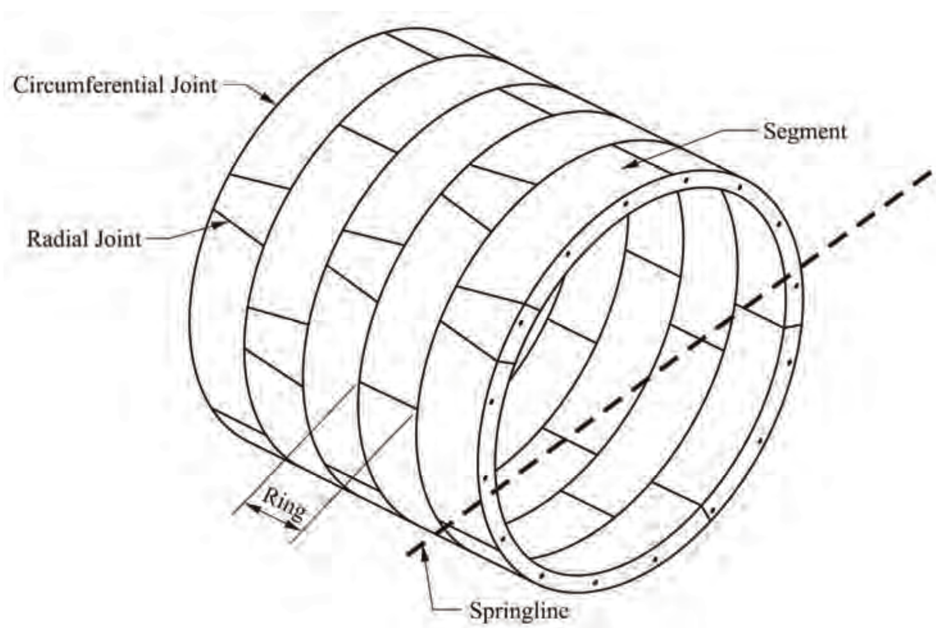


Figure 10-12 Schematic Precast Segment Rings

Figure 10-13 Mock-up of Precast Segment Rings

Joint design should consider the configuration of the gaskets. The gasket can eliminate much of the bearing area for load transfer between joints (See Figure 10-11 for example). Joints should be adequately reinforced to transfer load across the joints without damage.

The primary load carried by the precast segments is axial load induced by ground, hydrostatic and other forces acting on the circumference of the ring. The presence of the waterproofing systems precludes load sharing between the final lining and the initial ground support. A basic design assumption is that the final lining carries long term earth loads with no contribution from the initial ground support. Loads imposed during construction must also be accounted for in the design. Loads from the jacking forces of the TBM are significant and can cause segments to be damaged and require replacement. These forces are unique to each tunnel and are a function of the ground type and the operational characteristics of the TBM. Reinforcement along the jacking edges of the segments may be required to resist this force. The segments should be checked for bearing, compression and buckling from TBM thrust loads.

Lifting and erecting the segments also impose loads. The segments should be designed and reinforced to resist these loads. The dead weight of the segment with a dynamic factor of 2.0 applied to the dead weight is recommended for design to resist these loads. When designing reinforcement for these loads, the provisions of Chapter 5 of the LRFD specifications should be used.

Grouting pressure can also impose loads on the lining. Grouting pressures should be limited to reduce the possibility of damage to the ring by these loads. A value of 10 psi (69 kPa) is recommended as the maximum permissible grouting pressure. The anticipated grouting pressure should be added to the load effects of the earliest ground loads applied to the lining.

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

Concrete behavior in a fire event must also be considered. When heated to a high enough temperature, concrete will spall explosively. This produces a hazardous condition for motorists attempting to exit the tunnel and to emergency response personnel responding to the incident. This spalling is caused by the vaporization of water trapped in the concrete pores being unable to escape. Spalling is also caused by fracture of aggregate and loss of strength of the concrete matrix at the surface of the concrete after prolonged exposure to high temperatures. Reinforcing steel that is heated will loose strength. Spalling and loss of reinforcing strength can cause changes in the shape of the lining, redistribution of stresses in the lining and possibly structural failure.

The lining should be protected against fire. Both external and internal protection can be provided. External protection in the form of coatings or boarding is available commercially. These items can provide a measure of protection against relatively low temperature fires. These are specialty products and manufacturers should be consulted to ascertain the exact level of protection that they can provide. Including polypropylene fibers in the concrete mix can reduce vaporization of entrapped water. The fibers melt during a fire and provide a pathway for water to escape.

Appendix G presents a calculation example to illustrate the design process for precast segmental lining.

10.5.3 Materials

Concrete mixes for precast segments for initial linings do not require special designs and can generally conform to the structural concrete mixes provided in most state standard construction specifications. Strengths in the range of 4,000 to 5,000 psi (27 to 35 MPa) are generally adequate. These strengths are easily attainable in precast shops and casting yards. Curing is performed in enclosures and is well controlled. Air entrainment is desirable since segments may be stored outdoors for extended periods of time and final lining segments may be exposed to freezing temperatures inside the tunnel.

Steel fiber reinforced concrete has become a topic of discussion and research for precast tunnel linings. Theoretically, steel fibers can be used in lieu of steel reinforcing bars. The fibers can potentially eliminate the need for fabricating the steel bars to very tight tolerances, provide ductility for the concrete and make the segments tougher and less damage prone during construction. Unfortunately, there is no US design code for the design of steel fiber reinforced concrete. Papers have been written that propose design methods and several European countries have developed design methods. The recommended practice until further research is conducted and design codes are developed is to use steel fibers in segments where the design is conducted as detailed in this manual and the lining is found to be adequate without reinforcing. The steel fibers then can be included in the concrete to improve handling characteristics during construction. A testing program is required by the specifications to have the contractor prove via field testing that the fiber reinforced segments can withstand the handling loads imposed during construction. The fibers then can be used lieu of reinforcement that would be installed to resist the handling loads.

Reinforcing steel bars should conform to the requirements of ASTM A615 grade 60 and welded wire fabric when used should conform to ASTM A185.

Concrete mixes for one pass lining segments have strengths ranging from 5,000 psi to 7,000 psi (34 to 48 MPa). Higher strengths are easily obtainable in precast shops and assist in resisting handling and erection loads.

10.5.4 Construction Considerations

Initial Lining Segments Grout holes are required for contact grouting. Grout holes can also serve as lifting points for the segments. Locate the grout holes symmetrically so that the load to the lifting devices is evenly applied. Grout holes and lifting devices are usually designed by the contractor to loads and criteria specified by the designer. The construction industry is moving toward vacuum erection and handling equipment. This device does not rely on the grout holes to handle the segments. A device of this type can be seen in Figure 10-7. This devices relies on vacuum created between the segment face and the device to produce the reaction required to lift and erect the segments.
Segments should be cast and cured in accordance with the requirements of the standard specifications of the owner. In the absence of standard specifications, the requirements of the Precast Concrete Institute should be used to develop construction specifications for the precast segments. Segments should be stored in a manner that will not damage the segments. Support locations should be shown on the drawings and maximum stacking heights should be specified.

Segments should be detailed to facilitate jacking the rings at the crown or near springline after erection. Space for material to temporarily close off the gap to stop earth from coming into the tunnel is required. A means to jack the segments should be devised and the space remaining from the jacking should be backfilled with concrete and/or contact grouting to complete the ring. The ends of the segments that are used for jacking may require additional reinforcing or steel plates to protect them from the forces associated with jacking.

The number of segments for two pass systems is usually kept at a minimum, with the segments being slightly larger than for a one pass system and the joints in the rings will line up with joints in adjacent rings.

Final Lining Segments (One-Pass System) The same considerations as for initial lining segments apply to final lining segments. Final lining segments, however are not jacked at the crown after erection. Final lining segments must also be detailed to accommodate the gaskets required for waterproofing. Often, final lining segments also receive a waterproofing coating applied to the outside of the segment. This waterproofing coating should be a robust material such as coal tar epoxy since the segments slide along the shield as it advances and damage to the coating will occur.

10.6 STEEL PLATE LINING

Steel plate lining is a segmental lining system. It is sometimes used for circular tunnels in soft ground mined by TBM or other methods. Several curved steel elements or segments are assembled inside the tunnel or the TBM to form a complete circle. The segments are constructed from steel plates that are pressed into the required shape. The plates have flanges along all four edges. The flanges are used to bolt the segments together in the longitudinal and circumferential directions. Adjacent rings are rotated so that joints do not line up from ring to ring. The segments are fitted with gaskets along all the flanges that are compressed when the bolts are tightened. These gaskets are intended to provide waterproofing for the tunnel. Lining plate is manufactured in standard sizes and in widths of either 12” (25.4 cm) or 24” (50.8 cm). Only the radius changes to meet the requirements of the project. Figure 10-14 shows typical steel lining plate details.

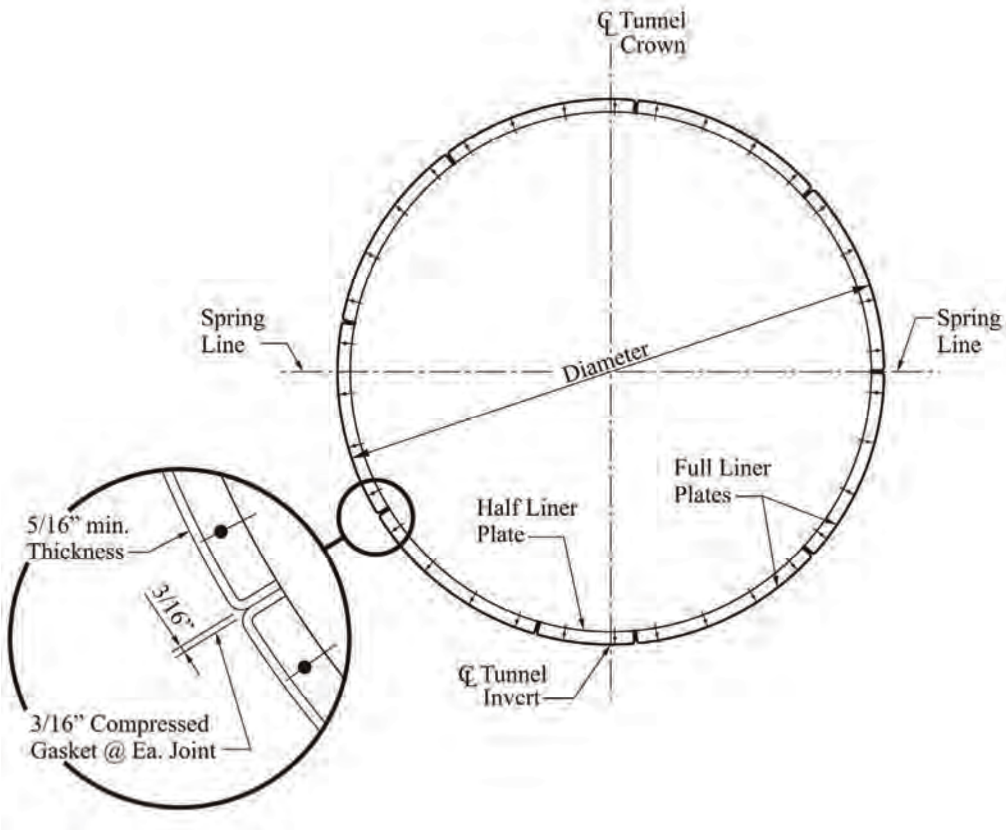


Figure 10-14 Typical Steel Lining Section

Advantages of a steel plate lining are as follows:

- Provides complete stable ground support that is ready for follow-on work.
- Materials are easily transported and handled inside the tunnel.
- No additional work such as forming and curing is required prior to being ready to use.

Disadvantages of a steel lining plate are as follows:

- Thrust applied from TBM must be limited to the capacity of the plate.
- Steel is subject to corrosion in the damp environment usually encountered in a tunnel.
- Fire can cause the lining plate to buckle and/or fail.
- Cast-in-place concrete will be needed for fire protection.

10.6.1 Design Considerations

Design of steel plate linings should be in accordance with Chapter 12 of the AASHTO LRFD specifications. The required checks for the service condition are included in that chapter. Typically, the design parameters and minimum dimensional requirements are specified on the drawings. The information on the drawings is developed from the AASHTO LRFD requirements. Lining plate manufacturers have standard products that can be selected for use on a project. The contractor will provide a specific product intended for use on the project and supply computations illustrating that the product meets the minimum requirements shown on the drawings.

The steel plate lining must be designed to resist jacking loads imposed by a TBM. A jacking ring or some other method of distributing these loads to the plates must be utilized to avoid damaging the plates during tunneling. Often, stiffeners are required at the center of the plates to resist the jacking loads. These stiffeners along with the flanges at the edges of the plates resist the bulk of this jacking force. The stiffeners and flanges are designed as columns to resist the anticipated jacking loads. Design of steel lining plate linings must include other loads induced by construction activities. Lifting and erection stresses should be checked by the contractor.

Curvature in horizontal and vertical alignments is accommodated with tapered segments just as with concrete segments.

Steel plate linings should be protected against corrosion. The exterior surface can be protected by a coating such as coal tar epoxy. The interior can be protected with coatings such as paint or galvanizing, but the most effective protection is a layer of unreinforced concrete. This concrete layer provides protection against corrosion and against heat damage due to fires. The protective concrete layer is placed after completion of the mining operation to avoid damage that can be caused by the jacking or the shield.

Gasket requirements for steel plate linings are similar to those for concrete segments. However, steel plate linings have far less surface area for gasket installation than do concrete segments.

10.7 SHOTCRETE LINING

As discussed in Chapter 9, shotcrete represents a structurally and qualitatively equal alternative to cast-in-place concrete linings. Its surface appearance can be tailored to the desired project goals. It may remain a rough, sprayer type shotcrete finish, or may have a quality comparable to cast concrete when trowel finish is specified. Shotcrete as a final lining is typically utilized in combination with the initial shotcrete supports in SEM applications when the following conditions are encountered:

- The tunnels are relatively short in length and the cross section is relatively large and therefore investment in formwork is not warranted, i.e. tunnels of less than 400-600 feet (150-250 m) in length and larger than about 25-35 feet (8-11 m) in springline diameter.
- The access is difficult and staging of formwork installation and concrete delivery is problematic.
- The tunnel geometry is complex and customized formwork would be required. Tunnel intersections, as well as bifurcations qualify in this area. Bifurcations are associated with tunnel widenings and would otherwise be constructed in the form of a stepped lining configuration and increase cost of excavated material.

When shotcrete is utilized as a final lining in dual shotcrete lining applications it will be applied against a waterproofing membrane as presented in Chapter 9. The lining thickness will be generally 10 to 12 inches (200 to 300 mm) or more and its application must be carried out in layers with a time lag between layer applications to allow for shotcrete setting and hardening. To ensure a final lining that behaves close to monolithically from a structural point of view it is important to limit the time lag between layer applications and assure that the shotcrete surface to which the next layer is applied is clean and free of any dust or dirt films that could create a de-bonding feature between the individual layers. It is typical to limit the application between the layers to 24 hours. Shotcrete final linings are applied onto a carrier system that is composed of lattice girders and welded wire fabric mounted to lattice girders toward the waterproofing membrane side. This carrier system also acts fully or partially as structural reinforcement of the finished lining. The remainder of the required structural reinforcing may be accomplished by rebars or mats or by steel or plastic fibers. The final shotcrete layer allows for the addition of micro poly propylene (PP) fibers that enhance fire resistance of the final lining.

Unlike the hydrostatic pressure of cast-in-place concrete during installation the shotcrete application does not develop pressures against the waterproofing membrane and the initial lining and therefore one must ensure that any gaps between waterproofing system and initial shotcrete lining and final shotcrete lining be filled with contact grout. As in final lining applications contact grout is accomplished with cementitious grouts but the grout takes are much higher. To assure a proper grouting around the entire lining circumference it is customary to use longitudinal grout hoses arranged radially around the perimeter. Figure 10-15 displays a typical shotcrete final lining section with waterproofing system, welded wire fabric (WWF), lattice girder, grouting hoses for contact grouting and a final shotcrete layer with PP fiber addition.

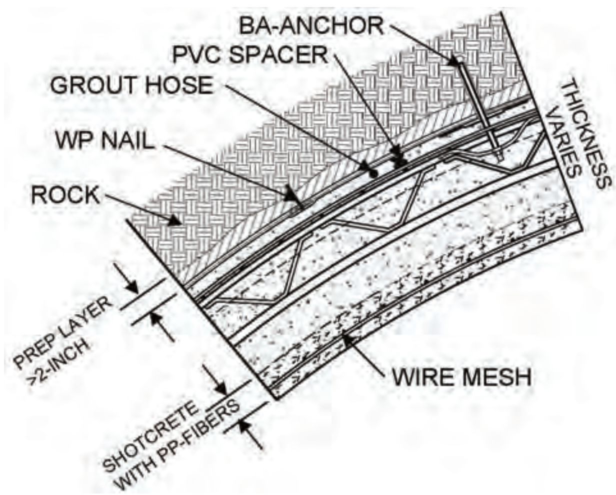


Figure 10-15 Typical Shotcrete Lining Detail

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

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

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

Chapter 9 presents detail discussions about shotcrete for initial support. Chapter 16 presents details about applying shotcrete for concrete repairs.

10.8 SELECTING A LINING SYSTEM

Each tunnel is a unique project and has its own combinations of ground conditions, opening size, groundwater condition, alignment and applicable construction technique. Given the wide range of combinations of these variables, guidance on the selection of a lining type can only be made using generalizations. The lining system designed for a project is selected based on the best judgment and experience of the designer. Once the project has been bid and awarded, it is not unusual for the contractor to request a change in the lining type, the mining method or both. The following paragraphs give conditions under which certain lining types make sense and offers caveats to be heeded when selecting a lining type for the project.

<u>Cast-in-Place Concrete</u> Cast-in-place concrete can be used in any tunnel with any tunneling method. It requires some form of initial ground support to maintain the excavated opening while the lining is formed, placed and cured. Cast-in-place concrete is usually used in hard ground tunnels mined using drill and blast excavation and soft ground tunnels mined using sequential excavation. Cast-in-place concrete can be formed into any shape so that the lining shape can be optimized to the required opening requirements.

Cast-in-place concrete is also used in both hard and soft ground tunnels excavated using a tunnel boring machine. Is these tunnels, the cast-in-place concrete lining is the final lining constructed after initial ground support is installed. Using cast-in-place concrete (two pass system) in a TBM tunnel can result in a larger excavated opening than if a single pass precast lining is used.

Cast-in-place concrete linings are cast against a waterproofing membrane. The membrane can be damaged during placement of reinforcing steel and forms. Forms must remain in place until the lining gains enough strength to support itself and curing must take place after forms are stripped.

<u>Precast Segmental Lining</u> Precast segmental linings are used exclusively in soft and hard ground tunnels excavated using a tunnel boring machine. This single pass system provides the ground support required during excavation and also forms the final lining of the tunnel. This system requires gaskets on each edge of the segments to provide a watertight lining. The segments must be manufactured to tight tolerances. The segments require specialized equipment to handle and erect inside the tunnel. Once erected and in place, the lining system is complete.

<u>Steel Plate Lining</u> Steel plate linings can be used in any ground condition with any mining method. The steel plates form the final lining and ground support once in place. This single pass system provides the ground support required during excavation and also forms the final lining of the tunnel. This system requires gaskets on the each edge of the segments to provide a watertight lining. The segments must be manufactured to tight tolerances. The segments require specialized equipment to handle and erect inside the tunnel. The segments are usually thin and not very stiff in the longitudinal direction. This lack of stiffness limits the amount of thrust that can be used to advance the tunnel boring machine. Difficult ground conditions that require high thrusts to advance the TBM may preclude the use of steel lining plate. Corrosion problems associated with steel linings can severely reduce the life of the lining.

CHAPTER 11
IMMERSED TUNNELS

11.1 INTRODUCTION

This chapter describes the structural design of immersed tunnels in accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO). The intent of this chapter is to provide guidance in the interpretation of the AASHTO specifications in order to have a more uniform application of the code and to provide guidance in the design of items not specifically addressed in AASHTO. The chapter begins with a basic description of immersed tunnel construction methodology.

Immersed tunnels consist of very large pre-cast concrete or concrete-filled steel tunnel elements fabricated in the dry and installed under water. More than a hundred immersed tunnels have been built to provide road or rail connections. They are fabricated in convenient lengths on shipways, in dry docks, or in improvised floodable basins, sealed with bulkheads at each end, and then floated out. Tunnel elements can and have been towed successfully over great distances. They may require outfitting at a pier close to their final destination. They are then towed to their final location, immersed, lowered into a prepared trench, and joined to previously placed tunnel elements. After additional foundation works have been completed, the trench around the immersed tunnel is backfilled and the water bed reinstated. The top of the tunnel should preferably be at least 5 ft (1.5 m) below the original bottom to allow for sufficient protective backfill. However, in a few cases where the hydraulic regime allowed, the tunnel has been placed higher than the original water bed within an underwater protective embankment.

Immersed tunnel elements are usually floated to the site using their buoyant state. However, sometimes additional external buoyancy tanks attached to the elements would be used if necessary. The ends of the tunnel elements are equipped with bulkheads (dam plates) across the ends to keep the inside dry, located to allow only about 6 to 8 ft feet (2 m to 2.5 m) between the bulkheads of adjacent elements at an immersion joint; this space is emptied once an initial seal is obtained during the joining process. The joints are usually equipped with gaskets to create the seal with the adjacent element. They are also equipped with adjustment devices to allow placement of the elements on line and grade. The tunnel elements will be lowered into their location after adding either temporary water ballast or tremie concrete. Figure 11-1 shows an illustration of the placement of an immersed tunnel.

Figure 11-1 Immersed Tunnel Illustration

11.1.1 Typical Applications

Immersed tunnels may have special advantages over bored tunnels for water crossings at some locations since they lie only a short distance below water bed level. Approaches can therefore be relatively short. Compared with high level bridges or bored tunnels, the overall length of crossing will be shorter. Tunnels can be made to suit horizontal and vertical alignments. They can be constructed in soils that would be a real challenge to a long-span bridge structure and under such conditions may be very cost competitive. However, immersed tunnels have potential disadvantages in term of environmental disturbance to the water body bed. They may have impact on fish habitats, ecology, current, and turbidity of the water. Furthermore, impacts on navigation in all navigable waterways should be considered and often extensive permitting would be required. In addition, many of the water bodies such as harbors or causeways have contaminated sediments requiring special handling. The use of immersed tunnel techniques might encounter such contaminated ground and would require its regulated disposal. For very long crossings where navigation is important, bridge-tunnel combinations can provide a most economical solution; long trestle bridges extend out from the shores through relatively shallow water to man-made islands at which the transition between bridge and tunnel is made, with the tunnel extending across the usually deeper navigation channels. The Chesapeake Bay Bridge-Tunnel in Norfolk, Virginia, was completed in 1964, is over 17 miles long and has immersed tunnels at each of the two main shipping channels, one of which is shown in Figure 11-2.

Figure 11-2 Chesapeake Bay Bridge-Tunnel

11.1.2 Types of Immersed Tunnel

Two main types of immersed tunnel have emerged, known as steel and concrete tunnels, terminology that relates to the method of fabrication. Both types perform the same function after installation. Steel tunnels use structural steel, usually in the form of stiffened plate, working compositely with the interior concrete as the structural system. Concrete tunnels rely on steel reinforcing bars or prestressing cables. The steel immersed tunnel elements are usually fabricated in ship yards or dry docks similar to ships, launched into water and then outfitted with concrete while afloat. Concrete immersed elements are usually cast in dry docks, or specially built basins, then the basin is flooded and the elements are floated out. Steel tunnels can have an initial draft of as little as about 8 feet (about 2.5 m), whereas concrete tunnels have a draft of almost the full depth. Tunnel cross-sections may have flat sides or curved sides.

Historically, concrete tunnels have predominantly been rectangular, which is particularly attractive for wide highways and combined road/rail tunnels. In Europe, Southeast Asia and Australia, virtually all immersed tunnels are concrete. In Japan, steel and concrete tunnels are in approximately equal numbers. Although most tunnels in North America are steel tunnels, there are also concrete immersed tunnels .

Steel tunnels have been circular, curved with a flat bottom, and rectangular (particularly in Japan), but the predominant shape in the US has been the double-shell tunnel, which is a circular shell within an octagonal shape. Most or all of the concrete in steel tunnels is placed while the steel shell is afloat, in direct contrast to concrete tunnels that are virtually complete before being floated out. The order in which concrete is placed for a steel tunnel is tightly controlled to minimize deformations and the resulting stresses. Steel immersed tunnels can be categorized into three sub-types: Single shell, double shell and sandwich.

11.1.3 Single Shell Steel Tunnel

In this type, the external structural shell plate works compositely with the interior reinforced concrete and no external concrete is provided. The shell plate requires corrosion protection, usually in the form of cathodic protection. The Hong Kong Cross-Harbour tunnel (Figure 11-3 and Figure 11-12) and San Francisco BART trans-bay tunnel are typical of this type.

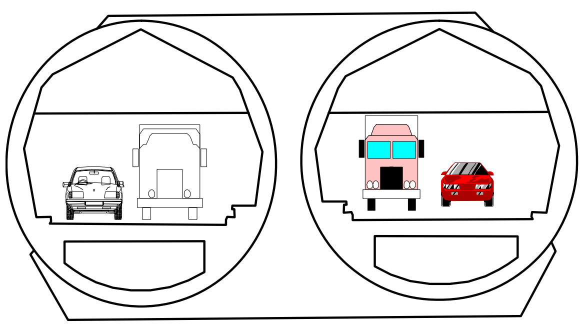


Figure 11-3 Cross Harbour Tunnel Hong Kong

Early examples of the single shell type are The Detroit River tunnel (1910), and the Harlem River tunnel (1914); both are rail tunnels and being the first two immersed transportation tunnels ever built, have similarities to single-shell tunnels. Of the eight existing single-shell immersed tunnels in the world, three are for rail in Tokyo, Japan, and three are for rail in the US. Two road tunnels have been constructed using the single shell method: The Baytown Tunnel in Texas (since removed) and the Cross Harbor Tunnel (Figure 11-12) in Hong Kong. Figure 11-4 shows the BART tunnel in San Francisco, a transit tunnel built in 1969. It is 5800m long and consists of 57 elements, all end launched.

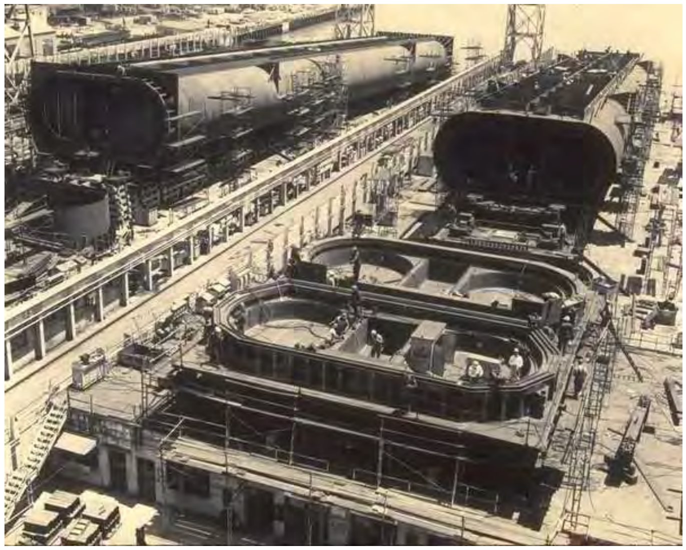


Figure 11-4 BART Tunnel, San Francisco

The initial draft of a single shell tunnel is less than that for other immersed tunnel types because of the elimination of the outer shell. However, leaks in the steel shell may be difficult to identify and seal; subdividing the surface into smaller panels by using ribs will improve the chances of sealing a leak. Great care and considerable testing is required to ensure that the welds are defect free. The risks of permanent leakage can be higher in single shell immersed tunnels than in other types. To avoid this, the external structural steel shell often requires a positive form of corrosion protection.

11.1.4 Double Shell

A double shell tunnel element is comprised of an internal structural shell that acts compositely with concrete placed within the steel shell. The top and invert concrete outside the structural shell plate is also structural. A second steel shell is constructed outside the structural steel shell to act as formwork for ballast concrete at the sides placed by tremie. In this configuration the interior structural shell plate works compositely with internal reinforced concrete while it is protected by external concrete placed within non-structural steel form plates. Figure 11-5 shows the cross section of the Second Hampton Roads Tunnel in Virginia. The steel portion of the double shell tunnel element is often fabricated at a shipyard. Prior to launching, the invert concrete may be placed to make the element more stable during towing and outfitting and to internally brace the steel elements. Due to the double shell configuration, this element is stiffer than the single shell section. However, due to the potential for rough conditions during towing and in particular during launching if not constructed in a dry dock, internal bracing may be required until the tunnel element is in its final position.

Multiple bores are created by linking sections with diaphragms. The diaphragms also serve to stiffen the steel shell. Diaphragms are spaced along the length of the tunnel element. Longitudinal stiffeners in the form of plates or T-sections are used in the longitudinal direction of the element between diaphragms to stiffen the shell. Figure 11-6 is a photograph of double shell tunnel elements constructed for the Fort McHenry Tunnel in Baltimore, Maryland.

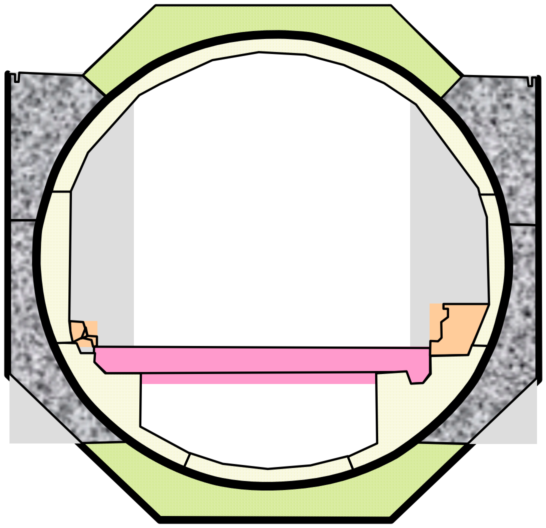


Figure 11-5 Double Shell - Second Hampton Road Tunnel, Virginia

11.1.5 Sandwich Construction

This construction type consists of a structural concrete layer sandwiched between two steel shells. Both the inner and outer shells are load carrying and both act compositely with the inner concrete layer. The concrete is un-reinforced and is formulated to be non-shrink and self consolidating. The inner surfaces of the steel shells are stiffened with plates and L-shaped ribs that also provide the connection required for composite action with the internal concrete. The internal concrete, once cured, carries compression loads and also serves to stiffen the steel shells. The steel shells carry the tension loads. Figure 11-7 shows a schematic of this type of construction.

Figure 11-6 Fort McHenry Tunnel, Baltimore

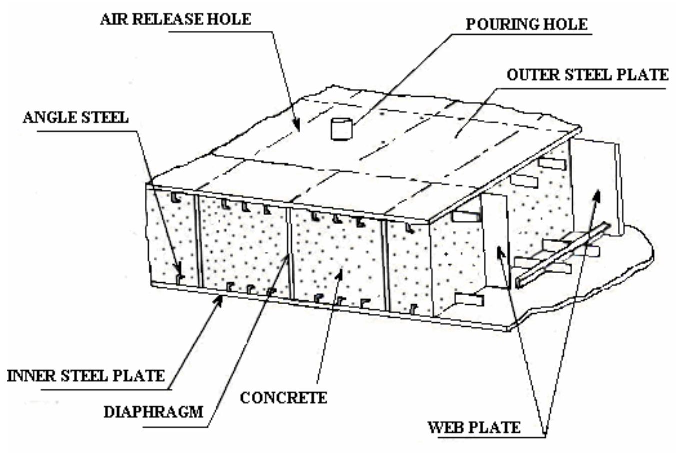


Figure 11-7 Schematic of Sandwich Construction

As with the other types, the steel shells are fabricated at a shipyard, launched and towed to the tunnel site. Internal diaphragms between the two shells stiffen the section sufficiently to resist the loads imposed during transport and outfitting. Once at the outfitting pier, the internal concrete is placed and the element draft increases. The element is towed to its location along the tunnel alignment and the final ballast and structural concrete is placed so that it can be lowered into place.

The steel sandwich construction provides a double layer of protection against leaks. However it is a very complex arrangement that requires carefully defined and executed procedures for fabrication and concreting. Distortion of the section during welding and poor quality welds can be costly mistakes for this type of construction. A recent example of a tunnel using this methodology is the Bosphorus crossing in Istanbul, Turkey where the end sections of each element are made in this way. Figure 11-8 shows two elements afloat while being outfitted. Several tunnels of this type exist in Japan.

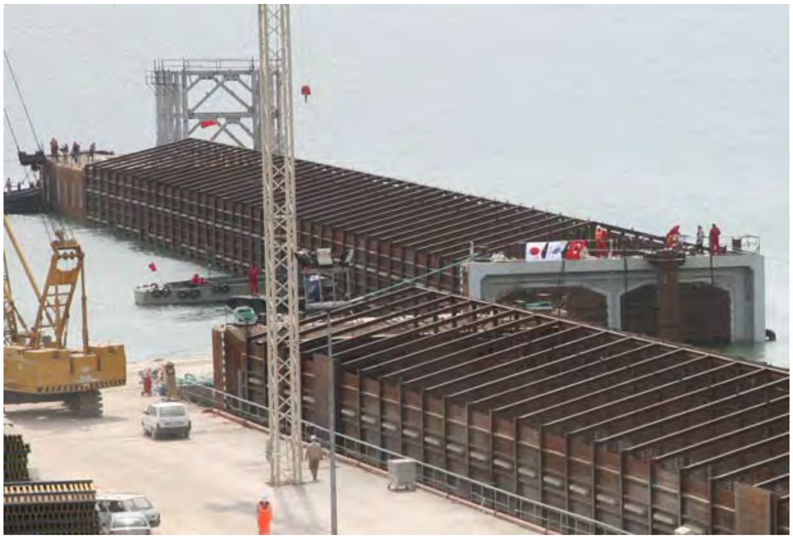


Figure 11-8 Bosphorus Tunnel, Istanbul, Turkey

11.1.6 Concrete Immersed Tunnels

Cast-in-place concrete is a versatile and durable material. It is easily formed into any shape or configuration to meet the needs of a specific project. Due to the fact that concrete is heavy, immersed tunnel elements constructed from concrete will float usually with very large drafts. In fact the freeboard for concrete elements is often less than a foot resulting in almost the entire element being underwater when being towed into position. This requires careful planning when using a concrete element. The path from the fabrication site to the tunnel alignment must contain water deep enough for the element to pass. Therefore, concrete elements are usually cast in a basin constructed close to the project site. A dredged channel may be required from the basin to the tunnel alignment. Once the concrete elements have been fabricated, the basin is flooded. The elements are towed out of the basin and to the tunnel alignment. Figure 11-9 shows construction of a concrete immersed tunnel crossing the Fort Point Channel in Boston.

Figure 11-9 Fort Point Channel Tunnel, Boston

Considerable development in the design and construction of concrete immersed tunnels has occurred over recent years, particularly in the use of materials and construction methods that reduce the number of construction joints. Water-cement ratio has been substantially reduced, and there have been efforts to reduce the heat of hydration, both of which result in fewer through-cracks during the curing of the concrete. Reducing the through cracks is key to making the sections waterproof. Figure 11-10 shows an above-ground fabrication facility and a transfer basin for the Øresund crossing in Denmark.

The length of concrete cast in a single operation for a full-width segment (bay) of a tunnel element has increased in length from some 30 ft (10 m) to about 60 ft (20 m) over the years, despite the very large volumes of concrete to place, and the expansion and contraction that occur during the first few days due to the heat of hydration.

To prevent cracking due to heat of hydration, mitigation measures have been used including concrete cooling using refrigerated pipes cast into the concrete, mix design, low heat cement such as ground granulated blast furnace cement, shielding from the elements and proper curing. Each of these measures has advantages and disadvantages. All aspects of these measures must be understood in order for them to be implemented. For example, high percentages of blast furnace slag will slow down the set of the concrete; pressure due to any additional height of liquid concrete needs to be considered in the formwork design.

Figure 11-10 Fabrication Facility and Transfer Basin, Øresund Tunnel, Denmark

Typically, the floor slab is cast first, followed by walls and then roof. Techniques have evolved permitting the outer walls and even the base slab to be cast with the roof slab, thereby reducing the number of construction joints in the exterior. Since construction joints are particularly susceptible to leakage, most often due to thermal restraint, it is most desirable to minimize their numbers.

Prestressing has been used in certain cases to resist bending moments and to reduce cracking. Some tunnels are prestressed transversely, and some have a nominal longitudinal prestressing applied. Careful detailing and good workmanship should be able to eliminate virtually all deleterious cracking in concrete.

11.2 CONSTRUCTION METHODOLOGY

11.2.1 General

The construction of an immersed tunnel consists of excavating an open trench in the bed of the body of water being crossed. Tunnel elements are fabricated off site, usually at a shipyard or in dry docks. Elements constructed on launching ways are launched similar to ships by sliding them into the water. Elements constructed in dry docks, are floated by flooding the dry dock. The ends of each element are closed by bulkheads to make the element watertight. The bulkheads are set back a nominal distance from the end of the element, resulting in a small space at the ends of the adjoining sections that is filled with water and will require dewatering after the connections with the previous element is made. After fabrication and launching, the elements are towed into position over the excavated trench, once positioned and attached to a lowering device (lay barge, pontoons, crane, etc), ballast is placed in or on the element so that it can be lowered to its final position. Sometimes ballasting of the element is achieved by water ballast in temporary internal tanks or by adding concrete. After placing the element in its position, connection is made between the newly placed element and the end face of the previously placed element or structure to which it is to be joined. Once the element is in its final position butted up against the adjacent element, the water within the joint between two elements is pumped out. After any remaining foundation work has been completed and locking fill is in place, the joint can completed and the area made watertight. Once locking fill is in position, another element can be placed. The bulkheads can then be removed, making the tunnel opening continuous. For safety reasons, the bulkheads at the joint to the most recently placed tunnel element are left in position. The tunnel is then backfilled and a protective layer of stone is placed over the top of the tunnel if required.

Variations in the construction method deal primarily with materials and location of the fabrication site at which the sections are constructed.

11.2.2 Trench Excavation

The most common method of excavation for immersed tunnels is the use of a clamshell dredger (Figure 11-11). Sealed buckets should be used for contaminated materials and/or to reduce turbidity in environmentally sensitive areas. Cutter suction dredgers have also been used and are able to remove most materials other than hard rock. Blasting may be required in certain areas, though it is highly environmentally undesirable.

Figure 11-11 Sealed Clam Shell Dredge

The tunnel trench should be dredged to longitudinal profiles and bottom widths taking potential sloughing of the sides and accuracy of dredging into account so that the necessary bottom width and profile can be maintained during lowering of the elements and placing of the foundation materials. Over-dredged areas should be refilled with materials conforming to design requirements for foundation materials. Dredging should be carried out in at least two stages: removal of bulk material; and trimming. The trimming should involve removal of at least the last 3 feet (1.0 m) above final dredge level. All silt or other material that may accumulate on the bottom of the trench should be cleared immediately before placing the element. Dredging methods and equipment should be designed to limit the dispersal of fine materials in the water. Turbidity or silt curtains or other measures should be used where appropriate. Methods, materials, and mitigation measures should be used to avoid or reduce to acceptable levels the impacts of excavation, filling and other operations on the marine environment.

Trench excavation in any waterway is an environmentally sensitive issue. Once the environmental conditions have been set by the planning and permitting process, extreme care should be taken to meet these conditions. Trench excavation underwater is a difficult and complex process that can be complicated by contaminated materials, tides, storms and construction restrictions in waterways due to environmental concerns associated with fish migration and mating patterns and with ecology and marine life. Scheduling of construction activities, environmentally friendly construction techniques and equipment and innovative methods of dealing with contaminants must be considered in the design of the excavation and backfill.

Locations, elevations and dimensions of all underwater utility lines and marine structures should be determined in the area of the dredging and protection should be provided if required. Excavations should be evaluated for stability using appropriate limit state methods of analysis. Temporary slopes offshore should be designed for a minimum factor of safety of 1.3. Side slopes of the trench should not be steeper than 2 horizontal to 1 vertical in soil, nor steeper than 1 horizontal to 4 vertical in rock provided the minimum specified factor of safety is achieved. The design should ensure that the bottom of any excavation is stable. The design should take into account excavation base stability against heave in any cohesive soils. Remedial measures such as ground improvement may be required to provide stability of the excavation base against heave.

Special requirements to handle the disposal of dredged materials are usually specified. Contaminated materials must be disposed of in special spoil containment facilities, while uncontaminated materials, if suitable, can be reused for backfill. Materials for reuse must be stored in areas where excess water can drain away. For most immersed tunnel projects where spoil containment facilities are required, the quality and quantity of the wet material are such that existing facilities are too small or unsuitable. A dramatic increase in dredging and disposal costs over the past three decades due primarily to continually tightening environmental restrictions present significant challenges to the disposal of unwanted material. Unique solutions were developed for various projects including: the use of the dredged materials to construct a manmade island such as for the Second Hampton Road Tunnel in Virginia or to reclaim a capped confined disposal facility (CDF) as a modern container terminal such as the case of the Fort McHenry Tunnel in Baltimore.

11.2.3 Foundation Preparation

Once the trench excavation is complete, installation of the foundation should begin. Two types of foundations are used in immersed tunnel construction, continuous bedding (screeded foundation or pumped sand) or individual supports.

<u>Continuous Bedding</u> Continuous bedding should consist of clean, sound, hard durable material with a grading compatible with the job conditions. These include applied bearing pressure, the method with which the bedding is placed and the material onto which the bedding is placed. The foundation thickness should not be less than 20 inches (500) mm and preferably less than 4.5 feet (1.4 m). The gap between the underside of the tunnel and the trench bottom should be filled with suitable foundation material. The foundation can be prepared prior to lowering the elements (screeded), or it can be completed after placing the elements on temporary supports in the trench (pumped sand); foundations formed after placement have included sand jetting, sand flow and grout. For a screeded foundation, the bedding is fine graded with a screed to the line and grade required for section placement, or a stone bed may be placed with a computer-controlled tremie pipe (“scrading”). Settlement analyses for the immersed tunnel should be performed and should consider compression of the foundation course placed beneath the tunnel elements. Analyses should also be performed to estimate the longitudinal and transverse differential settlement within each tunnel element, between adjoining tunnel elements, and at the transitions at the ends of the immersed tunnel. Measures should be taken to prevent sharp transitions from soil to rock foundations. Varying the thickness of the continuous bedding can accomplish this. Alternately the tunnel structure should be designed to resist the load effects from the potential differential settlement of the sub-foundation material.

Individual Supports Individual supports usually consist of driven piles. Pile foundations should be designed in accordance with generally recognized procedures and methods of analysis. The piles should be designed to fully support all applied compression, uplift and lateral loads, and any possible down-drag (negative friction) loads from compressible soil strata. The load-bearing capacity, foundation settlement and lateral displacement should be evaluated for individual piles and for pile groups, as appropriate. The load capacity for bearing piles should be confirmed by static and/or dynamic pile load testing in accordance with recognized standards. The piles and tunnel sections are usually detailed to be adjustable in order to fine tune the horizontal and vertical placement of the tunnel. Once the tunnel sections are in their final potions, the adjustment is locked off and a permanent connection between the tunnel and pile may be made. The space between the bottom of the tunnel section and the bottom of the trench below the tunnel section is then filled with granular material. This process must be carefully controlled so that the bottom of the trench is not disturbed and that the void is completely filled. Since in most cases, the weight of the tunnel section being placed is less than the weight of the soil it is replacing, pile foundations are rarely used.

11.2.4 Tunnel Element Fabrication

For steel tunnels, fabrication is usually done by modules, each module being in the range of 15ft (5m) long, spanning between diaphragms. The modules are then connected and welded together to form the completed shell of the tunnel element. Electro-slag and electro-gas welding are not permitted, and all groove and butt welds are full-penetration welds. Measures need to be taken to eliminate warping and buckling of steel plates resulting from their local overheating during welding. Welds must be tested by non-destructive methods; it is recommended that ultrasonic testing be supplemented by X-ray spot-check testing. In some cases, stress relieving may be necessary. The placing of keel concrete should be done in such a way that it avoids any overstressing or excessive deflections in the bottom shell and its stiffeners. All length and angular measurements for tolerances need to be made while the structure is shielded from direct sunlight to eliminate errors due to warping from differential temperatures. Figure 11-12 shows the completed fabrication of a tunnel element for the Hong Kong Cross Harbor Tunnel, almost ready to be side launched.

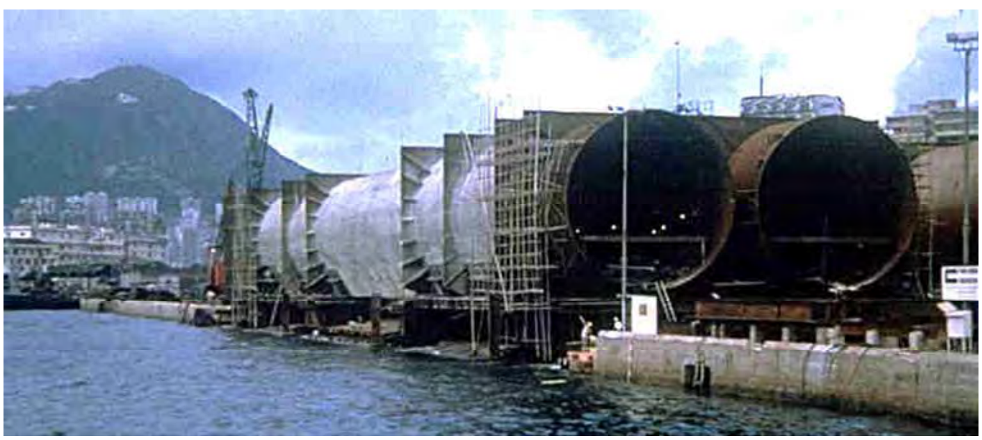


Figure 11-12 Hong Kong Cross Harbour Tunnel is Nearly Ready for Side Launching.

Concrete tunnel elements are usually constructed in a number of full-width segments to reduce the effects of shrinkage. The segment joints may be construction joints with reinforcement running through them, or they may be movement joints. All joints must be watertight. Tight controls on casting and curing must be maintained to minimize cracking. Differential heat of hydration can be controlled by the use of high percentages of blast furnace slag to replace Portland cement or by using internal cooling system. Where concrete segments are cast with movement joints, they are joined together using temporary or permanent post-tensioning to form complete elements at least during transportation and installation. Care must be taken to ensure that long-term movements of short segments free to move are acceptable.

Tunnel elements are generally fabricated to be approximately 300 to 400 feet in length each. The actual length is a function of the capacity of the fabrication facility, restrictions along the waterway used to float the elements to the construction site, restrictions at the tunnel including accommodation of marine traffic during construction, currents, element shape and the availability of space for an outfitting pier, and the capacity of the equipment used to lower the elements into place.

All construction hatches, openings, etc., need to be sealed, by welding or other secure means, upon completion of concreting or other works for which they were required. Before the launching or floating of elements, bulkheads, manholes and doors, etc. should be inspected to ensure that they are secure and watertight. When no longer needed, any temporary access manholes through the permanent structure should be closed and a permanent seal made.

As tunnel elements are installed, the actual installed length of tunnel and position should be monitored so that any changes to the overall length of future tunnel elements and the orientation of the end faces can be adjusted as required to ensure fit with the actual surveyed positions of installed tunnel elements. This is especially important prior to fabrication and placement of the closure (last) element.

11.2.5 Transportation and Handling of Tunnel Elements

The stability of tunnel elements must be ensured at every stage of construction, especially when afloat. In checking tunnel elements for stability while floating, due attention must be paid to effects of variations in structural dimensions, including results of thermal and hydrostatic effects. Items to consider include:

- Sufficient freeboard for marine operations, so that tunnel elements are relatively unaffected even when waves run over the top. A positive buoyancy margin exceeding 1% is recommended to guard against sinking due to variations in dimensions and the densities of both tunnel materials and the surrounding water.
- Lateral stability of the element using cross-curves of stability analysis should have a factor of safety in excess of 1.4 of the area under the righting moment curve against the heeling moment curve. A positive metacentric height (static stability) exceeding 8 inches (200 mm) is also recommended.

When a storm warning is issued, or forecast wave heights are expected to exceed operational limits, all marine operations should be ceased temporarily; marine plant and floating tunnel elements should be sent to their designated storm moorings or shelters. It is recommended that an emergency berth be identified for tunnel elements, preferably within or close to the placement site. Special measures may be required to control tunnel elements in areas with currents or navigation channels. Figure 11-13 shows the transportation of a tunnel element to its final position.

11.2.6 Lowering and Placing

After outfitting at their final destination, immersed tunnel elements are prepared for immersion and lowering onto prepared foundations in a trench in the bed. The equipment used may typically be provided on a purpose-built catamaran straddling the element (Figure 11-14). Other methods include the placement of pontoons on top of elements (Figure 11-13), or cranes have sometimes been used. In Boston, for the

Fort Point Channel tunnel, vertical buoyancy tubes were attached to the top of the elements and immersion by progressively adding water ballast was done.

Figure 11-13 Osaka Port Sakishima Tunnel Element Transported to Site with Two Pontoon Lay Barges

Figure 11-14 Catamaran Lay Barge

To lower an element to its final position, it is usual for either a temporary ballasting system to be used or for the element weight to be such that the element will itself have sufficient negative buoyancy. The method of immersion must:

- Maintain stability, including control over the tunnel element, while it is lowered to its final position.

- Enable the negative buoyancy to be increased as necessary so that a minimum factor of safety against flotation and overturning of 1.025 is obtained immediately after lowering.
- Enable the negative buoyancy to be increased to give a minimum factor of safety against flotation and overturning of 1.04 within a few hours of lowering and placing, ignoring assistance from adjacent elements.
- Maintain a vertical downward load of not less than 112.5 kips (500 kN) on every temporary seabed support, if used, until the element is placed on its final foundation.

The calculation of the factor of safety may include items such as external ballast, for example concrete blocks or internal ballast water tanks.

Lowering equipment should be designed to enable the lowering operation to be effectively controlled from a central control point and to make available at the central control accurate information on the position of the element and the loads on the lowering and the holding lines.

Elements are lowered and butted up to preceding elements. Thereafter, the joint between them is dewatered. A typical joint between elements includes watertight bulkheads (dam plates); watertight access bulkhead doors; joint seal and gaskets, dewatering equipment including any pumps and piping; location devices to guide the element horizontally and vertically into place relative to the preceding element, provision for shear keys (horizontal and vertical) and vertical and horizontal adjustment devices such as wedges, jacks and shims.

Tunnel elements should be installed at an elevation that considers an allowance for settlement such that after completion of the foundation works and all backfilling, they will be expected to be located within a tolerance of 2 inches (50 mm) laterally and vertically from their theoretical location, or any such lower figure on which the design methods are based. The allowance for settlement included in the determination of the installation level should be determined before installation. Notwithstanding the above, the relative location laterally and vertically should not be more than 1 inch (25 mm) across any joint. The relative location vertically across the terminal joints to other structures should not exceed 2 inches (50 mm).

Where foundation pads are used for temporarily supporting tunnel elements, any requirements for preloading and all subsequent behavior of the pads should be determined. The effect of potential hard spots beneath the tunnel element created by the foundation pads should be evaluated. Settlement of the foundation pads should be measured from the time of installation through any period of preloading until the tunnel element no longer requires support by the pads.

Permanent survey markers are needed within and on top of each element so that at any time its position relative to its position at time of casting is known. Survey towers or other markers or systems are needed so that the position of the element during lowering and placing is accurately known.

11.2.7 Element Placement

Element placement is the most delicate of all operations involving immersed tunnel elements. The needed duration of weather windows must be defined as well as “go / no-go” hold points. Some recent tunnels where prevailing currents could affect placing operations have used a weather-forecasting modeling system to forecast the required window; this may require monitoring of the hydrological and meteorological conditions concurrently to develop a forecasting model. Such a model should provide an understanding of the relationship between observed flow and meteorological and hydrological conditions. The last “go / no-go” decision should be based upon the current waves, and other physical conditions staying below the designed upper limits with a statistical probability of more than 90%. In all cases, the actual current at the element position should be checked immediately before lowering and continuously observed during the lowering and placing operation.

The element should have sufficient negative buoyancy to maintain stability and control of the tunnel element during immersion, so that the element can be lowered safely to its final position. The design should enable the negative buoyancy to be increased, if required, to give the minimum factors of safety given in Clause 11.2.6. Figure 11-15 shows the placement of a tunnel element using a catamaran lay barge.

Figure 11-15 A Tunnel Element is Being Placed.

Valves for dewatering of immersed joints should be operated from inside the previously placed tunnel element. No watertight doors or hatches should be opened until it can be confirmed that there is no water on the other side. Access must be maintained to the inside of the first element that is placed from the time when the element is placed until completion of permanent access through one of the terminal joints. Where hydrostatic pressure exists on a temporary bulkhead, the next two bulkheads should remain in place (one at the remote end of the same element, and the immediately adjacent one in the next tunnel element). Watertight doors in these bulkheads should remain closed at all times when the last tunnel element is unoccupied by personnel. Watertight doors should not be opened until the absence of water on the far side has been confirmed. The stability of the installed immersed tunnel elements during removal of temporary ballast and joint dewatering must be controlled to ensure that necessary factors of safety are maintained for the element as a whole, not only for the ends and for the sides, and so that the bearing pressure on the foundation remains approximately uniform.

After lowering and initial joining of each immersed tunnel element, its position should be precisely surveyed before the next element is placed. Settlement monitoring of tunnel elements should be carried out using the survey markers installed inside the elements. Levels should be recorded weekly until completion of backfilling of the subsequent element to ensure no remedial action is required and monthly thereafter until settlement becomes negligible.

11.2.8 Backfilling

The design should take into account the suitability of excavated material for use as backfill. The design should ensure that backfill placed next to the immersed tunnel is placed uniformly on both sides of the structure to avoid imbalanced lateral loads on the structure. The maximum difference in backfill level outside such structures above the locking fill should be 3 ft (1 m) until the lower side has been filled to its final level. Elements with more than 3 ft (1 m) difference in backfill level should be designed to accommodate the resulting transverse loads.

All fill materials subject to waves and currents should be designed to prevent scour and erosion. All underwater filling and rock protection material should be placed in a way that avoids damage to the waterproofing membranes (if present) or to the structure from impact or abrasion. The material should be placed in even layers on either side of the tunnel to avoid unequal horizontal pressures on the structures, and should be placed by means of buckets or tremie.

Prior to and during the placing of fill, the trench should be checked for sediment. Sediment that is detrimental to the performance of the material being placed should be removed.

Backfill should be provided around the tunnel. In seismic areas where there is a risk of liquefaction, the foundation and backfill should be designed as free-draining to prevent the development of excess pore-water pressure during and following a seismic event. Armor protection, if needed, should be provided to prevent long-term loss of backfill at the sides and on top of the tunnel.

The backfill usually consist of the following:

- <u>Selected locking fill</u> to secure the elements laterally
- <u>General backfill</u> to the sides and top of the tunnel structure, also providing an impact-absorbing / load-spreading layer above the tunnel
- <u>A rock protection blanket</u> generally above and adjacent to the tunnel to provide scour protection;
- <u>Rock-fill anchor-release bands</u> at both sides of the tunnel are sometimes provided.

11.2.9 Locking Fill

Selected locking fill is placed in the trench to a minimum level of half the height of each element after the joint to the adjacent tunnel has been dewatered. Locking fill should extend at least 6 ft (2 m) horizontally from the tunnel element before being allowed to slope down not steeper than 1:2. Locking backfill is placed in layers of uniform thickness not exceeding 2 ft (600mm), such that lateral and vertical forces on the tunnel element are minimized and no displacement of the element occurs. Placement of locking backfill proceeds from the inboard (jointed) end of tunnel elements and progresses towards the outboard end of tunnel elements in a manner that produces a uniformly dense backfill bearing tightly against the tunnel periphery.

The locking fill must be a granular, clean, sound, hard, durable material that will compact naturally and that will remain stable under both non-seismic and seismic conditions (where required). It may include crushed sound rock or gravel. Well graded sub-angular sand may be included. Sand fill, if used, must be free-draining.

11.2.10 General Backfill

General backfill should be used to fill the remainder of the trench above the selected locking fill up to the underside of any protection layer, or to the pre-existing seabed level if no protection layer is used. General backfill should be placed by a method that avoids segregation or misplacement of the fill.

The properties of general fill must suit the proposed design and method of placing. General fill may comprise soft cohesionless material that will remain stable. General fill must be free from clay balls and be chemically inert. Often the dredged materials for the trench are suitable as general backfill.

11.2.11 Protection Blanket

The elevation of the top of the protection layer should approximate pre-existing seabed levels unless instructed otherwise. However in certain situations, the top of the tunnel can extend above the original seabed in an underwater embankment if permitted. In this situation, the protective blanket shall be provided above the embankment backfill.

Rock protection blanket material should consist of hard inert material, usually sound, dense, newly quarried rock in clean angular pieces, well graded between 1 inch to 10 inches (25 mm and 250 mm). The material should be durable for at least the design life of the tunnel. The method of placing this material must ensure that the large-size stones do not penetrate the general backfill and must cause no damage to waterproofing of the tunnel (if used). The protection layer should not be placed by bottom dumping.

11.2.12 Anchor Release Protection

In navigable waters, anchor release protection should be provided, if required, and if the tunnel cover extends above the bed. Rock armor for anchor release bands should be of sound, dense, newly quarried rock in clean angular pieces and well graded. The intent of the anchor release protection is to bring the anchor to the surface and choke the gape (the space between the hook and the shank). The size of anchor should be for vessels plying those waters. The material needs to be durable for at least the design life of the tunnel.

11.3 LOADINGS

11.3.1 General

For the assessment of loads, the density of materials should be based on actual measurements made on samples from the same source as will be used for construction. For the design of individual sections, the least favorable loading should be used. The design should take into account the fact that the specific gravity of water may vary according to depth, prevailing weather conditions and season. The effect of suspended material should be taken into account in determining the specific gravity of water. The maximum hydrostatic load should be used for structural calculations. To ensure flotation (during launching or floating of the elements), the minimum relevant specific gravity of water should be used, and to prevent flotation (after placement of the element) the maximum should be used. The maximum and minimum values for each material used must be specified. The design must take into account any particular current regimes expected. This must include consideration of current speed, depth, direction, any interface between contra-flowing currents and the turbulence engendered thereby.

11.3.2 Loads

The loads to be considered in the design of structures along with how to combine the loads are given in Section 3 of the AASHTO LRFD specifications. It divides loads into two categories: Permanent Loads and Transient Loads. Paragraph 3.3.2 "Load and Load Designation" of the AASHTO LRFD specifications defines following permanent loads that are applicable to the design of immersed tunnels:

DC = Dead Load: This load comprises the self weight of the structural components as well as the loads associated with nonstructural attachments. Nonstructural attachments can be signs, lighting fixtures, signals, architectural finishes, waterproofing, etc. Typical unit weights for common building materials are given in Table 3.5.1-1 of the AASHTO LRFD specifications. Actual weights for other items should be calculated based on their composition and configuration. [These items have essentially well defined weights.]

DW = Dead Load: This load comprises the self weight of wearing surfaces and utilities. Utilities in tunnels can include power lines, drainage pipes, communication lines, water supply lines, etc. Wearing surfaces can be asphalt or concrete. Dead loads of wearing surfaces and utilities should be calculated based on the actual size and configuration of these items. [The weights of these items are generally less well defined, may be removed or replaced, and have different load factors.]

EH = Horizontal Earth Pressure Load. This load is generated by the backfill material and any armoring located above the backfill. The properties of the backfill material should be well defined. The value of the horizontal earth pressure should be calculated based on the properties of the specified backfill material. At-rest pressures should be used in the design of immersed tunnels.

EL = Accumulated locked-in force effects resulting from the construction process including secondary forces from post tensioning.

ES = Earth surcharge load. This is the vertical earth load due to fill over the structure that was placed above the original ground line. This load may be generated by armoring that is placed over the backfill.

EV = Vertical pressure from the dead load of the earth fill. This is the vertical earth load due to fill over the structure up to the original ground line. The properties of the backfill material should be well defined. The value of the vertical earth pressure should be calculated based on the properties of the specified backfill material.

Paragraph 3.3.2 "Load and Load Designation" of the LRFD specifications defines following transient loads that are applicable to the design of immersed tunnels:

CL = Construction Load: These loads are not explicitly defined in the AASHTO LRFD specifications, but must be considered when designing immersed tunnels. They include loads imposed when the tunnel section is launched, transporting loads such as loads imposed when towing the sections, wave action on the floating section, current loads when the section is being outfitted or placed, the loads imposed when the section is floating and concrete is placed in or on the section, wind on the section when it is being towed or when it is moored and being outfitted.

CR = Creep: Creep can be a factor in the design of concrete immersed tunnels and should be considered accordingly.

CT = Vehicular Collision Force: Inside the tunnel, this load would be applied to individual components of the tunnel structure that could be damaged by vehicular collision. Typically, tunnel walls are very massive or are protected by redirecting barriers so that this load need be considered only under very unusual circumstances. It is preferable to detail tunnel structural components so that they are not subject to damage from vehicular impact.

CV = Vessel Collision Force: This load could is generated by a sinking ship coming to rest over the tunnel. The magnitude of this load is a function of the type and size of vessels using the waterway over the tunnel. A study of the vessel traffic should be performed and the load determined based on the results of that study. Another category of this load is anchor impact. Should a ship drop its anchor in the vicinity of the tunnel, this will impart a significant load on the tunnel. This load should not be applied in combination with the vessel collision force. The following section provides guidelines for computing these loads.

EQ = Earthquake. This is a load that should be considered in areas where seismic activity is expected. This is discussed in Chapter 13.

IM = Vehicle dynamic load allowance: This load can apply to the roadway slabs of tunnels. An equation for the calculation of this load is given in paragraph 3.6.2.2 of the AASHTO LRFD specifications.

LL = Vehicular Live Load: This load can apply to the roadway slabs of tunnels and should be applied in accordance with the provisions of paragraph 3.6.1.2 of the AASHTO LRFD specifications.

PL = Pedestrian Live load. Pedestrian are typically not permitted in highway tunnels, however, there are areas where maintenance and inspection personnel will need access, areas such as ventilation ducts when transverse ventilation is used, plenums above false ceilings, and safety walks. These loads are transmitted to the lining through the supporting members for the described features.

SE = Settlement: Allowance should be taken of immediate settlements during the first week or so after placement of the element due to compaction of the foundation material (this could easily be 1 inch or 25 mm), expected long term movements due to placement of backfill and subsequent movements of the underlying materials, and movements resulting from the placement and backfilling of adjacent tunnel elements. Lateral movements can occur in soils that are non-uniform laterally and where the soil surface is sloping. Proper preparation of the foundation and placement of the backfill can minimize these effects. For the typical highway tunnel, the overall weight of the structure is less than the soil it is replacing. As such, unless backfill in excess of the original ground elevation is placed over the tunnel, settlement will not be an issue for immersed tunnels. If settlement is anticipated due to poor subsurface conditions or due to the addition of load onto the structure or changing ground conditions along the length of the tunnel, it is recommended that a pile foundation be used.

SH = Shrinkage: Shrinkage usually results in cracking. In the case of concrete immersed tunnels, detailing and construction methods should be employed to minimize shrinkage in order to minimize the resulting cracking. Shrinkage can also occur in the concrete placed as part of steel shell tunnel sections. The effect of this force should be accounted for in the design or else the structure detailed to minimize the effect of shrinkage.

SL = Support Loss: This loading is not defined in AASHTO since it is unique to immersed tunnels. It should include loss of support (subsidence) below the tunnel or to one side, and storms and extreme water levels with a probability of being exceeded not more than once during the design life (considering appropriate static and dynamic effects for each). A loss of support of not less than 10% of the length of an immersed tunnel element and uneven support from the foundation over the full width of the tunnel element should be considered.

TG = Temperature Gradient. Concrete immersed tunnel elements are typically massive members that have a large thermal lag. Combined with being surrounded by an insulating soil backfill that maintains a relatively constant temperature, the temperature gradient across the thickness of the members can be measurable. This load should be examined on a case by case basis depending on the local climate and seasonal variations in average temperatures. Steel tunnel sections may be thinner and would have a smaller thermal lag, which would help reduce this effect. However, it is recommended that this load be studied for all tunnel types. Paragraph 4.6.6 of the AASHTO LRFD specifications provides guidance on calculating this load. Note that paragraph C3.12.3 allows the use of engineering judgment to determine if this load need be considered in the design of the structure.

TU = Uniform Temperature. This load is used primarily to size expansion joints in the structure. If movement is permitted at the expansion joints, no additional loading need be applied to the structure. Since the structure is very stiff in the primary direction of thermal movement, the effects of the friction force resulting from thermal movement can be neglected in the design.

WA = Water load. This load represents the hydrostatic pressure expected outside the tunnel structure. Immersed tunnel structures are typically detailed to be watertight. Hydrostatic pressure acts normal to the surface of the tunnel. The design should take into account the specific gravity of the water which can be saline. Both maximum and minimum hydrostatic loads should be used for structural calculations as appropriate to the member being designed. The designer should take into account the fact that the specific gravity of water may vary according to depth, prevailing weather conditions and season. The effect of suspended material should be taken into account in determining the specific gravity of water. The maximum hydrostatic load should be used for structural calculations. To ensure flotation (during launching or floating of the elements), the minimum relevant specific gravity of water should be used, and to prevent flotation (after placement of the element) the maximum should be used. Two water levels should be considered: normal (observed maximum water level) and extreme, 3 ft (1 m) above the 200-year flood level. The buoyancy force should be carefully evaluated to ensure that the applied dead load effect is larger than the applied buoyancy effect. Frequently, structural member sizes will have to be increased to ensure that the buoyancy is completely resisted by the dead load or ballast added to the tunnel to counteract the buoyancy effect. The net effect of water pressure on the tunnel, i.e., the buoyancy, is the difference between hydrostatic loads on upward and downward facing surfaces. The total uplift force is equal to the weight of water displaced. Friction effects (the theoretical force required to dislodge the wedge of material over the tunnel) of backfill should not be taken into account, but the weight of material vertically above the structure may be taken into account.

Some of the loads shown in paragraph 3.3.2 of the LRFD specifications are not shown above because they are not applicable to the design of highway immersed tunnels as described below.

BR = Vehicular Breaking Force: This load would be applied only under special conditions where the detailing of the structure requires consideration of this load. Under typical designs, this force is resisted by the mass of the roadway slab and need not be considered in design.

CE = Vehicular centrifugal force: This load would be applied only under special conditions where the detailing of the structure requires consideration of this load. Under typical designs, this force is resisted by the mass of the roadway slab and need not be considered in design.

DD = Downdrag: This load comprises the vertical force applied to the exterior of the tunnel that can result from the subsidence of the surrounding soil due to the subsidence of the in-situ soil below the bottom of the tunnel. This load would not apply to immersed tunnels since it requires subsidence or settlement of the material below the bottom of the structure to engage the downdrag force of the tunnel. For a typical immersed tunnel, the overall weight of the structure is usually less than the soil it is replacing. As such, unless backfill significantly in excess of the original ground elevation is placed over the tunnel, settlement will not be an issue.

FR = Friction. The structure is very stiff in the direction of thermal movement. Thermal movement is the source of the friction force. In a typical tunnel, the effects of friction can be neglected.

IC = Ice load. Since the tunnel is not subjected to stream flow and unlikely to be exposed to the weather in a manner that could result in an accumulation of ice or icebergs, this load does not apply to immersed tunnel design.

LS = Live Load Surcharge: This load would be generated by vehicles traveling over or adjacent to the tunnel. Since immersed tunnels are constructed under water, this load does not apply.

WL = Wind on live load. The tunnel structure is not exposed to the environment, so it will not be subjected to wind loads.

WS = Wind load on structure. The tunnel structure is not exposed to the environment, so it will not be subjected to wind loads when in service, however, when the tunnel section is being towed to the tunnel site, this is a potential loading. See construction loads (CL) listed above.

Section 3 of the LRFD specifications provides guidance on the methods to be used in the computations of these loads. Loadings unique to immersed tunnels such as anchor and ship impact are calculated as follows.

11.3.3 Ship Anchors

The effect of an anchor impacting the underwater tunnel structure directly or being dragged across the line of the tunnel structure should be considered. Either the tunnel structure should be designed to resist the full loading imposed by the design anchor system, or the backfill / armor system should be designed to mitigate the loading, in which case the tunnel structure should be designed for the demonstrable reduced load. Rupture of the waterproofing membrane should not occur. The design anchor should be selected as appropriate to shipping using or expected to use the waterway, based on the relevant section of Lloyd's Rules.

The penetration depth of a falling anchor through tunnel roof protection material should be estimated. The formulae given in CEB Bulletin d'Information No 187, August 1988, reproduced for reference below provide a good design method to calculate the anchor penetration depth in granular material:

Penetration Depth of a Falling Anchor through Granular Material:

$$x = 10 N_{pen} d_e$$
$$N_{pen} = \sqrt{\frac{m_w}{E_r d_e^3}} \cdot v_i$$
$$d_e = \sqrt{\frac{4A}{\pi}} \qquad \text{11-1}$$
$$A = 0.6 + 0.2\frac{m_a}{1000}$$

where x penetration depth (m)

N_{pen} penetration parameter

d_e equivalent diameter of striking area of anchor (m)

m_w mass of anchor reduced by the mass of the displaced water (kg)

m_a mass of anchor in air (kg)

E_r modulus of elasticity in the longitudinal direction of the layer (N/m²)

v_i impact velocity of anchor (m/s)

A cross-sectional striking area of anchor (m²)

The calculated maximum penetration depth should not exceed 90% of the total thickness of the protection layer covering the tunnel using the 5% fractile value for E_r. The dynamic load factor (DLF) ratio of the static equivalent load on the tunnel roof to the triangular dynamic load pulse $F = m_w v_i / T_d$ may be obtained from Figure 11-16 below using the minimum duration of impact $T_d = x / v_i$ (where x is calculated with the 95% fractal value for E_r), and the natural period T_0 of the affected element.

11.3.4 Ship Sinking

The primary sunken ship design case should be assumed to consist of a ship of the size approximating those using or expected to use the waterway. The imposed loading of a ship on the tunnel should be taken as an appropriate uniform loading over an area not exceeding the full width of the tunnel times a length as measured on the longitudinal axis of the tunnel of 100 feet (30 m). Collision impact loading should not be considered.

If appropriate, a secondary sunken ship design case should be assumed to consist of a smaller vessel, such as a ferry or barge, sinking and impacting the tunnel structure with the stem or sternpost in a manner similar to that of a dropped anchor. A static equivalent concentrated load of 225 kips (1,000 kN) working on an area of 3.3 x 6.6 ft^2 (1x2 m²) directly on the tunnel roof should be considered.

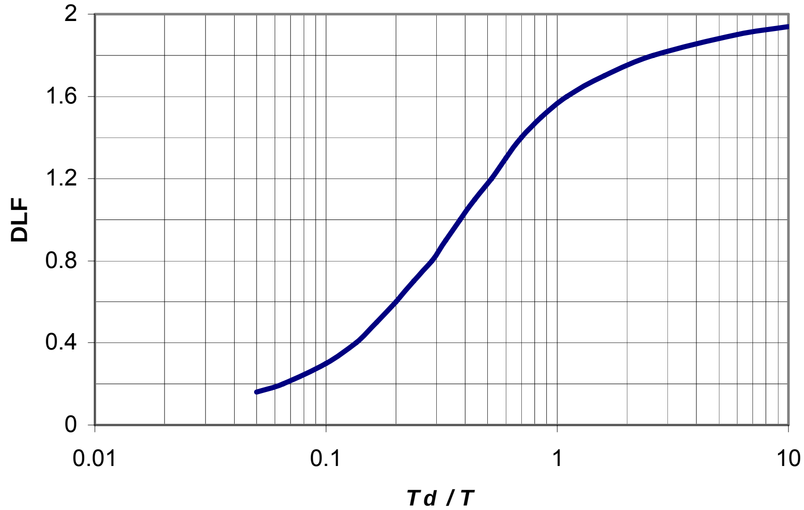


Figure 11-16 Graph of Dynamic Load Factor (DLF) Against Td / T

The intensity of uniformly distributed loading from a sunken ship should be determined by methods such as that outlined in Chapter 6 of the State-of-the-Art Report, 2nd Edition, International Tunnelling Association Immersed and Floating Tunnels Working Group, Pergamon, 1997. In the absence of data to the contrary, it may be assumed that the ship will exert a pressure of 1 ksf (50 kN/m^2).

11.3.5 Load Combinations

The loads described above should be factored and combined in accordance with the LRFD specification and applied to the structure. Paragraph 12.5.1 gives the limit states and load combinations that are applicable for buried structures as Service Limit State Load Combination I and Strength Limit State Load Combinations I and II. These load combinations are given in Table 3.4.1-1. In some cases, the absence of live load can create a governing case. For example, live load can reduce the effects of buoyancy. Therefore, in addition to the load cases specified in Section 12 of the AASHTO LRFD specifications, the strength and service load cases that do not include live load should be used, specifically Strength III and IV and Service IV. In addition, vessel collision forces and earthquake forces must be considered in the design of immersed tunnels. These loads are contained in the Extreme Event I and II load combinations. Combining the requirements of Section 12 and Section 3 as described above results in the following possible load combinations shown in Table 11-1 for use in the design of immersed tunnels:

When developing the loads to be applied to the structure, each possible combination of load factors should be developed. Assessment can then be used to eliminate the combinations that obviously will not govern.

Table 11-1 Permanent In-Service Load Combinations

Load Comb. Limit State	DC		DW		EH* EV# SL		ES		EL	LL, IM	WA	TU, CR, SH, CL		TG	EQ ** CT CV	SE***
	Max	Min	Max	Min	Max	Min	Max	Min					Max	Min		
Strength I	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	1.75	1.00	1.20	0.50	0.00	0.00	γ_{SE}
Strength II	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	1.35	1.00	1.20	0.50	0.00	0.00	γ_{SE}
Strength III	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	0.00	1.00	1.20	0.50	0.00	0.00	γ_{SE}
Strength IV	1.50	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	0.00	1.00	1.20	0.5	0.00	0.00	0.00
Extreme Event I	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	γEQ	1.00	0.00	0.00	0.00	0.00	γ_{SE}
Extreme Event II	1.25	0.90	1.50	0.65	1.35	0.90	1.50	0.75	1.00	0.5	1.00	0.00	0.00	0.00	0.00	γ_{SE}
Service I	1.00		1.00		1.00		1.00		1.00	1.00	1.00	1.20	1.00	0.5	0.50	γ_{SE}
Service IV	1.00		1.00		1.00		1.00		1.00	0.00	1.00	1.20	1.00	1.00	1.00	1.00

* The load factors shown are for at-rest earth pressure. At-rest earth pressure should be used for all conditions of design of immersed tunnel structures.
# The load factors shown are for rigid frames. All immersed tunnel structures are considered rigid frames.
** EQ is used only in Extreme Event I, CT and CV are used, one at a time in Extreme Event II.
*** γ_{SE} is computed is considered on project specific basis. It should be determined based on the certainty that anticipated settlements can be accurately predicted.

11.3.6 Loads during Fabrication, Transportation and Placement

During fabrication, load effects caused by placement of concrete while the element is afloat or by settlements of the foundation (in case of concrete elements), and other items should be evaluated. Some of these loads may cause locked-in stresses that must be considered together with stresses due to external loads.

Particular care must be taken during the placement of concrete while an element is afloat to ensure not only that stresses stay within limits, but also that the deflected shape due to the weight of the new concrete is within acceptable limits. At all times when the element is afloat, stresses due to waves should be checked to ensure that all limit states are satisfied; the wave height and length used in design must be specified for each stage of construction and for towing so that measures can be taken to move the element to a place of safety when forecasts predict conditions that exceed allowable limits. If the freeboard is such that waves could run over the top of an element, this loading should also be taken into consideration.

During transportation and while moored at the outfitting pier or elsewhere and even while in the fabrication yard, a tunnel element can be subject to wind loads that should be considered.

The tunnel element may be suspended from lifting hooks during immersion and may be placed on temporary supports in the final location pending completion of the foundation. All limits states must be satisfied. Temporary supports if used should be released before backfill is placed. When adjacent tunnel elements are connected by shear keys, the effects due to relative differential settlements of each tunnel element during progressive backfilling operations must be taken into account.

Paragraph 3.4.2 of the AASHTO LRFD specifications provides guidance for minimum load factors to be used when investigating loads that occur during construction. The following Table 11-2 reflects the load combinations and load factors to be used when evaluating immersed tunnel sections for construction loads.

Table 11-2 Construction Load Combinations

	DC	EL	WS	CL	WA
Strength I	1.25	1.00	0.00	1.5	1.00
Strength II	1.25	1.00	0.00	1.5	1.00
Strength III	1.25	1.00	1.25	1.5	1.00
Strength IV	1.25	1.00	0.00	1.5	1.00
Service I	1.00	1.00	1.25	1.5	1.03
Service IV	1.00	1.00	0.00	1.5	1.05

11.4 STRUCTURAL DESIGN

11.4.1 General

Historically there have been three basic methods used in the design of immersed tunnels:

- Service load or allowable stress design which treats each load on the structure equally in terms of its probability of occurrence at the stated value. The factor of safety for this method is built into the material's ability to withstand the loading.
- Load factor design accounts for the potential variability of loads by applying varying load factors to each load type. The resistance of the maximum capacity of the structural member is reduced by a strength reduction factor and the calculated resistance of the structural member must exceed the applied load.
- Load and resistance factor design takes into account the statistical variation of both the strength of the structural member and of the magnitude of the applied loads.

The fundamental LRFD equation can be found in paragraph 1.3.2.1 of the AASHTO specification. This equation is:

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \qquad \text{11-2}$$

(AASHTO Equation 1.3.2.1-1)

In this equation, η is a load modifier relating to the ductility, redundancy and operation importance of the feature being designed. The load modifier η is comprised of three components:

η_D = a factor relating to ductility = 1.0 for immersed tunnels constructed with conventional details and designed in accordance with the AASHTO LRFD specification.

η_R = a factor relating to redundancy = 1.0 for immersed tunnel design. Typical cast in place and prestressed concrete structures are sufficiently redundant to use a value of 1.0 for this factor. Typical detailing using structural steel also provides a high level of redundancy.

η_I = a factor relating to the importance of the structure = 1.05 for immersed tunnel design. Tunnels usually are important major links in regional transportation systems. The loss of a tunnel will usually cause major disruption to the flow of traffic, hence the higher importance factor.

γ is a load factor applied to the force effects (Q) acting on the member being designed. Values for γ can be found in Table 11-1 above.

ϕ is a resistance factor applied to the nominal resistance of the member (R) being designed. The resistance factors are given in the AASHTO LRFD specifications for each material in the section that covers the specific material. Specifically, Section 5 covers Concrete Structures and in general, the resistance factors to be used in concrete design can be found in Section 5. However, Section 12 of the AASHTO LRFD specifications gives the following values to be used for ϕ in Table 12.5.5-1:

For Reinforced Concrete Box Structures:

ϕ = 0.90 for flexure
ϕ = 0.85 for shear

Since the walls, floors and roofs of immersed tunnel elements will experience axial loads, the resistance factor for compression must be defined. The value of ϕ for compression can be found in Section 5.5.4.2.1 of the AASHTO LRFD specification given as:

ϕ = 0.75 for compression

Values for ϕ for precast construction are also given in Table 12.5.5-1. However, only rarely under unusual circumstances will a casting yard be set up to create the same controlled conditions that exist in a precast plant. Therefore, it is recommended that the ϕ values given for cast-in-place concrete be used for the design of immersed tunnels.

Structural steel is also used in immersed tunnel construction. Structural steel is covered in Section 6 of the AASHTO LRFD specification. Paragraph 6.5.4.2 gives the following values for steel resistance factors:

For Structural Steel Members:

ϕ_f = 1.00 for flexure
ϕ_v = 1.00 for shear
ϕ_c = 0.90 for axial compression for plain steel and composite members

R_r is the calculated factored resistance of the member or connection.

11.4.2 Structural Analysis

Structural analysis is covered in Section 4 of the AASHTO LRFD specifications. Section 4 describes a number of analysis methods that are permitted. It is recommended that classical force and displacement methods be used in the structural analysis of concrete immersed tunnel elements. Other methods (as described below) may be used, but will rarely yield results that vary significantly from those obtained with the classical methods. The modeling should be based on elastic behavior of the structure as per AASHTO paragraph 4.5.2.1. Steel immersed tunnels can also be analyzed using the same structural model except that the efficiency of any curvature of the steel members will not be fully utilized. Most general purpose structural analysis programs have routines based on these principles for dimensional models.

Since all members of a concrete immersed tunnel element are subjected to bending and axial load, the secondary effects of deflections on the load affects to the structural members should be accounted for in the analysis. AASHTO LRFD specifications refer to this type of analysis as “large deflection theory” in paragraph 4.5.3.2. Most general purpose structural analysis software have provisions for including this behavior in the analysis. If this behavior is accounted for in the analysis, no further moment magnification is required. Alternatively finite element models can be used. These models can identify load sharing, account for secondary effects and identify load paths

Steel immersed tunnel elements, are complex assemblies of plates that might be curved, stiffeners and diaphragms. Simplifying these systems to the point where classical methods of analysis can be used often undermines the efficient use of materials that can result from complex load paths. Steel structures lend themselves well to sophisticated computer modeling such as finite element models. These models can identify load sharing, account for secondary effects and identify load paths. It is recommended that these models be used in the analysis of steel immersed tunnel sections.

Paragraph 4.5.1 of the AASHTO LRFD specifications states that the mathematical model used to analyze the structure should include “…where appropriate, response characteristics of the foundation”. The response foundation for an immersed tunnel element can be modeled through the use of a series of non-linear springs placed along the length of the bottom of the section. These springs are considered non-linear because they should be specified to act in only one direction, the downward vertical direction. This model will provide the proper distribution of loads to the bottom of the model and give the designer an indication if buoyancy is a problem. This indication is seen in observing the calculated displacements of the structure. A net upward displacement of the entire structure indicates that there is insufficient resistance to buoyancy.

Structural models for computer analysis are developed using the centroids of the structural members. Due to the thickness of the walls and the slabs of an immersed tunnel, it is important when calculating the applied loads, that the loads are calculated at the outside surface of the members. The load is then adjusted according to the actual length of the member as input. For example, if the out to out bottom width of a tunnel structure is 90 feet and the bottom of the bottom slab is located 15 feet below the water table, the buoyancy force on the bottom slab is calculated as:

62.4pcf x 15ft x 1ft (along length of tunnel) = 936plf for a total load on the bottom of the tunnel of:

936plf x 90ft = 84,240lbs

If the outside walls of the tunnel are 4ft thick, then the length of the structural model is 90ft – 4ft = 86ft

Using 86ft, the applied buoyancy force is 936plf x 86ft = 80,496lbs. This computation underestimates the buoyancy force by 5 percent. Given that the load factor for the buoyancy force is 1.00, this could result in a buoyancy problem with the tunnel. The solution would be to apply the actual calculated load as follows: 84,240lbs / 86ft = 980plf. This results in a slightly conservative estimate of the load for bending and shear, but an accurate estimate of the buoyancy effect including the axial load in the side walls.

This problem is not as prevalent in a finite element model. However, the designer should be careful that sufficient load is being applied to the model to be sure that the actual conditions are being modeled as closely as possible.

11.5 WATERTIGHTNESS AND JOINTS BETWEEN ELEMENTS

11.5.1 External Waterproofing of Tunnels

External waterproofing for tunnel elements should be considered for both steel tunnels and concrete tunnels. The waterproofing should envelop every part of the element exposed to soil or water with materials impervious to the surrounding waters. For steel tunnels the outer steel membrane would act as waterproofing membrane, while for concrete elements either steel or synthetic membrane should be used. For steel waterproofing membranes used on either concrete or steel elements, an appropriate corrosion protection and monitoring system should be used to ensure that the minimum design thickness is maintained during the life of the facility or an added sacrificial thickness should be provided. Non-structural steel membranes should be no less than 1/4 in (6 mm) thick.

The membrane should be watertight. Typical materials used for concrete elements include two coats of a spray-applied elasticized epoxy material; steel plates; and flexible PVC waterproofing sheet. Minimum thickness should be no less than 0.06 inch (1.5 mm), and anchored to the concrete using T-shaped ribs. The materials of the waterproofing system should have a proven resistance to the specific corrosive qualities of the surrounding waters and soils. The materials of the system should be flexible and strong enough to span any cracks that may develop during the life of the structure. Bituminous membranes are not recommended. The waterproofing system should preferably adhere at every point to the surfaces to which it is applied so that, if perforated at any one location, water may not travel under it to another. The areas of free water flow between the membrane and the underlying concrete in case of leakage should be limited to no more than 100sf ($10m^2$). For a steel tunnel, the membrane could be the external steel shell, provided that an adequate corrosion protection is provided either by cathodic protection or additional sacrificial thickness. Steel plates should be joined using continuous butt welds. All welds should be inspected and tested for soundness and tested for watertightness. Notwithstanding the provision of a membrane, the underlying structural concrete should be designed to be watertight.

Depending upon the type of waterproofing used, it may require protection on the sides and top of the tunnel elements to ensure that it remains undamaged during all operations up to final placement and during subsequent backfilling operations.

11.5.2 Joints

Joints between immersed tunnels elements can be classified as described below.

Immersion Joint (or Typical Joint) The immersion joint is the joint formed when a tunnel section is joined to a section that is already in place on the seabed. After placing the new element, and joining it with the previously placed element, the space between the bulkheads (dam plates) of the two adjoining elements is then dewatered. In order to dewater this space, a watertight seal must be made. A temporary gasket with a soft nose such as the Gina gasket (Figure 11-17) is most often used. In addition an omega seal is also provided after dewatering the
joint from inside the joint.

Figure 11-17 Gina-Type Seal

For immersion joints, the primary compression or immersion seal is usually made of natural or neoprene rubber compounds. The most common cross-section used today is the “Gina” type. This consists of a main body with designed load/compression characteristics and an integral nose and seating ridge. The materials used should have a proven resistance to the specific corrosive qualities of the water and soils and an expected life no shorter than the design life of the tunnel unless the gasket is considered temporary. For flexible joints, a secondary seal is usually required in case of failure of the primary seal. It is usually manufactured from chloroprene rubber to an overall cross-section corresponding to that known as an “Omega” type (Figure 11-18), the materials having proven resistance against the specific corrosive qualities of the water and soils, oil, fungi and micro-organisms, oxygen, ozone and heat.

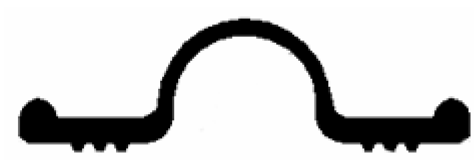


Figure 11-18 Omega Type Seal

Figure 11-19 shows a typical immersion joint. It is essential that immediately after dewatering of the chamber between the two bulkheads, an inspection of the primary seal is made so that any lack of watertightness can be remedied. Similarly, the secondary seal of a flexible joint should be pressure tested up to the expected maximum service pressure via a test pipe and valve to ensure that it too can function as required; after a successful testing, the chamber between the seals should be de-watered.

Closure or Final Joint: Where the last element has to be inserted between previously placed elements rather than appended to the end of the previous element, a marginal gap will exist at the secondary end.

This short length of tunnel sometimes is completed as cast-in-place and is known as the closure or final joint.

The form of the closure or end joint is dependent on the sequence and method of construction. Closure joints may also be immersion joints, although details may need to be different. Potential options for the closure joints include:

Figure 11-19 Gina-type Immersion Gasket at Fort Point Channel, Boston, MA

- Place the last element between two previously placed elements and dewater one joint between the newly placed element and the one of the previously placed elements. Then insert under water closure form plates and place tremie concrete around the closure joint to seal it. The joint can then be dewatered and interior concrete can be completed from within the joint. Other methods such as telescopic extension joints and wedge joints have been developed to make the closure joint similar to the immersion joint.
- Construct both end (terminal) joints first, lay the tunnel elements outwards from these and complete the immersed tunnel with a special closure (final) joint.
- Construct one terminal joint first and lay all the immersed tunnel elements outwards from that side and backfill over the top of the final element, using a soil-cement mixture (or other reasonably watertight material) in the vicinity of the second terminal joint. Construct the structures abutting the second terminal joint after the immersed tunnel is complete.
- Lay and complete the immersed tunnel with or without a special closure joint and backfill at the terminal elements using a soil-cement mixture (or other reasonably watertight material) in the vicinity of both terminal joints. Construct the structures abutting the both terminal joints after the immersed tunnel is complete.

Earthquake Joint This may be an immersion joint of special design to accommodate large differential movements in any direction due to a seismic event. It also applies to a semi-rigid or flexible joint strengthened to carry seismic loads and across which stressed or unstressed prestressing components may be installed.

<u>Segment or Dilatation Joint</u> Moveable segment joints must be able to transmit shear across the joint and well as allowing dilatation and rotation. The joints contain an injectable rubber-metal waterstop as well as neoprene and hydrophilic seals.

11.5.3 Design of Joints between Elements

All immersed tunnel joints must be watertight throughout the design life, and must accommodate expected movements caused by differences in temperature, creep, settlement, earthquake motions, method of construction, etc. Displacements in any direction should be limited so that the waterproof limits of a joint are not exceeded. Joint shear capability should take into account the influence of normal forces and bending moments on the shear capacity of the section; the design should take account of shear forces generated where the faces of the joints are not normal to the tunnel axis. Joints must be ductile in addition to accommodating longitudinal movements. Tension ties may be used to limit movement so that joints do not leak or break open, especially during a seismic event.

The axial compression of tunnel elements and bulkheads due to depth of immersion should be taken into account in determining joint dimensions at installation.

The design of primary flexible seals at tunnel joints must be designed to take into account the maximum deviations of the supporting frames relative to their theoretical location, the maximum deviation of the planes of the frames, and any relaxation of the seal. The seal is required to have a minimum compression of 3/8 inch (10 mm) greater than the compression required to maintain a seal. Just in case an initial seal is not obtained after immersion and joining, it may be advisable in some cases for the immersion joint to be designed so that a backup method of obtaining an initial seal is available.

For flexible joints, a secondary seal (omega) capable of carrying the full water pressure should be fitted across the inside of the joint and should be capable of being inspected, maintained and replaced. The seal should be capable of absorbing the long-term movements of the joint. The secondary seals should be provided with a protective barrier against damage from within the tunnel. All joints in the tunnel should be finished to present a smooth surface.

The metal hardware in joints should have a design life adequate to fulfill its purpose throughout the design life of the joint. Nuts and bolts for primary and secondary seals should be stainless steel. Plate connections between elements should be corrosion-protected to ensure that the design life is obtained.

The mounting procedure or the mounting surface for the primary seal of immersion joints must allow for fine adjusting and trimming of the seal alignment in order to compensate for construction tolerances. It is recommended that the gasket be protected from accidental damage until the time of immersion. All embedded parts, fixings, including the bolts and their corrosion protection system, mating faces, clamping bars and other fixings, must have a design life at least equal to that of the tunnel structure. Where clamping bars and other fixings are used for the secondary seal, these need to have a design life at least equal to that of the secondary seal. The gasket assembly should have provision for injection in case of leakage.

CHAPTER 12
JACKED BOX TUNNELING

12.1 INTRODUCTION

Jacked box tunneling is a unique tunneling method for constructing shallow rectangular road tunnels beneath critical facilities such as operating railways, major highways and airport runways without disruption of the services provided by those surface facilities or having to relocate them temporarily to accommodate open excavations for cut and cover construction (Chapter 5). Originally developed from pipe jacking technology, jacked box tunneling is generally used in soft ground at shallow depths and for relatively short lengths of tunnel, where TBM mining would not be economical or cut-and-cover methods would be too disruptive to overlying surface activities.

Jacked box tunneling has mostly been used outside of United States (Taylor et al, 1998) until it was successfully applied to the construction of three short tunnels beneath a network of rail tracks at South Station in downtown Boston. These tunnels were completed and opened in 2003 as a part of the extension of Interstate I-90 for the Central Artery/Tunnel (CA/T) Project. Figure 12-1 shows the opening ceremony for the completed I-90 tunnels. Since CA/T Project represents the most significant application to date of the jacked box tunneling in the US, it will be used to demonstrate the method throughout this Chapter.

Figure 12-1 Completed I-90 Tunnels

12.2 BASIC PRINCIPLES

Figure 12-2 illustrates the basic jacking sequence of jacked box tunneling under an existing railway. The box structure is constructed on jacking base in a jacking pit located adjacent to one side of an existing railway. A tunneling shield is provided at the front end of the box and hydraulic jacks are provided at the rear. The box is advanced by excavating ground from within the shield and jacking the box forward into the opening created at the tunnel heading. In similar fashion to pipe jacking, lengths of tunnel that would exceed the capacities of jacks situated at the rear of the box structure can be successfully advanced into place by dividing the box structure into sections and establishing intermediate jacking stations. The box structure shown in Figure 12-2 is divided into two sections with an intermediate jacking station set up in between them.

In order to maintain support to the tunnel face, excavation and jacking normally carried out alternately in small increments, typically in the range of 2 to 4 feet. In most cases, the soft ground must be treated by means of ground improvement techniques such as ground freezing, jet grouting, etc. as discussed in Chapter 7 Soft Ground Tunneling to enhance its stand up time. Refer to Chapter 5 for discussions about temporary excavation support systems.

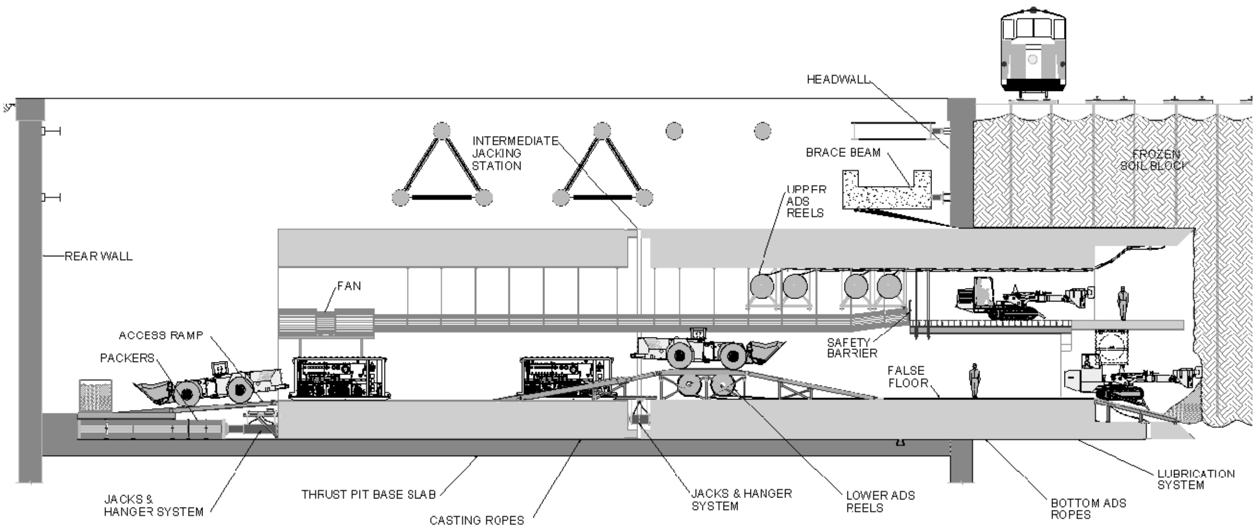


Figure 12-2 Typical Jacked Box Tunneling Sequence under an Existing Rail Track

12.3 CENTRAL ARTERY/TUNNEL (CA/T) PROJECT JACKED BOX TUNNELS

The use of the jacked box tunneling method on the CA/T Project in Boston is described by van Dijk et al. (2000) and van Dijk et al., (2001). A major component of the CA/T project was the extension of Interstate I-90 eastward to Boston's Logan International Airport. This extension required three crossings of the network of tracks leading into South Station, a regional transportation hub used by Amtrak and the Massachusetts Bay Transportation Authority (MBTA) for hundreds of train movements daily. The critical surface use of the site, the large spans of the underground openings required to accommodate a multi-lane highway, the relatively shallow cover dictated by the roadway profile, and the poor soils in combination with the high groundwater level at the site led to tunnel jacking being selected as the preferred tunneling method over staged cut-and-cover and conventional tunneling techniques.

The three crossings of the tracks consisted of box structures for the eastbound lanes of I-90, the westbound lanes, and westbound exit ramp that provided access to Interstate I-93. The box structure for the I-90 EB lanes was the longest of the three, at 379 feet. It was constructed in 3 sections, with cross-sectional dimensions of 36 feet high by 79 feet wide, and a total weight of approximately 32,500 tons. The other two box structures were 38 feet high by 78 feet wide and were each constructed in two sections. The I-90 WB tunnel was 258 feet long and weighed approximately 27,000 tons, while the exit ramp tunnel was 167 feet long and weighed 17,000 tons.

Subsurface Condition and Ground Freezing: As shown in Figure 12-3, the geologic conditions through which the three box tunnel structures were jacked included (at the top of the subsurface profile) a layer of miscellaneous fill 20 to 25 feet thick, primarily a medium dense silty sand. This fill layer contained a number of obstructions related to the more than 150 years use of the site for rail road, industrial and waterfront infrastructure, which included granite block seawalls, rock filled timber cribwalls, brick and masonry structure foundations, a buried trackway, and an abandoned brick-lined sewer. Below the historic fill material was a deposit of weak organic sediments 10 to 15 feet thick, consisting of organic silt with some fine sand and peat. Underlying the organic layer were lenses of alluvial sand and inorganic silt deposits, generally less than 5 feet thick. The remaining part of the profile through which the tunnel boxes were jacked consisted of marine clay, consisting of clay and silt that was soft, except for the upper 15 feet, which was somewhat stronger and less compressible. Groundwater at the site was generally 6 to 10 feet below track level, resulting in the tunneling horizon in each case being completely submerged.

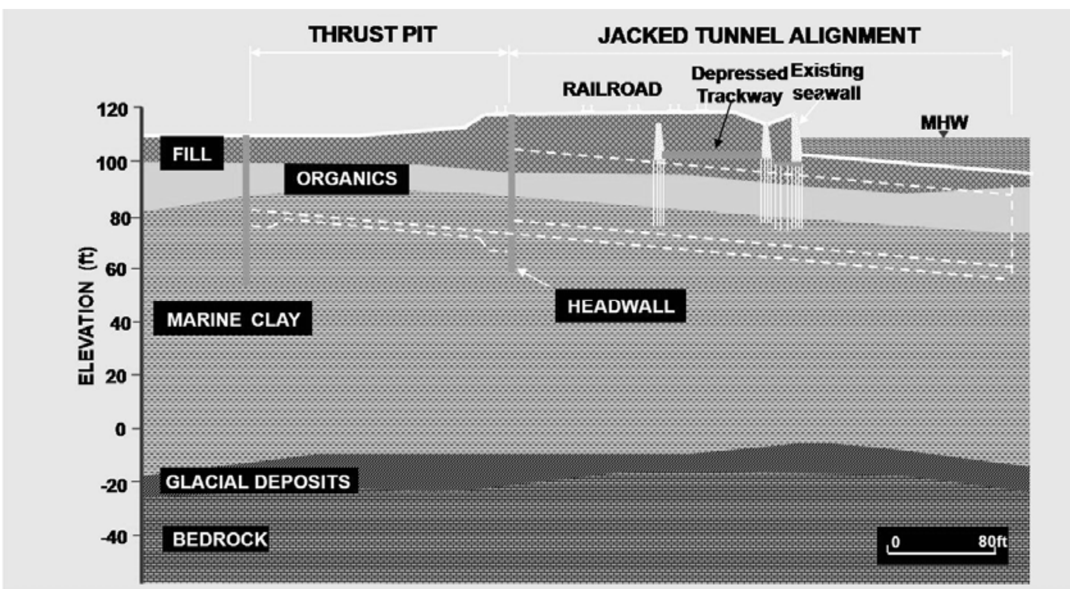


Figure 12-3 Generalized Subsurface Profile for the I-90 Jacked Box Tunnels

The success of the box jacking operation depended critically on maintaining the stability of the soils through which the tunnels passed. For the existing condition of weak soils below the groundwater table, shallow cover over the tunnel boxes, and the large spans required, there were serious concerns about loss of ground at the headings, the potential for significant settlement of the overlying track structures, and loss of alignment control during jacking. Therefore, ground improvement was required to enable the tunneling to be performed effectively and safely. The original design for the tunnels had called for ground treatment consisting of a combination of dewatering and chemical grouting in the miscellaneous fill materials; horizontal jet grouting in the organic sediments; and soil nailing of the marine clay layer. The Contractor was concerned about the potential risks associated with the implementation of this combination of soil stabilization methods, and consequently made a value engineering proposal to substitute ground freezing for all of the methods. This proposal was accepted, and a large scale freezing operation was performed that encompassed all three tunnel alignments, as further discussed below in Section 12.5.

<u>Box Casting Operation:</u> Each tunnel box structure was constructed in a “jacking pit” immediately adjacent to the west side of the South Station track network (Figure 12-4). The jacking pits were constructed by slurry wall methods, with post-tensioning of the sidewalls and the formation of a low-level improved soil “strut” formed by jet grouting below the base slab to reduce the number of bracing levels required so that the boxes could be constructed without interference from a cross-lot bracing system. The concrete base slab of each jacking pit was placed with a tight tolerance on grade, since it served as the casting surface for the box structures and also established the starting profile to ensure that the tunnel sections were jacked to the required alignment.

Figure 12-4 Tunnel Structure Construction Operation

After the base slab of each jacking pit was completed, a series of steel wire ropes were installed longitudinally on the slab and steel plates covering the entire footprint of the box sections were placed on the wire ropes. Shear studs were welded to the base plates to anchor the plates to the concrete slab, so that when jacking started, the frictional resistance that the jacks needed to overcome to move the box structures would result from steel plates sliding over steel wire ropes, rather than concrete sliding on concrete. Figure 12-4 shows the construction of the I-90 WB tunnel box structures.

The structural design of these tunnel sections had to consider not only the long term loads from the overburden and railroad surcharge loads, but also the construction phase jacking loads. Each tunnel was constructed in sections (2 sections for the I-90 WB exit ramp tunnel and the I-90 WB tunnel itself, and 3 sections for the I-90 EB tunnel) to reduce the jacking forces required to move the tunnels into their final positions by using intermediate jacking stations in addition to the jacks positioned at the rear. To prevent soil from entering into the gap between adjacent box sections, a system of transversely continuous sliding overlapping steel "bridge" plates were used. Once jacking was completed, the jacks were removed and the intermediate jacking station areas were filled with concrete.

The external surfaces of the box structures could not be waterproofed because the waterproofing material would have been torn away during jacking. Water seepage control was achieved by using low permeability concrete mixes to construct the boxes and grouting the interface between the boxes and the surrounding ground through grout ports cast into the walls and roof slab after tunneling and jacking were completed.

A cellular concrete shield was constructed at the front of each lead box section to support the excavation operation by establishing multiple access points to the face that could be closed off if stability problems developed. A beveled steel knife edge was provided at the perimeter of the shield that was flared a small amount to ensure that the opening into which the tunnel box structures would be jacked could be closely controlled, but also excavated large enough to prevent the boxes from getting stuck as they were pushed forward.

Tunnel Excavation: Mining of the frozen soils at the tunnel face, which had estimated uniaxial compressive strengths in the range of 700 to 1400 psi, was done primarily with roadheaders, working at two levels within the shield. Figure 12-5 shows a typical view of the roadheader mining operation.

Figure 12-5 Excavation of the Frozen Ground at the Front of the Tunnel Shield by Roadheader

The roadheaders also proved to be effective at removing the numerous timber piles that were encountered. For removing masonry obstructions, which were firmly bound in place in the frozen soil mass, hydraulic hammers were used. The excavated material dropped to the bottom of the shield during the mining operation, where it was collected using a Gradall machine and a loader. A wheel-mounted scoop tram was used to shuttle the material to the rear of the tunnel box structure and dump it into a skip bucket, which was lifted out of the pit by crane and stockpiled for loading onto haul trucks. Figure 12-6 shows the scoop tram loading the skip bucket.

Figure 12-6 Scoop Tram Loading Excavated Material into Skip Bucket for Removal

Based on typical mining production rates, stand-up time for the unsupported frozen ground, the volume of excavated material to be handled, the design of the jacking system, and the shift schedule, the Contractor determined that incremental excavation advance for efficient, consistent progression of the jacking operation was approximately 3 feet. Depending on the amount of obstructions encountered in a particular round, the advance rate achieved was generally one to two rounds per day, or 3 to 6 feet. At the completion of each excavation increment, the Contractor had to check the shield perimeter to ensure that all obstructions, including abandoned freeze pipes, were cut back sufficiently to be clear of the tunnel box.

<u>Anti-Drag System</u> As discussed previously, an anti-drag system was installed above and below the tunnel box structure to reduce the frictional resistance between the box structure and the surrounding ground. The system worked to even out the friction acting over the roof and bottom surface areas of the box, which contributed to alignment control during jacking, and also reduced the potential for surface settlement and lateral movement of the shallow overburden over the tunnel by separating the interface between the box concrete and the soil. This was achieved by installing a series of greased ¾-inch diameter wire ropes that were anchored to the jacking pit and threaded through slots in the shield into the interior of the tunnel box structure, where they were stored on slings mounted on the soffit of the roof slab and on reels on the base slab located inside the tunnel. The system was configured so that as the tunnel moved forward, the wire ropes were run out from the storage units to cover the portion of the top and

bottom surfaces of the box structure that was embedded in the ground beyond the thrust pit. More discussions of the Anti-Drag System (ADS) are provided in Section 12.4.1.

Tunnel Jacking Operation: At the completion of each excavation round, the tunnels structures were jacked into the space created at the face. This was accomplished by a group of 25 hydraulic jacks positioned at base slab level at the rear of the tunnel box, and additional groups of 26 to 32 jacks situated in the intermediate jacking stations. Each jack had a working capacity of 533 tons at a working pressure of 6100 psi, and could deliver a maximum thrust of 889 tons at a pressure of 10,200 psi. The maximum stroke of the rear jacks was 42 inches, while the stroke of the intermediate station jacks was limited to 16.5 inches. At each jacking station, the individual jacks were connected in nine clusters of 2 to 4 jacks each. This simplified the hydraulic control, and also enabled some horizontal steerage capability through variable operation of the clusters. The required thrust reaction for the jacks was transferred to a heavily reinforced concrete block wall at the rear of the jacking pit through a series of steel pipe sections referred to on the project as “packers.” The loads exerted on the reaction block wall were in turn transferred into the surrounding ground through the pit base slab and rear wall. More discussions about the jacking operations are included in Section 12.4.2.

12.4 LOAD AND STRUCTURAL CONSIDERATIONS

In most aspects, the structural loading and design considerations for jacked box tunnels are similar to those for cast-in-place cut and cover tunnels as discussed in Chapter 5. Readers are referred to Sections 5.3 for detailed discussions about structural framing, design, buoyancy, waterproofing, etc., and Sections 5.4 about loads and load combination. Section 5.5 provides discussions about structural design procedures and considerations for a box tunnel.

However, in addition to the typical design loads discussed in Section 5.4, jacked box design can be dominated by two unique loads during construction: jacking thrust loads and interface drag loads.

12.4.1 Ground Drag Load and Anti-Drag System (ADS)

Ground drag, resulted from the contact pressures between soil and box structure is calculated and multiplied by appropriate friction factors, and is used to estimate drag loads at frictional interfaces; an appropriate adhesion value is used at the interface between the box and cohesive ground. Simplifying assumptions are made in developing ADS loads and modeling box/ADS/soi1 interaction, the validity of which is done by back-analyses of loads and other historical data. To reduce such an enormous drag load, an anti-drag system (ADS) is used to separate the external surface of the box from the adjacent ground during tunnel jacking.

As described in Section 12.3, the CA/T tunnels utilized an ADS consisting of an array of closely-spaced wire ropes which are initially stored within the box with one end of each rope anchored at the jacking pit. As the box advances, the ropes are progressively drawn out through guide holes in the shield and form a stationary separation layer between the moving box and the adjacent ground. The drag forces are absorbed by the ADS and transferred back to the jacking pit. In this manner the ground is isolated from drag forces and remains largely undisturbed. Readers are also referred to Ropkins 1998 for more discussions for other ADS applications.

12.4.2 Jacking Load

The ultimate bearing pressures on the face supports and on the shield perimeter are used to calculate the jacking load required to advance the shield. Note that the face pressure must be analyzed using the treated soil properties. In addition, jacking load also includes the ADS loads as discussed above.

Jacking thrust is provided by means of specially built high capacity hydraulic jacking equipment. Jacks of 500 tons (4,448 kN) or more can be utilized on large tunnels. As discussed in Section 12.3, jacks with a capacity of 533 tons at a working pressure of 6100 psi (42 MPa) were used in the I-90 tunnels (Figure 12-7). For jacking a large size road tunnel structure, multiple jacks are required to provide sufficient jacking thrust to counter the face pressure. In addition, using multiple jacks offers some steerage control redundant capacity in the event of possible underestimates of the required jacking loads.

Reaction to the jacking thrust developed is provided by either a jacking base or a thrust wall, depending on the site topography and the relative elevation of the tunnel. An example of a heavily reinforced thrust block wall is also shown in Figure 12-7. These temporary structures must in turn transmit the thrust into a stable mass of adjacent ground. A thrust wall is normally stabilized by passive ground pressure. In developing this reaction, the wall may move into the soil and this movement must be taken into account when designing the jacking system. When a thrust wall is used in a vertical sided jacking pit, care is required to ensure that movement of the thrust wall under load does not cause any lack of stability elsewhere in the pit.

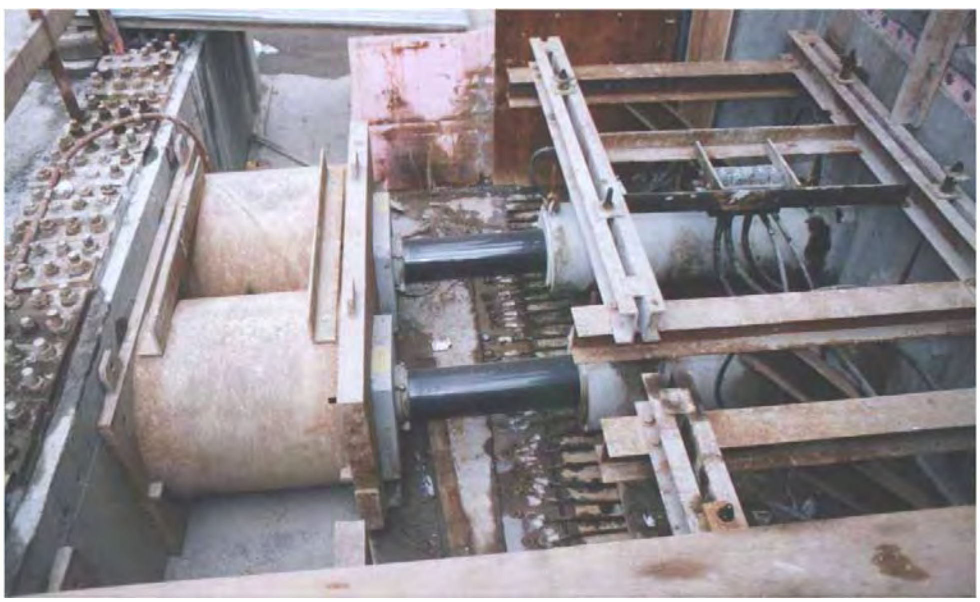


Figure 12-7 Close Up of High Capacity Hydraulic Jacks, Reaction Blocks, and Packers

As the tunnel box and the jacks were gradually advanced away from the thrust wall, the Contractor needed to come up with a method to continue to transfer the jacking reaction force back into the thrust block wall. This was done by installing a series of 3-ft diameter structural steel pipe sections (i.e., packers) to bridge the gap between the jack pistons and the thrust block wall. Figure 12-7 also shows Initial short packer sections installed once the tunnel box structure had been jacked away from the rear reaction block a distance exceeding the maximum stroke of the jacks. The packers were connected together with 1-inch thick diaphragm plates that were anchored to the base slab in the thrust pit. Three views of the packer installations are shown in Figure 12-7, Figure 12-8, and Figure 12-9.

A jacking base is normally stabilized by shear interaction with the ground below and on each side. Where the interface is frictional, the interaction may be enhanced by surcharging the jacking base by means of pre-stressed ground anchors or compacted tunnel spoil. The jacking base is also stabilized by both the top and bottom ADS which are anchored to it.

Figure 12-8 Installation of Packer Sections and Connecting Diaphragm Plates

Figure 12-9 Progressive Installation of Packer Sections and Connecting Diaphragm Plates

12.5 GROUND CONTROL

As discussed previously, the soft ground most likely will need to be pre-treated to provide sufficient stand-up time during jack tunneling. In addition, ground may need to be stabilized in advance to control surface settlement must be controlled when tunnel jacking at such a shallow depth.

Techniques for stabilizing ground for jacked box tunneling include: grouting, well point dewatering, and freezing which are presented in Sections 7.6.5 “Grouting Methods”, 7.6.6 “Ground Freezing”, and 7.6.7 “Dewatering”. Ground freezing is discussed hereafter to demonstrate how a ground control measure is used for jacked box tunneling.

12.5.1 Ground Freezing for CA/T Project Jacked Tunnels

As discussed in Section 12.3 above the Contractor made a value engineering proposal to replace the various soil stabilization methods indicated in the Contract with ground freezing. This alternative approach offered several advantages, including the ability to completely stabilize the soil mass through which the tunnel box structures were jacked. In contrast, the horizontal jet grouting and soil nailing methods in the original design, would have required tunnel jacking to be interrupted periodically to permit installation of the ground improvement measures from the heading. Ground freezing also offered: 1), the advantages of improved face stability, which made breasting of shield compartments unnecessary, 2), better encapsulation of obstructions which otherwise had the potential to suddenly ravel into the heading when exposed, and 3) the avoidance of windows of untreated ground.

The freezing system was installed entirely from the ground surface overlying each tunnel alignment, within the track network. The Contactor selected a conventional brine freezing system, with an ammonia plant providing the refrigeration. In the freeze plant, ammonia gas was compressed, condensing it to a liquid, then evaporated to chill the brine to an average temperature range of -25°C to -30°C. The brine used to cool and eventually freeze the ground was circulated through circuits of vertical freeze pipes as shown schematically in Figure 12-10.

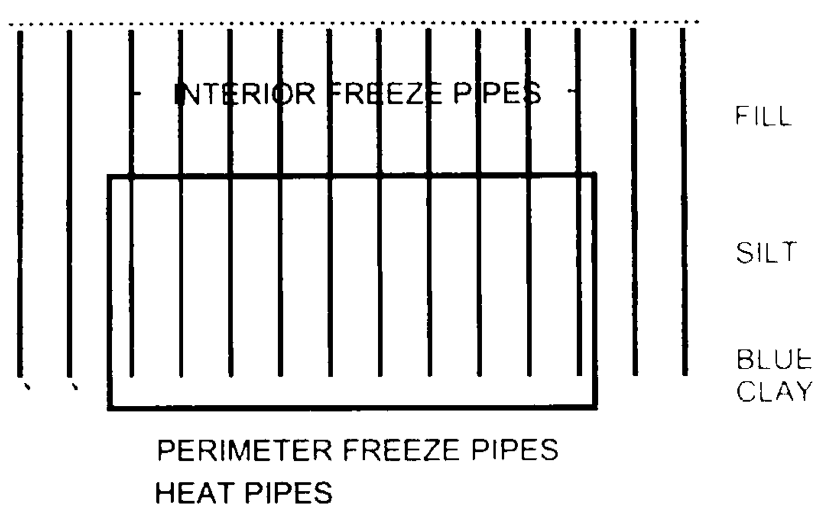


Figure 12-10 Schematic Arrangement of Freeze Pipes to Freeze Ground Mass Prior to Tunnel Jacking

Each individual freeze pipe consisted of a 4.5-inch diameter steel pipe closed at the end, with a 2-inch diameter plastic pipe inserted in it that was open at the bottom. As shown in Figure 12-11, the chilled brine was pumped from a supply header line into the inner pipe, where it exited at the bottom and rose up in the annulus between the inner and outer pipe, cooling the surrounding ground in the process. At the top of the pipe, the brine was sent to the next freeze pipe for cooling circulation, as part of a circuit of 4 to 7 pipes. After passing through all of the pipes in the circuit, the brine was pumped back to the freeze plant for re-chilling through a return header pipe. The brine was circulated continuously in this manner through all of the circuits comprising the freeze zone in what was a closed system. The temperature of the ground mass was gradually lowered over a period of 4 to 5 months until the soil froze and an average target temperature of -10°C was reached.

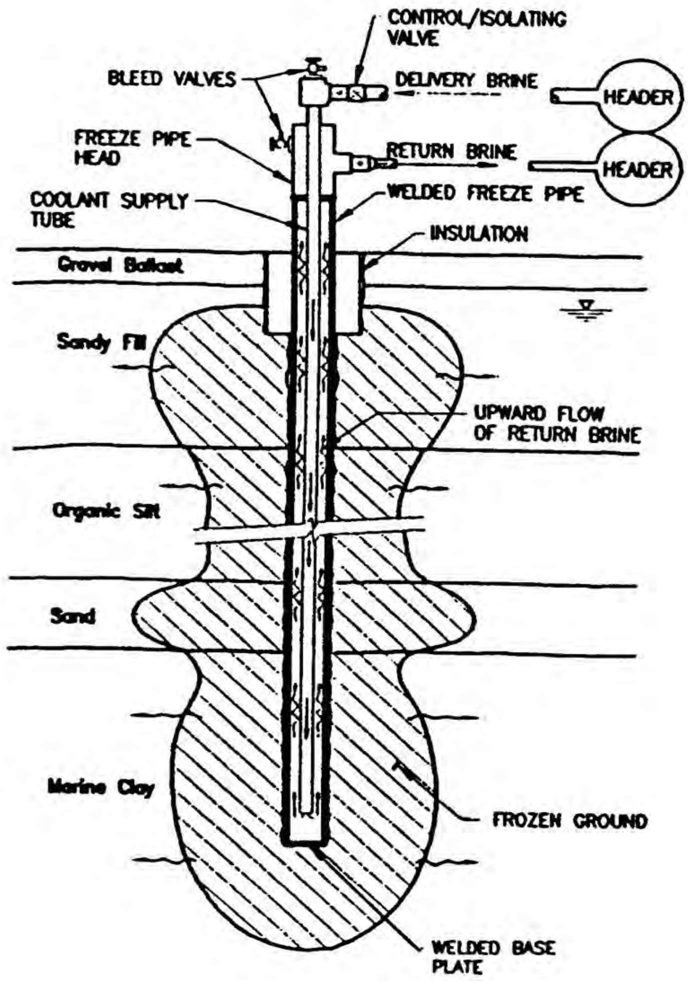


Figure 12-11 Arrangement of an Individual Freeze Pipe showing Brine Circulation

The freeze pipes were installed within the track area using a sonic type drill rig, which used a vibratory coring bit to advance a starter hole through the miscellaneous fill material and any obstructions contained within it, and then vibrated the outer steel freeze pipe into place in a dry drilling process that displaced the underlying organic sediments and marine clay deposits. The drill rig was mounted on a turntable on the back of a high-rail truck vehicle, which provided flexibility for locating the pipes between rails and outside of the timber ties and switching and signal equipment. Most of the drilling work was done at night by using a series of carefully coordinated track outages with the Railroad, and the sonic drilling method proved to be very effective for installing the freeze pipes quickly with relatively little drilling spoils being generated. Figure 12-12 shows the system in operation while commuter trains continued to run through the freezing area. A total of nearly 1800 freeze pipes were used on the project to freeze over 3.5 million cubic feet of soil.

Figure 12-12 Ground Freezing System in Operation while Commuter Trains Run Through the Area

The ground freezing method was very effective at providing a stable face over the entire tunnel cross-sectional face area, as shown in Figure 12-13. The one significant disadvantage of the method was the expansion of water when it freezes caused the overlying track area to heave. The amount of heave varied considerably over the alignment of each tunnel, depending on the variation in moisture content of the underlying soil profile. Typically the maximum deformation, which was monitored daily by detailed surveys of rail elevations, was in the range of 4 to 7 inches. The heave tapered to the original ground elevation over distances that extended laterally from tunnel centerline to approximate distances of about 50 to 70 feet beyond the edge of the tunnel box structure. The magnitude of this deformation required periodic re-profiling of the tracks by the Railroad to ensure that their rail geometry requirements for safe operation of their trains were maintained.

The temperature of the frozen soil mass was monitored by a series of temperature probes installed at each freeze site. After the target temperature was reached, the freeze system was adjusted to maintain that temperature, which controlled the stability of the soils at the tunnel face. As the excavation progressed for each tunnel, the freeze circuits were shutdown and the brine and inner pipes removed from the outer steel pipes, which were left in place. This progressive shut-down and dismantling of the freeze system was timed to avoid any significant warming of a section of the soil mass prior to it being exposed in the tunnel heading. When the abandoned steel freeze pipes were encountered, they were removed by cutting them out with a torch.

The Tunnel Designer should ensure that ground treatment measures do not in themselves cause an unacceptable degree of ground disturbance and surface movement.

Figure 12-13 Frozen Face Seen from Shield at Front of Jacked Box Structure

12.5.2 Face Loss

Design should also include provisions for controlling face loss which occurs when the ground ahead of the shield moves towards the tunnel as a result of reduction in lateral pressure in the ground at the tunnel face. With face loss, as the tunnel advances, a greater volume of ground is excavated than that represented by the theoretical volume displaced by the tunnel advance.

In cohesive ground, face loss is controlled by supporting the face at all times by means of a specifically-designed tunneling shield and by careful control of both face excavation and box advance. The shield is normally divided into cells by internal walls and shelves which are pushed firmly into the face. Typically 0.5 ft (150mm) of soil is trimmed from the face following which the box is jacked forward 0.5 ft (150mm). This sequence is repeated until the tunneling operation is complete, thus maintaining the necessary support to the face.

12.5.3 Over Cut

Design should also include provisions for controlling overcut in soft ground, by ensuring that the shield perimeter is kept buried and cuts the ground to the required profile. However, a degree of over-cut at the roof and sides beyond the nominal dimensions of the box is required for three reasons:

1. The hole through which the box travels must be large enough to accommodate irregularities in the external surfaces of the box.
2. It is desirable to reduce contact pressures between the ground and the box, to reduce drag.
3. Overcutting may be required to fully remove obstructions at the perimeter of the shield.

The amount of over-cut required should be minimized if unnecessary ground disturbance and surface settlement is to be avoided. This demands that the external surfaces of the box be formed as accurately as possible. Typical forming tolerances are: ± 0.4 in (10mm) at the bottom and ± 0.6 in (15mm) at the walls and roof.

12.6 OTHER CONSIDERATIONS

12.6.1 Monitoring

The jacked box tunneling operation must be carefully monitored and controlled to ensure proper performance and safety. Throughout the tunneling operation, movements at the ground surface over the area affected by the tunneling operation, jacking forces and vertical and horizontal box alignment are all regularly monitored and compared to predicted or specified values.

Chapter 15 presents a variety of available instrument for monitoring ground surface movement (Section 15.2). Section 15.7 discusses overall instrumentation management considerations. Daugherty (1998) also provides detail discussions about the instrumentation design for the CA/T C09A4 tunnels.

12.6.2 Vertical Alignment

Design should also include provisions for controlling vertical alignment. A long box has directional stability by virtue of its large length to depth ratio. The box is guided during the early stages of installation by its self weight acting on the jacking base. Beyond the jacking base, the bottom ADS 'tracks' maintain the box on a correct vertical alignment. As the pressure on the ground under the 'tracks' is normally less than or similar to the pre-existing pressure in the ground and as localized disturbance of the ground is eliminated, no settlement of the tracks can occur. Any tendency for the box to dive is thereby prevented.

In the case of a short box or series of short boxes, it is necessary to steer each box by varying the elevation of the jacking thrust. This is done by arranging groups of jacks at each jacking station at different elevations within the height of the box and by selectively isolating individual groups. The jacking process is complicated by the need to check, at each stage of the operation, the alignment of all box units and if necessary to employ a suitable steering response at all jacking stations.

12.6.3 Horizontal Alignment

Design should also include provisions for controlling horizontal alignment. As discussed previously under vertical alignment, a long box has a degree of directional stability by virtue of its length to width ratio, and is normally guided during the early stages of installation by fixed guide walls located on the jacking base along both sides of the box. Where appropriate, steerage may also be used and is normally provided by selectively isolating one or more groups of thrust jacks located across the rear of the box. Depending on the ground conditions, some adjustment in horizontal position can also be obtained by controlling the amount of undercut/overcut of the excavation on one side of the heading relative to the other.

In the case of a short box or series of short boxes, fixed side guides are also appropriate but more reliance has to be placed on steerage.

Appendix F

Sequential Excavation Method Example

Appendix F – Sequential Excavation Method Example

The calculation example involves the tunneling analysis and lining design of a typical two-lane highway tunnel using the finite element code Phase2 by Rocscience, Inc. The calculation is carried out in stages and follows the approach laid out in 9.7.2.3 above and evaluates ground reaction as indicated in 9.7.2.4 and evaluates support elements as described in 9.7.2.5 and 9.7.2.6.

In this example, homogeneous, isotropic ground conditions are assumed. The constitutive model is based on the Mohr-Coulomb failure criterion. Table 9-6 displays each calculation stage in a left column, typical output graphics in the middle column and further explanations and comments in the right column. The calculation is for a SEM tunnel that uses a top heading and bench excavation sequence. After each excavation step (top heading and bench) the initial support elements are installed and consist of rock dowels and an initial shotcrete lining.

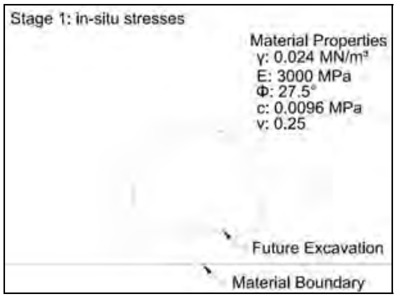
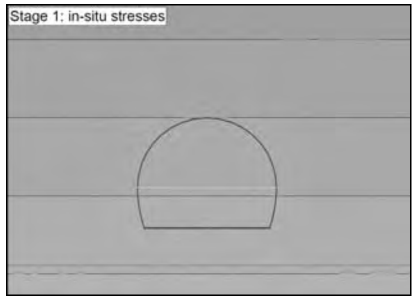
After establishing the initial, geostatic stress conditions in Stage 1, the excavation and installation of initial support is carried out in stages 2 through 5. The tunnel final lining installation occurs in stage 6. For simplicity, it is assumed that the initial lining will deteriorate completely and all ground loads will be imposed onto the final lining in stage 6. No other loading conditions such as ground water loads or seismic loading are included in this example.

The structural capacity of the initial and final linings is evaluated using so called Capacity Limit Curves or “CLCs.” The calculated section force combinations N-M, i.e., initial or final lining normal forces N and lining bending moments M are graphed onto charts where the CLCs denote the capacity of the structural lining section in accordance with ACI 318.

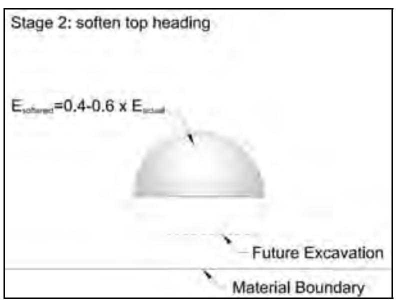
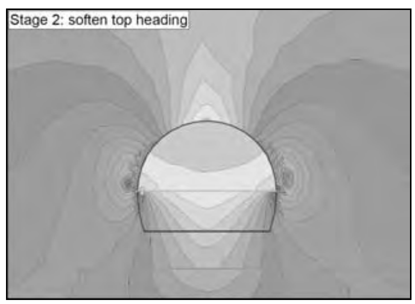
Section force combinations N-M are obtained from each finite element included in the representation of the lining (beam or shell) in the numerical modeling. The capacity of the lining is displayed in accordance with ACI 318 considering lining thickness, concrete (shotcrete) design strength, and structural reinforcement of the lining section. Steel fibers are used for the structural reinforcement of the shotcrete initial lining and conventional, deformed bars are used for the reinforcement of the concrete linings.

The example is presented in a tabulated format (Table F-1) as follows. Note the last row of Table F-1 represents the capacity limit curves for both the initial shotcrete and final concrete linings. All N-M (normal force-bending moment) combinations represented by dots fall well within the enveloping CLCs indicating that in this example the linings as designed will provide sufficient capacity for the anticipated ground conditions and associated ground loads.

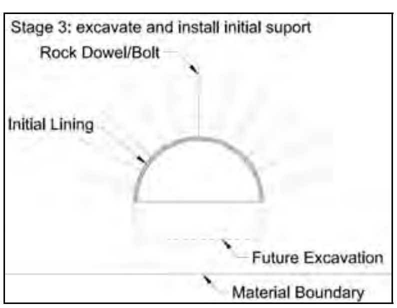
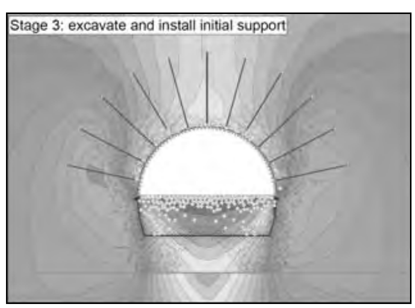
TABLE F-1 SEM Calculation Example for a Two-Lane Highway Tunnel in Rock

<table>
<tr><td colspan="3">Stage 1: This stage assesses the in-situ, geostatic stress conditions prior to the tunnel construction. It considers the unit weight of the ground material, lateral loads dictated by the lateral earth pressure coefficient, any tectonic stresses and overburden loads. Main input parameters involve unit weight (γ), modulus of elasticity (E), friction angle (φ), cohesion (c) and Poisson's Ratio (ν). This stage is considered to be the 'initial stage' of the model prior to any tunnel excavation.</td></tr>
<tr><td>
</td><td>
</td><td>Stage 1: Geostatic Stress Conditions

Output Options: Ground Stresses and Deformations

Output Shown: Major Principal Ground Stress Sigma 1</td></tr>
<tr><td colspan="3">Stage 2: The tunnel excavation causes ground relaxation and ground deflection related to it occurs ahead of the advancing tunnel construction face and around the tunnel. While this relaxation causes ground deflection and surface settlements near excavations, the ground movement also mobilizes shear resistance in the ground. This ground relaxation due to the excavation process before support installation associated with the excavated round length is approximated by "softening" the material within the top heading; the ground material within the top heading is softened by reducing the stiffness of the material by 0.4 – 0.6 times the actual ground modulus (E_{actual}).</td></tr>
<tr><td>
</td><td>
</td><td>Stage 2: Excavation of the Top Heading

Output Options: Ground Stresses and Deformations

Output Shown: Major Principal Ground Stress Sigma 1 (Note stress relaxation above the tunnel and stress concentration around top heading sidewalls and temporary invert)</td></tr>
<tr><td colspan="3">Stage 3: In this step the ground elements in the top heading are removed and the initial support elements including shotcrete and rock dowels/bolts are inserted. This leads to a new equilibrium where the initial support elements support the tunnel opening. The shotcrete is modeled using beam elements and the dowels/bolts are modeled using elements that may be loaded in axial loading only. To simulate the early age of the shotcrete its elastic modulus is reduced to one third (1/3) of its final, 28-day design strength. The shotcrete reaches its full strength in the next stage. The initial shotcrete lining capacity is verified in accordance with ACI 318 using Capacity Limit Curves.</td></tr>
<tr><td>
</td><td>
</td><td>Stage 3: Installation and Loading of Initial Support in the Top Heading (Shotcrete and dowels/bolts)

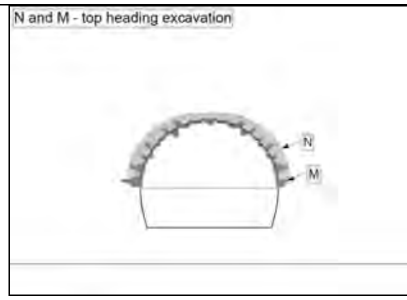
Output Options: Ground Stresses, Deformations of Ground and Linings, Section Forces (N, M) in Shotcrete Lining, Loads in Rock Dowels/Bolts

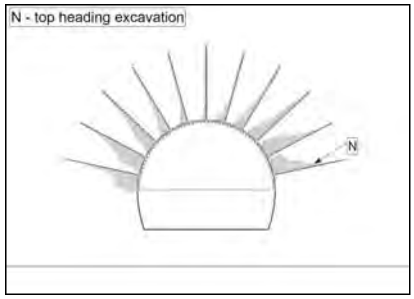
Output Shown:

- Major Principal Ground Stress Sigma 1

- Shotcrete Lining Force Diagram:
- N – Axial Force
- M – Bending Moment

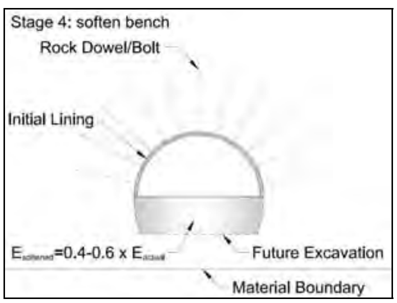
- Dowel/Bolt Forces:</td></tr>
</table>

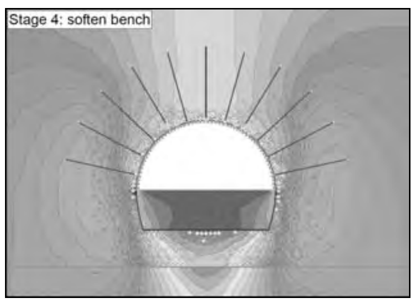




- N – Axial Force

Stage 4: Similar to Stage 2 the tunnel excavation in the bench will cause ground relaxation and ground deflection. This ground relaxation due to the excavation process before support installation is approximated by "softening" the material within the bench; the ground material within the bench is softened by reducing the stiffness of the material by 0.4 – 0.6 times the actual ground modulus (E_{actual}).



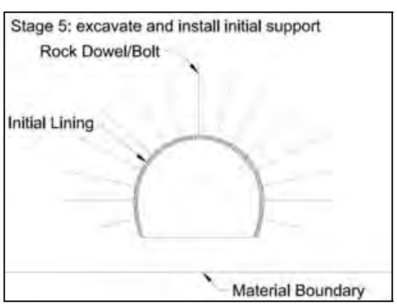


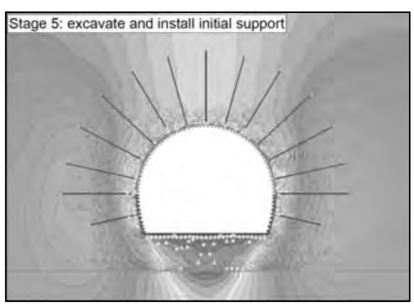
Stage 4: Excavation of the Top Heading

Output Options: Ground Stresses, Deformations of Ground and Linings, Section Forces (N, M) in Shotcrete Lining (Top Heading), Loads in Rock Dowels/Bolts (Top Heading)

Output Shown: Major Principal Ground Stress Sigma 1

Stage 5: Similar to Stage 3 the ground elements in the bench are removed and the initial support elements including shotcrete and rock dowels/bolts are inserted in the bench.





Stage 5: Installation and Loading of Initial Support in the Bench (Shotcrete and dowels/bolts)

Output Options: Ground Stresses, Deformations of Ground and Linings, Section Forces (N, M) in Shotcrete Lining, Loads in Rock Dowels/Bolts

Output Shown:

- Major Principal Ground Stress Sigma 1

- Shotcrete Lining Force Diagram:
  - N – Axial Force
  - M – Bending Moment

- Dowel/Bolt Forces:
  - N – Axial Force

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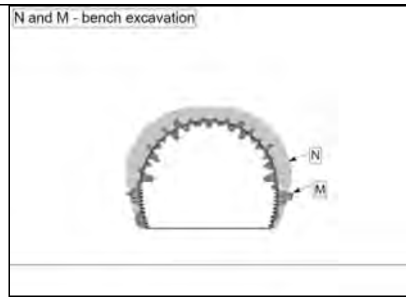
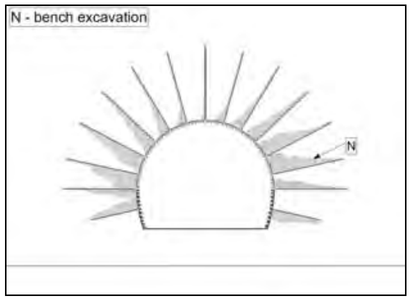
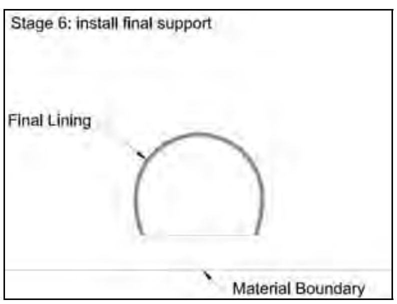
<td>


</td>
<td></td>
</tr>
<tr>
<td colspan="3">Stage 6: This stage involves installation of the concrete final lining beam elements. These are inserted into a stress free state as all ground loads are supported by the initial support elements. A “slip” layer is simulated between the shotcrete and concrete lining beam elements. This layer will allow transfer of radially acting forces only thus representing the waterproofing membrane layer between the linings that is incapable of transferring shear forces. In this example it is assumed that over time, the initial shotcrete lining and rock dowels/bolts deteriorate and all loads need to be supported by the final lining. To simulate this phenomenon, the initial lining elements (i.e. shotcrete and rock dowels/bolts) are removed from the model thus loading the final lining. The final concrete lining capacity is verified in accordance with ACI 318 using Capacity Limit Curves.</td>
</tr>
<tr>
<td>
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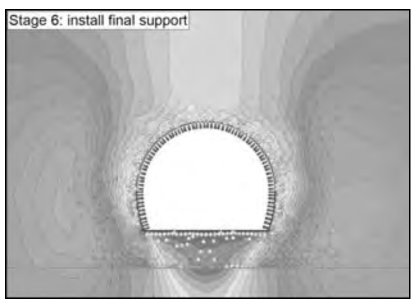
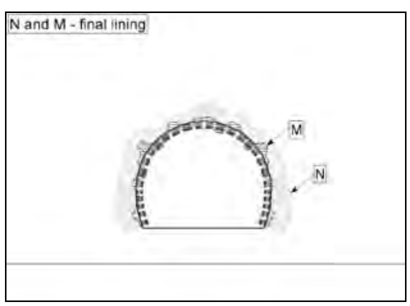
<td>


</td>
<td>Stage 6: Installation and Loading of Final Concrete Lining by Removing all Initial Support elements in the Top Heading and Bench (Shotcrete and dowels/bolts)

Output Options: Ground Stresses, Deformations of Ground and Linings, Section Forces (N, M) in Shotcrete Lining

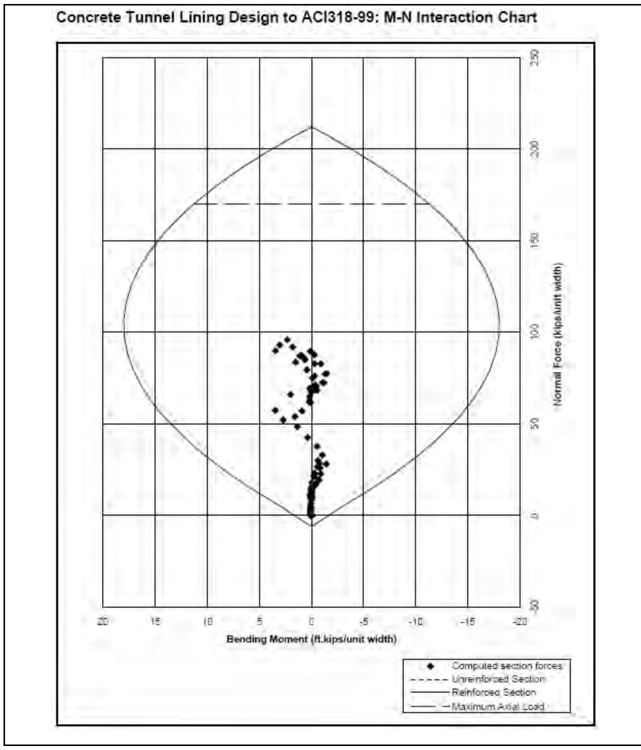
Output Shown:

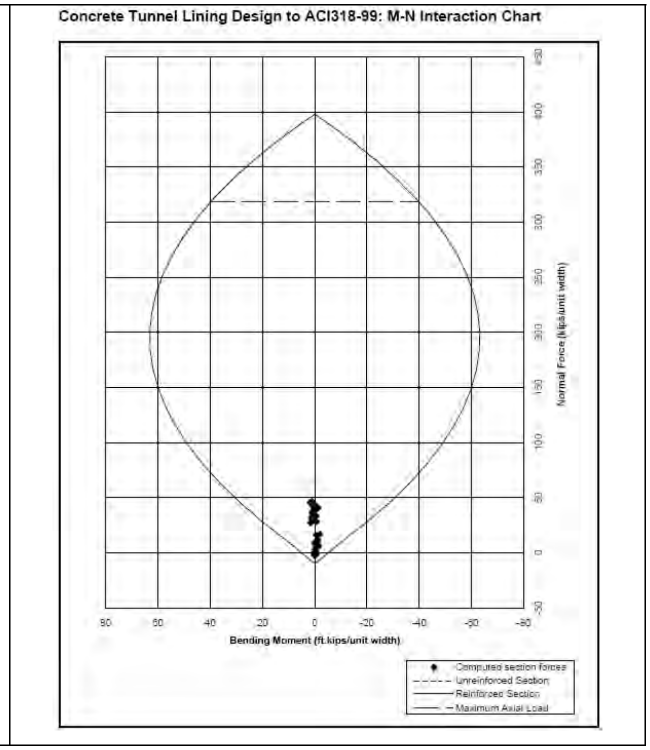
- Major Principal Ground Stress Sigma 1

- Concrete Lining Force Diagram:
- N – Axial Force
- M – Bending Moment

- Dowel/Bolt Forces:
- N – Axial Force</td>
</tr>
</table>

Stage 3: Initial Lining Limit Capacity Curve as per ACI 318-99.	**Stage 6:** Final Lining Limit Capacity Curve as per ACI 318-99.


Concrete Tunnel Lining Design to ACI318-99: M-N Interaction Chart
Bending Moment (ft.kips/unit width)
Normal Force (kips/unit width)
Computed section forces
Unreinforced Section
Reinforced Section
Maximum Axial Load


Concrete Tunnel Lining Design to ACI318-99: M-N Interaction Chart
Bending Moment (ft.kips/unit width)
Normal Force (kips/unit width)
Computed section forces
Unreinforced Section
Reinforced Section
Maximum Axial Load

Appendix G

Precast Segmental Lining Example

APPENDIX G
PRECAST SEGMENTAL LINING DESIGN EXAMPLE

INTRODUCTION

The following design example is intended to illustrate the application of the AASHTO LRFD Specifications to the design of a precast segmental concrete tunnel lining. The design scenario involves a tunnel constructed insoft ground using a tunnel boring machine. The roadway typical section approaching the tunnel is a 4-lane highway with full shoulders a median. The four lanes will be accomodated in two openings, each carrying two lanes of traffic. The tunnel section therefore will be sized to carry two 12'-0" traffic lanes with reduced shoulders on both sides. A 3'-3"" wide walkway for maintenance will be included in the typical section. Emergency egress will be accomodated either at the roadway level using the shoulders provided or through the adjacent bore. Access to the adjacent bore will be gained through cross passages located every 500' along the tunnel alignment. The tunnel will utilize jet fans in a longitudinal ventilation system. The jet fans will be suspended from the tunnel liner.

The analysis of the liner structure will be performed using the beam-spring model described in paragraph 10.??.??.

Figure 10E-1 provides the details of the typical section used in the example.

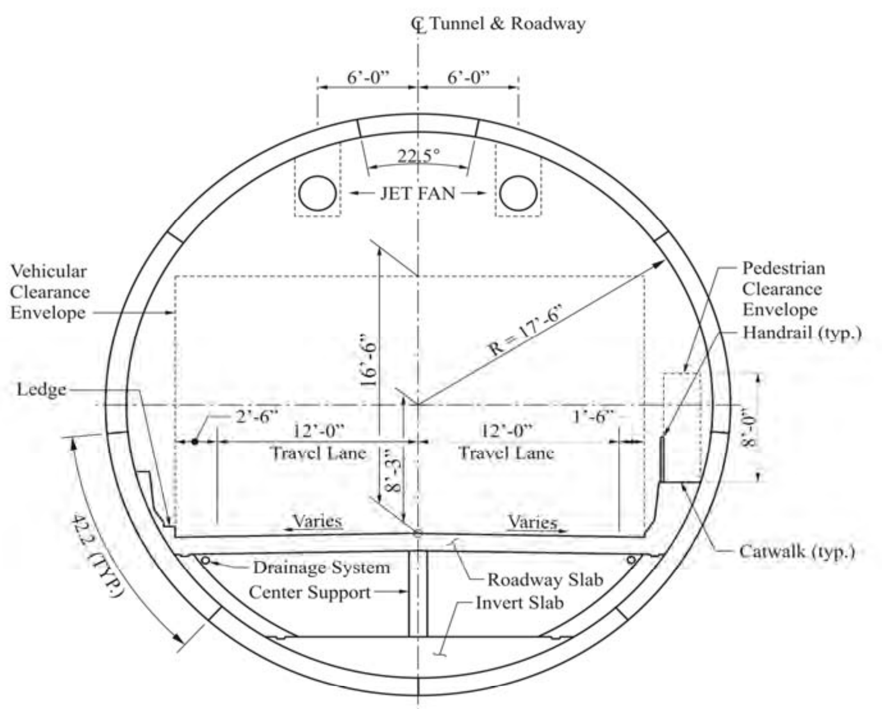


Figure 10E-1
Design Example Typical Section

A typical dimension along the longitudinal axis of the tunnel for the segments is 5'-0". The structural analysis and modeling shown in the following sections of this design example will be based on a five foot length of tunnel. As usch applied loads and spring constants will be multiplied by 5 to account for the fact that the design section is five feet long.

DETERMINE NUMBER OF SEGMENTS

Each segment must be fabricated at a casting yard or precast plant. Once it is fabricated, it must be stripped from the forms and moved to a curing area then to a storage yard. It must be transported form the storage yard to the tunnel site where a stockpile of segments is usually kept. At the tunnel site, it is loaded onto a materials cart that will transport the segment through the tunnel to the tunnel face where it will be erected to form part of ring. The segment must pass through all of the trailing gear associated with a tunnel boring machine on its way to the face. Segments are typically manufactured in advance of the mining operation so that there are suffecient segments on hand to allow the mining operation to proceed without stopping. It is not usual for segments to be damaged during handling and installation, so the number of segments produced is usually more than the total number of segments used in the tunnel. Therefore, segments must be handled several times, stored in at least two seperate locations, transported between the two seperate locations and transported through the tight space found inside a tunnel under construction.

Understanding this process helps to understand how determining the number of segments is a judgmnet decision that should balance minimizing the number of pieces in ring, keeping the length of each segment short enough that it can be practically stored, shipped and handled and making the piece light enough to be handled by the type and size machinery availble inside the tunnel to erect the segments.

Note that it is not unusual for a contractor to suggest a different arrangement of segments than that shown in the contract documents. Most owners allow the contractor to submit changes that are more in line with the means and methods used by a contractor.

For this example, the Inside Diameter = **35.00** ft Segment Length = **5** ft

Assume **8** Segments and a key segment. Key segment subtends: 22.50 degrees
Other segments subtend: 42.188 degrees

Length of non-key segment along inside face of tunnel = 12.885 ft This seems to be a reasonable length.

Number of joints = 9

This example problem will assume that the segments extend along 5 feet of the tunnel length. If **16 in.** is assumed to be the thickness of the segments, then the weight of each segment is calculated as follows:

Length of segment along the centroid of the segment = 13.131 ft

Weight = 13.1309 x 5 x 150 = 9848.2 lbs = 4.92 tons

For a tunnel of this diameter, it should be practical to have equipment large enough to handle these segments at the face of the tunnel.

The example will follow through using 5 feet as the length of the lining along the length of the tunnel. As such input paramters including section properties, spring constants and loads will be based on a 5 foot length of lining being designed.

DETERMINE MODEL INPUT DATA

This section illustrates the development of the data required by most general purpose structural analysis programs. This type of program is required for the beam spring analysis used in this design example. Note that paragraph 4.4 of the AASHTO LRFD specifications describes the acceptable methods of structural analysis. The computer model used in this example for the analysis utilizes a matrix method of analysis which falls into the classical force and displacment category listed in paragraph 4.4. Paragraph 4.5 of the AASHTO LRFD specification describes the mathematical model requirements for analysis. This paragraph states that the model shall include loads, geometry and material behavior of the structure. The input required for these lements will be described below and include the calculation of loads, joint coordinates, the magnitude of the load at each joint, the modulus of elasticity of the concrete and the cross sectional area and moment of inertia of the liner segments.

Paragraph 4.5.1 of the AASHTO LRFD specifications also says that the model shall include the response characteristics of the foundation where appropriate. Since the surrounding ground is an integral part of the structural lining, the response characteristic of the ground is modeled by the springs installed in the model.

CALCULATE JOINT COORDINATES:

Joint coordinates are calculated along the centroid of the lininig segments. In order to calculate the joint coordinates for the initial analysis runs, a lining thickness must be assumed. If the lining thickness changes as a result of the design process, the analysis should be re-run using the parameters associated with the revised lining thickness. This process continues until the lining thickness will support the loads effects from the analysis.

Assume a lining thickness = 16 " Radius to centroid of the lining (r_o) = 18.17 ft

Joint coordinates are claculated as:

Y coordinate = r_o x $\sin\alpha$ X coordinate = r_o x $\cos\alpha$ See Figure 2

In order to keep the model mathematically stable, use a chord length between joint coordinates approximately equal to 1.5 times the thickness of the liner. See paragraph 10.? of the manual.

For a radius r_o = 18.17 ft the angle subtended by chord length of c = $2\sin^{-1}(c/2r_o)$

For chord length = 2.00 ft subtended angle = 6.31 degrees

Number of joints = 360 / 6.31 = 57 say 72 joints at 5 degrees between joints.

72 joints was selected to provide analysis results at the invert, crown and springlines.

Tabulation of Joint Coordinates at the centroid of the lining:

Joint	α (deg)	x (ft)	y (ft)	Joint	α (deg)	x (ft)	y (ft)
1	0	18.17	0.00	37	180	-18.17	0.00
2	5	18.10	1.58	38	185	-18.10	-1.58
3	10	17.89	3.15	39	190	-17.89	-3.15
4	15	17.55	4.70	40	195	-17.55	-4.70
5	20	17.07	6.21	41	200	-17.07	-6.21
6	25	16.46	7.68	42	205	-16.46	-7.68
7	30	15.73	9.08	43	210	-15.73	-9.08
8	35	14.88	10.42	44	215	-14.88	-10.42
9	40	13.92	11.68	45	220	-13.92	-11.68
10	45	12.85	12.85	46	225	-12.85	-12.85
11	50	11.68	13.92	47	230	-11.68	-13.92
12	55	10.42	14.88	48	235	-10.42	-14.88
13	60	9.08	15.73	49	240	-9.08	-15.73
14	65	7.68	16.46	50	245	-7.68	-16.46
15	70	6.21	17.07	51	250	-6.21	-17.07
16	75	4.70	17.55	52	255	-4.70	-17.55
17	80	3.15	17.89	53	260	-3.15	-17.89
18	85	1.58	18.10	54	265	-1.58	-18.10
19	90	0.00	18.17	55	270	0.00	-18.17
20	95	-1.58	18.10	56	275	1.58	-18.10
21	100	-3.15	17.89	57	280	3.15	-17.89
22	105	-4.70	17.55	58	285	4.70	-17.55
23	110	-6.21	17.07	59	290	6.21	-17.07
24	115	-7.68	16.46	60	295	7.68	-16.46
25	120	-9.08	15.73	61	300	9.08	-15.73
26	125	-10.42	14.88	62	305	10.42	-14.88
27	130	-11.68	13.92	63	310	11.68	-13.92
28	135	-12.85	12.85	64	315	12.85	-12.85
29	140	-13.92	11.68	65	320	13.92	-11.68
30	145	-14.88	10.42	66	325	14.88	-10.42
31	150	-15.73	9.08	67	330	15.73	-9.08
32	155	-16.46	7.68	68	335	16.46	-7.68
33	160	-17.07	6.21	69	340	17.07	-6.21
34	165	-17.55	4.70	70	345	17.55	-4.70
35	170	-17.89	3.15	71	350	17.89	-3.15
36	175	-18.10	1.58	72	355	18.10	-1.58

Figure 10E-2 shows the arrangement of joints and members for the computer model.

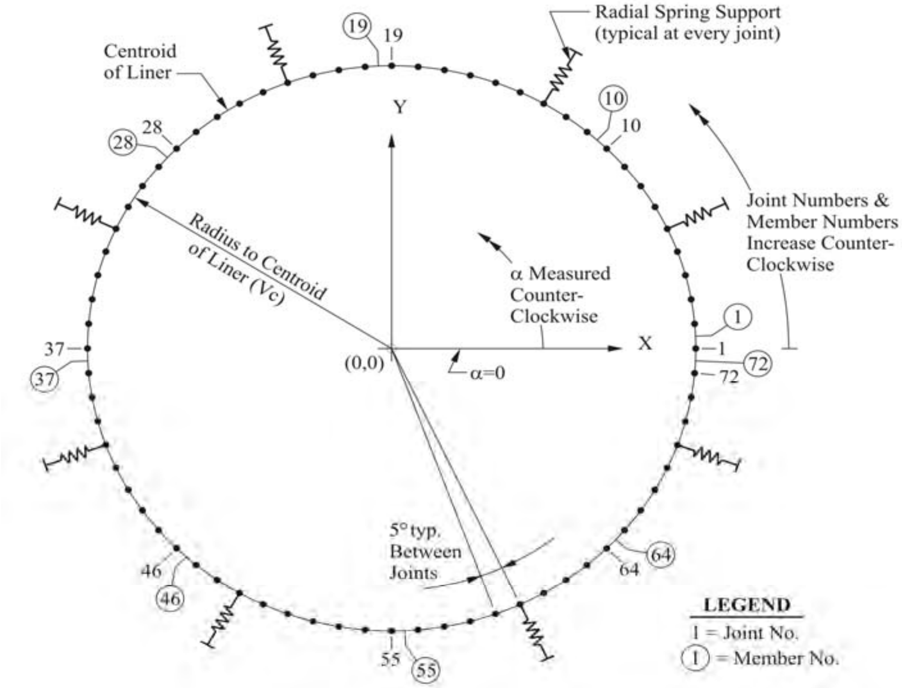


Figure 10E-2
Joints & Members - Computer Model

CALCULATE SPRING CONSTANTS

The subsurface investigation revealed that the tunnel alignment traverses a very stiff clay.

The modulus of subgrade reaction of the clay supplied by the subsurface investigation program is **22 kcf.**

Spring constants can be determined based on tributory projections on the x and y axis of each joint or alternately, if the analysis software being used supports the use of radial springs, then all spring constants will be the same. The following formulas can be used to to determine spring constants.

For orthoganal springs:

Spring constant in the Y direction = $K_s(X_n + X_{n+1})/2$

Where:

$X_n = |(x_n - x_{n+1})|$

$X_{n+1} = |(x_{n+1} - x_{n+2})|$

Spring constant in the X direction = $K_s(Y_n + Y_{n+1})/2$

Where:

$Y_n = |(y_n - y_{n+1})|$

$Y_{n+1} = |(y_{n+1} - y_{n+2})|$

In the above equations:

The coordinates for joint N = (x_n, y_n)

The coordinates for joint N+1 = (x_{n+1}, y_{n+1})

The coordinates for joint N+2 = (x_{n+2}, y_{n+2})

APPENDIX G
PRECAST SEGMENTAL LINING DESIGN EXAMPLE

Figure E10-3 is a graphic representation of the above calculations of orthogonal spring constants.

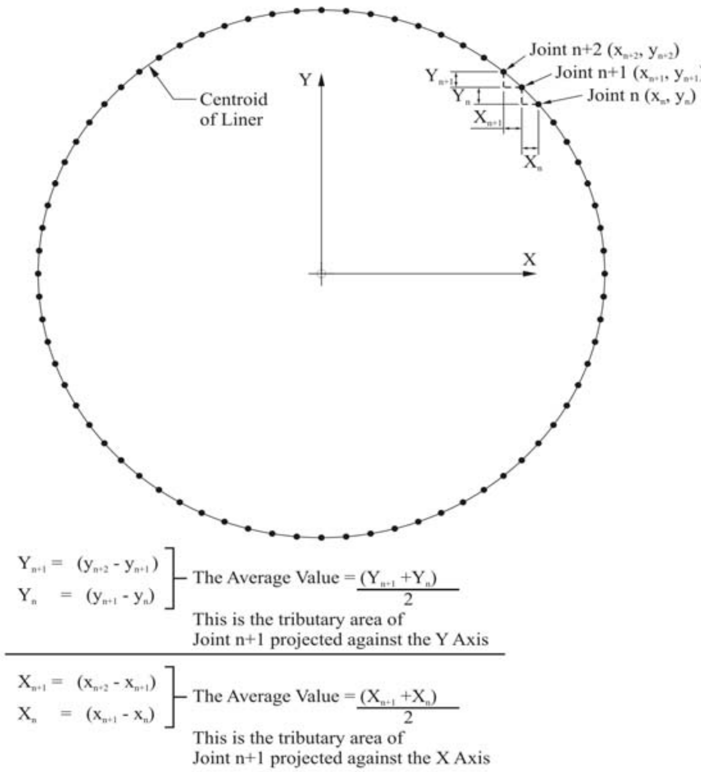


Figure E10-3
Spring Constant Computation

The above computation for orthogonal spring constants uses the coordinates of the joints calculated as input for the model. Since these joints lie along the centroid of the lining and not the outside face where the contact with the surrounding ground occurs, the spring constants calculated using this method should be modified to more closely approximate the resistance provided by the surrounding ground. The modification factor would be the ratio of the outside radius to the radius at the centroid. For this example, the modification factor would be calculated as follows:

Radius at centroid = r_c =	18.17 ft					
Radius at outside face = r_o =	18.83 ft					
Modification factor = r_o / r_c =	18.83	/	18.17	=	1.04	

For radial springs, since a one foot length of tunnel is being modeled, the computation of the tributary area for each joint is the same and is the length of the arc between joints.

This tributary area can be calculated as $\pi r_o \alpha / 180$

Where:

r_o =	Radius to the outside face of the lining	=	18.83 ft
α =	Angle subtended between joints		

It is important to use the outside radius of the tunnel when calculating spring constants since this is the face that is in contact with the surrounding ground.

For this example, the tributary area = 3.14159 x 18.83 x 4 / 180 = 1.31481 ft^2

Clay	E_s (kcf)	Radial Spring Constant (k/ft)	
Gneiss	4000	5259.3	Run analysis using the values shown for
Marble	2500	3287	Gneiss and Marble to bracket the actual
Schist	750	986.11	ground conditions.

When running the computer model, only springs that are in compression are considered active. A spring is in compression if the joint displacements at the location of that spring indocate movement away from the center of the tunnel. Joint displacements toward the center of tunnel indicatemovement away from the ground and the spring at that location should not be active in the model. The analysis is performed with an initial assumption of active and non active springs. The results of the analysis, specifically the joint displacements are examined to determine if the spring assumptions correspond with the output values. If the correspondence does not match, then the asumptions for the springs is adjusted and the analysis re-run. This procedures continues until a solution is obtained where the input values for the springs matches the output values for the joint displacements.

Many computer programs will perform this iterative process automatically. For programs that do not support an automatic adjustment, it is useful to model the springs as orthoganl springs. Modeling the sprngs this way makes it easier to determine if a joint is moving toward or away from the center of the tunnel since each component of the meovement (x and y) can be examined seperately and the direction of the movement ascertained by inspection. When using orthogonal springs, each spring component is adjusted separately.

CALCULATE LINER SECTION PROPERTIES

Segment Thickness = 16 in Segment Length = 5 ft = 60 in

As described in section 10.?? the joints in the liner segments will act to reduce the stiffness of the ring.

Formula for reducing stiffness is as follows: $I_e = I_j + I*(4/n)^2$ (Formula 10 - ??)

where I_e is modified I
n is number of joints (more than 4)
I_j is joint stiffness - conservatively taken as zero

Unmodified Moment of Inertia = $60 \times 16^3 / 12 = 20480$ in^4

Number of Joints = 9

Reduced Moment of Inertia = $20480 \times (4/9)^2 = 4045.4$ in^4

Segment Area = $(16.0 / 12) \times 5 = 6.67$ ft^2

Assume concrete strength = **5000** psi

AASHTO LRFD specification paragraph 5.4.2.4 provides the method for the calculation of the modulus of elasticity.

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'}$$

Where:

K_1 = 1.0
w_c = 145 pcf (AASHTO LRFD specification Table 3.5.1-1)
f_c' = 5000 psi

E_c = 4074281 psi

Poisson's Ratio is given in AASHTO LRFD specification paragraph 5.4.2.5 as 0.2.

CALCULATE LOADS:

The soil load and the hydrostatic pressure are applied to the outside face of the tunnel lining. The structural model is built at the centroid of the lining. Therefore, the surface area to which the rock and hydrostatic loads are applied is larger than the surface area along the centroid the model. The surface area at the location of the centroid is directly proportional to the surface area at the outside face in the ratio of the radius of the outside face to the radius at the centroid. To account for this difference between the modeled area and the actual area and to include the full magnitiude of the applied loads, multiply the rock and hydrostatic loads by the ratio of outside radius to centroidal radius.

Radius to Centroid (r_c) = 18.17 ft
Radius to Outside face (r_o) = 18.83 ft

Multiply Loads Applied to Outside of Tunnel by ro/rc: 18.83 / 18.17 = 1.037

Calculate Hydrostatic Loads:

Hydrostatic head at the tunnel invert = **40** ft = 2.50 ksf

Hydrostatic Load from ground water is applied to the outside of the tunnel.

Value at the invert = 2.50 ksf
Applied amount = 2.50 x 1.037 x 5 = 12.94 ksf Where 5' is the length of the segment

The water pressure magnitude at each joint is calculated based on the distnace of the joint from the invert:

Magnitude of the hydrostatic pressure at joint j = [Value at invert $-|(y_{invert} - y_j)|$ x 62.4] x r_o/r_c x segment length

Where:
y_{invert} = the y coordinate of the joint at the tunnel invert
y_j = the y coordinate of the joint at which the hydrostatic pressure is being calculated

Since the hydrostatic pressure is applied perpendicular to the face of the tunnel, it may be necessary or convenient, dependening on the software being used, to calculate the horizontal and vertical components of the hydrostatic pressure at each joint. This value can be calculated at joint j as follows.

X component of Hydrostatic Pressure at joint j = Magnitude at joint j times $\cos(\alpha_j)$

Y component of Hydrostatic Pressure at joint j = Magnitude at joint j times $\sin(\alpha_j)$

Figure 10E-4 is the hydrostatic pressure loading diagram and also includes a depiction of j and α.

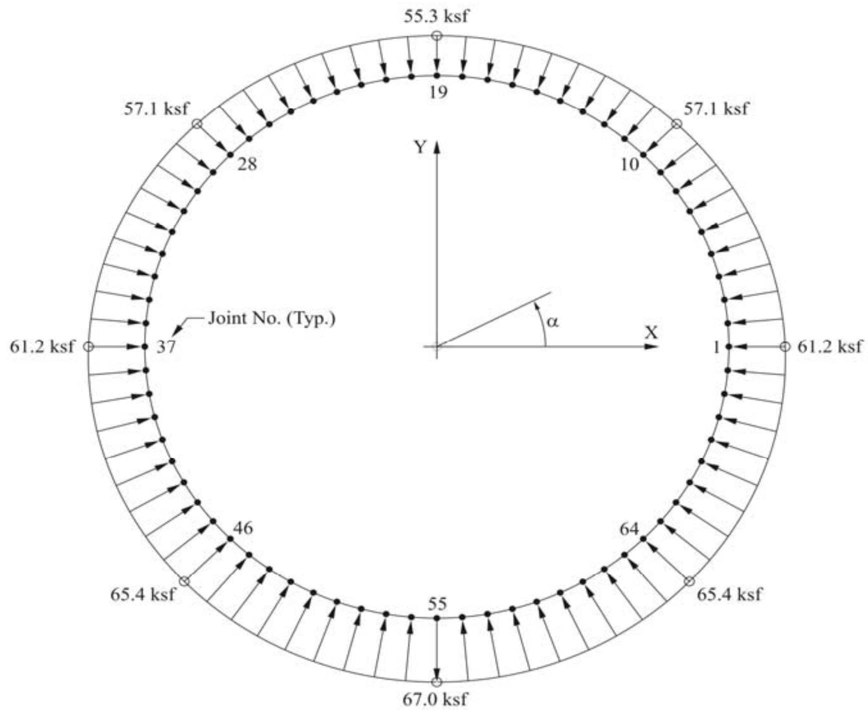


Figure 10E-4
Hydrostatic Pressure Loading Diagram

Tabulation of Hydrostatic Pressure Input Loads

Joint	α	Joint Coordinates x	y	Magnitude	X Component	Y Component
	(deg)	(ft)	(ft)	(ksf)	(ksf)	(ksf)
1	0	18.167	0.000	61.21	-61.21	0.00
2	5	18.098	1.583	60.70	-60.47	-5.29
3	10	17.891	3.155	60.19	-59.28	-10.45
4	15	17.548	4.702	59.69	-57.66	-15.45
5	20	17.071	6.213	59.20	-55.63	-20.25
6	25	16.465	7.678	58.73	-53.22	-24.82
7	30	15.733	9.083	58.27	-50.47	-29.14
8	35	14.881	10.420	57.84	-47.38	-33.18
9	40	13.916	11.677	57.43	-44.00	-36.92
10	45	12.846	12.846	57.06	-40.34	-40.34
11	50	11.677	13.916	56.71	-36.45	-43.44
12	55	10.420	14.881	56.40	-32.35	-46.20
13	60	9.083	15.733	56.12	-28.06	-48.60
14	65	7.678	16.465	55.88	-23.62	-50.65
15	70	6.213	17.071	55.69	-19.05	-52.33
16	75	4.702	17.548	55.53	-14.37	-53.64
17	80	3.155	17.891	55.42	-9.62	-54.58
18	85	1.583	18.098	55.36	-4.82	-55.15
19	90	0.000	18.167	55.33	0.00	-55.33
20	95	-1.583	18.098	55.36	4.82	-55.15
21	100	-3.155	17.891	55.42	9.62	-54.58
22	105	-4.702	17.548	55.53	14.37	-53.64
23	110	-6.213	17.071	55.69	19.05	-52.33

24	115	-7.678	16.465	55.88	23.62	-50.65
25	120	-9.083	15.733	56.12	28.06	-48.60
26	125	-10.420	14.881	56.40	32.35	-46.20
27	130	-11.677	13.916	56.71	36.45	-43.44
28	135	-12.846	12.846	57.06	40.34	-40.34
29	140	-13.916	11.677	57.43	44.00	-36.92
30	145	-14.881	10.420	57.84	47.38	-33.18
31	150	-15.733	9.083	58.27	50.47	-29.14
32	155	-16.465	7.678	58.73	53.22	-24.82
33	160	-17.071	6.213	59.20	55.63	-20.25
34	165	-17.548	4.702	59.69	57.66	-15.45
35	170	-17.891	3.155	60.19	59.28	-10.45
36	175	-18.098	1.583	60.70	60.47	-5.29
37	180	-18.167	0.000	61.21	61.21	0.00
38	185	-18.098	-1.583	61.72	61.49	5.38
39	190	-17.891	-3.155	62.23	61.29	10.81
40	195	-17.548	-4.702	62.73	60.59	16.24
41	200	-17.071	-6.213	63.22	59.41	21.62
42	205	-16.465	-7.678	63.69	57.73	26.92
43	210	-15.733	-9.083	64.15	55.55	32.07
44	215	-14.881	-10.420	64.58	52.90	37.04
45	220	-13.916	-11.677	64.99	49.78	41.77
46	225	-12.846	-12.846	65.37	46.22	46.22
47	230	-11.677	-13.916	65.71	42.24	50.34
48	235	-10.420	-14.881	66.02	37.87	54.08
49	240	-9.083	-15.733	66.30	33.15	57.42
50	245	-7.678	-16.465	66.54	28.12	60.30
51	250	-6.213	-17.071	66.73	22.82	62.71
52	255	-4.702	-17.548	66.89	17.31	64.61
53	260	-3.155	-17.891	67.00	11.63	65.98
54	265	-1.583	-18.098	67.06	5.84	66.81
55	270	0.000	-18.167	67.04	0.00	67.04
56	275	1.583	-18.098	67.06	-5.84	66.81
57	280	3.155	-17.891	67.00	-11.63	65.98
58	285	4.702	-17.548	66.89	-17.31	64.61
59	290	6.213	-17.071	66.73	-22.82	62.71
60	295	7.678	-16.465	66.54	-28.12	60.30
61	300	9.083	-15.733	66.30	-33.15	57.42
62	305	10.420	-14.881	66.02	-37.87	54.08
63	310	11.677	-13.916	65.71	-42.24	50.34
64	315	12.846	-12.846	65.37	-46.22	46.22
65	320	13.916	-11.677	64.99	-49.78	41.77
66	325	14.881	-10.420	64.58	-52.90	37.04
67	330	15.733	-9.083	64.15	-55.55	32.07
68	335	16.465	-7.678	63.69	-57.73	26.92
69	340	17.071	-6.213	63.22	-59.41	21.62
70	345	17.548	-4.702	62.73	-60.59	16.24
71	350	17.891	-3.155	62.23	-61.29	10.81
72	355	18.098	-1.583	61.72	-61.49	5.38

CALCULATE EARTH LOADS

Roof Load = **4.55** ksf

Applied Load = 4.55 x 5 x 1.04 = 23.58 ksf

The horizontal load is given as 1.0 times the vertical load = 23.58 ksf

This load is applied vertically to the lining members. Care should be taken in the input of this load to be sure that it is modeled correctly. The total applied load should be equal to the *Roof Load* times the *Outside Diameter of the Tunnel* times the *Length of the Segment* . Figure 10E-5 shows the loading diagram for this load.

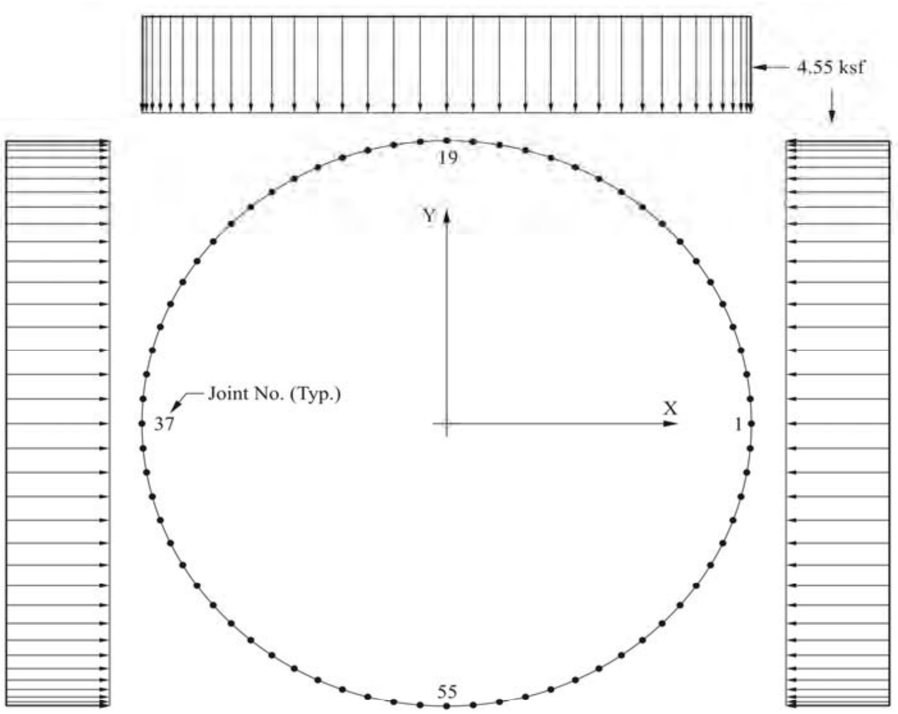


Figure 10E-5
Rock Loading Diagram

APPURTENANCE DEAD LOAD

For this example, the appertenances consist of the jet fans, the drainage system and the roadway slab. The jet fans and the roadway slab are considered as DC loads or the drainage syatem is considered a DW load as given in paragraph 3.3.2 of the AASHTO LRFD specifications.

Jet Fans:

Jet fan load consists of dead load and a dynamic allowance for when the fan starts operation. The dynamic allowance does not need to be treated separately from the dead load. The total anticipated load from the jet fans is 2,000 pounds applied vertically.

Using figure 1, the jet fan load is applied at a location that is 6'-0" on either side of the center line of tunnel. Assume that the supports for the jet fan lie 1'-0" on either side of the centerline of the jet fan. Apply the load as a joint load to the joints that x coordinates are closest to ±2.00 and ±4.00. The load applied at each of these joints will be one half of the jet fan load shown above. For this example, the loads will be applied at joints 15, 16, 22 & 23.

Drainage System:

The drainage system consists of a 6" diameter standard weight steel pipe. Conservatively assume that the pipe is full of water to calculate the dead load.

Pipe weight =	18.97 plf							
Inside Diameter =	6.065 in							
Inside Area =	6.065	x	3.14159	/	2.00	=	9.53	in^2
Weight of water in pipe =	9.53	/	144	x	62.4	=	4.13	plf
Load Applied to Liner =	(18.97	+	4.13)	x	5	=	115.49	pounds

The pipe weight will be applied at the end of the roadway slab. Referring to Figure 1 shows that the intersection of the center of roadway slab and the tunnel wall is located approximately at approximately 9.2 feet below the center of the trunnel. (Assuming a 15" thickness for the roadway slab.) Therefore, in this model the drainage system load can be applied at joints 42 and 68 to approximatel the effect of this load.

Roadway Slab:

The roadway slab consists of three components, the slab, the vertical center support and the barrier/walkway shapes.

Slab: Assume thickness of roadway slab and center support = 15 in

The intersection of the center roadway slab and the tunnel wall is located approximately 9.2 feet below the center of the tunnel. Therefore in this model, the slab load should be applied at joints 42 and 68 to approximate the effect of this load.

The approximate length of the roadway slab would be the distance between joints 42 and 68 = 32.93 ft

Weight of roadway slab = 1.25 x 150 x 32.93 x 5 = 30871 lbs

Since the roadway slab is continuous and supported in the center, assume that 40% of this load is applied at the side walls and 60% is applied at the center support.

Load applied to the side walls =	30871.1	x	0.20	=	6174	lbs				
Load applied to center support =	30871.1	x	0.60	=	18523	lbs				
Weight of center support =	1.25	x	150	x	7.50	x	5	=	7031	lbs
Total load from center support =	18523	+	7031	=	25554	lbs				

Because fo the invert slab, the load from the center support will be distributed over several joints. Apply this load to joints 51 to 59.

LIVE LOAD

Live load from the roadway slab will be the result of the application of the design truck or design tandem coincident with the lane load as per paragraph 3.6.1.2 of the AASHTO LRFD specifications. The minimum spacing of the truck load axles is 14'. The maximum truck axle load is 14'. This means with a 5' long segment, only one truck axial can be on a ring at any time. The maximum truck axle load is 32 kips. The tandem axles are spaced at 4'-0" and weigh 25 kips each. Using the 4-foot spacing, both tandem axles for a total of 50 kips can be on a single ring at a time. Therefore, use the tandem axle arrangement for this example.

The dynamic load allowance (IM) for the limit states used in the tunnel of tunnel linings (i.e., all limit states except fatigue and fracture) is given in AASHTO LRFD specifications in Table 3.6.2.-1 as 33%. The dynamic load allowanc is applied only to the design tandem and not to the lane load. The computation of the live load effect then is as follows:

Live Load Case 1 - One Traffic Lane:

50.000	kips	x	1.33	x	1.20	=	79.8	kip
0.640	klf	x	5.00	x	1.20	=	<u>3.84</u>	kip
Total:							83.64	kip

Where the value of 1.20 is the Multiple Presence Factor (m) given in the AASHTO LRFD specifications in Table 3.6.1.1.2-1

Where the value of 5.00 is the length of a single ring.

Assign 40% of this value to joint 42 and 60% of this value to joints 51 to 59.

Load applied at joint 42 =	33.5	kip
Load applied to each of joints 51 to 59 =	5.6	kip

Live Load Case 2 - Two Traffic Lanes:

50.000	kips	x	1.33	x	1.00	=	66.5	kip
0.640	klf	x	5.00	x	1.00	=	<u>3.2</u>	kip
Total:							69.7	kip

Where the value of 1.00 is the Multiple Presence Factor (m) given in the AASHTO LRFD specifications in Table 3.6.1.1.2-1

Where the value of 5.00 is the length of a single ring.

Assign 40% of this value to joint 42 and 68 and 60% of this value to joints 51 to 59.

Load applied at joints 42 and 68 = 27.9 kip
Load applied to each of joints 51 to 59 = 4.6 kip

LOAD COMBINATIONS

The following table represents the load combinations associated with the limit states to be investigated and the associated load factors. These load cases were entered into the structural analysis software to obtain the results that are presented below.

	Limit State	DC	DW	EV	LL	WA
Strength I	Strength Ia1	1.3	1.5	1.4	1.8	1
	Strength Ib1	0.9	1.5	1.4	1.8	1
	Strength Ic1	1.3	0.65	1.4	1.8	1
	Strength Id1	0.9	0.65	1.4	1.8	1
	Strength Ie1	1.3	1.5	0.9	1.8	1
	Strength If1	0.9	1.5	0.9	1.8	1
	Strength Ig1	1.3	0.65	0.9	1.8	1
	Strength Ih1	0.9	0.65	0.9	1.8	1
	Strength Ia2	1.3	1.5	1.4	1.8	1
	Strength Ib2	0.9	1.5	1.4	1.8	1
	Strength Ic2	1.3	0.65	1.4	1.8	1
	Strength Id2	0.9	0.65	1.4	1.8	1
	Strength Ie2	1.3	1.5	0.9	1.8	1
	Strength If2	0.9	1.5	0.9	1.8	1
	Strength Ig2	1.3	0.65	0.9	1.8	1
Strength II	Strength Ih2	0.9	0.65	0.9	1.8	1
	Strength IIa1	1.3	1.5	1.4	1.4	1
	Strength IIb1	0.9	1.5	1.4	1.4	1
	Strength IIc1	1.3	0.65	1.4	1.4	1
	Strength IId1	0.9	0.65	1.4	1.4	1
	Strength IIe1	1.3	1.5	0.9	1.4	1
	Strength IIf1	0.9	1.5	0.9	1.4	1
	Strength IIg1	1.3	0.65	0.9	1.4	1
	Strength IIh1	0.9	0.65	0.9	1.4	1
	Strength IIa2	1.3	1.5	1.4	1.4	1
	Strength IIb2	0.9	1.5	1.4	1.4	1
	Strength IIc2	1.3	0.65	1.4	1.4	1
	Strength IId2	0.9	0.65	1.4	1.4	1
	Strength IIe2	1.3	1.5	0.9	1.4	1
	Strength IIf2	0.9	1.5	0.9	1.4	1
	Strength IIg2	1.3	0.65	0.9	1.4	1
	Strength IIh2	0.9	0.65	0.9	1.4	1
SERVICE	Service I1	1	1	1	1	1
	Service I2	1	1	1	1	1
	Service II	1	1	1	N/A	1

The designation 1 & 2 in the above table indicates the number of live load lanes.

Design will be performed for the following load cases:

1. Maximum moment (M_{max}) and associate axial load (P).
2. Maximum axial load (P_{max}) and associated moment (M).
3. Maximum shear (V_{max}).

The following are the results:

Schist:	M_{max} =	367.1	ft-kip	P =	524.1	kip	Strength IIa1 Joint 19
	P_{max} =	1496.1	kip	M =	173.2	ft-kip	Strength IIa1 Joint 38
	V_{max} =	93.5	kip				Strength IIa1 Joint 15

Appendix G Segmental Concrete Design Example

DESIGN PROCESS CALCULATIONS

References: AASHTO LRFD Bridge Design Specifications, 3rd Ed., 2004

Data: Segmental lining dimensions:

Segment Length =	5.00	ft
Lining Thickness =	1.33	ft

1. Structure Design Calculations

1.1 Concrete Design Properties:

			AASHTO LRFD Reference
Es =	29000	ksi	5.4.3.2
f_y =	60	ksi	
f'_c =	5	ksi	
γ_c=	145	pcf	Table 3.5.1-1
β_1 =	0.80		5.7.2.2

1.2 Resistance Factors

AASHTO LRFD Reference 5.5.4.2

Flexure =	0.75	(ø) varies to 0.9
Shear =	0.90	
Compr. =	0.75	

1.3 Limits for Reinforcement

AASHTO LRFD Reference 5.7.4.2

For non-prestressed compression members, the maximum area of reinforcement is given by AASHTO LRFD Specification Equation 5.7.4.2-1 as:

$$\frac{A_s}{A_g} \leq 0.08 \Longrightarrow A_s \leq 76.8 \ in^2$$

For non-prestressed compression members, the minimum area of reinforcement is given by AASHTO LRFD Specification Equation 5.7.4.2-3 as:

$$\frac{A_s f_y}{A_g f'_c} \geq 0.135 \Longrightarrow A_s \geq 10.8 \ in^2$$

Where:

A_s = Area of nonprestressed tension steel (in^2)
A_g = Gross area of the concrete section (in^2)
f_y = Specified yiled strength of the reinforcing bars (ksi)
f'_c = Specified compressive strength of the concrete (ksi)

2. Check for One Lining Segment

2.1 Following a Design calculation check will be performed for one lining segment:

2.2 Slenderness Check (LRFD 5.7.4.3):

$k =$	0.65				$\beta_1 =$	0.85
$l_u =$	5.00 ft	=	60 in		ds =	13.75 in
$d =$	1.33 ft	=	16.0 in		d's =	2.25 in
$I =$	4096 in^4				#8 bar dia. =	1.00 in
$r =$	4.62 in					

$$I = \frac{12 \cdot 30^3}{12} \qquad r = \sqrt{\frac{I}{12} \cdot d}$$

From analysis output:

$$k \cdot \frac{lu}{r} = 8.44$$

$$34 - 12\left(\frac{M_1}{M_2}\right) = 23.55$$

where $M_1 =$	58.8 kip-ft	$P_1 =$	2864.9 kip
$M_2 =$	67.5 kip-ft	$P_2 =$	2864.9 kip

Where M_1 and M_2 are smaller and larger end moments

Neglect Slenderness

$k \cdot \frac{l_u}{r}$ is bigger than $34 - 12\left(\frac{M_1}{M_2}\right)$

2.3 Calculate EI (LRFD 5.7.4.3):

$$E_c = 33000 \cdot (\gamma_c)^{1.5} \cdot (f'_c)^{0.5}$$

$E_c =$ 4074.28 ksi

$I_g =$ 4096 in^4

$c =$ 5.5 in

$$I_s = 2\left(\pi \cdot \frac{dia^4}{64} + A_s \cdot c^2\right)$$

$I_s =$ 363.10 in^4

$M_{no} =$ 67.50 kip-ft

$M_2 =$ 67.50 kip-ft

$$\beta_d = \frac{M_{no}}{M_2} = \quad 1.00$$

$$EI = \frac{\left(E_c \cdot \frac{I_g}{5} + E_s \cdot I_s\right)}{(1 + \beta_d)}$$

EI = 6933748.9 kip-in^2

$$EI = \frac{\left(E_c \cdot \frac{I_g}{2.5}\right)}{(1 + \beta_d)}$$

EI= 3337650.74 kip-in^2

2.4 Approximate Method (LRFD 4.5.3.2.2)

The effects of deflection on force effects on beam-columns and arches which meet the provisions of the LRFD specifications and may be approximated by the Moment Magnification method described below.

For steel/concrete composite columns, the Euler buckling load P_e shall be determined as specified in Article 6.9.5.1 of LRFD. For all other cases, Pe shall be taken as:

$$P_e = \frac{\pi^2 \cdot EI}{(k \cdot l_u)^2} \quad \text{(LRFD eq. 4.5.3.2.2b-5)}$$

Where:

l_u = unsupported length of a compression member (in)

k = effective length factor as specified in LRFD Article 4.6.2.5

E = modulus of elasticity (ksi)

I = moment of inertia about axis under consideration (in^4)

P_e = 44992.35 kip

From LRFD section 4.5.3.2.2b:

Moment Magnification:

(The components for sidesway will be neglected. Bracing moment will not include lateral force influence)

The factored moments may be increased to reflect effects of deformations as follows:

LRFD eq. (4.5.3.2.2b-1):

$M_c = \delta_b * M_{2b} + \delta_s \cancel{* M_{2s}}$ $M_u =$ 67.50 kip-ft

$M_c =$ 68.89 kip-ft

where $M_{2b} =$ 67.50 kip-ft

in which:

$$\delta_b = \frac{C_m}{\left(1 - \frac{P_u}{\varphi P_e}\right)} \geq 1$$ LRFD eq. (4.5.3.2.2b-3)

$\delta_b =$ 1.020656

Where:

P_u= factored axial load (kip)

Pe= Euler buckling load (kip)

M_{2b}= moment on compression member due to factored gravity loads that result in no appreciable sideway calculated by conventional first-order elastic frame analysis; always positive (kip-ft)

Φ= resistance factor for axial compression

$P_u =$ 2864.9 kips

For members braced against sidesway and without transverse loads between supports, C_m:

$$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right)$$ LRFD eq. (4.5.3.2.2b-6)

$C_m =$ 0.95

Where:

M_1= smaller end moment

M_2= larger end moment

Factored flexural resistance:

(From LRFD section 5.7.3.2.1)

The factored resistance M_r shall be taken as:

$M_r = \Phi M_n$

Where:

M_n= nominal resistance (kip-in)

Φ = resistance factor

The nominal flexural resistance may be taken as:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) - A'_s \cdot f'_y \cdot \left(d'_s - \frac{a}{2}\right) \quad \text{(LRFD eq. 5.7.3.2.2-1)}$$

Do not consider compression steel for calculating M_n

M_n =	3754.15	kip-in	
M_n =	312.85	kip-ft	
Φ =	0.9		
ΦM_n =	281.56	kip-ft	=> OK
M_r=	281.56	kip-ft	Mr > Mc

Where:

A_s= area of nonprestresses tension reinforcement (in^2)

f_y= specified yield strenght of reinforcing bars (ksi)

d_s= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (in

a= $c\beta_1$; depth of equivalent stress block (in)

β_1= stress block factor specified in Article 5.7.2.2 of LRFD

c= distance from the extreme compression fiber to the neutral axis

$$c = \frac{(A_s \cdot f_y)}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} \quad \text{LRFD eq. (5.7.3.1.2-4)}$$

from which:

A_s=	6.0	in^2	
f_y=	60.0	ksi	
f'_c=	5.0	ksi	
β_1=	0.80		LRFD 5.7.2.2
b=	12.0	in	
c =	8.30	in	
a =	$\beta_1 \cdot c$		
a =	6.64	in	

Create interaction diagram

As_{min} = 10.8 in^2

As_{prov} (total) = 12.00 in^2 **Choose #7 at 6 both faces**

E_s = 29000 ksi

β_1 = 0.85

Y_t = 8 in

$0.85 \cdot f'_c$ = 4.25 ksi

A_g, in^2 = 960 in^2

$A_s = A'_s$ = 6.0 in^2

At zero moment point — From LRFD eq. (5.7.4.5-2):

$$P_o = 0.85 \cdot f'_c \cdot (A_g - A_{st}) + A_{st} \cdot f_y$$

P_o = 4415 kip

ΦP_o = **3311 kip**

Where:

Φ = 0.75

At balance point calculate P_{rb} and M_{rb}

c_b = 8.25 in

a_b = 7.01 in $\quad a_b = \beta_1 \cdot c_b$

f'_s = 63 ksi $\quad f'_s = E_s\left[\left(\frac{0.003}{c}\right)\cdot(c - d')\right]$

f's>fy; set at fy

A_{comp} = 420.75 in^2 $\quad A_{comp} = c \cdot b$

$y' = \frac{a}{2}$ = 3.50625 in

$$\phi P_b = \phi\left[0.85 \cdot f'_c \cdot b \cdot a_b + A'_s \cdot f'_s - A_s \cdot f_y\right]$$

ΦP_b = **1341 kip**

ΦM_b = 9046 kip-in

ΦM_b = **754 kip-ft**

At zero 'axial load' point (conservatively ignore compressive reinforcing)

a = 0.3 in $\qquad a = \dfrac{A_s \cdot f_y}{(0.85 \cdot f'_c \cdot b)}$

ΦM_o = 3674.4 kip-in

ΦM_o = **306 kip-ft**

At intermediate points

a, in	$c = a/\beta_1$	A_{comp}, in^2	f'_s,ksi	f_s,ksi	f_y, ksi	$\Phi M_{n,}$ k-ft	$\Phi P_{n,}$ kips
						306	**0**
2	2.5	120	45	270	60	**439**	**363**
3	3.8	180	59	180	60	**557**	**555**
4	5.0	240	66	135	60	**632**	**746**
5	6.3	300	70	108	60	**688**	**937**
6	7.5	360	73	90	60	**729**	**1128**
7	8.8	420	75	77	60	**754**	**1320**
8	10.0	480	77	67	60	**762**	**1511**
10	12.5	600	79	54	60	**732**	**2005**
11	13.8	660	79	49	60	**693**	**2201**
						0	**3311**
					End 1	**367**	**524**
					End 2	**173**	**1496**
						68	3000

Φ may decrease from 0.90 to 0.75 as "a" increases

Note: from 0.0 to ab. Use 0.75 to be conservative.

Where:

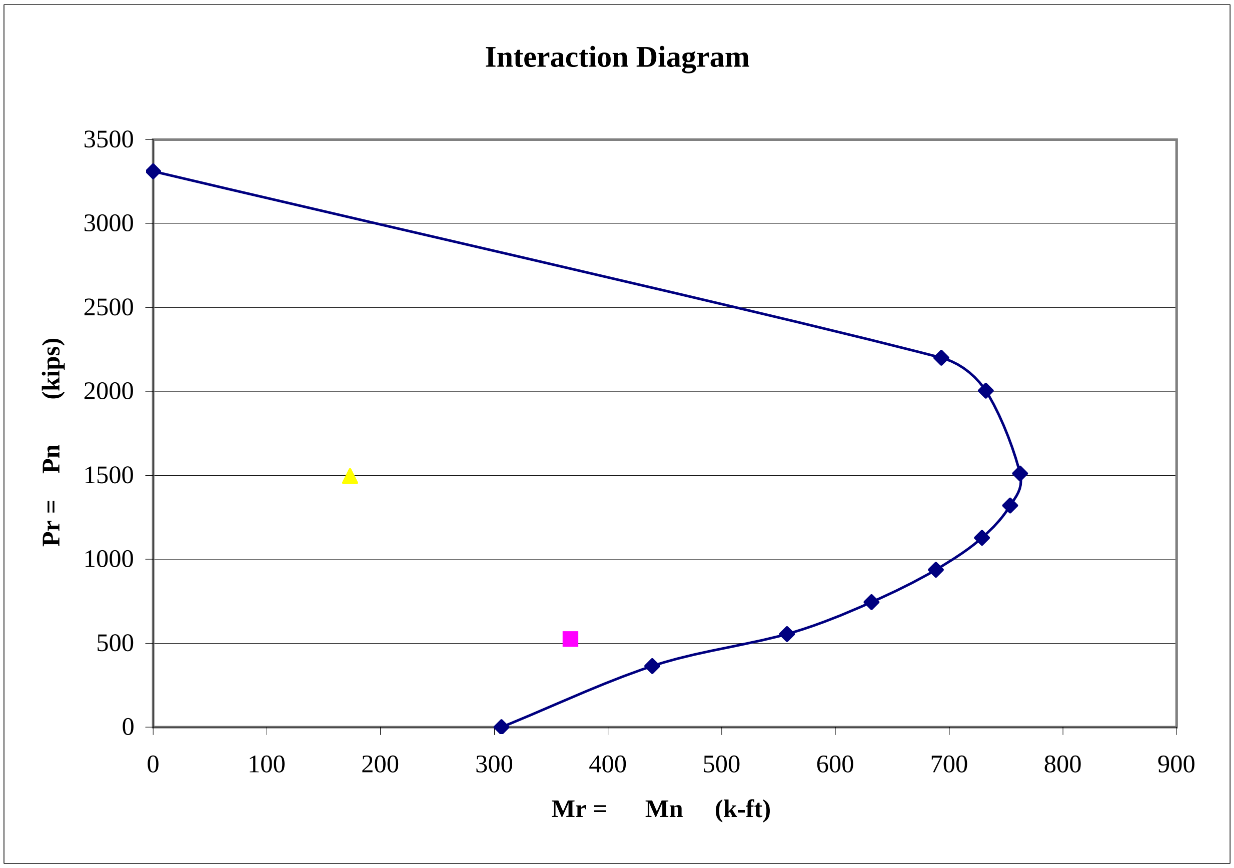
$$A_{comp} = a \cdot 60 \quad \text{in}^2$$

$$f'_s = E_s \cdot \left(\frac{0.003}{c}\right) \cdot (c - A'_s) \quad \text{ksi}$$

$$f_s = E_s \cdot \left(\frac{0.003}{c}\right) \cdot (c - A_s) \quad \text{ksi}$$

$$\phi M_n = \frac{\phi\left[\left(A_{comp} - A'_s\right) \cdot \left(y_t - \frac{a}{2}\right) \cdot 0.85 \cdot f'_c + A_s \cdot f_y\left(d - y_t\right) + A'_s \cdot f'_s\left(y_t - d'\right)\right]}{12} \quad \text{k-ft}$$

$$\phi P_n = \phi\left(A_{comp} - A'_s\right) \cdot 0.85 \cdot f'_c + A'_s \cdot f'_s - A_s f_y \quad \text{kips}$$


Interaction Diagram
3500
3000
2500
2000
1500
1000
500
0
0
100
200
300
400
500
600
700
800
900
Pr = Pn (kips)
Mr = Mn (k-ft)

3. Shear Design (LRFD section 5.8.3.3)

The nominal shear resistance, V_n shall be determined as the lesser of:

LRFD eq. 5.8.3.3-1: $V_n = V_c + V_s$

LRFD eq. 5.8.3.3-2: or $V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v$

NOTE: V_p is not considered

in which:

For slab concrete shear (V_c), refer to LRFD Section 5.14.5:

$$V_c = \left(0.0676\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126 \cdot \sqrt{f'_c}\,bd_e$$ LRFD eq. (5.14.5.3-1)

where $\dfrac{V_u \cdot d_e}{M_u} \leq 1.0$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s}$$ LRFD eq. (5.8.3.3-4)

$\alpha = 90^o$; $\theta = 45^o$ $\quad V_s = \dfrac{A_v \cdot f_y \cdot d_v}{s}$

Where:

A_s= area of reinforcing steel in the design width (in^2)

d_e= effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in)

V_u= shear from factored loads (kip)

M_u= moment from factored loads (kip-in)

b= design width (in)

b_v= effective web width taken as the minimum web width within the depth d_v (in)

d_v=effective shear depth taken as the distance, measured perpendicular to the neutral axis (in)

A_v= area of shear reinforcement within a distance s (in^2)

s= spacing of stirrups (in)

$d_v = 0.9 \cdot d_e$ or $0.72 \cdot h$ (LRFD section 5.8.2.9)

$d_v =$ 12.38 in

$d_e =$ 27.75 $\qquad A_v =$ 0 in^2

$\dfrac{V_u \cdot d_e}{M_u} =$ 6.68 $\qquad s =$ 12 in

Use $\dfrac{V_u \cdot d_e}{M_u} =$ 1.00

Max. shear and associated moment from analysis output:

V_u =	32.8	kip			
M_u =	67.5	kip-ft			
V_c =	80.14	kip			
or V_c =	46.49	kip	**Controls**		
V_s =	0.00	kip			
V_n =	46.49	kip	V_n =	185.63	kip
therefore V_n =	46.49	kip			
Φ= 0.9					
ΦV_n =	41.84	kip	> **Vu OK**		

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

1. **The sequential excavation method is also commonly referred to as the NATM. What does NATM stand for?**

   - New Australian Tunneling Method
   - New Austrian Tunneling Method
   - New Advanced Tunneling Method
   - New American Tunneling Method

2. **What is the purpose of the initial shotcrete lining?**

   - It allows a controlled ground deflection to mobilize the inherent shear strength in the ground
   - Initiate load redistribution
   - None of the above
   - Both A and B

3. **Which is not a model used to summarize the process from the ground investigation to the final definition of the ground support?**

   - Geological Model
   - Geotechnical Model
   - Ground Support Model
   - Tunnel Support Model

4. **What are the two main types of ground?**

   - Rock and hard
   - Rock and soft
   - Sand and Clay
   - Hard and soft

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

5. **What is the purpose of shotcrete?**

   - Provides immediate support after excavation
   - Reduces the potential for relative movement of rock bodies or soil particles
   - It fills small openings, cracks and fissures
   - All of the above

6. **Which is not a type of rock reinforcement?**

   - Rock bolts
   - Rock dowels
   - Rock anchors
   - Rock hangers

7. **Pre-support measures are generally used to:**

   - Limit over-break
   - Reduce ground deflection
   - Increase stand-up time
   - All of the above

8. **Ground improvement measures are primarily aimed at:**

   - Adding a tunnel to the landscape
   - Making the tunnel area safer for the environment
   - To decrease the soils stiffness
   - Modifying the ground matrix to increase shear and compressive strengths

9. True or False: An experienced SEM tunnel designer must monitor and evaluate the recorded data for any distress during the excavation:

   - True
   - False

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

10. **Which is not a type of deformation component that needs to be monitored by instruments installed in the tunnel?**

    - Latitudinal
    - Longitudinal
    - Horizontal
    - Vertical

11. **What must be considered additionally when constructing a tunnel in an urban setting?**

    - Future loads on the tunnel
    - Noise pollution within the tunnel
    - Traffic
    - Weather

12. True or False: Ground freezing is a type of ground improvement measure that should be considered first:

    - True
    - False

13. **Which of the following types of lining can take on any geometric shape?**

    - Precast concrete
    - Steel Plate
    - Foam blocks
    - Cast-in-place

14. True or False: The tunnel lining is frequently less flexible than the ground:

    - True
    - False

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

15. **Historically, which is a basic method used in the design of a structure?**

- Load factor design
- Load and resistance factor design
- Service load or allowable stress design
- All of the above

16. **This type of model is useful in analyzing all geometric shapes.**

- Beam spring model
- Empirical method for hard ground
- SEM model
- Classic model

17. **For circular tunnels in soft ground, the validity of_________ has been highly criticized.**

- Beam spring model
- Three dimensional models
- The resistant model
- Numerical method

18. **For cast-in-place tunnel linings, a water proofing system is typically placed _____ the initial ground support and the cast-in-place concrete lining?**

- Before
- Between
- Diagonal
- Parallel

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

19. **What is a disadvantage of a steel lining plate?**
   - Fire can cause the lining plate to buckle
   - Steel is subject to corrosion
   - Cast-in-place concrete will be needed for fire protection
   - All of the above

20. True or False: Bored tunnels have less of an environmental impact than immersed tunnels:
   - True
   - False

21. **Which is not a main type of immersed tunnel?**
   - Shotcrete
   - Concrete
   - Steel
   - None of the above

22. **Which is the most common method of excavation for immersed tunnels?**
   - Cutter section dredgers
   - Blasting
   - Manual digging
   - Clamshell dredger

23. **Which types of foundations are used in immersed tunnel construction?**
   - Continuous bedding and individual supports
   - Multiple supports and continuous bedding
   - Individual supports and multiple supports
   - Hard rock

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

24. **Backfill usually consists of:**

- Locking fill
- Rock-fill anchor-release bands
- Rock protection blanket
- All of the above

25. True or False: External waterproofing should only be considered for steel tunnels?

- True
- False

26. **Which is not a type of joint used in the construction of immersed tunnels?**

- Open joint
- Immersion joint
- Closure joint
- Earthquake joint

27. **Which type of tunneling is used to avoid disruption of major surface facilities such as railways, airport runways and major highways?**

- Cut-and-cover
- Immersion
- Mixed face tunneling
- Jacked box

28. **The ground freezing method was very effective at providing a stable face over the entire tunnel cross- sectional face area, as shown in Figure 12-13. What was one significant disadvantage with the freeze method?**

- The expansion of water when it freezes caused the overlying track area to heave
- Major deformation of the overlying track area
- Uncontrolled face loss
- None of the above

Design & Construction of Road Tunnels: Part 3 Design and Detailing Quiz

29. **When does face loss occur?**

- When the tunnel boring machine loses its capacity due to ground freeze
- When the ground ahead of the shield moves away from the tunnel as a result of reduction in lateral pressure in the ground at the tunnel face
- When the ground ahead of the shield moves towards the tunnel as a result of reduction in lateral pressure in the ground at the tunnel face
- When the opening of the tunnel has to be built smaller than the rest of the tunnel due to ground conditions

30. **When jacked box tunneling, why would it be necessary to install an anti-drag system?**

- Increase the frictional resistance between the box structure and the surrounding ground
- Reduce the frictional resistance between the box structure and the surrounding ground
- To protect the tunnel from freezing
- All of the above