RECLAMATION

Managing Water in the West

Design Standards No. 3

Water Conveyance Facilities, Fish Facilities, and Roads and Bridges

Chapter 4: Tunnels, Shafts, and Caverns
Phase 4 (Final)

U.S. Department of the Interior
Bureau of Reclamation

April 2014

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Design Standards Signature Sheet

Design Standards No. 3

Water Conveyance Facilities, Fish Facilities, and Roads and Bridges

DS-3(4)-3: Phase 4 (Final)
April 2014

Chapter 4: Tunnels, Shafts, and Caverns

Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 3

Water Conveyance Facilities, Fish Facilities, and Roads and Bridges

Chapter 4: Tunnels, Shafts, and Caverns

DS-3(4)-3:[1] Phase 4 (Final)
April 2014

Chapter 4 – Tunnels, Shafts, and Caverns is an existing chapter within Design Standards No. 3 and was revised to include the addition of:

- Added requirement for Geotechnical Baseline Report
- Added range of design rugosities for lined and unlined tunnels
- Added typical velocities based on tunnel function and lined and unlined tunnels
- Updated safety requirement on use of timber
- Added design guidance on tunnels in squeezing ground
- Expanded horizontal curve requirements
- Redefined terms “deviation” and “tolerances” to match current definitions
- Updated design code references

[1] DS-3(4)-3 refers to Design Standards No. 3, chapter 4, revision 3.

- Expanded seismic requirements for tunnel design
- Added list of typical construction methods and initial support for shafts
- Decreased acceptable station marker spacing
- Expanded and updated estimated average daily advance rates of hard rock tunneling, shaft sinking and raise bore.
- Expanded and updated excavation methods capabilities
- Added description and figure of American lake tap method
- Added criteria for filling and draining tunnels.

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Tunnels, Shafts, and Caverns

4.1 Introduction

4.1.1 Purpose

These design standards outline preferred design practices, methods of analysis, loadings, factors of safety, and structural and equipment types for projects and/or construction designed by the Bureau of Reclamation (Reclamation). All Reclamation design work, whether performed by the Technical Service Center, another Reclamation office, or an architectural/engineering firm, should conform insofar as practicable to these design standards. This chapter covers Reclamation underground structures. References in these design standards to specific products or services do not imply that those are the sole or best products or services.

These design standards are not intended to provide cookbook solutions to complex engineering problems. Strict adherence to a handbook procedure is not a substitute for sound engineering judgment. The designer should be aware of and use state-of-the-art procedures.

4.1.2 Scope

This chapter provides considerations for civil engineering designs. While it does discuss other subjects as they pertain to civil engineering, it does not purport to be standards for those fields.

4.1.3 Deviations from Standards

Whenever a design engineer deviates from these standards, the designer should note the deviation, and the rationale for it should be approved and documented. The designer should initiate action to inform the Water Conveyance Group of the deviations that are required if these design standards no longer reflect current design practices.

4.1.4 Revisions of Standards

Send your comments on this chapter to the Water Conveyance Group. These standards will be revised periodically as appropriate.

4.1.5 Applicability

Apply the guidelines in this chapter to all Reclamation underground structures (tunnels, shafts, and caverns) and their appurtenances.

4.1.6 General

Underground structures are located in rock, soil, or any combination thereof. The inherent peculiarities of in situ materials hinder determining their properties and time-dependent behaviors. The design of underground structures is not as precise as the design of aboveground structures, which will be constructed with manmade materials having known properties. Very often, lacking precise design parameters, underground structural design is based on previous experiences and/or judgment.

Restrictive access in underground construction, maintenance, and operation makes underground work labor, time, and cost intensive. Selecting an underground structure, rather than a surface structure, should be carefully evaluated.

Underground structures are preferred in many situations. These situations include when:

- Construction requires deep cuts or involves unstable slopes
- Effects of vandalism or sabotage of structures above ground have high replacement and service interruption costs
- Protection is required against variations in temperatures
- Reduction of visual and environmental impacts is required

4.2 Design Data and Interpretation

4.2.1 Data Collection

Reclamation Manual, Directives and Standards, FAC 03-03 addresses design data collection. The *Reclamation Manual* can be found at http://www.usbr.gov/recman/. Recommended data to be collected for the preparation of appraisal investigations, feasibility designs and specifications (final) designs are addressed in Chapters 2, 3, and 4, respectively, of Design Data Collection Guidelines (September 2007). These guidelines can be found at http://intra.do.usbr.gov/~tsc/guidance/design/designdata.html.

Site and subsurface investigations are important and costly. The U.S. National Committee on Tunneling Technology recommends that the typical total length of boreholes equal 1.5 times the tunnel length, and a typical geotechnical investigation expenditure of 3 percent of the estimated project cost [1].

The U.S. National Committee on Tunneling Technology recommendation is presented as a general guide for planning a geologic investigation program. Additional expenditures to achieve closer borehole spacing may be warranted where the geology is complex or the tunnel will be excavated beneath a dense urban setting. The design team should weigh the cost of gaining data against the cost of solving possible design and construction problems.

Data collection for design of caverns or deep tunnels should include determination of magnitude and directions of the principal stresses of the in situ rock.

4.2.2 Geotechnical Baseline Report

A Geotechnical Baseline Report (GBR) [2] shall be prepared for all final designs. The GBR interprets anticipated or assumed geotechnical conditions to be encountered underground. These contractual statements (baselines) are to be included in the contract documents. The GBR should be a collaborative effort between the project's geotechnical engineer, geologist, and tunnel designer.

4.3 Functional Design

4.3.1 Portals and Intakes

Tunnel intakes in reservoir designs must consider cleaning debris from the trashracks and isolating the tunnel from the reservoir to allow draining the tunnel for inspection and maintenance. Stoplogs, bulkheads, or gates in the intake may be used to isolate the tunnel from the reservoir. Stoplogs or bulkheads should not be used to isolate tunnel reaches longer than 820 feet (ft) (250 meters [m]). Intakes should be designed to protect fish and other wildlife. Unprotected reaches should be as short as possible. Design trashracks, fish screens, and bypass devices in consultation with the appropriate technical service providers.

The intake design should consider vortex formation, cross circulation, stagnant or eddy zones, and high velocity zones. Submerge intakes of pressure tunnels a minimum of 0.25 ft (76 millimeters [mm]) plus $1.5*H_v$ (H_v = velocity head). Consult with engineers experienced in sedimentation and river hydraulics about any possible sedimentation problems.

Fish have difficulty swimming vertically. Design submerged intakes to draw water in horizontally, not vertically. At times, physical modeling may be necessary.

If future measurement of a tunnel's hydraulic losses may be needed, each water tunnel portal should have a stilling well and piping, or pressure tap, to find water surface height, or pressure. Highway tunnel entrance portals may need to have their surfaces painted to aid the motorist's eyes in adapting to the sudden lack of light upon entering the tunnel. Pedestrian tunnel portals may require canopies to protect pedestrians from falling rocks or drainage.

4.3.2 Transitions

A transition section is normally used in tunnels conveying water at changes in cross section. Transitions reduce flow losses and are usually at the inlet or outlet portals and wherever the tunnel cross section changes shape. Although transitions are beneficial, they pose concrete-forming difficulties and may require hand mining or special blasting. Transitions should be situated in areas that do not require special excavating or blasting whenever possible. Transitions are normally two tunnel diameters (or widths) long.

The design of the stretch of road leading to a highway tunnel should take into consideration the perspective of motorists. Drivers must be able to see into the tunnel to detect obstacles without reducing their speed (see Section 4.3.7, "Lighting").

4.3.3 Tunnel Hydraulics

A complete discussion on hydraulic design and head-loss calculations is covered in reference [3].

Sudden changes in tunnel cross section may create undesirable eddy currents, turbulent flows, or transient wave reflections; therefore, design any convergence and divergence to create minimal hydraulic impacts. Hydraulically, long radius bends are preferable over short radius bends. Transitions, bends, grade changes, bifurcations, and manifolds for dividing flows should achieve a smooth change in velocity, an absence of swirls and vortices, and a minimum head loss.

Consider performing laboratory model testing when designing bifurcating or manifolding tunnels. The term "manifolding" can be either: (1) the splitting of a single tunnel into multiple tunnels or (2) the combining of multiple tunnels into a single tunnel. The term "bifurcating" specifically refers to dividing a single tunnel into two tunnels.

The freeboard for free-flow water tunnels is based on a flow depth equal to 0.82 times the internal tunnel diameter; however, the freeboard should never be less than l.5 ft (0.46 m). The distances between the water surface and electrical cables or other items hung in the crown shall be 0.75 ft (0.23 m) or greater.

The Froude number should not exceed 0.7 for subcritical flow water tunnels, ensuring flow stability at designed discharge capacity. The Froude number is given by the equation:

$$\boldsymbol{F} = \frac{V}{(g * y)^{1/2}}$$

and

$$y = \frac{A}{T}$$

where:

F = Froude number
V = Average flow velocity
g = Acceleration due to gravity
y = Hydraulic depth
A = Cross sectional area of flow
T = Top width of flow area

Concrete-lined water conveyance tunnels are designed with a usual flow velocity of about 10 feet per second (ft/s) (3.0 meters per second [m/s)] and a maximum velocity of 20 ft/s (6.1 m/s). The design velocities of Reclamation's unlined water conveyance tunnels, excavated by drilling and blasting, have ranged from 3.5 ft/s (1.1 m/s) to 5.16 ft/s (1.57 m/s). Design velocities used by Reclamation for unlined tunnels, bored by a tunnel boring machine (TBM), have ranged from 6.78 ft/s (2.07 m/s) to 6.89 ft/s (2.10 m/s). Erodibility of the rock is the primary consideration in the selection of the design velocity for a nonpower unlined tunnel. Erosion potential can be evaluated using reference [4].

A pressure tunnel forms part of a conveyance system (power conduit) that transports water to a powerhouse. Pressure tunnels are either joined to a steel penstock that is then manifolded to the powerhouse turbines, or the tunnel itself may be designed to serve as the penstock. Unlined pressure tunnels, with unpaved inverts, constructed by drilling and blasting, have been designed for a range of maximum flow velocities from 3.5 ft/s (1.1 m/s) to 5.5 ft/s (1.7 m/s) [5]. Unlined pressure tunnels, with paved inverts, excavated by drill and blast techniques, have been designed for a range of maximum flow velocities from 3.5 ft/s m (1.1 m/s) to 9.0 ft/s (2.7 m/s) [5]. An allowable velocity of 10.5 ft/s

(3.20 m/s) was selected for design of an unlined tunnel constructed by a TBM [6]. Both erodibility of the rock and the cost of power are primary considerations in the selection of design velocity for an unlined pressure tunnel.

Design Standard No. 14, Appurtenant Structures for Dams (Spillways and Outlet Works) [7], gives design velocities for Reclamation spillway and outlet works tunnels. Tunnels conveying water containing sand or gravel have high potential for invert erosion [8]. Smooth flowing clear water will not damage good quality concrete, even at high velocities, unless the velocity or direction of velocity is abruptly changed or when surface irregularities or misalignment of joints reduce the pressure in flowing water to its vapor pressure [8].

Cavitation damage may occur when flowing water contacts rough surfaces [9]. For example, at sea level, with a depth of flow of 8.0 foot (2.4 m), and a ¼-inch (6.4 mm) offset, incipient cavitation would occur at an average velocity of approximately 36 ft/s (11 m/s) [10].

Calculate hydraulic head losses for tunnels using the Darcy-Weisbach equation:

$$h_f = f * \frac{L}{D_H} * \frac{V^2}{2 * g}$$

where:

h_f	=	Hydraulic head loss
f	=	Friction factor
L	=	Length of tunnel
D_H	=	Hydraulic diameter ($4*A/WP$)
V	=	Average velocity of flowing water
g	=	Acceleration due to gravity
A	=	Cross sectional flow area
WP	=	Wetted perimeter

When a tunnel is intermittently lined, convergence and expansion losses at the upstream and downstream end, respectively, at each of these lined sections should be considered. For intermittently lined tunnels, calculate convergence and expansion losses as follows:

$$H_L = k * \frac{V_{lined}^2}{2 * g}$$

where:

H_L	=	Convergence or expansion loss
V_{lined}	=	Average flow velocity in lined section of tunnel

k = Coefficient of contraction or coefficient of expansion as appropriate
g = Acceleration due to gravity

The coefficient of contraction can be taken as varying linearly between 0.11, for an angle between the tunnel axis and the upstream taper of the tunnel of 5.75 degrees (°), and 0.78, for a 45° angle (page. 3-20 in reference [11]) where $k = 1/C^2 - 1$.

Coefficients of expansion can be taken from the table below (θ = twice the angle between the axis of the tunnel and the downstream flare of the tunnel in degrees) (table 65 of reference [11]).

Coefficients of expansion

θ	10	20	25	30	35	40	45	50	60
k	.03	.1	.13	.16	.18	.19	.20	.21	.23

Table 4.3.3-1 gives design values of rugosity. Tunnels should be hydraulically designed assuming both rough (old and eroded or tuberculated) surfaces and smooth (new) surfaces. The design rugosities are intended to be used only for tunnels conveying clean (raw) water. They are not to be used for tunnels conveying sewage water. The design rugosities should be used in conjunction with the Colebrook-White equation to determine the Darcy-Weisbach friction factor.

Table 4.3.3-1. Tunnel design rugosities

	Smooth feet (mm)	**Rough feet (mm)**	**Reference**
Cast-in-place concrete lining	0.00020 (0.061)	0.0020 (0.61)	[12]
Precast segmental lining	0.0013 (0.41)	0.0033 (1.0)	[13]
Centrifugally applied, mortar-coated steel liner	0.00015 (0.046)	0.0010 (0.30)	[12]
Unlined drill and blast tunnels:			
Beds of slate diagonal to tunnel alignment		2.00 (600)	[14], [15], [16]
Homogenous rocks or horizontal beds of slate	0.14 (43)	1.00 (300)	
Shotcrete-lined drill and blast tunnels:			
Beds of slate diagonal to tunnel alignment		2.00 (600)	
Homogenous rocks or horizontal beds of slate	0.14 (43)	1.00 (300)	
Unlined bored tunnel	0.0073 (2.2)	0.11 (34)	[17], [17], [18]
Shotcrete-lined bored tunnel		0.054 (16.5)	[18]

The design rugosities shown in table 4.3.3-1 for unlined drill and blast tunnels are based on the design (“A” line) dimensions (see Section 4.4.1.2, “Size,” for the definition of “A” line). Note that for unlined drill and blast tunnels, the design rugosities have been adjusted from the rugosities that were calculated using data from the references indicated in the table. For example, a design rugosity, using hydraulic properties based on the design (“A” line) dimensions, gave the same head loss per unit length as was attained using the actual rugosity and the hydraulic properties based on the actual excavated dimensions. Rugosities of unlined or shotcrete supported drill and blast tunnels are quite variable and must be carefully determined. They are dependent on the rock type, rock quality, strike and angle of dip of bedding planes or foliation, and the blasting plan used. Shotcrete linings for drill and blast tunnels will have rugosities similar to those for unlined tunnels excavated by the drill and blast method.

The rugosities shown in table 4.3.3-1 for unlined bored tunnels are based on the design (“A” line) dimensions. Rock types, and their extents along the tunnel, must be considered when selecting a hydraulic roughness for unlined TBM excavated tunnels because rugosities of different rock types have been shown to vary widely [17] [18]. Most rock types will have rugosities less than 0.052 ft (16 mm). The type of outer cutter used on the TBM can also significantly affect the rugosity [6].

Consider all possible operational conditions, ensuring that all critical hydraulic conditions have been studied. The effort of performing operation and surge studies during design helps prevent costly problems in the future.

Show the hydraulic design parameters on the tunnel profile drawing(s) of the specification. The hydraulic properties should include: the flow rate, the flow area, the average velocity, the hydraulic radius, the rugosity, the invert slope, and the normal depth (for free-flow tunnels).

4.3.4 Control Structures

Control structure equipment such as gates, valves, checks, fish ladders, fish screens, and sedimentation control devices may be required to ensure proper operation. Consult with engineers who are experienced in mechanical equipment design, and engineers experienced in sedimentation and river hydraulic analysis, to ensure that the design of the control structure is acceptable. To assist design, physical hydraulic models may be desirable.

4.3.5 Water Measurements

Operations may necessitate water measurements at or near the tunnel portals. For free-flow tunnels, the flow rate and the water level or depth of flow should be measured. For pressure tunnels, the flow rate and pressure should be measured.

The flow rates of water encountered during construction must be frequently measured and recorded. The quality of the water collected from a tunnel under construction should be checked, and the water should be treated as required prior to being discharged into the surrounding environment. Equipment design should be coordinated with the appropriate specialty organization.

4.3.6 Ventilation

Ventilation includes heating, air-conditioning, filtration, and fire protection and is governed by local codes and other applicable regulations. Current issues of the American Society of Heating, Refrigeration and Air Conditioning Engineer's Handbooks (*HVAC Applications*, *Fundamentals*, *Refrigeration* and *HVAC Systems & Equipment*) provide a complete reference on the subject. The contractor is responsible for ventilation during construction and is governed by *Reclamation Safety and Health Standards* [19].

Designs must provide ventilation for normal inspection and maintenance. Tunnels, shafts, and caverns, not mechanically ventilated, are considered confined spaces and must be treated as such. Where a tunnel, shaft, or cavern is infrequently entered, the cost of permanent mechanical ventilation can be weighed against the cost and inconvenience of temporarily ventilating a confined space.

Consult mechanical engineers who are experienced in designing permanent ventilation systems. The ventilation system is designed to comply with the Office of Safety and Health Administration's (OSHA) Standard 29 Code of Federal Regulations (CFR) 1926.800. When designing these systems, a minimum of 200 cubic feet per minute of fresh outside air, per expected employee, must be delivered to the space, although more may be required based on the application.

4.3.7 Lighting

Adequate lighting during construction must comply with *Reclamation Safety and Health Standards* [23] and is the responsibility of the contractor. Water tunnels do not require permanent lighting. Other structures, such as access adits, do require permanent lighting. The Illuminating Engineering Society of North America gives the lighting requirements for tunnels designed for various purposes [20].

Highway tunnel lighting requirements depend on several factors such as length, geographic orientation, geometric design, traffic volume, vehicular speed, light source, and modes of light application.

Consult electrical engineers who are experienced in the design of lighting systems.

4.3.8 Safety

4.3.8.1 General

Design Standards No. 3 - Water Conveyance Facilities, Fish Facilities, and Roads and Bridges, Chapter 13, "Safety Requirements," addresses functional design safety requirements for underground structures. Contractors are responsible for safety during construction. Area offices are responsible for safety during operation and maintenance, and they should ensure conformance with the applicable Reclamation criteria.

4.3.8.2 Timber

Use of timber supports and/or wood lagging is currently prohibited by Section 23.10.7 of *Reclamation Safety and Health Standards* [19]. A waiver of Section 23.10.7 must be obtained where use of timber supports and/or lagging is anticipated. All timber should be treated with fire retardant unless an approved fire prevention or suppression system is provided. OSHA (Standard No. 1926.800 (m) (12)) requires that all underground structures be constructed of materials with a fire resistance rating of at least 1 hour.

4.4 General Design Considerations

4.4.1 Geometric Design

4.4.1.1 Shape

The most common shapes for Reclamation tunnels are circular, horseshoe, and modified horseshoe. Other shapes, such as elliptical and rectangular, are not common, but may be used if justified. Justification must include consideration of possible increased concrete forming costs. Shafts are typically circular or rectangular in shape.

Underground structures' shapes are usually influenced by the intended function, geology, geotechnical properties of the in situ ground, and the construction method.

Considerations based on intended function include:

- Free-flow water tunnels may be designed using any of the three common shapes. The circular shape is best for pressure flow or power tunnels.

- Access tunnels require a flat invert or floor; therefore, the modified horseshoe shape is most appropriate. Other shapes with curved inverts can be used but require building up the invert to level or providing a walkway.

- Highway and road tunnels require a relatively flat invert or road bed; therefore, the modified horseshoe shape is most appropriate. Long highway tunnels may require ventilation, lighting, and maintenance spaces that may be accommodated better by other shapes, (e.g., circular).

Considerations based on geology and geotechnical properties include:

- A benefit of the circular shape and the horseshoe shape is that loadings from the ground or groundwater place the lining in compression. A circular shape is usually the best shape to resist squeezing and swelling ground loads, as well as external hydrostatic pressures.

- Moments from ground and external hydrostatic loads can result in the allowable tensile capacity of the concrete in the sidewall and the invert of the lining of a modified horseshoe shape being exceeded, requiring concrete reinforcement.

Considerations based on construction method include:

- It is easier to drill, blast, and muck a horseshoe or modified horseshoe shape than a circular shape.

- Tunnel boring machines are limited to circular shapes. Similarly, bored shafts are circular.

- Roadheaders can excavate most shapes; however, it is easier to work on flatter inverts.

- Both unreinforced and reinforced concrete may be used to line all shapes of tunnels. Precast concrete segments are generally used for circular tunnels constructed by TBMs.

Computing hydraulic properties of circular sections and modified horseshoe sections is straightforward. This is not true for the horseshoe cross section (figure 4.4.1.1-1). The equations used to calculate the area and wetted perimeter depend on the location of the water surface relative to the tunnel cross section. The horseshoe section is divided into three zones. The region bounded by the wetted perimeter of the tunnel and a water surface in the invert is defined as Zone 1. The region bounded by the wetted perimeter of the tunnel and a water surface above the invert and below or at the springline is defined as Zone 2. The region bounded by the wetted perimeter of the tunnel and a water surface above the springline is defined as Zone 3. As used above, “invert” is defined as the curved floor of a tunnel [21], and “spring line” is defined as the meeting of the roof arch and the sides of a tunnel [28].

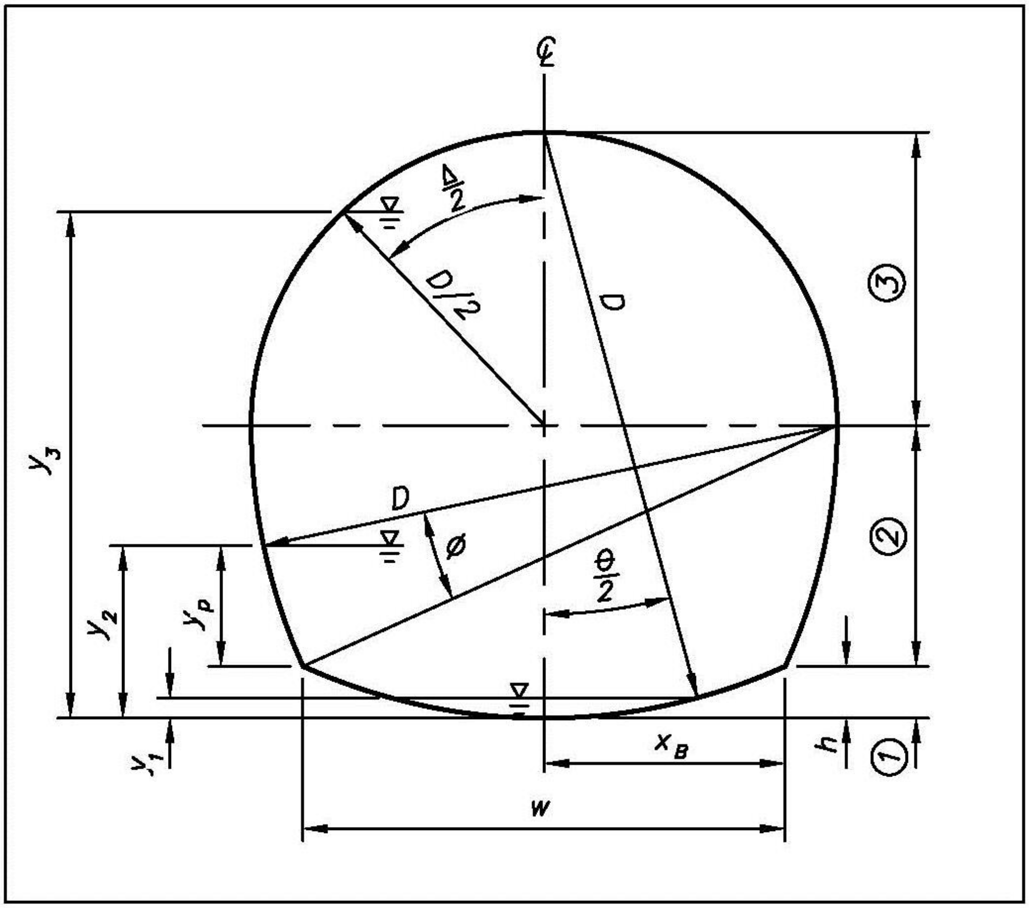

Figure 4.4.1.1-1. Horseshoe section geometry.

In Zone 1 ($0 [y_1 [h$), the first three equations are the same as for a circular cross section:

$$A = \frac{D^2}{2} * (\theta - sin\,\theta)$$

$$WP = \theta * D$$

$$\theta = 2 * \cos^{-1}\left(1 - \frac{y_1}{D}\right)$$

$$h = \left(\frac{3 - \sqrt{7}}{4}\right) * D$$

$$w = 0.822875656 * D$$

where:

y_1	=	The depth of flow when water surface is in Zone 1
D	=	As shown on figure 4.4.1.1-1
A	=	Area of flow
WP	=	Wetted perimeter
Θ	=	Angle as shown on figure 4.4.1.1-1, in radians
h	=	Height as shown on figure 4.4.1.1-1
w	=	Width as shown on figure 4.4.1.1-1

In Zone 2 ($h < y_2 [D/2$):

$$A = 0.049031040 * D^2 + (\emptyset - \sin\emptyset) * D^2 + 2 * y_p * x_B + \left[\left(2 * D * \sin\left(\frac{\emptyset}{2}\right)\right)^2 - {y_p}^2\right]^{0.5} * y_p$$

$$WP = (0.848062080 + 2 * \emptyset) * D$$

$$\emptyset = \sin^{-1}\left(\frac{\sqrt{7}-1}{4}\right) - \sin^{-1}\left(\frac{1}{2} - \frac{y_2}{D}\right)$$

$$x_B = \left(\frac{\sqrt{7}-1}{4}\right) * D$$

$$y_p = y_2 - h$$

where:

y_2	=	The depth of flow when water surface is in Zone 2
$\emptyset$	=	Angle as shown on figure 4.4.1.1-1, in radians
x_B	=	Half-width as shown on figure 4.4.1.1-1
y_p	=	As shown on figure 4.4.1.1-11

In Zone 3 ($D/2 < y_3 [D$):

$$A = 0.829323333 * D^2 - (\Delta - \sin\Delta) * \frac{D^2}{8}$$

$$WP = \left(3.266920487 - \frac{\Delta}{2}\right) * D$$

$$\Delta = 2 * cos^{-1}\left(\frac{2 * y_3}{D} - 1\right)$$

where:

y_3 = The depth of flow when water surface is in Zone 3
Δ = Angle as shown on figure 4.4.1.1-1, in radians

For a full or complete section, these equations simplify to:

$$A = 0.829323333 * D^2$$

$$WP = 3.266920487 * D$$

4.4.1.2 Size

The finished size of an underground structure depends on its functional requirement and, in the case of power tunnels, economic sizing studies. The excavated size is a function of the geology, the construction method and the contractor's preferences for that method, the type and magnitude of the ground loading, the type and size of any needed support, and the thickness of any required lining.

Section 4.3.3, "Tunnel Hydraulics," covers sizing free-flow water conveyance tunnels. Power (pressure) tunnels require economic sizing studies.

The size of highway and road tunnels should meet the American Association of State Highway Transportation Officials (AASHTO) policy on geometric design [22]. In accordance with AASHTO, the minimum vertical clearance over the entire roadway width, including the width of any shoulders, is 16 ft (4.9 m) for freeways and 14 ft (4.3 m) for other highways. The vertical clearance should be increased 6 inches (15.2 centimeters [cm]) to allow for future repaving. Additionally, the minimum clear height should not be less than the maximum height of load that is legal in a particular State.

Dam exploratory and drainage tunnels are a minimum of 5 ft (1.5 m) wide and 7 ft (2.1 m) high (from finished floor to crown). The thickness of the floor depends on the drainage channel invert. The size of access tunnels depends on the largest piece of equipment that must go through it or work within the tunnel.

The ground loading is a function of the width of the excavated opening. In some geology, high in situ stresses or low rock quality may necessitate two or more tunnels where one would otherwise have been sufficient in better ground.

When tunneling through squeezing or swelling ground, excavation dimensions are given special attention and are determined on a case-by-case basis. For guidance on drill and blast mining through squeezing or swelling ground, see reference [23]. For considerations on TBM tunneling through squeezing or swelling ground, see reference [24]. When unexpected squeezing or swelling ground is encountered during construction, the cross section of a tunnel excavated by drilling and blasting can be increased to allow for inward deformation of the ground, unlike the diameter of the bore excavated by a TBM, which is predetermined. If a significant length of a tunnel to be excavated by a TBM is expected to be in squeezing or swelling ground, then selecting a larger diameter bore could be considered.

The maximum width of hand-mined soil tunnels is about 63 ft (19.2 m). Caverns with a 203-ft (62-m) span have been excavated in rock. To date, the largest size tunnel excavated by a soft ground TBM is 57.5 ft (17.5 m). The present maximum size of a tunnel excavated by a rock TBM is 47.2 ft (14.4 m). Tunnels as small as approximately 6.6 ft (2 m) can be excavated in rock or soft ground by TBMs. Small boring machines between 4.5 ft (1.4 m) and 6.5 ft (2.0 m) can excavate limited distances. Pipes as small as 20 inches (50 cm) in diameter can be installed limited distances by the microtunneling method.

Designers should consider allowing, at the contractor’s option, a larger finished tunnel size than is functionally required, but at no additional cost to the Government. Tunnels that are larger than originally specified in the design must meet all the required criteria of the designed tunnel. The designer should consider probable sizes during design and may state the maximum sizes allowed in the specifications.

The size of the excavation must accommodate the thickness of the lining and any needed support system beyond the finished dimensions.

The finished dimension of unlined tunnels should be 8 ft 0 inch (2.4 m) or larger, and lined tunnels should be 7 ft 0 inch (2.1 m) or larger.

The “A” line dimension defines the minimum limits of an excavation, as discussed in Section 4.7.2.2, “Minimum Thickness.” The “B” line is the payline when tunnel excavation is paid for by the cubic yard. The “A” line to “B” line dimension is predicated on providing sufficient space for the expected support system (including lagging where steel sets are specified) and allowing for reasonable overbreak beyond the minimum excavation theoretically required for the installed support system.

The procedure for determining the dimension between the “A” line and “B” line was previously quite elaborate. It was a function of “A” line diameter, tunnel shape, installed support, and excavation method. Basing the dimension on so many factors led designers to believe that it was a precise and important dimension, which it is not. Although based on historical data [25], the predicted amount of overbreak is limited to the specific geology and rock quality of the tunnels studied, and it only provides an estimate for tunnels constructed in other geology and rock quality. Additionally, the contractor typically chooses to excavate the ground to other than the “B” line (payline).

Unit price bidding gave the contractor incentive to submit an unbalanced bid and to install structural steel supports where they were clearly not required to support the ground. Paying for the excavation and support of the tunnel by the linear foot of tunnel (for a given rock classification) and simplifying the method for determining the “B” line alleviate these problems. When per linear foot bidding is used, the “B” line only serves to provide reasonable quantities of excavation for estimating purposes and can otherwise be eliminated from the specifications. Establishment of a per linear foot pay option should be a team decision. A common “B” line throughout a tunnel may also be an option.

Roadheader or drill and blast excavated tunnels with finished diameters between 7 ft and 20 ft (2 m and 6 m) have “A” line to “B” line dimensions of 16 inches (40.6 cm). Tunnels with finished diameters between 20 ft and 50 ft (6 m and 15 m) have “A” line to “B” line dimensions of 18 inches (45.7 cm).

The minimum bore (excavated) diameter for unsupported or rock-bolted TBM tunnels is the “A” line dimension plus 8 inches (200 mm). For rib-supported TBM tunnels, it is the “A” line dimension plus 12 inches (300 mm).

4.4.1.3 Gradient

Reclamation water tunnels usually have very gentle, constant slopes. The minimum tunnel slope is 0.0001, based on drainage. Tunnel reaches less than 100 ft (30 m) long may be flat. The minimum slope for free-flow tunnels is usually based on hydraulics but should never be less than 0.0001. The maximum slope for free-flow tunnels is based on Froude number as explained in Section 4.3.3, “Tunnel Hydraulics.”

Permanent access tunnel grades should never be greater than 10%. Construction adit grades may be up to 18%. Steep grades usually reduce access tunnel length but greatly increase unit construction costs.

The practical limit of grade for muck handling depends on equipment and is 3% for steel rail equipment, 10% for rubber-tired equipment, and 36% for belt conveyors. A slusher (cable-assisted box-type scraper) can be used for grades to about 58% when excavating upslope and with no limitation of grade when excavating downslope. Grades around 100% and steeper are considered self-mucking.

4.4.1.4 Horizontal Curves

Straight tunnel alignment is preferred, but bends are sometimes required. To ensure the movement of locomotives and cars with long wheel bases, along curves with widely spaced rails, without binding, the centerline radius of curves should not be less than 50 ft (15 m). The minimum bend radius of a tunnel conveying water is three times (four times preferred) the finished diameter of the tunnel. The minimum radius for open shield driven tunnels is 600 ft (183 m) (see Chapter 6 of reference [26]). The minimum radius required for tunnels excavated by a TBM varies slightly with the size of the TBM. For example, a minimum radius of 500 ft (152 m) would be required for a 10-ft- (3.0-m-) diameter TBM, while a minimum radius of 600 ft (183 m) would be required for a 20-ft- (6.1-m) diameter TBM. For tunnels that use a continuous belt conveyor system to remove the muck, a minimum radius of 900 ft (274 m) would be required. If it is assumed that a continuous belt conveyor system will be used, then in order to minimize energy consumption, any required curve(s) in the tunnel alignment should be located as far away from the portal (where the tunnel muck will be removed) as practical.

4.4.1.5 Typical Sections

The “A” line is the line (shown on the typical tunnel sections of the specification drawings) within which no unexcavated material of any kind (i.e., timbering, metallic, or other support elements) shall be permitted to remain. However, structural steel rib supports, steel pipe spreaders, and tie rods may extend within the “A” line if allowed by the designer.

If the bidding schedule unit for excavation in tunnel quantities is cubic yards, then the “B” line is the payline for excavation. As such, the measurement for payment for excavation of the tunnel will always be made to the “B” line, regardless of whether the actual extents of the excavation fall inside or outside the “B” line and regardless of the size of steel supports used.

If the bidding schedule unit for excavation in tunnel quantities is linear feet, then the “B” line serves only as the average line of excavation used for estimating material quantities, and the need to show it on the typical tunnel sections of the drawings is at the designer’s discretion.

4.4.1.6 Deviations and Tolerances

For concrete-lined tunnels, the limits of departure (deviation) from established alignment (centerline) should be ±1 inch (25 mm). The limits of departure from finished grade for short tunnels (length not exceeding 1,000 ft [300 m]) should be ± 1/2 inch (12 mm). The limits of departure from finished grade for long tunnels (length exceeding 1,000 ft [300 m]) should be ±1 inch (25 mm). The limits of variation of the specified inside dimension are ±0.5 percent, provided that the thickness of lining at any point is never less than that specified.

The invert of a cast-in-place tunnel lining is typically placed prior to the rest of the lining. For tunnels conveying water, a cast-in-place invert should have a U3 (steel troweled) finish with a corresponding T3 surface irregularity tolerance. For tunnels that do not convey water, a cast-in-place invert should have a U2 (floated) finish with a corresponding T3 surface irregularity tolerance (see chapter VI of Reclamation's *Concrete Manual* [27] for description of U2 and U3 surfaces of concrete).

4.4.2 Ground Cover Requirements

For excavation purposes, the minimum cover over the tunnel is a function of the rock quality. For very good rock quality, the cover should not be less than the largest cross-sectional dimension of the tunnel. Portals of tunnels excavated in rock of lesser quality should have increased cover. An open cut is usually more economical where cover is less than two diameters in rock and three diameters in soil. Caverns should have cover equal to or greater than the largest cross-sectional dimension. Section 4.7.3.3, "Confinement," discusses cover related to pressure tunnels.

4.4.3 Ground Settlement

Subsidence of the ground surface must be considered and avoided when excavating under existing structures. The amount of settlement is influenced by the size and depth of the opening, soil properties, groundwater conditions, method and speed of excavation, and method and timing of support installation. Settlement is increased by large-sized openings, noncohesive soils, permanent lowering of the groundwater table, slow rates of advance of the tunnel, slow or delayed installation of supports, and loss of ground at the heading.

Based on elastic theory, the maximum settlement of the ground surface (in ft) above a shield-driven tunnel in clay is given by the following equation (See Eq. 7.9 in reference [28]):

$$S_{max} = (1 - \nu^2) * (P/E) * (4 * r^2 * h)/(h^2 - r^2)$$

where:

S_{max} = Maximum settlement of the ground surface
ν = Poisson's ratio of soil
P = External pressure on tunnel lining
E = Modulus of elasticity of soil
r = Radius of opening
h = Cover measured from tunnel springline

If a tunnel is driven where there are no structures, then based solely on construction costs, the volume of the settlement trough per unit length of tunnel should not exceed 50 percent of the excavated volume of the tunnel per unit length [29]. The risk of ground caving all the way (daylighting) to the surface should also be considered.

Numerical methods can also be used to predict settlement. Where practical, the use of earth pressure balance or slurry TBMs will minimize settlement.

4.4.4 Portals

Portals are entrance or exit sections of a tunnel that need special design considerations. Tunnel portals are designed to: (1) sustain the existing overburden (frequently of highly weathered material), (2) prevent outside drainage water from entering the tunnel, and (3) create an acceptable visual effect.

Portals need to be supported and to have a canopy during construction (see section 23.10.1 of reference[19]).

For portals excavated in hard and intact rock, a minimum of one tunnel diameter of cover can be considered. Typically, for portals excavated in rock of lesser quality, a minimum of two to three diameters of cover is necessary to develop the ground arch. Economy usually dictates that the cover over the portals be at least 20 ft (6 m) or, preferably, two times the tunnel diameter in rock and 30 ft (9 m) or three times the tunnel diameter in soil. Ten-ft- (3-m-) wide benches that provide for stages of construction for slope stability, and to retain rockfall and raveling material, are required every 30 ft (10 m) vertically at portals. Very wide portals require a transition section between the portal and the tunnel. The transition design should be hydraulically efficient and structurally safe.

4.5 Codes and Manuals for Design

Except as noted, the following codes and manuals will be used for design:

- The design of steel will conform to the American Institute of Steel Construction (AISC) [30], except for the design of structural steel sets for initial support (see section 4.7.1.1.1) and the design of steel liners (see sections 4.7.3.5 and 4.7.3.6).

- The design of reinforced concrete will conform to American Concrete Institute (ACI) 350-06 [31].

- The design of unreinforced concrete will conform to ACI 318-08 [32].

- The design of prestressed concrete will conform to ACI 350-06 [31].

4.6 Design Loads

4.6.1 General

Ground and groundwater loads acting on initial support and final lining shall be based on the "B" line.

Internal hydraulic loads acting on linings shall be based on:

- The finished face for unreinforced linings
- The "A" line for reinforced linings
- The location of the impermeable membrane for such linings

4.6.2 Ground Classification Systems

Several ground classification systems are available to predict the support loading or the required total support system. Ground classification systems for rock are different from those for soil. Some rock classification systems used by Reclamation are: Rock Quality Designation (RQD) [33], Rock Mass Rating (RMR) [34], Rock Structure Rating (RSR) [35], Q [36], and Terzaghi's [23] classification systems. For soft ground tunneling, the Terzaghi soil classification system [37] is used by Reclamation. Employ at least two classification systems where possible. Compare their results to any analytical or numerical analysis used.

4.6.3 Rock Tunnels

Rock tunnel initial support design falls into one of the two following categories: (1) the initial support resists the total load, or (2) the initial support and rock together share the total rock load as a composite system. The rock loading may be due to loosening (gravitational), swelling, or squeezing loads.

4.6.4 Soil Tunnels

Shallow tunnels (cover between 1.0 (B + H_t) to 1.5 (B + H_t)) in cohesionless soil are designed such that the supports resist the load from the full depth of cover:

$$P_v = \gamma * H_d$$

where:

B = Excavated width of the tunnel
H_t = Excavated height of the tunnel (except H_t = 0 for a circular tunnel)

P_v = Vertical pressure from overburden
γ = Average unit weight of soil above the roof of the tunnel
H_d = Height of soil above the roof of the tunnel

For deep tunnels (cover more than 1.5 (B + H_t)) in cohesionless soils, the value of H_d (see table 7-2, reference [29]) in the above equation is taken as the lesser of the actual cover or the effective height (between 0.3 (B + H_t) and 2.0 (B + H_t)) which is dependent on the characteristics of the soil, whether the tunnel is above or below the groundwater table, and the dimensions and shape of the tunnel.

Pressures increases from the swelling of clay have been estimated at 50,000 pounds force per square foot [23]. Tests should be performed on such soils to obtain a competent value.

For cohesive soils, the stability number (*N*) is defined as:

$$N = (P_z - P_a)/S_u$$

where:

P_z = Overburden pressure at tunnel centerline
P_a = Compressed air pressure above atmospheric pressure
S_u = Undrained shear strength of the cohesive soil (usually one-half of unconfined compressive strength of the clay)

If N is less than 4, no unusual tunneling difficulties would be expected; if N is greater than 5, hand mining would become difficult; and if N is greater than 7, steering a shield would become difficult (see chapter 5, reference [38]), and compressed air tunneling methods may be required. Compressed air tunneling is expensive, time consuming, and hazardous. In such cases, consider using special ground stabilization techniques such as freezing or jet grouting.

Use of an earth pressure-balance (EPB) TBM or a slurry TBM should be considered for either cohesive or cohesionless soil types, respectively.

4.6.5 Seismic Considerations

For tunnels in rock or reasonably stiff soils, the tunnels are constrained by and move with the surrounding ground. Tunnels are not subject to significant vibratory amplification as are surface structures. Tunnels have generally performed better than above-ground structures during seismic events. For tunnels in rock, no damage was found where the peak acceleration at the surface was less than 0.19 g [39] (where “g” is the acceleration due to gravity), and no further seismic analysis is typically required.

Design of typical water conveyance tunnels, shafts, and caverns should be based on an earthquake with a 10-percent probability of exceedance in an exposure time of 50 years (approximately a 475-year frequency). Tunnels associated with dams or highways may require higher return periods because of the importance of the structure.

The peak ground acceleration can be calculated using the "Java Ground Motion Parameter Calculator" application found on the U.S. Geological Survey Web site: <http://earthquake.usgs.gov/hazards/designmaps/javacalc.php>. For tunnels in soil that are less than 100 ft deep, site adjustment of the peak ground acceleration, similar to those found in AASHTO Load and Resistance Factor Design (LRFD) [40], should be made. Other seismically related data, such as peak ground particle velocity, effective wave propagation velocity (for shear [S-wave], compressional [P-wave], and Rayleigh waves), and an accelogram, should be included in the design data.

The analysis of underground structures should be based on the displacement/ deformation of the lining imposed by the ground or interaction of the ground and structure. Tunnels shall be analyzed for axial (longitudinal), curvature, and ovaling deformations.

Where the structure is flexible relative to the surrounding ground, such as tunnels in rock or stiff soil, the axial and curvature strain can be analyzed by assuming the structure experiences the same strains as the surrounding ground. The most critical angle between the axis of the tunnel alignment and the direction of wave propagation (angle of incidence) should be assumed. Table 4.6.5-1 gives the maximum values of strain [41] (for the critical angle of incidence) for the various wave types.

Table 4.6.5-1. Maximum strain value for various types of waves

Wave type	Longitudinal strain (axial)	Curvature
P-wave	V_P/C_P	$0.385*A_P/C_P^2$
S-wave	$V_S/(2*C_S)$	A_s/C_S^2
Rayleigh wave (compressional component)	V_{RP}/C_R	$0.385*A_{RP}/C_R^2$
Rayleigh wave (shear component)		A_{RS}/C_R^2

Notes:
V_P, V_S, V_{RP} are the peak particle velocities for the P-wave, S-wave, and the compressional component of the Rayleigh wave, respectively.
C_P, C_S, C_{RP} are the effective propagation velocities for the P-wave, S-wave, and the compressional component of the Rayleigh wave, respectively.
A_P, A_S, C_{RS} are the effective propagation velocities for the P-wave, S-wave, and the shear component of the Rayleigh wave, respectively.

The allowable combined axial (longitudinal) and curvature strain of a linear tunnel or underground structure should be less than a compressive strain of 0.002. Tension cracks are acceptable when they do not adversely affect the tunnel's function or shorten the life of the structure.

Where a linear structure is located in soft soil, the structure will be relatively stiffer (in the longitudinal direction) than the surrounding soil, and ground deformations will be resisted by the lining. When a linear structure is located in soft soil, making the assumption that the structure will have strains equal to those of the surrounding ground, will produce unreasonable results. The internal moments, shears, and axial forces for this case can be solved using numerical methods [42], [43]. A simplified procedure [44] based on tunnel ground interaction is recommended as a check of the numerical modeling.

Shear waves cause deformation of circular linings in the transverse direction (ovaling). Analytical procedures by numerical methods can be used to determine the diametric strain. The maximum allowable diametric strain, where the lining has little stiffness relative to the ground, is given by the following equation:

$$\frac{\Delta D}{D} = \pm 2 * \gamma * (1 - \mu_m)$$

where:

ΔD = Maximum diametral deformation of the lining
D = Diameter of the tunnel
γ = Maximum free field shear strain (radians)
μ_m = Poisson's ratio of the ground

and

$$\gamma = \frac{V_S}{C_S}$$

where:

V_S = Peak particle velocity
C_S = Effective shear wave propagation velocity

When the lining has a stiffness equal to or greater than the ground, the maximum allowable diametric strain is given by the following equation:

$$\frac{\Delta D}{D} = \pm \frac{\gamma}{2}$$

The numerical solution for diametric strain can be checked by the following equation [44]:

$$\varepsilon_{total} = \frac{V_s}{C_s} * \left[3 * (1 - \mu_m) * \frac{t}{R} + \frac{1}{2} * \frac{R}{t} * \frac{E_m * (1 - \mu_L^2)}{E_L * (1 + \mu_m)}\right]$$

where:

ε_{total}	=	Maximum combined bending strain and thrust compression strain
μ_m	=	Poisson's ratio of the ground (medium)
μ_L	=	Poisson's ratio of lining material
t	=	Thickness of tunnel lining
R	=	Radius of tunnel
E_m	=	Modulus of elasticity of the ground (medium)
E_L	=	Modulus of elasticity of the tunnel lining

The potential for seismically related damage increases where: (1) an underground structure passes through active fault zones, (2) a shallow tunnel is located in rugged terrain or landslide areas, (3) a deep tunnel is in a highly pressurized rock zone, or (4) ground or structure characteristics dramatically change, such as at the portals. These situations should be carefully evaluated regardless of the magnitude of the peak ground acceleration.

Seismic events can create ground shaking and ground failure. Types of ground failure are faulting, landslides, liquefaction, and tectonic uplift and subsidence. While attempts may be made to design for large fault movements, it is highly recommended that tunnel alignments through active faults be avoided where possible. While preventive measures against large-scale ground failure are impractical, some preventive measures are possible against small-scale ground failure. It is usually possible to design to accommodate ground shaking.

Protection against liquefaction should be provided for underground structures and portals in soils vulnerable to severe strength loss as a result of ground shaking. Protective measures against liquefaction are extremely sensitive to site conditions. For controlling liquefaction, investigate densification, removal and replacement of existing soil with clay-rich soil, or installation of drainage systems. In near-surface underground structures, existing soil can be made denser by dynamic compaction. For deep underground structures, consider applying compaction grouting or chemical grouting to control liquefaction.

4.6.6 Shafts

Shaft design must include provisions for stability of the opening against both horizontal and vertical movements. Selecting the proper construction method, initial support and final lining for a shaft is dependent on the ground characteristics, the groundwater, as well as the shape, size and depth of the shaft. Vertical stability of the lining may be achieved by supporting it on pillars or foundation blocks, by hanging it from a collar (ring) beam on the surface, or keying or anchoring it to the ground. Construction methods and initial support of shafts excavated in soft ground include:

- Soldier piles and lagging or steel plates
- Liner plate with horizontal steel sets
- Horizontal steel sets and vertical lagging
- Sheet piles with horizontal steel sets
- Corrugated metal or steel casing used with drilled shafts
- Secant piles
- Cutter soil mixing
- Slurry walls
- Freezing
- Cast in place concrete caisson
- Precast segmental concrete caisson

Initial support of shafts excavated in hard rock includes:

- Steel sets with liner plate
- Steel sets with lagging
- Rock bolts with wire mesh
- Shotcrete
- Cassions can also be used in hard rock conditions

4.6.7 Caverns

Exploration adits are usually necessary for conducting geotechnical investigations to determine the magnitudes and orientations of the principal stresses, and to explore for anomalies that are not otherwise detectable. For deep exploration, where construction of an adit is not practical, rock stress can be estimated using a method of hydraulic fracturing or strain gages down a borehole. Thorough exploration is critical for cavern design, as undetected anomalies can negate an otherwise good design. Designers never know for sure what rock is present until it is encountered. Careful geologic evaluation during excavation is important.

Caverns should be located in favorable geology. If the magnitudes of the horizontal in situ principal stresses differ significantly, the cavern should be oriented so that its longest dimension is parallel to the direction of the larger horizontal in situ principal stress, if possible. Orienting the longest cavern dimension parallel to the direction of the larger horizontal principal stress minimizes the loading on the supports. The designer must weigh the problems such an orientation alleviates against higher design and construction costs. Three-dimensional analyses, such as analytical, numerical, photoelastic, or model simulation, are normally used in cavern design.

4.6.8 Intersections

Analyses of intersections are conducive to numerical design methods.
Finite element analyses are useful for geometrically intricate intersections. Alternatively, although more expensive and time consuming than numerical methods, model analyses and photoelastic analyses (currently less common) could be used. Other methods may be used where appropriate.

In finite element analyses, the model dimensions should not be less than three times the largest cross-sectional dimension of the opening in any direction from the intersection.

4.6.9 Adjacent Tunnels

Excavation of an underground opening may result in increased stresses (expressed by stress concentration factors) around existing adjacent openings. It has been found [45] that if a pillar between two adjacent openings is equal to or greater than the sum of the diameters of the adjacent openings, there are usually no increases in stresses or deformation compared to those of a single opening.

4.7 Support Systems

The selection and design of the support and lining are dependent on the in situ pressures and stresses, geological and engineering characteristics of rock or soil, standup time, and selected excavation methods. For competent rock, no support is required from a structural point of view. Rock reinforcement, structural steel, shotcrete, steel liner plates, and precast concrete segments are commonly used for initial support.

Historically, the final lining for Reclamation water conveyance tunnels has usually been a cast-in-place concrete lining, although precast concrete segmental linings have also been used in tunnels constructed by tunnel boring machines. Precast concrete segments can sometimes serve as both initial support and final lining (one-pass system), and eliminate the need for an additional cast-in-place lining (two-pass system), which can result in significant cost savings. Steel linings may be required in the reaches of a power tunnel near the portals and in pressurized portions of outlet works tunnels downstream of the gate chamber near the dam axis. Lining may not be required in competent rock.

4.7.1 Initial Support and Ground Control

4.7.1.1 Structural Steel Supports

4.7.1.1.1 Steel

The design of structural steel used as initial support shall be based on American Society for Testing of Materials (ASTM) A 36 steel. A 992 steel meeting the requirements of A 36 steel may be substituted when A 36 is not practically available; however, the design will still be based on A 36 steel. The allowable section is the wide flange section (AISC W shape). The allowable axial and bending stresses for initial support structural steel sections are 0.75 F_y, where F_y is the yield strength of the steel. The allowable shear stress for initial support steel sections is 0.4 F_y.

4.7.1.1.2 Spacing

The standard spacing of structural steel supports for reaches of tunnels with light rock loads is 5 ft (1.5 m) to 6 ft (1.8 m), with moderate rock loads is 4 ft (1.2 m), and with heavy rock loads is 2 ft (0.6 m) to 3 ft (0.9 m). The minimum spacing for structural steel supports in all tunnels is 1 ft (300 mm) between flanges. The usual practice is to pick a spacing and design a shape that resists the loads. Structural steel support spacing is usually a function of the rock quality and associated rock loads, the steel shape size, and the maximum size of the steel shape that can be bent to the required radius. Both the cost of the steel supports and the cost of the lagging between these supports must be considered to determine the most economical spacing.

4.7.1.1.3 Blocking and Wedging

Blocking and wedging is a method of holding sets in place. Wood blocks, with wedges tightly driven between the blocks and the rock, transfer the rock loadings to the structural steel supports (see figures 4.7.1.1.6-1 and 4.7.1.1.6-23). Closer spacing of the blocking increases the capacity of a rib to resist rock loadings. Blocking shall be placed at the springline and on both sides of the butt joint at the crown. Above springline, the center to center spacing of blocking shall not be greater than 2 ft (600 mm). The maximum center to center spacing of blocking shall be 1 ft (300 mm) before using jump sets. Blocking is not required with expanded ribs, provided that the ribs contact the rock at least every 2 ft (600 mm).

4.7.1.1.4 Bend Radius of Cold Formed Steel Ribs

The minimum radius that 4-inch- and 6-inch-deep structural steel shapes can be cold bent is eight times the depth of the section. The minimum radius that larger structural steel shapes can be cold bent varies from 10 to 14 times the depth of the section.

4.7.1.1.5 Spreaders and Tie Rods

Spreaders and tie rods are used to stabilize the structural steel support about its minor axis. Their spacing should be determined by design. The maximum spacing is 60 times the radius of gyration about the minor axis 23].

4.7.1.1.6 Lagging and Foot Blocks

Lagging (figure 4.7.1.1.6-1) consists of timber (where approved), steel, or precast concrete members that span between and transfer intermediate rock loadings to the adjacent structural steel supports. The area covered by timber lagging is limited to no more than 50% of an area bounded by adjacent steel sets and any 3-ft length measured along the outside of the sets. Where more than 50% of the area defined above needs lagging, steel or precast concrete lagging must be used for that portion of the lagging. Foot blocks (figure 4.7.1.1.6-2) distribute the loadings from the structural supports to the floor of the tunnel.

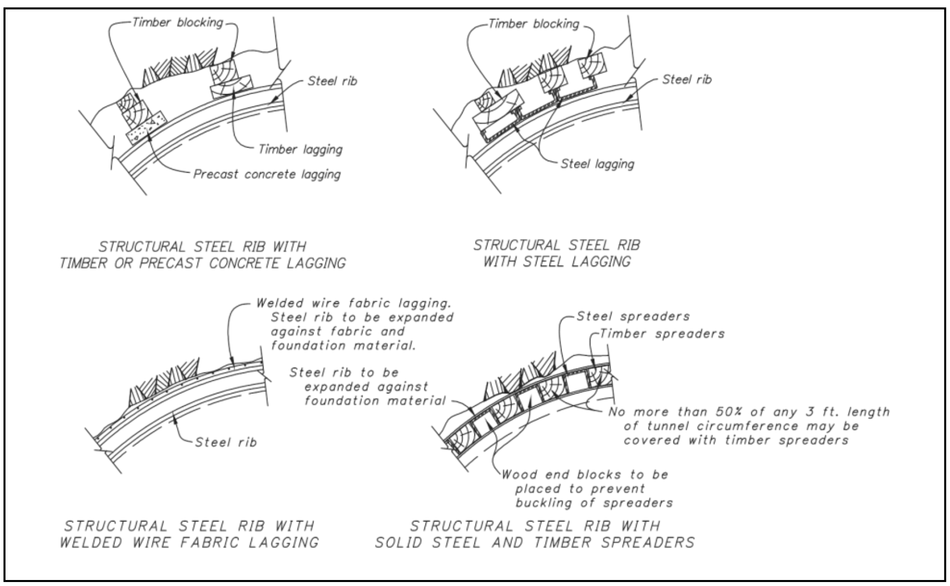

Figure 4.7.1.1.6-1. Lagging and blocking details.

4.7.1.1.7 Steel Support Size

The size of the structural steel support is determined by structural design. Unless otherwise designed, shop drawings should be checked to ensure that the deflected shape of the steel support remains outside the A-line.

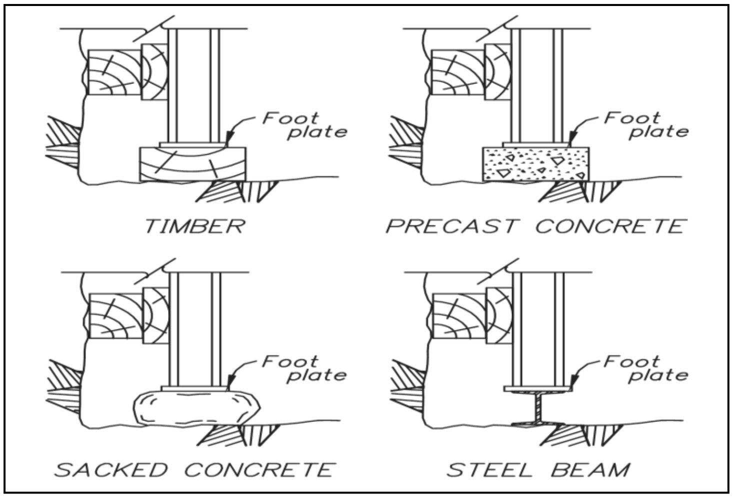

Figure 4.7.1.1.6-2. Typical foot blocks detail.

4.7.1.2 Rock Reinforcing

Geotechnical engineers typically design rock reinforcement. Reclamation uses three kinds of rock reinforcement (figure 4.7.1.2-1): rock anchors, rock bolts, and tendons. Rock anchors are not tensioned and are used where some rock movements can be tolerated and ground shear strength enhancement is not required. Rock bolts are tensioned and are used where no rock movements could be tolerated and the rock shear strength needs to be enhanced. Tendons are long, tensioned cables used when rock movement needs to be minimized and the excavated opening is narrow or small. They are also used or economically justified because of the reinforcement length.

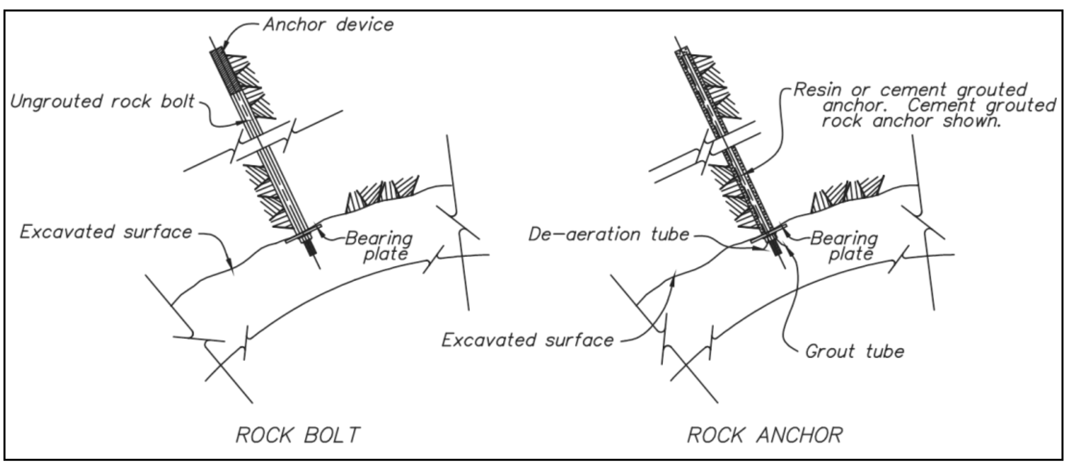

Figure 4.7.1.2-1. Rock reinforcement details.

Rock reinforcement can be used as either an initial or final support depending on rock characteristics and type of rock reinforcement. Rock reinforcement used as final support must have a projected life at least as long as the life of the project. Fully grouted reinforcement will usually provide this life. Epoxy-coated steel or fiberglass rock bolts may be used in corrosive environments.

Spot or pattern rock reinforcement can be used by itself or in combination with metal straps (mine roof mats), chain link fabric, welded-wire fabric, and shotcrete. Typical reinforcing patterns range from 2-ft by 2-ft (0.6-m by 0.6-m) to 6-ft by 6-ft (1.8-m by 1.8-m). Reinforcement should be placed to span over prominent joints.

Solid or groutable hollow rock bolts or anchors, and rock stabilizers (e.g., Split Sets® or Swellex®) can be designed to sustain the load using principles of (1) suspension, (2) arching, or (3) beam theory [46].

In suspension theory, the weight of a critical (key block) and potentially unstable rock block is carried by a single bolt or multiple bolts with a usual factor of safety of 2. In arching theory, at an indeterminate height above an opening, an arch forms as a result of stress redistribution following excavation of the opening. Any loosened rock between the crown or roof of the opening and this natural arch must be supported. For heavily jointed rock, by using rock bolts in conjunction with shotcrete or mesh, an artificial arch near the crown or roof of the opening carries the loosened rock loading. In beam theory, the rock and the bolts form a laminated beam which carries the rock load. Rock bolts increase the shear capacity of the laminated rock beam.

The minimum length of rock reinforcement for arch theory is the greatest of the following:

1. Two times the bolt spacing.

2. Three times the width of critical and potentially unstable rock blocks.

3. One-half of the span of the opening for openings up to 20 ft (6 m).

4. One-fourth of the span for openings between 60 ft (18 m) and 100 ft (30 m).

5. One-half to one-fourth of the span for openings between 20 ft (6 m) and 60 ft (18 m), respectively.

Maximum spacing of rock reinforcement is the least of the following:

1. One-half the length of bolt.

2. One and one-half times the width of critical and potentially unstable rock blocks.

3. Six ft (1.8 m).

The minimum spacing of rock reinforcement is 2 ft (0.6 m).

4.7.1.3 Shotcreting

Shotcrete can be used as either an initial or a final support. A 1.5-inch-thick layer (38 mm) of shotcrete may be specified as a protective coating to preserve rock surfaces subject to slaking and is considered not to carry any load. Support shotcrete must be 3 inches (76 mm) thick or greater.

The usual Reclamation specified compressive strength is 3,000 pounds per square inch (lb/in^2) (21 megapascals [MPa]); however, the designer may specify higher strengths. Current industry standard strengths are 4,000 lb/in^2 tested at 28 days. Testing for shotcrete strengths is typically done using test panels instead of cast cylinders.

4.7.1.4 Grouting

Cement, microsilica, pozzolan, and foam grouts can be used for ordinary, consolidation, compaction, jet, and pressure grouting. Chemical grout is used for special purposes such as stabilizing soil or cutting off water. Grout life is highly dependent on geological and hydrogeological factors. Some chemical grouts are short lived. Cement grouts should not be used in a low pH (4 or less) environment.

4.7.1.5 Freezing

Freezing is a costly method of ground stabilization that is most appropriate where the dewatering or grouting of a cohesionless soil is not feasible. When freezing is planned as a method for tunnel, shaft, or cavern excavation, the amount of heave that is likely to occur should be properly controlled. Ground freezing in areas containing steam, water, or sewage lines should be avoided, as insulating these utility lines significantly increases the costs.

4.7.1.6 New Austrian Tunneling Method

The New Austrian Tunneling Method (NATM) has not been specified by Reclamation; thus, no standards exist. Some of its elements such as shotcreting, rock bolting, and observation by use of closure points have been incorporated into Reclamation designs.

4.7.2 Final Lining

4.7.2.1 General

The functions of final (permanent) tunnel linings can be one or more of the following:

- Maintain the long-term (design life) structural integrity of the tunnel for external or internal loadings
- Provide a smoother hydraulic surface than that of an unlined tunnel
- Prevent decomposition of erosive rock
- Minimize water loss through exfiltration
- Limit infiltration of deleterious water and contaminant
- Provide a roadbed for vehicular travel

4.7.2.2 Minimum Thickness

The minimum thickness of an unreinforced concrete lining is measured from the inside face of the concrete lining to the “A” line and is dependent on the method of excavation. For drill and blast excavated tunnels, the minimum lining thickness should equal 3/8 inch (10 mm) to 1/2 inch (13 mm) per foot of internal tunnel diameter but not less than 4 inches (100 mm). For roadheader excavated tunnels, the minimum lining thickness is 1 inch (25 mm) more than if the tunnel were excavated by drill and blast methods. The minimum lining thickness for TBM excavated tunnels is the same as for roadheader excavated tunnels but not less than 6 inches (150 mm) if expanded ribs are used. The smaller minimum lining thickness for the drill and blast method of excavation is allowed because greater overbreak is anticipated with only isolated points of rock protruding to the “A” line.

Ordinarily, because of a dam’s importance, the minimum thickness of concrete linings associated with dam tunnels (e.g., diversion, spillway, and outlet works tunnels) is twice (3/4 inch to 1 inch per foot of internal diameter) that of other drill and blast excavated tunnels and should not be less than 6 inches [47]. Tunnel linings associated with dams are frequently reinforced.

For reinforced concrete lining, the minimum cover over the reinforcement on the ground side is 3 inches (75 mm). The minimum cover over the reinforcement on the opening side is 2 inches (50 mm) for No. 8 or smaller bars, and 3 inches (75 mm) for No. 9 or larger bars. Temperature and shrinkage reinforcement is based on the “B” line or minimum bore diameter.

4.7.2.3 Loadings

Lining calculations must account for existing geological conditions including anomalies such as shear zones intercepting the tunnel alignment at a skew angle, ground loads, internal and external hydrostatic pressures, loadings from backfill and ring grouting, tensile strength of concrete for unreinforced linings, and reinforcement. Final lining structural design should ignore the strength contribution of initial supports. Final lining loading and design are based on "A" line dimensions. Initial support loadings and design are based on "B" line dimensions.

Segmental linings support the ground during and after construction. Segmental lining design must include consideration of the following: loads from storage (stacking) of the segments, loads from handling and installation (lifting) of the segments, single shield TBM thrust forces during the mining cycle, grouting pressures, and ground loads.

It is important to ensure that the rock above the crown of the tunnel will not loosen and to avoid point loadings on the lining. Backfill grouting (see figure 4.7.5-1) at 15 to 30 lb/in^2 (100 to 210 kilopascals [kPa]) above existing external water pressure is used to fill any voids above the crown not filled during placement of the lining concrete. Filling these voids ensures uniform distribution of the ground loading. Unless unusual grouting procedures are specified, lining should be designed to withstand the grout pressure of 30 lb/in^2 (210 kPa) over the upper 90º of tunnel lining applied at the "A" line.

4.7.2.4 Water Tightness Design

Tunnels may or may not be designed for water tightness. Most water tunnels designed to convey irrigation water are designed assuming that water leaks into and out of the tunnel. Potable water tunnels must not allow any infiltration. Waterproofing requirements of highway and access tunnels depend on their particular use.

Water can infiltrate or exfiltrate through lining joints. Steel liner joints are welded and watertight. Joint water tightness is ensured through nondestructive testing or pressure testing. There are several joint types in a concrete lining: construction, contraction, control, and expansion. These joints can be sources of infiltration or exfiltration. The joints can be made waterproof to some degree. In some cases, with proper design and construction control, 100% water tightness can be obtained.

A lining can be waterproofed externally or internally. External waterproofing uses an impervious membrane, preferably a thermoplastic seal, which is installed at the lining-ground interface. In the internal method of waterproofing the lining, the impervious membrane is installed at the innermost face of the lining or is sandwiched between the outer and inner lining layers. Any sealing method must be tested in place.

Individual joints in a concrete lining are waterproofed using waterstops, gaskets, sealants, or caulking compounds. Waterstop joints can be sources and locations of long-term deterioration and should be avoided if possible. The proper embedment of water stops (at least 3 inches [75 mm]) ensures effective performance.

4.7.2.5 Cracking of Concrete Tunnel Lining

Allowable crack widths depend on the functional requirements of the tunnel, the geology, and the environmental exposure. In addition to a loss of conveyed water, seepage through cracks in the lining of a pressure tunnel can result in lining instability, slope stability failures, and hydrofracturing. A crack width of 0.01 inch (0.25 mm) can pipe silt and sand into the tunnel under 2 lb/in^2 (0.014 MPa) of pressure [48].

Use of steel reinforcement in concrete will not stop cracking of the concrete; however, well distributed reinforcement, within the area of maximum tensile stress, will result in more cracks of reduced width instead of a few large cracks [49]. Where a concrete lining of a pressure tunnel is to be reinforced to resist tensile hoop stresses, the crack width and accompanying leakage are minimized by use of reduced reinforcement stresses (see section 4.7.3.4.). Where absolute water tightness is required, a steel lining or other impermeable lining should be used. To determine the distribution of reinforcement, where a concrete lining is designed to resist flexural stresses, the maximum allowable stress in the reinforcement at service loads will be selected according to the appropriate environmental exposure condition. ACI 350 [31] defines normal environmental exposure as exposure to liquids with a pH greater than 5, or exposure to sulfate solutions of 1,000 parts per million or less. Severe environmental exposures are conditions that exceed these limits. The environmental exposure should consider both the quality of the water conveyed and the quality of the groundwater encompassing the tunnel.

The area and distribution of shrinkage and temperature reinforcement shall be in accordance with section 7.12 of ACI 350. The thickness of the concrete section for design shall be the distance from the finished surface of the lining to the "B" line (see Section 4.4.1.2, "Size") for the definition of "B" line.

4.7.3 Pressure Tunnels

4.7.3.1 General

Pressure tunnels are tunnels that convey water with a hydraulic gradeline above the tunnel crown. Power (penstock) tunnels (conduits), outlet works tunnels or portions thereof, and some spillway tunnels and water tunnels require pressure tunnel design.

A pressure tunnel lining must not leak when the tunnel passes through or under an embankment dam or in other situations where seepage through the tunnel lining may create instability or internal erosion of the surrounding materials. In such cases, the lining must be made watertight by using a steel liner or impermeable membranes such as high density polyethylene (HDPE) or steel sheets.

Internal pressure may be resisted by the lining alone or the lining in combination with the ground. If the ground is weak and incapable of sustaining any load, the lining must withstand the full internal head without any help from the surrounding ground.

In addition to internal fluid pressure, designs must also account for external water pressures. With time, external water pressures can build around pressure tunnels and pressure shafts. The external water pressures can be reduced by using French drains, relief drains, weep holes, or drainage galleries. Systems combining the use of drainage and grouting are sometimes effective to control external water pressures. The possibility of calcification or other sources plugging the drains should be anticipated, and provisions should be made to maintain the drains. Such designs should be used with extreme caution, if at all.

Designs should not be based on the premise that the internal pressure offsets the external loadings. Although the internal pressure resists some portion of the external loadings, when the tunnel is dewatered (e.g., for maintenance), the internal pressure does not exist, and the lining must resist the full external loadings. Similarly, the external hydrostatic pressure should not be assumed to resist part or all of the internal pressure unless the magnitude of the external head can be ensured.

Typically, a pressure tunnel lining should be designed for:

- External rock loading and external hydrostatic loading without internal pressure

- Internal pressure without external hydrostatic loading

If air valves are present and a soft seat is not specified, then the minimum head over the air valve must be 12 ft (4 m).

4.7.3.2 Lining

Pressure tunnels are usually constructed with a steel liner, reinforced concrete lining, unreinforced concrete lining, or a combination of these types. Prestressed concrete may be used for pressure tunnel lining. Other design organizations have used linings of plastic membrane sandwiched between separate concrete layers or placed at the rock and concrete lining interface. Other organizations have used grouting to prestress the concrete lining to counterbalance internal pressure, but with mixed success. Reclamation has not commonly used unlined pressure tunnels but considers them where rock conditions are favorable.

In a pressure tunnel, where only some reaches of tunnel have a steel liner, a grout ring (figures 4.7.3.2-1 and 4.7.3.2-2) is required at each end of the steel liner to reduce seepage. To ensure smooth stress transfer from steel-lined reaches to

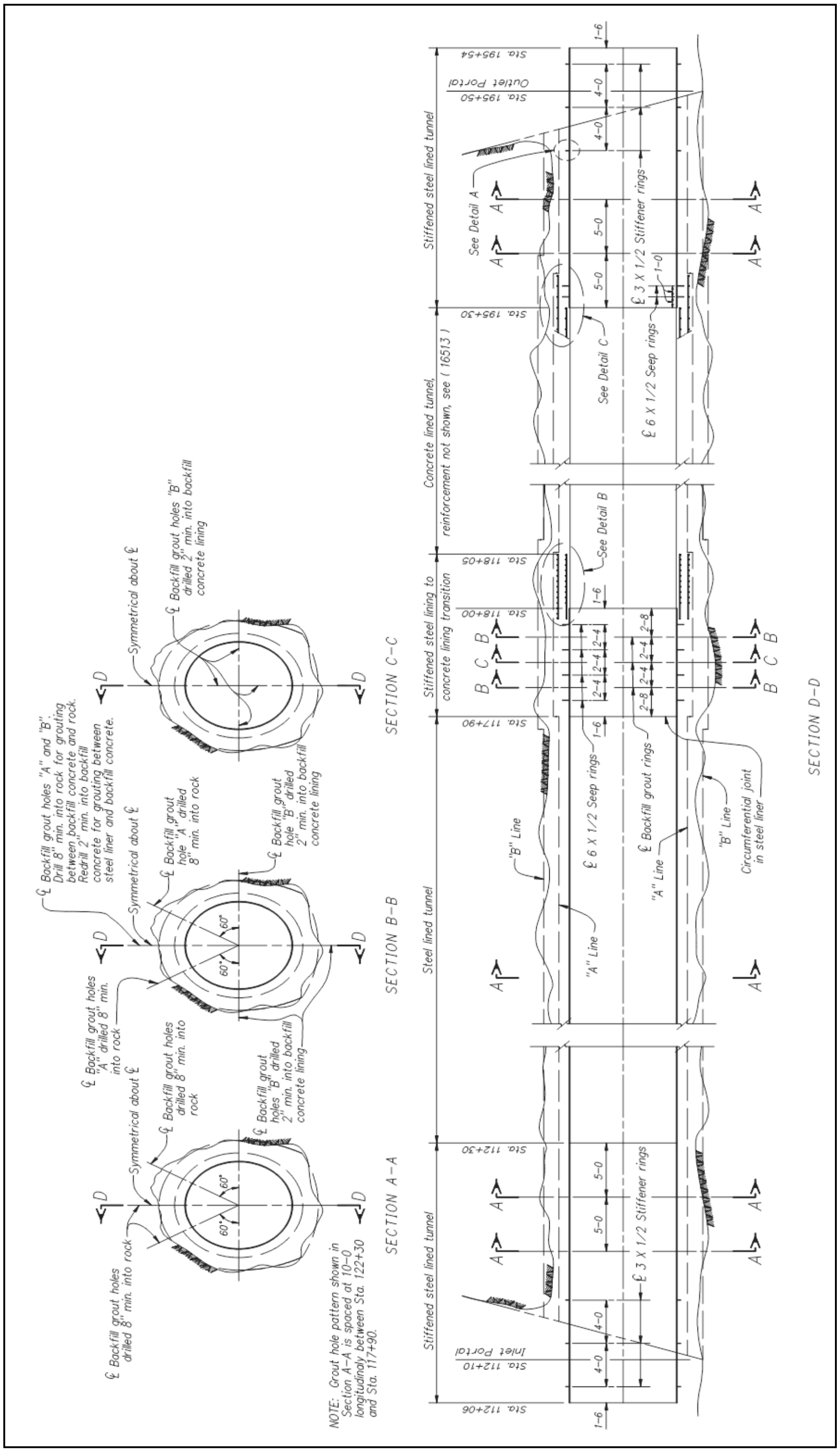

Figure 4.7.3.2-1. Steel-lined tunnel – sections.

nonsteel-lined portions, install transition sections (figures 4.7.3.2-3 and 4.7.3.2-4) of reinforced concrete of adequate length (at least two times the finished tunnel diameter).

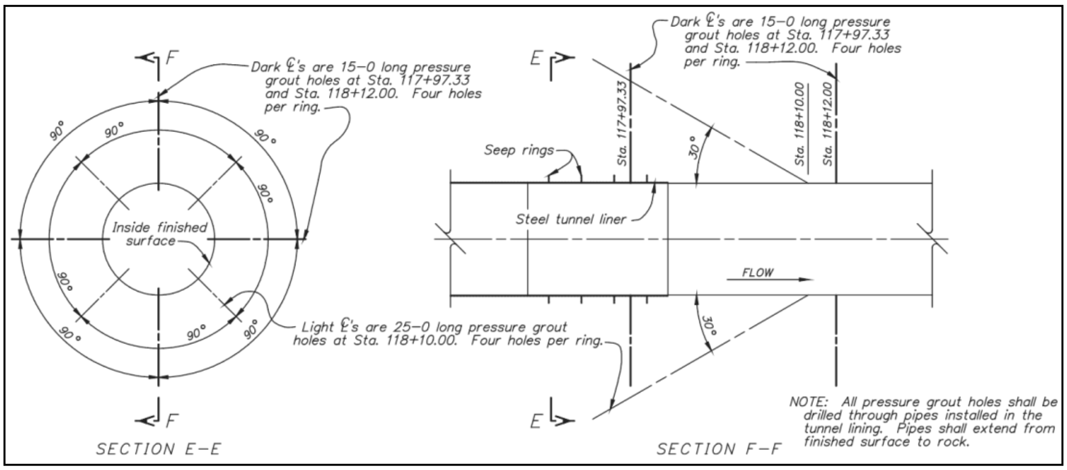

Figure 4.7.3.2-2. Pressure grouting detail.

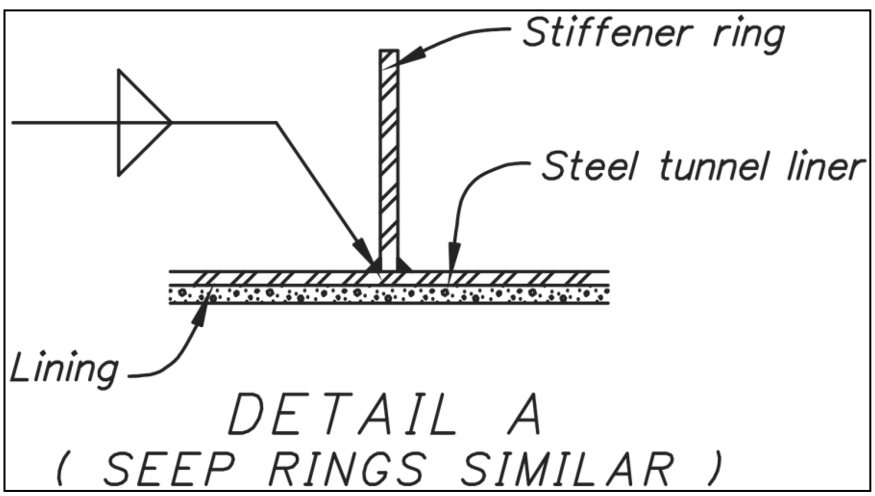

Figure 4.7.3.2-3. Stiffened steel liner.

4.7.3.3 Confinement

A tunnel must be steel lined to control leakage in reaches where either of the following conditions exist: (1) the internal hydraulic head is greater than 150 ft (46 m) and the effective rock cover above the tunnel is less than 40 to 50 percent of that hydraulic head, or (2) where the minimum in situ principal stress of the surrounding rock mass is less than the internal hydraulic pressure. Hydrofracturing will occur when the internal hydraulic pressure is sufficient to exceed the minimum in situ principal stress plus the tensile strength of the rock.

Hydraulic jacking of a joint will occur if the internal hydraulic pressure exceeds the in situ stress acting normal to that joint. While these rules are used by many designers, their validity is dependent on the unit weight of the rock, the fracture density, and the fracture openness.

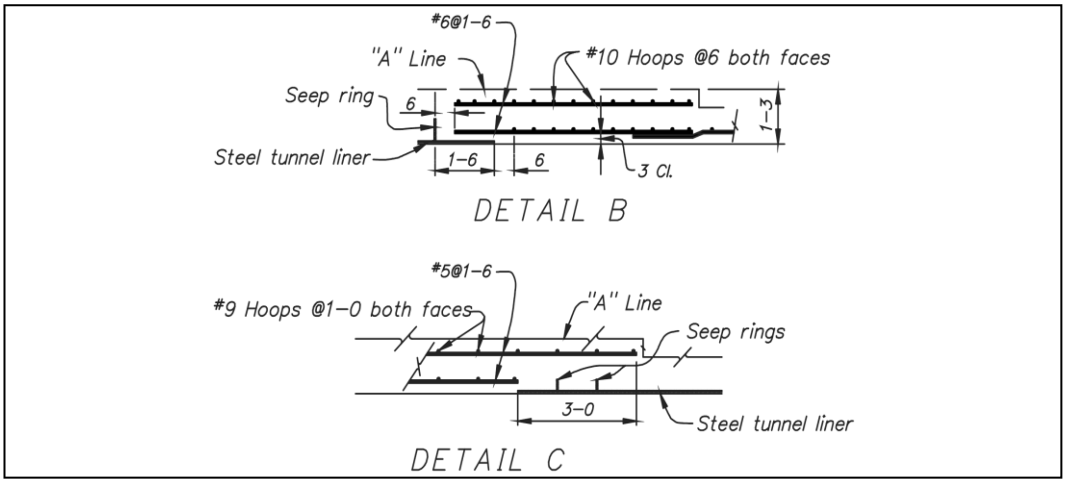

Figure 4.7.3.2-4. Steel-lined tunnel – transition reach.

For internal hydraulic heads less than or equal to 150 ft (46 m), reinforced concrete lining may be used in reaches of tunnel where the effective rock cover is insufficient (less than 40 to 50% of the internal hydraulic head) or where the minimum in situ principal stress of the rock mass surrounding the tunnel is less than the internal pressure.

The internal hydraulic head used in the confinement criteria above is defined as the normal operating head plus any head rise due to transients. The effective rock cover should not include the depth of soil or loose rock or any weathered rock which may be readily removed by erosion.

4.7.3.4 Design for Internal Pressure (Reinforced Concrete Lining)

Depending on the modulus of deformation of the rock surrounding the tunnel, a reinforced concrete section can be designed to withstand the full internal hydraulic head with or without load sharing with the rock. The modulus of deformation can be computed from either the RQD [50][51]or RMR [52] [53] rock classifications. The allowable stress in concrete reinforcement (σ_{ALL}) in concrete lining is:

$$\sigma_{\mathrm{ALL}} = (17{,}000\,\mathrm{lb/in^2}) - \left(35\,\frac{\mathrm{lb/in^2}}{\mathrm{ft}}\right) *\ \mathrm{H} \qquad \text{(H in ft)}$$

or:

$$\sigma_{ALL} = (117\ \mathrm{MPa}) - (0.79\ \frac{\mathrm{MPa}}{\mathrm{m}}) * \mathrm{H} \qquad \text{(H in m)}$$

but not less than 12,000 lb/in^2 (83 MPa)

where:

H = Hydraulic head (includes head rise from transients)

The maximum circumferential or longitudinal reinforcement spacing shall not be greater than 12 inches (30.5 cm).

4.7.3.5 Design for Internal Pressure (Steel Liner)

For massive rock with very few joints, the backfill concrete and surrounding rock can be considered to share part of the internal pressure load where the tension in the rock at a depth of 33 ft (10 m) below the ground surface is less than the compression stress at this location [54]. For lesser quality rock and wider joints, there is no accepted design approach. The designer should evaluate each instance and take a conservative approach (requiring more cover before assuming load sharing).

Where load sharing is not assured, the steel liner is designed to resist the full load from internal pressure without any consideration of load transfer to the backfill concrete and surrounding rock. The allowable tensile stress of the steel is 50% of the yield stress.

Where the backfill concrete and the surrounding rock can be considered to share part of the internal pressure load, two conditions are checked: (1) the internal pressure is assumed to be resisted by the steel liner alone, with an allowable tensile stress of 75% of the yield stress of the steel and (2) the load from the internal pressure is distributed (use load-sharing formulas) to the backfill concrete and surrounding rock where the allowable tensile stress of the steel is of 50% of the yield stress.

The hydraulic head used to design the steel liner is the normal operating head plus any head rise due to transients. It is permissible to use stiffeners (see figure 4.7.3.2-3), although this is discouraged because of construction difficulties, or drain pipes to reduce the wall thickness of a steel liner.

The thickness for a steel liner by bursting analysis is given by the equation:

$$t_{SL} = \frac{P_i R_S}{\sigma}$$

where:

t_{SL} = Thickness of steel liner
P_i = Internal pressure
R_s = Radius of steel liner
σ = Allowable stress

For a liner with a diameter less than or equal to 6.6 ft (2.0 m), the minimum thickness for handling (t_{min}) is given by the equation:

$$t_{min} = \frac{D + 20}{400}$$

where:

t_{min} = Minimum thickness in inches
D = Diameter in inches

or:

$$t_{min} = \frac{D + 508}{400}$$

where:

t_{min} = Minimum thickness in mm
D = Diameter in mm

When the diameter of the liner is greater than 6.6 ft (2.0 m), use the appropriate curve [55] for the specified strength of steel to select the minimum thickness for handling.

When a steel liner is encased in backfill concrete, an annular gap (figure 4.7.3.5-1) may form near the interface of the steel liner and backfill concrete. The gap is caused by a combination of drying shrinkage of the concrete and the difference between the maximum temperature of the concrete reached during curing and the lowest temperature of the conveyed water during operation. The formation of an annular gap is dependent on the temperature variation and is given by equation:

$$\Delta_G = \alpha * \Delta_T * R_S$$

where:

Δ_G = Gap thickness
α = Coefficient of thermal expansion of steel
Δ_T = Range of temperature variation
R_S = Outside radius of steel liner

Δ_G usually does not exceed 1 x 10^{-3} times the radius of the steel liner, and the gap generally lies between 2 x 10^{-4} and 4 x 10^{-4} times the radius of the steel liner.

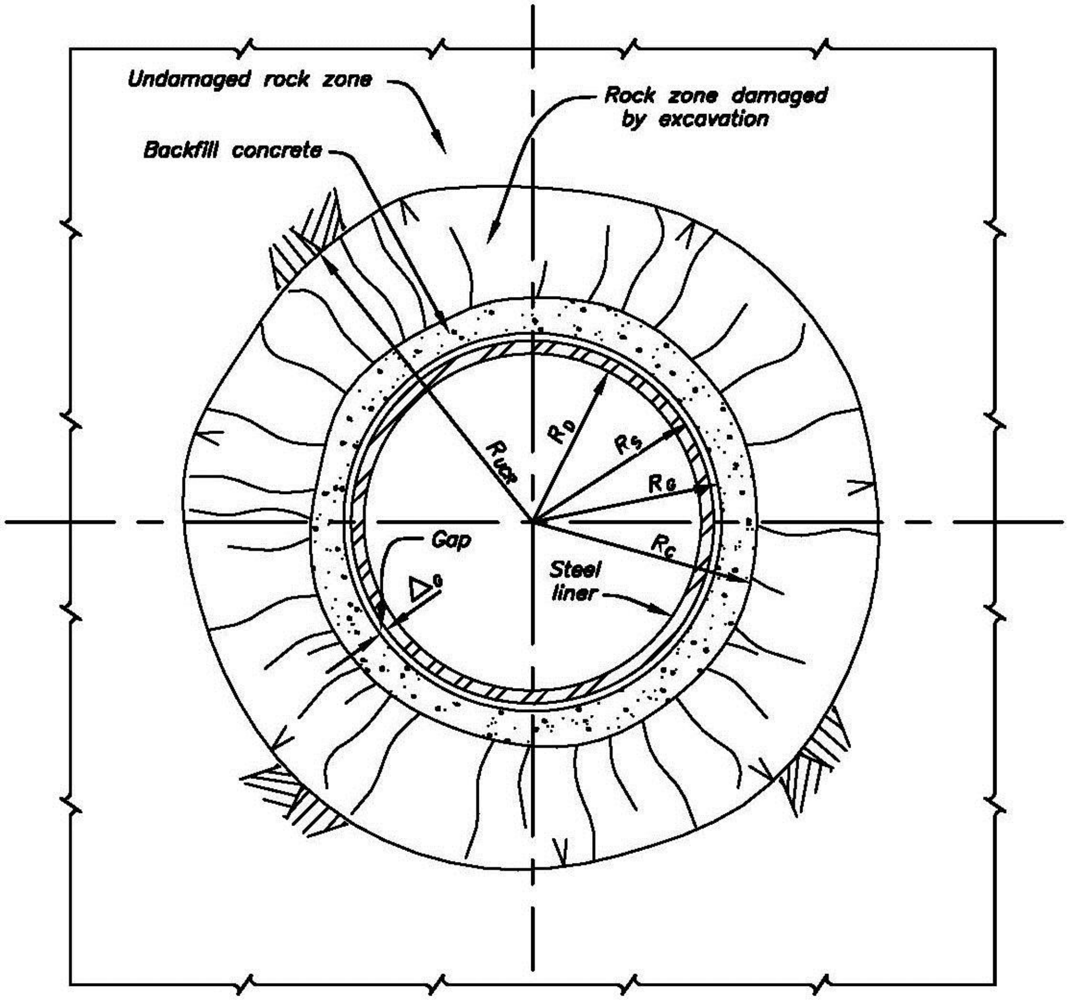

Figure 4.7.3.5-1. Load sharing between steel liner, backfill concrete, and surrounding rock.

To maintain contact at the interfaces, the following equation must be satisfied:

$$\Delta_S = \Delta_G + \Delta_C + \Delta_{cr} + \Delta_{ucr}$$

where:

Δ_S = Deformation of steel lining
Δ_G = Gap width
Δ_C = Deformation of the concrete lining
Δ_{cr} = Deformation of the rock zone damaged by excavation
Δ_{ucr} = Deformation of the rock outside the damaged zone

Considering steel liner to act as a thin cylinder, but the concrete lining and rock to act as thick cylinders of elastic media and plane strain conditions, the pressure taken by steel liner, P_S is given by the formula [42]:

$$P_S = \frac{\left\{\alpha * \Delta_T * R_S + 0.5 * P_i * \left[\left(\frac{1-v_C^2}{E_c}\right) * \frac{1}{R_C} * (R_C^2 - R_G^2) + \frac{R_G}{R_{ucr} * R_c} * \frac{(1-v_{cr}^2)}{E_{cr}} * (R_{ucr}^2 - R_C^2) + 2 * \left(\frac{1+v_{ucr}}{E_{ucr}}\right) * R_G\right]\right\}}{\left\{R_S^2 * \frac{(1-v_S^2)}{t_{SL} * E_s} + 0.5 * \left[\frac{(1-v_C^2)}{E_c} * \frac{1}{R_C} * (R_C^2 - R_G^2) + \left(\frac{R_G}{R_{ucr} * R_C}\right) * \left(\frac{1-v_{cr}^2}{E_{cr}}\right) * (R_{ucr}^2 - R_C^2) + 2 * \left(\frac{1+v_{ucr}}{E_{ucr}}\right) * R_G\right]\right\}}$$

where the new terms are:

E_c = Modulus of elasticity of concrete
E_s = Modulus of elasticity of steel
E_{cr} = Modulus of deformation of the rock damaged by excavation
E_{ucr} = Modulus of deformation of the rock outside of the damaged zone
v_C = Poisson's ratio of concrete
v_S = Poisson's ratio of steel
v_{cr} = Poisson's ratio of the rock damaged by excavation
v_{ucr} = Poisson's ratio of the rock outside the damaged zone
R_D = Radius to the inside of the steel liner
R_S = Radius to the outside (gap side) of the steel liner
R_G = Radius to the outside of gap
R_C = Radius to the contact between the backfill concrete and the rock
R_{ucr} = Radius to the rock undamaged by excavation
t_{SL} = Thickness of steel lining
Δ_T = Change in temperature between constructed and operating conditions
α = Coefficient of thermal expansion
P_i = Internal hydrostatic pressure plus pressure rise due to hydraulic transients
P_S = Pressure taken by steel liner

The steel liner can be designed once P_S is found. The backfill concrete around the steel liner is designed to withstand the pressures (P_i - P_S).

Another load-sharing formula [56] is:

$$P_C = \frac{\left[\frac{P_i * R_D^2}{t_{SL} * E_S} * (1 - v_S^2) - \alpha * \Delta_T * R_D\right]}{\left[\frac{R_D^2}{t_{SL} * E_S} * (1 - v_S^2) + \frac{R_G}{E_C} * log_e\left(\frac{R_C}{R_G}\right) + \frac{R_G}{E_{CR}} * log_e\left(\frac{R_{ucr}}{R_C}\right) + \frac{R_G}{E_{ucr}} * (1 + v_{ucr})\right]} \text{ and}$$

$$P_S = P_i - P_C$$

where:

P_C = Pressure at inside of backfill concrete
(Other symbols were defined previously.)

Although there are several other load-sharing formulas available (see table B-14, reference [57]), the above two formulas are considered adequate for Reclamation design.

A review of other load-sharing formulas reveals that there are significant differences between formulas. These differences are attributed to the different assumptions adopted for their derivations.

Since neither rock nor concrete follow elastic theory exactly, for a more realistic analysis, it is recommended that at least a two-dimensional finite element analysis is made. A three-dimensional finite element analysis is more realistic and more expensive to perform than a two-dimensional analysis but may be desirable.

Cellular concrete may be used for backfill of steel liners. The properties of cellular concrete must be considered in the above equations where appropriate. The effect on the buckling equations given below is unknown and, therefore, should not be used where external hydrostatic pressure is a concern. The benefits of cellular concrete increase when the steel liner is designed to resist the entire head. Cellular concrete ameliorates buoyancy problems that are common with normal backfill concrete.

4.7.3.6 Design for External Pressure (Steel Liner)

Pressure tunnel linings must be designed to resist external water pressure. The actual external pressure is difficult to predict. The design pressure is the greater of the maximum potential groundwater table head or the internal static head. The lining must be checked for elastic stability (buckling). Increasing the thickness of a lining or using stiffener rings increases the resistance to buckling. The use of stiffener rings should be weighed against the requirements for larger excavations and the concrete placement difficulties that stiffener rings create. For a comprehensive treatment of buckling of a steel liner, consult references [58] [59] [60].

Drainage galleries, drainage pipes, or drains can be used to reduce the external water pressure behind the lining. Do not use drainage in a design if maintenance (cleaning) of the drains is problematic.

The minimum design external hydraulic pressure on steel linings should be 75 lb/in^2 (520 kPa) (includes safety factor) or the designed backfill grout (see 4.7.5) pressure above the existing external water pressure at the time of construction, whichever is greater.

Jacobsen [61] developed equations that must be solved simultaneously to determine the critical buckling pressure. One set of equations is for steel liner encased (constrained buckling) in concrete; the other set of equations is for ring-stiffened liners.

For steel liner without stiffener rings but backfilled with concrete, solve the following three equations simultaneously for α, β, and P:

$$\frac{r}{t}=\sqrt{\frac{\left(\frac{9*\pi^2}{4*\beta^2}-1\right)*\left[\pi-\alpha+\beta*\left(\frac{\sin\alpha}{\sin\beta}\right)^2\right]}{12*\left(\frac{\sin\alpha}{\sin\beta}\right)^3*\left\{\alpha-\left(\frac{\pi*\Delta}{r}\right)-\beta*\left(\frac{\sin\alpha}{\sin\beta}\right)*\left[1+\frac{\tan^2(\alpha-\beta)}{4}\right]\right\}}}$$

$$\frac{p}{E}=\frac{\frac{9}{4}*\left(\frac{\pi}{\beta}\right)^2-1}{12*\left(\frac{r}{t}\right)^3*\left(\frac{\sin\alpha}{\sin\beta}\right)^3}$$

$$\frac{\sigma_y}{E}=\frac{t}{2*r}*\left[1-\left(\frac{\sin\beta}{\sin\alpha}\right)\right]+\left(\frac{p*r*\sin\alpha}{E*t*\sin\beta}\right)*\left[1+\frac{4*\beta*r*\sin\alpha*\tan(\alpha-\beta)}{\pi*t*\sin\beta}\right]$$

where:

α, β	=	Angles used in the buckling equations
E	=	$(E_{steel}/(1-\nu^2))$ for unstiffened pipe, where ν = Poisson's ratio for steel, and E_{steel} is modulus of elasticity of steel

p = Critical buckling pressure (can be due to external pressure and/or vacuum pressure)

r = Internal radius of concrete backfill envelope

t = Steel liner thickness

Δ = Width of gap between steel liner and concrete backfill envelope

s_y = Yield stress of steel

Jacobsen's equations for determining the critical buckling pressure for stiffened liners (see figure 4.7.3.2-3, Stiffened steel liner) are:

$$\frac{r}{\sqrt{\frac{12*J}{F}}} = \sqrt{\frac{\left(\frac{9*\pi^2}{4*\beta^2}-1\right)*\left[\pi-\alpha+\beta*\left(\frac{\sin\alpha}{\sin\beta}\right)^2\right]}{12*\left(\frac{\sin\alpha}{\sin\beta}\right)^3*\left\{\alpha-\left(\frac{\pi*\Delta}{r}\right)-\beta*\left(\frac{\sin\alpha}{\sin\beta}\right)*\left[1+\frac{\tan^2(\alpha-\beta)}{4}\right]\right\}}}$$

$$\frac{p*k}{E*F}*\sqrt{12*\frac{J}{F}} = \frac{\frac{9*\pi^2}{4*\beta^2}-1}{\frac{r^3*\sin^3\alpha}{\frac{J}{F}*\sqrt{12*\frac{J}{F}}*\sin^3\beta}}$$

$$\frac{\sigma_y}{E} = \frac{h*\sqrt{\frac{12*J}{F}}}{r*\sqrt{\frac{12*J}{F}}}*\left[1-\left(\frac{\sin\beta}{\sin\alpha}\right)\right]+\left(\frac{p*k*\sqrt{\frac{12*J}{F}}*r*\sin\alpha}{EF*\sqrt{\frac{12*J}{F}}*\sin\beta}\right)*$$

$$\left[1+\frac{8*\beta*h*r*\sin\alpha*\tan(\alpha-\beta)}{\pi*\sqrt{\frac{12*J}{F}}*\sqrt{\frac{12*J}{F}}*\sin\beta}\right]$$

where:

E = Modulus of elasticity for stiffened pipe

F = Combined cross sectional area of the stiffener rings and the steel liner (shell) between the two stiffeners

J = Moment of inertia of a composite section consisting of the stiffener rings and a portion of the liner (shell). The assumed carrying width of the liner = $1.56*(r*t)^{1/2}$

h = Distance to the outer fiber of the stiffener ring from the neutral axis of the composite section

k = Distance between stiffener rings

4.7.4 Water Control

4.7.4.1 General Concepts

Water may infiltrate or exfiltrate a tunnel during or after construction. Inflowing water can carry finer materials into the tunnel, resulting in voids behind the lining and the potential for lining instability. Infiltrating waters can also carry dissolved gases, such as hydrogen sulfide, which can potentially create a toxic work environment. Acidic waters can corrode a lining.

Infiltrating water will lower the water table and may result in increased settlement beneath surface structures about the tunnel. Wells influenced by the groundwater drawdown will incur higher pumping costs. Infiltrating water may also carry heavy metals and other deleterious substances that can contaminate the quality of the tunnel water. Infiltrating clean water at small flow rates is usually acceptable.

Exfiltrating water effects, particularly for a pressure tunnel, should be evaluated. Such flows can result in hydraulic jacking or slope stability failures at locations of low cover. Exfiltration may result in contractual water deliveries not being met if conveyance system losses have not been adequately predicted.

There are two philosophical methods of water control: exclusion and extraction. Exclusion methods prevent water entry and exit by building some kind of impermeable barrier, such as a steel liner. Barriers can also be formed by pressure grouting the rock surrounding the tunnel, filling joints with caulking compounds, water stops, or by installing plastic sheets having thermoplastic splices.

Extraction methods that drain water away during construction may include pumping, well-point ejectors, sump pumps, and panning. Drainage following construction is by weep holes.

The principal geologist estimates the rate of infiltration to establish a water control unit bid cost during construction. Detailed analysis is required when water inflow is critical to the design.

4.7.4.2 Drainage

While small circular tunnels may be economically designed to withstand considerable external water pressure without drainage relief, the design of concrete linings of larger tunnels, especially noncircular shaped tunnels, should consider the use of weep holes or drainage holes to reduce external hydrostatic pressure on the concrete lining. Weep holes are not appropriate where: (1) a steel liner is used; (2) the internal pressure of a power tunnel must be contained (e.g., near the portals); (3) a lowering of the water table is unacceptable; (4) the water table is lower than the hydraulic gradeline of conveyed flow, and increased leakage out of the tunnel would be unacceptable; and (5) where previously unwetted expansive rock/soil is encountered.

No weep holes or drainage holes should be used on pressurized portions of outlet works tunnels. Additionally, to prevent the piping of fines, where a pressurized reach of a tunnel transitions to a nonpressure reach of tunnel (such as sometimes occurs on outlet works tunnels where the hydraulic control is at the dam axis), weep holes or drainage holes should not be used on the nonpressurized reach of the outlet works tunnel from 2.0 to 5.0 tunnel diameters downstream of the control. Further, if the nonpressure reach of tunnel beyond 2.0 to 5.0 diameters downstream of the control is in erodible ground, either design the lining without weep holes or provide weep holes with drain pack to prevent the piping of fines.

Carbonate precipitates from the rock surrounding the tunnel or from the concrete lining itself can plug weep holes or drainage holes with a resultant increase in external hydrostatic pressure. The tunnel should be periodically inspected to monitor the condition of the weep holes and drainage holes. Any plugged weep holes or drainage holes should be reopened. Depending on the rate of plugging and the interval between inspections, drawdown procedures may need to be modified to accommodate this condition. Often, shrinkage cracks in a concrete lining serve as relief for external pressure; however, they should not be considered to relieve external hydrostatic pressures during drawdown. Routine inspection of the weep holes is required to ensure that they are not plugged and that external pressure behind the lining is not building up. An economic design of free-flow tunnel and, occasionally, pressure tunnel linings requires that the designed drainage will be maintained in good working order.

4.7.5 Grouting

“Backfill” grouting (figure 4.7.5-1), also known as “contact” grouting, is required to ensure uniformity of load transfer between the ground and a concrete lining. “Skin” grouting is used to increase the buckling resistance of a steel liner by filling any gap between the backfill concrete and the steel liner. “Consolidation” grouting would be incorporated into the design of a pressure tunnel to increase the modulus of deformation of the rock, thus increasing the rock’s ability to share the loading from the internal hydrostatic pressure. “Ring” grouting is incorporated into the design of an outlet works tunnel or near the transition from a steel liner to concrete lining of a pressure tunnel to reduce the flow of water along the tunnel. “Ring” grouting can also be used to consolidate the rock surrounding a pressure tunnel. At a minimum, grouting of the rock above the crown of the opening (the rock that is most disturbed by excavation) should be used to decrease the flow of water along an outlet works tunnel.

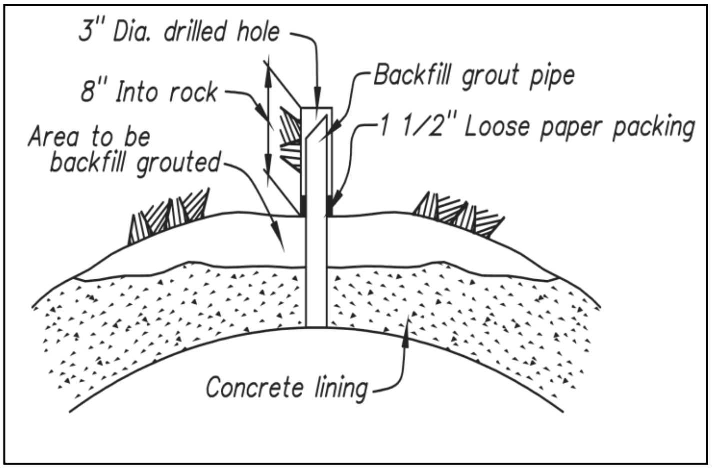

Figure 4.7.5-1. Backfill grout pipe detail.

Figure 4.7.5-2 illustrates how grouting can be placed behind a steel liner. Alternative grouting methods that do not require penetration and repair of the steel liner's protective lining should be considered.

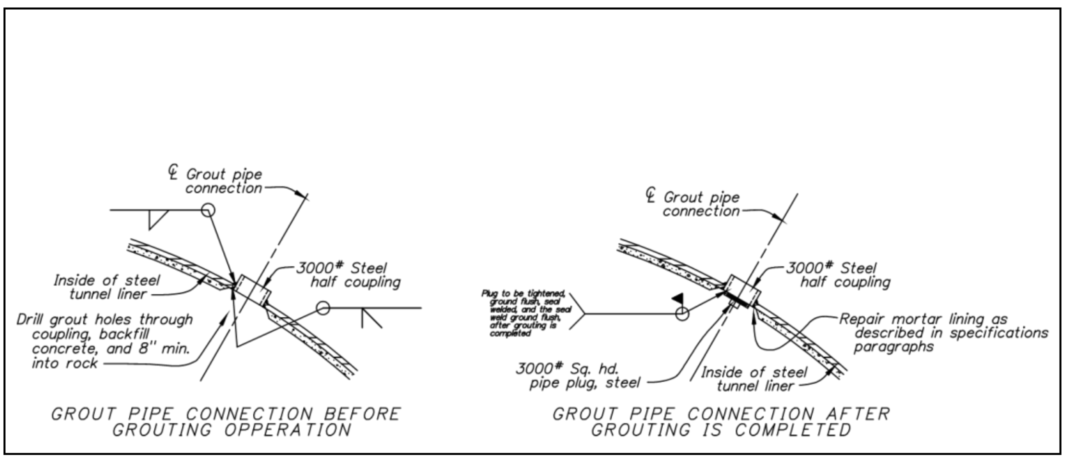

Figure 4.7.5-2. Grout pipe connection details for steel liner.

For a drill and blast tunnel, four backfill grout holes are required per approximately 20 linear ft (6 m) of tunnel. The grout holes are located at overbreak high spots. For TBM-driven tunnels, the number is reduced to three grout holes per approximately 20 linear feet (6 m) of tunnel. The usual backfill pressure is 15 to 30 lb/in^2 (105 to 210 kPa) above the existing groundwater pressure. Grouting proceeds upslope along the grade and from the lower-lying grout holes to higher-lying holes.

4.7.6 Bifurcation and Manifold

Consult mechanical engineers who are experienced in designing bifurcations and manifolds.

4.7.7 Special Features

4.7.7.1 Staging Area

The staging areas must provide sufficient work space for the contractor and the Government. The staging area's size and shape depend on the selected method of construction, the contractor's space requirements, and the terrain. Terrain constraints such as steep grades, proximity to steep slopes, or narrow valleys tend to require an elongated staging area.

Normally, a 500-ft by 300-ft (150-m by 90-m) area adequately serves as a staging area. An additional 150-ft by 100-ft (45-m by 30-m) area is required for treating water recovered during tunneling before it enters the natural drainage.

The staging area must be accessible; otherwise, an access road is required. If required, the Government must obtain right-of-way for the staging area and other facilities before the start of actual construction.

4.7.7.2 Station Markers

Permanent station markers should be installed in tunnels to facilitate maintenance and inspections. Station marker spacing should not exceed 1,000 ft or 500 m. Closer spacing of either 200 ft or 100 m may be warranted to ensure that workers are always within a standard survey tape length (100 ft or 50 m) of a station marker.

4.7.7.3 Instrumentation

The instrumentation plans, objectives, and justifications should be prepared by a knowledgeable design engineer and coordinated with appropriate field personnel, the cost estimator, specification writer, construction management personnel, and the personnel responsible for instrumentation. Extensometers are used to monitor deformations of underground openings or tunnel linings. Such convergence measurements are typically required for outlet works tunnels and should be considered for water conveyance tunnels where squeezing or swelling ground is anticipated or encountered.

4.7.8 Construction Schedules

Construction schedules shall be made whenever an accurate construction duration is required. The critical path shall be found. All expected submittals, holidays, and seasons of the year shall be taken into account. For excavating and lining, the schedule should be based on 24 hours a days, 7 days a week. This allows for maintenance and surveying during the weekends.

Based on working days, the estimated average daily advance for hard rock tunneling should not be greater than:

- 2.1 times the "B" line width for drill and blast excavation of supported tunnels less than 18 ft (5.5 m) in width; the lessor of 1.2 times the "B" line width, and 26 ft (8 m) for drill and blast excavation of supported tunnels between 18 ft (5.5 m) and 23 ft (7 m) in width.

- 154 ft (47 m) for TBM excavating. The penetration rate is dependent on the strength of the rock, the amount of fracturing, the angle of the bedding, the tensile versus compressive properties of the rock, and the machine design. The advance rate is a function of the penetration rate, the time to reset the machine, the time to erect a ring of segments, and the percent of time the TBM is used.

- 35 ft (11 m) per day for roadheader excavation of supported tunnels. The cutting rate is dependent on the compressive strength of the rock joint spacing, anistrophy, clay content, and machine design. The advance rate is a function of the cutting rate, the cross sectional area being excavated, and the percent of time the roadheader is being used.

Based on working days, the average daily advance used for shaft excavating should be no greater than:

- 14.8 ft (4.5 m) for drill and blast. The advance rate is dependent on the diameter and depth of the shaft.

- 65 ft (20 m) for raise bore. The advance rate is dependent on rock strength and highly dependent on mucking rates. The drilling of the pilot hole precedes the reaming, and its average daily advance should be no greater than 225 ft (69 m) for this operation.

Based on working days, the estimated average daily advance for soft ground tunneling should not be greater than:

- 75 ft (23 m) for TBM excavating.

Allow 10 to 11 months to manufacture a 20-ft (6-m) diameter or smaller TBM and 13 months to 15 months to manufacture a 39-ft to 49-ft (12-m to 15-m) diameter TBM. Delivery time is an additional 4 to 5 weeks. Onsite assembly requires 2 to 3 months.

4.7.9 Excavation Methods

4.7.9.1 Drill and Blast

The drill and blast excavating method is suitable for excavating any hard ground type or shape. Methods of excavating hard rock tunnels include full-face, heading-and-bench, top-heading, side-drift, and multiple-drift (a combination of side drifts and top drift). Line drilling, presplitting (also known as preshearing), cushion blasting, and smooth blasting are controlled blasting techniques used to control vibration and reduce crack propagation and overbreak during blasting. Presplitting is not usually used for underground works.

Where morainal deposits, sand and gravel, crushed rock, fault gouge, mud, and silt or clay are encountered during drill and blast excavation, forepoling using spiles may be required. Forepoling is a support method in which sharpened planks or steel sections are driven into the soft ground ahead of the face of the tunnel (figure 4.7.9.1-1) to protect against sloughing ground.

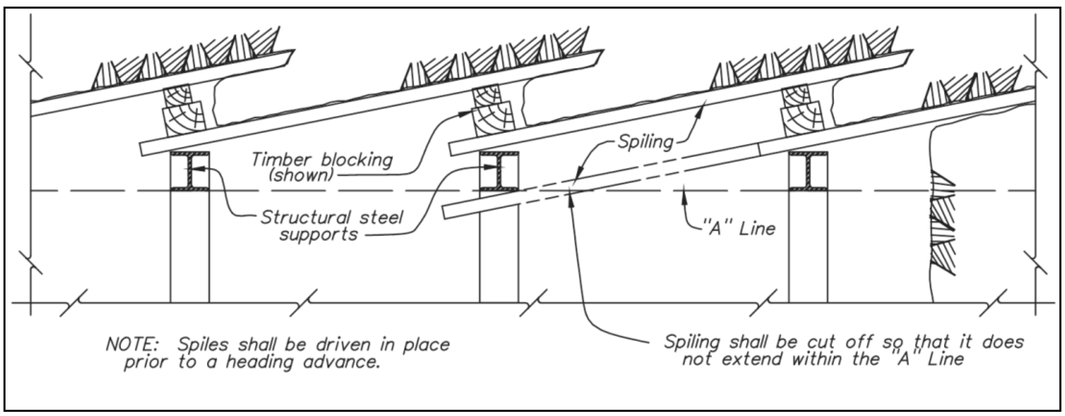

Figure 4.7.9.1-1. Drill and blast excavation using spiles.

The designer should be familiar with applicable portions of *Reclamation Safety and Health Standards* [19] related to drilling and blasting. While drill and blast excavating may be used to excavate any hard ground, it may not be economical for long tunnels. Consequently, drill and blast excavation need not be specified for tunnels longer than 6,560 ft (2,000 m).

4.7.9.2 Tunnel Boring Machines

4.7.9.2.1 General

TBMs are generally designed to excavate only hard or soft ground. Hybrid TBMs can be designed to optimize excavation through mixed ground (both hard and soft ground) conditions. Selecting a TBM type requires careful consideration of geology, groundwater conditions, soil properties, and soil chemistry.

Reclamation specifications that allow the use of TBMs should include contingency plans to complete the excavation if the selected TBM fails to advance the tunnel face in a manner or at a rate acceptable to Reclamation.

4.7.9.2.2 Hard Rock

The three types of hard rock tunnel boring machines are the main-beam, single-shield and the double-shield. The main-beam TBM does not have a shield, and its use is advantageous where the rock is stable with minimal underground water. In reaches of tunnel where the rock is not competent to stand unsupported, the design of a main-beam TBM can be modified to provide continuous support concurrent with its mining (excavation) cycle. Either a single-shield or double-shield TBM can be used in ground requiring the safety of a shield to erect a segmental lining. The advantages of using a single shield TBM (compared to a double-shield TBM) are: (1) its short shield length minimizes the shield's surface exposure to squeezing or swelling ground forces that could entrap the shield, and (2) the short shield length enables the TBM to turn a smaller radius. The disadvantage of using a single-shield TBM is that the mining cycle must be suspended while a segmental lining is erected within the shield. The advantage of using the longer double-shield TBM is that the mining cycle is concurrent with the erection of a ring of precast concrete segments within the tail shield, which enables a larger advance rate (progress rate).

The size of hard rock TBMs ranges from about 6 ft to about 47.2 ft (2 m to 14.4 m) in diameter. The average progress rate can range anywhere from 17.4 ft to 154 ft per working day (5 m to 47 m per day) with maximum daily progress rates of 98 ft to 565.5 ft (30 m to 172.4 m) [62].

The advantages of TBMs include high daily footage of excavation, minimum damage to the rock surrounding the tunnel, minimum volume of overbreak, and, therefore, minimum amount of concrete lining. A disadvantage of TBMs is that they require a long lead time for machine delivery (see Section 4.7.8, "Construction Schedules").

Generally, a new TBM requires at least a 10,000-ft (3,000-m) tunnel to be economically competitive with the drill and blast method. However, an increasing supply of used but proven machines of adaptable size may be available for shorter length tunnels and be competitive with the drill and blast method. Designers should consider designing for a TBM option when the tunnel length is longer than 3,280 ft (1000 m.

TBMs, in general, should have the following requirements specified:

- Be capable of drilling probe holes to determine water and rock conditions ahead of the tunnel face

- Be capable of drilling holes for ground consolidation

- Have methane and other gas detectors
- Shut down automatically if the lower explosive limit for methane or other flammable gases or vapors is detected
- Have explosion-proof components as required
- Have an automatic fire suppression system
- Be capable of successful operation (including muck handling) in water (inflow or construction water)

Main-beam TBMs should have the capability of roof drills and ring beam erectors to install support if needed.

4.7.9.2.3 Soft Ground

Slurry shields and earth pressure balance (EPB) shields are used for soft ground tunneling. The tunnel face is stabilized, and settlement of the ground surface above the tunnel is minimized, by maintaining a steady positive pressure on the face. Disc cutters can be added if hard ground is anticipated. The EPB method is usually considered more efficient in cohesive soils. A slurry shield can typically be used up to 7 bars of hydrostatic pressure, while an EPB can be used up to 3.5 bars. A fully shielded TBM has been designed for 17 bars (247 lb/in^2) of hydrostatic pressure. A 57.5-ft (17.5-m) diameter earth pressure balance TBM is the largest built to date.

4.7.9.3 Roadheaders

Roadheaders are boom-type (arm-type) mechanical excavators. Machine weight currently ranges from approximately 7 tons (6 tonnes) to 149 tons (135 tonnes). Heavier roadheaders are required to excavate harder rock. Water-jet assisted roadheaders can also be used for harder rocks. Roadheaders have excavated rock with compressive strengths of 22,000 lb/in^2 (150 MPa); however, they are generally most effective excavating rock that is less than 5,000 lb/in^2 (30 MPa). The practicality of using a roadheader for tunnel excavation is dependent not only on the compressive strength of the rock, but also on the joint spacing, anistrophy, clay content, and quartz content, as well as the size (weight) of roadheader that will fit within the tunnel.

Roadheaders are economical for tunnel lengths similar to those of drill and blast excavating (see Section 4.7.9.1, “Drill and Blast”).

4.7.9.4 Compressed Air

Reclamation has never used the compressed-air method of excavation; thus, no Reclamation standards exist.

4.7.9.5 Tunnel Jacking

The tunnel jacking process consists of pushing one pipe or pipe-like section of tunnel after another by hydraulically operated jacks while excavation is taking place at the tunnel face within a shield. Open face excavation can be done manually, with an excavator (back-actor) or a cutter boom. Excavation that requires a closed face can be accomplished using a compressed air shield. In addition, closed face excavation of pipes can be accomplished by using a microtunnel boring machine (MTBM). The two basic types of microtunnel boring machines are the slurry type and the auger type. A reaction frame or thrust block with proper anchorage is constructed before the start of tunnel jacking to provide necessary reaction for hydraulically operated jacks.

Where tunnel jacking is specified, an alternative method, such as shield tunneling with liner plate, should be included in the specifications.

4.7.9.6 Other Methods of Tunnel Construction

Soft ground (earth) tunnels can be excavated using an open shield or be hand mined using unitary excavation with steel liner plate. The cut-and-cover method is sometimes used to construct transportation tunnels in urban settings. Tunneling under reservoirs, rivers, or large canals is hazardous work. Such projects should be approached with caution. Canal reaches should be drained if practical.

4.7.9.7 Shaft Excavation

A shaft can be excavated in rock by the following methods:

- Drill and blast sinking
- Raise boring (drilling a pilot hole down followed by reaming up)
- Raise boring plus slashing
- Down boring (drilling a pilot hole up followed by reaming down)
- Blind boring
- Shaft boring machines (SBM) or V-mole.

A crane with a clamshell bucket is typically used to excavate shafts in soil. The economical minimum “A” line diameter for drill and blast excavation is 15 ft (4.6 m). The current largest diameter by raise boring is 23.3 ft (7.1 m). V-moles have been used to bore shafts up to 27.9 ft (8.5 m) in diameter.

4.7.9.8 Cavern Excavation

Access tunnels or shafts to a cavern, such as for an underground powerhouse or a gate chamber, must be excavated first. The cavern is then excavated using drill and blast methods, roadheaders, or other mechanical excavators. Usually, the cavern is excavated and supported in sections until the full excavation is completed.

Completed caverns that are occupied full time should have at least two accesses.

4.7.9.9 Lake Taps

Large lake taps can be accomplished by the American lake tap method [63]. The American lake tap method (figure 4.7.9.9-1) involves the following steps:

1. Excavation of the tunnel (running tunnel) only to a safe distance from the lake
2. Excavation of an underwater bench
3. Excavation of the shaft
4. Lowering a steel liner into the shaft
5. Filling the annulus between the shaft and rock with mortar
6. Placing low-strength concrete into the debris chamber using a tremie
7. Placing a bulkhead, with piping through it, on top of the shaft
8. Dewatering the shaft through the piping
9. Excavation of the remaining length of tunnel (connecting tunnel) into the tremie concrete
10. Lining the tunnel and debris chamber as required
11. Filling the tunnel and shaft through the piping
12. Removing the bulkhead
13. Installing the intake structure on the top of the shaft

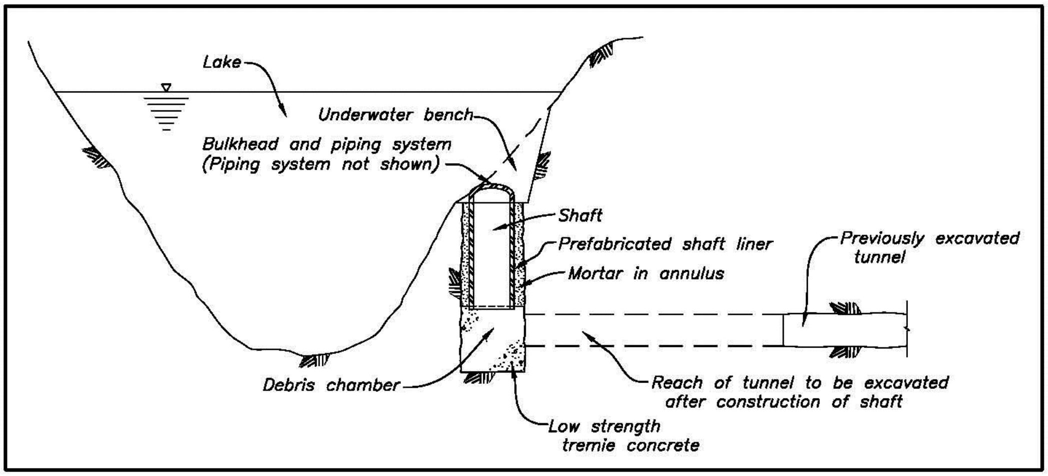

Figure 4.7.9.9-1. Lake tap.

Smaller sized lake taps can frequently be constructed by microtunneling into the lake from a shaft.

4.7.9.10 Material Handling

Material handling methods that are used during construction are generally a contractor option but their feasibility and efficiency should be a design concern. The design of muck disposal areas should address environmental concerns, (including visual esthetics), as well as slope stability, and drainage.

4.7.10 Filling Rate

The filling rate of a power tunnel must ensure that the stability of the tunnel is not compromised, that the lining will not be damaged, and that the release of air from the tunnel will not blow the air valve shut. The determination of a filling rate depends on multiple factors, such as the velocity of water filling the tunnel, the erosion potential of an unlined tunnel, the potential for cavitation of an unlined or lined tunnel, water hammer pressures, safe air escape velocity, and permissible noise level of escaping air. Rapid filling rate has caused unwarranted cracking of a concrete tunnel lining [64].

A safe-escape air velocity of 100 ft/s (30 m/s) is recommended [65]. Manufacturers' catalogs usually list air inflow rates compared with valve sizes, but the air escape velocity may exceed the above recommendation.

To increase the filling rate, larger or multiple air valves may be used in parallel. When air valves are not used, the recommended maximum filling rate is 5% of the design flow rate unless studies show that higher filling rates are acceptable. When air valves are used, the recommended maximum filling rate is 15% of the design flow rate.

4.7.11 Draining Rate

A pressure tunnel should be drained at a rate that does not allow the vacuum pressure to fall below 7 lb/in^2 absolute. When weep holes have been incorporated into the design of a pressure tunnel, the location of the phreatic surface above the tunnel and if maintenance of the weep holes has been performed should be considered when determining the drainage rate. If the weep holes have become plugged, rapid draining may subject the lining to more external head than it was designed for.

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