Chapter 2

DESIGN OF THE METROWEST PRESSURE TUNNEL*

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ABSTRACT

The Massachusetts Water Resources Authority is currently improving its primary water supply system for the Boston Metropolitan area. The improvements planned by the Authority, which provides water for 46 communities and over 2.5 million people in eastern Massachusetts, include the MetroWest Water Supply Tunnel (MWWST) to provide redundancy in the transmission system, a large capacity water treatment plant and extensive covered storage facilities.

The 28.3 km (17.6 miles) long, mostly 4.3 m (14 ft.) I.D. MetroWest Water Supply Tunnel will allow repair of the aging Hultman Aqueduct, which for 56 years has served as the transmission main from the water supply reservoirs to Boston. On completion of the tunnel, the old pipeline aqueduct will be repaired and brought back into service as a parallel, redundant transmission main.

This paper describes design considerations and the design process for pressure tunnels in general and the MWWST in particular. It provides an overview of how the design of pressure tunnel and shaft components progresses.

INTRODUCTION

Planning, design and construction of a major water transmission main facility in an urban/suburban area is a complex undertaking which may take 10 years, or more, to implement. The planning and design efforts may take five or more years of work by a wide variety of engineering and non-engineering disciplines to define, obtain required governmental approval and gain public acceptance of the required facility. Long range planning to identify the need for a facility early enough to have it completed when needed is consequently an ongoing challenge for metropolitan water supply agencies.

*This article represents the opinions and conclusions of the authors and not necessarily those of the MWRA. This article shall not be used as evidence of design intent, design parameters or other conclusions which are contrary to express provisions in contract documents for the MWRA MetroWest Water Supply Tunnel.

The design of major facilities such as the MWWST facility proceeds through logical, progressive steps of development, and reviews. Considerations are given to a wide variety of planning and design issues. These issues include definition of serviceability requirements, rights-of-way, community relations, environmental, regulatory, geological/geotechnical, construction costs and financial planning. Each major step of development culminates in production of reports and increasingly detailed plans and specifications, finally leading to production of construction contract documents, bidding and construction of the facility. At selected milestones, major reviews are made by independent expertise from specific design disciplines or formally assembled review boards.

In 1989 the Massachusetts Water Resources Authority (Authority) issued a planning and design contract for a second transmission main to provide redundancy for the Hultman Aqueduct. As originally conceived, the project consisted of a tunnel combined with reconstruction of an existing, unused gravity aqueduct, the Sudbury Aqueduct. During feasibility studies, it was recognized that costs and environmental and community impact issues related to reconstruction of the Sudbury Aqueduct through an urban/suburban area compared unfavorably with a full length tunnel in rock, deep under existing structures and facilities. The alignment of the tunnel, which is currently under construction, generally coincides with the existing Hultman Aqueduct and is constructed in permanent underground easements below several hundred private properties.

The full length, unreinforced concrete lined, pressure tunnel design concept was selected and the facility was named the MetroWest Water Supply Tunnel. The extensive planning and environmental studies required for this $540 million project have been described in two earlier papers presented by design team members, Caspe, et. al., 1994a and 1994b.

The final horizontal and vertical alignments for the MWWST facility are shown in Figure 1 and Figure 2, respectively. Figure 1 also shows the existing Hultman Aqueduct, Wachusett Reservoir and tunnels in the Authority's water transmission system.

TUNNEL DESIGN

The essential design criteria which must be incorporated into design of pressure tunnels constructed in rock may be categorized as follows:

- Serviceability Criteria
- Geotechnical Conditions
- Constructability Considerations
- Pressure Tunnel Design Principles

These categories of design criteria are discussed in general, as well as specifically for the MetroWest Water Supply Tunnel, in the following paragraphs.

Serviceability Criteria

A number of essential serviceability criteria requires early definition during design of a pressure tunnel to serve as a water transmission main. These are:

- Tunnel Termination and Connection Points and Intermittent Connections
- Static Hydraulic Gradeline (HGL)
- Design Transient Pressure
- Tunnel and Shaft Conduit Diameters
- Maximum Operating Flow Velocities
- Leakage Design Considerations

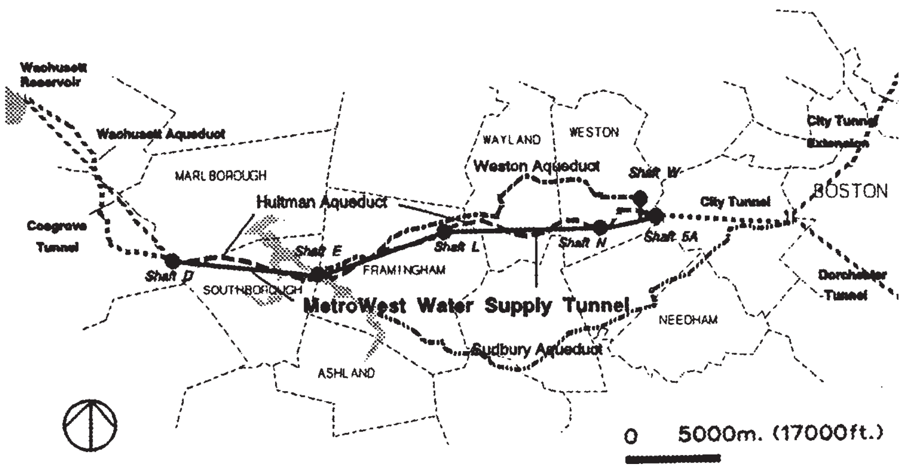

Figure 1. Existing transmission system and proposed MetroWest Water Supply Tunnel

<u>Tunnel Termination and Connection Points</u>

Termination point criteria for a specific water tunnel facility is often simple and straight forward. In the case of the MWWST, it was determined that the tunnel would be redundant to the existing Hultman Aqueduct by connecting Shaft C on the existing Cosgrove Tunnel in the west with Shaft 5 on the existing City Tunnel, and with the Weston Aqueduct Terminal Chamber Area in the east end. These criteria resulted in terminal connections at Shafts D, 5A and W, respectively, and definition of the general alignment of the tunnel facility. Additional major connections to existing Authority transmission facilities resulted in Shaft E, Shaft L, Shaft NW and Shaft NE. Connections to serve community clients resulted in the five small diameter riser shafts to supply water to customers along the tunnel alignment. The locations of these existing transmission system connection points and the resulting horizontal alignment of the facility are shown on Figure 1.

<u>Static Hydraulic Gradeline (HGL)</u>

Definition of the design static HGL is one of the most critical design elements of a pressure tunnel, because of its importance in determining the vertical alignment (depth) of the

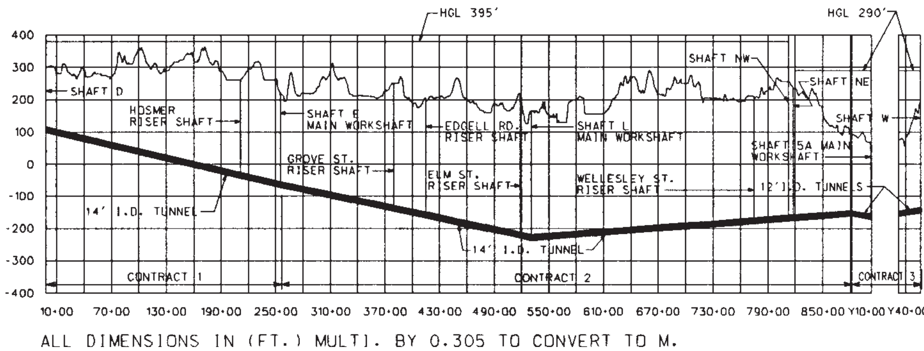

Figure 2. MetroWest Water Supply Tunnel — Profile

constructed tunnel. It is essential that long term planning of a water transmission system be considered in the selection of the HGL so as to prevent development of undesirable “bottlenecks” which may restrict future improvements to the system. This may be considered of particular importance in “upstream” portions of a transmission main system. It is suggested that the HGL should be set at the highest probable level which may be considered for operation in a 100 year (or longer) period of planning, as the useful life of a deep pressure tunnel constructed in rock may be extended for very long periods by periodically performing limited repairs/upgrades of specific components of the overall system.

For the MWWST facility, an HGL corresponding to the maximum water surface elevation of the Wachusett Reservoir, feeding the proposed facility by gravity from an overflow elevation of 120.4 m (395 ft. Boston City Base Datum, BCB), was selected by the Authority. A 120.4 m (395 ft.) HGL was used for design of the tunnel from Shaft D to Shaft NW at the Norumbega Reservoir where water will continue to be stored for distribution to the Boston area. The remaining downstream portion of the MWWST facility, Shaft NE to Shaft 5A and Shaft W, is designed for an HGL of El. 88.4 m (290 ft. BCB). Both HGLs used for design are shown in Figure 2.

Design Transient Pressure

The design transient pressure is determined by hydraulic analysis of the overall future transmission system, including accounting for characteristics of control mechanisms, e.g., shutting times for major valves or catastrophic type changes in flow rate. The resulting design transient pressure for the MWWST facility was established to correspond to 15.2 m (50 ft.) of head increase, or approximately 137.9 KPa (20 psi). Transient pressure is used for design of impermeable lining system components, i.e., localized steel linings in the shafts and tunnels.

Conduit Diameters

The final shaft and tunnel conduit sizes (Internal Diameter or I.D.) were also selected largely from the results of the hydraulic analysis. In determination of conduit sizes, long term planning type flow rates are commonly used to avoid creation of future bottlenecks due to under-sizing of conduits in the system. The design flow rate used in hydraulic analysis for the MWWST facility was 1.893 x 10^6 m^3 per day (500 MGD).

In addition to meeting the fundamental hydraulic requirements of operation, such as flow capacities and operating hydraulic characteristics, additional important operations and maintenance provisions require identification and incorporation in the design, including provisions for future reentry into the tunnels and associated unwatering and safe future access for men and some essential equipment. Considerations of filling and unwatering (e.g., air release/vacuum valves, filling/unwatering procedures and requirements for pumping), as well as the need and means for metering of flows during operation, also must be incorporated into the design. Future access to inspect and maintain the tunnels must be accommodated through the shaft conduits making sizing of manholes and access hatches, invert gradients (the tunnel must generally slope toward the access shafts) and locations of metering devices important design considerations.

The selected I.D. of the MWWST tunnel conduit is 4.3 m (14 ft.) from Shaft D to Norumbega Reservoir and 3.7 m (12 ft.) for the remainder to Shaft 5A and Shaft W. The shaft conduits vary from 3.7 m (12 ft.) to 2.4 m (8 ft.), depending on the location, metering, hydraulic and future access requirements. The five small diameter community riser shafts were selected to be 0.51 m (30 in.) I.D.

Maximum Flow Velocities

Maximum flow velocities are calculated to assess if these are acceptable for the various final lining materials incorporated in the facility. For the MWWST tunnel conduits, all projected flow velocities are less than 3 m (10 ft.) per second and thus acceptable for all envisioned types of lining systems.

Leakage Design Considerations

Issues related to leakage of groundwater into completed gravity flow or low pressure water tunnels, or leakage from the completed pressure tunnel, may affect tunnel final lining criteria and, in effect, the overall design approach.

The MWWST facility is designed to be lined with a cast-in-place, unreinforced concrete final lining, which, in principle, is considered a permeable lining not specifically intended to prevent or control leakage from the tunnel into the surrounding rock mass (exfiltration) during operation of the facility (see Moore, 1989). For the MWWST, leakage is controlled by the rock mass which serves as an integral part of the lining system. For a specific tunnel, the loss of water is largely controlled by the ability of the host rock mass to contain pressurized water, and implementation of localized, special lining treatment where required. Total leakage rates cannot be accurately estimated prior to commissioning of a concrete lined or unlined pressure tunnel.

Geotechnical Conditions

As mentioned earlier, systematically concrete lined and unlined pressure tunnels utilize the host rock mass as the main lining system component to contain the pressurized water within the tunnel and shaft conduits. Several geotechnical characteristics of the host rock mass are critical for design and operation of such facilities:

- In situ stress
- Topographical characteristics
- In situ hydraulic conductivity (permeability)
- Location of groundwater table
- Modulus of deformation

Additional geotechnical characteristics are important for other design aspects of design of tunnels constructed in rock, e.g., rock boreability, rock mass quality and durability. However, the scope of this paper is limited to the discussion of the aspects listed above and their applicability for design of pressure tunnels and the MWWST facility.

<u>In Situ Stress</u>

The prevailing state of minimum principal stress in the rock mass is critical to design of concrete lined and unlined pressure tunnels, because of the long-term exposure of the rock mass to water pressures equal to the internal static pressure. In flat terrain, the vertical stress is generally directly proportional to depth below ground surface and corresponds to the weight of the overburden. Although horizontal stresses will often be less than the vertical stresses, horizontal stresses may be higher due to tectonic conditions, i.e., the vertical stress may be the minor principal stress. This has been reported to be the case in the northeastern United States (Zoback and Zoback, 1980). However, site specific geotechnical exploration is required to verify in situ stresses along the alignment of a pressure tunnel.

The issue of in situ stress in regards to design of a concrete lined or unlined pressure tunnels is associated with the phenomenon of ”hydraulic jacking,” i.e., opening of existing rock mass joints by the action of pressurized water introduced into the joints. Hydraulic jacking phenomena result in dramatically increased permeability and leakage out of a concrete lined or unlined pressure tunnel through the rock mass. In addition to increased leakage, secondary, undesirable environmental effects may also result.

As mentioned above, hydraulic jacking of joints occurs when the water pressure in a joint exceeds in situ stress normal to the joints in the rock mass opening these joints. Critical stress across joints which may be subject to hydraulic jacking may be different from the minor principal stress due to differences in orientation of principal stresses and joints. Complete exploration of in situ principal stresses (i.e., magnitudes and directions) at different depths in numerous boreholes along a tunnel alignment is expensive, impractical and unnecessary. However, assessment of “hydraulic jacking pressure” at different depths, and at various locations along a proposed pressure tunnel is required for design.

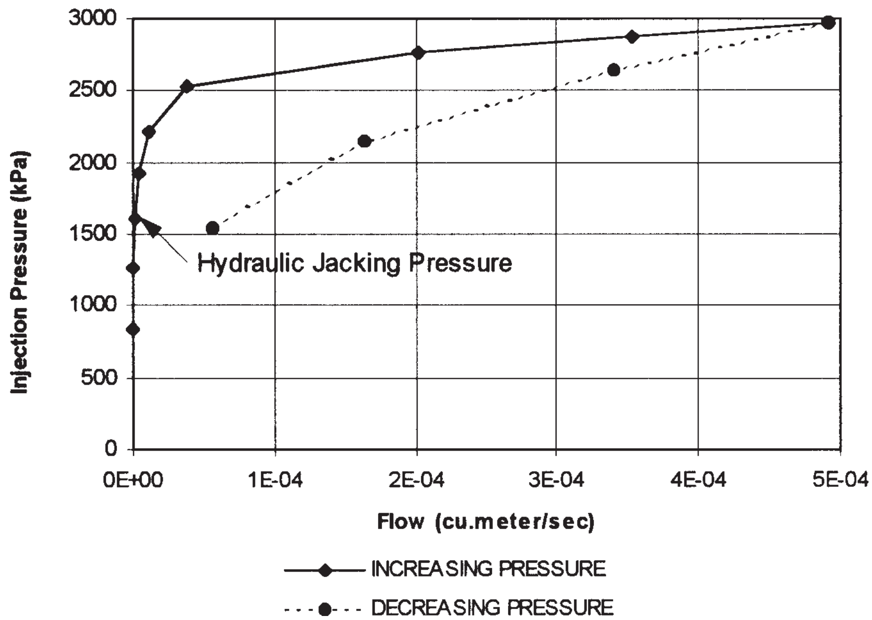

Figure 3. Typical Hydraulic Jacking Test Diagram

The "hydraulic jacking pressure" of critical concern for design is the minimum water pressure which causes opening of joints in the rock mass. These pressures are investigated by hydraulic jacking testing and the results compared to the static hydraulic pressures in the tunnel and shafts corresponding to the HGL. Hydraulic jacking tests are conducted by injecting water at high pressures between straddle packers in exploratory borings along the tunnel. Numerous hydraulic jacking tests were performed at several depth intervals in approximately two dozen deep borings along the alignment of the MWWST facility alignment to investigate the effects of various orientations of joints. This work was carried out with the assistance of a specialty geotechnical consultant, Dr. Gregg Korbin. These tests have become state-of-practice and are necessary for reliable design of pressure tunnels.

An illustration of an actual test conducted in a boring, drilled for MWWST, is shown in Figure 3, where the dramatic decrease of curve slope at pressures beyond the point of hydraulic jacking is directly related to the increase of rock mass permeability due to the opening of joints in the rock mass. Hydraulic jacking testing is described in detail in Hamilton, et. al., 1987, and Johannesson, 1988.

Topographical Characteristics

Localized variations of in situ stress due to topographic phenomena, e.g., steeper slopes, incision-like geological features such as gorges, or varying depth of overburden types of critical features and their effects on the vertical alignment, is the presence of a “buried valley,” or depression of the top-of-rock elevation, located east of Shaft L along the MWWST facility (see Figure 2). This geological feature resulted in locating the deepest point of the tunnel in this area.

In Situ Permeability

The prevailing natural, or in situ, hydraulic conductivity (permeability) of the rock mass is an essential design consideration for any tunnel in rock. Groundwater inflows encountered during construction must be assessed, and is also of particular importance for the feasibility of unlined and concrete lined pressure tunnels. Quantitative data for this assessment is obtained by packer pressure testing during preconstruction design, in conjunction with the hydraulic jacking tests described above. Important descriptive information about permeability characteristics is also obtained by observing drilling fluid behavior during exploratory drilling, and during construction by observing groundwater occurrence in the tunnel. Rock mass permeability assessments along the MWWST alignment, data from previous tunnel construction, and satisfactory performance of existing Boston pressure tunnels indicated that a systematically concrete lined pressure tunnel is feasible. This is discussed in more detail under *Pressure Tunnel Design Principles* later in this paper.

Groundwater Table

The location of the natural groundwater table, relative to the depth of the tunnel, may be of importance for the performance of a concrete or unlined pressure tunnel. An extreme and potentially adverse design situation in regards to leakage from the tunnel would exist where the applicable HGL is high above the ground level, and where permeability of the rock mass is high and the in situ groundwater level low relative to the invert level of a proposed tunnel. For the MWWST facility the groundwater level is near the ground surface and favorable for design of a concrete lined pressure tunnel.

Modulus of Deformation

The magnitude of the modulus of deformation (deformability or stiffness) of the host rock mass may be of significance for design of pressure tunnel lining systems in rock. Deformability of the rock affects any load sharing required for design of impermeable steel linings and performance of reinforced concrete linings designed to control leakage from the tunnel. However, design of steel linings is often governed by external loading due to groundwater pressure on an empty conduit, rather than hoop stress due to internal pressure and any potential load-sharing criteria. Lining design considerations for the MWWST facility are discussed in greater detail under *Pressure Tunnel Design Principals*, later in this paper.

Constructability Considerations

Design of tunnels generally requires thorough consideration of construction methods, costs, logistics, scheduling, sequence, support and lining requirements, boreability, muck disposal, groundwater issues, power requirements, heading length and numerous issues specific to a particular facility due to its environmental or geographical setting or specific serviceability requirements. The following paragraphs will only discuss the most significant constructability issues related to design of urban water supply tunnels in rock and the MWWST facility.

Shaft Sites

Construction of water supply tunnels in areas of generally flat terrain is performed through shafts, which inherently tend to increase in depth with increasing hydraulic head and application of a permeable concrete tunnel lining system. The main construction or work shaft is the focal point of most above ground construction activities in support of underground construction, such as transportation of materials, manpower access, administration, utilities and other services. Identification of suitable shaft sites along the alignment of a long tunnel in an urban/suburban area may present challenges, particularly in residential neighborhoods, due to regulatory and environmental impact considerations.

For the MWWST facility, suitable sites for construction shafts that were already owned or controlled by the Authority were found available along the general alignment of the proposed tunnel, at two locations (Shaft E and Shaft 5A) as shown on Figure 1. The third site, Shaft L, was privately owned, requiring a temporary construction type easement and lease arrangements. The site was selected to allow for construction of two headings, approximately five miles long, from a single construction shaft, following thorough engineering evaluations, including a Value Engineering effort.

Invert Gradient (Slope)

A gentle "uphill" invert slope is always desirable in tunnel excavation and lining to allow for gravity drainage of groundwater inflows away from the excavation heading or area of concrete placement. This also allows for "downhill" transport of tunnel muck. A maximum invert gradient of 0.01 is desirable from an economical and safety aspects of rail haulage during construction. From water tunnel design considerations, including consideration of future unwatering and manned access for inspection and maintenance, a minimum invert gradient of 0.001 is required for gravity drainage in the completed facility. This invert slope should be toward a shaft where suitable disposal of drainage water with minimum preparation and treatment requirements could be arranged at occasions of future entry into the tunnels. These considerations must be balanced with considerations of construction costs by avoiding unnecessary depths of the tunnels and shafts. For the MWWST facility, overall design considerations allowed invert gradients to range between 0.002 and 0.007.

Lining Construction

Where reinforcing of concrete linings is required, it is undesirable to require multiple layers, close spacing, and large bar sizes of reinforcing steel. A minimum concrete lining thickness of 0.3 m (12 in.) is required to achieve the necessary high quality concrete placement, while allowing for all anticipated types of primary support, including steel ribs embedded in the concrete and installation of localized steel lining where required. Changes of lining systems, e.g., from unreinforced concrete to steel linings, is also undesirable, as high incremental costs are associated with such changes. However, special lining treatments and changes in lining type are often necessary as a result of localized ground conditions. It is also important to protect the fresh concrete from groundwater inflows during placement of the concrete lining to reduce the frequency of circumferential shrinkage cracking.

Excavated Tunnel Diameter

The excavated tunnel diameter must accommodate all anticipated types of primary support, while allowing sufficient clearance for installation of any lining system that is anticipated along the tunnel alignment. This dimension should be kept to a minimum, but is most efficiently left to the actual contractor. This is because the contractor has numerous construction considerations that are related the excavated diameter, such as excavation methods, ventilation, transportation and costs of the final lining.

Construction Methods

Feasible tunnel and shaft construction methods are assessed during design to identify any effects on structural components and overall construction costs. Excavation by tunnel boring machine (TBM) and drilling and blasting, and feasible primary support systems are evaluated at a minimum. Ground characteristics which may require consideration in structural design, or development of special lining treatment components (e.g. grouting) are also identified.

Excavation by TBM, and primary support by installation of rock dowels, are anticipated for most of the length of the MWWST. Construction shafts are anticipated to be excavated by conventional shaft sinking methods while the riser shafts will be excavated by raise boring or a combination of raise boring and slashing to minimize construction activities at ground surface.

Pressure Tunnel Design Principles

The specific principles applied to the design of a systematically concrete lined pressure tunnel such as the MWWST facility is discussed briefly in the following paragraphs.

Vertical Alignment

A preliminary vertical alignment of a proposed tunnel is required early during design to provide depths for exploratory borings. To establish this depth for a pressure tunnel several important preliminary design criteria, need to have been already established:

- Design HGL
- Tunnel Horizontal Alignment
- Ground Surface Profile

- Overburden Depth (Range)
- Presence of any “Buried Valleys”
- Locations of Construction Shafts

The minimum depth envelope along a proposed concrete lined or unlined pressure tunnel is established to avoid hydraulic jacking in the rock mass at any location along the tunnel. In flat or gently sloping terrain, and assuming that vertical stress is critical, this is accomplished by application of the “0.5 Cover Criterion” as follows:

$$CR = \frac{TOR - INV}{HGL - INV}$$

where:

where:	CR	=	cover ratio (0.5)
	TOR	=	top of rock elevation (modified to account for overburden depth if needed)
	INV	=	elevation of the tunnel invert
	HGL	=	elevation corresponding to the Static Hydraulic Gradeline

This expression may also be summarized in the following statement: “The internal pressure head must be balanced in the vertical direction by half this amount of rock cover.” Application of this criterion results in an apparent factor of safety against the occurrence of hydraulic jacking of approximately 1.3, assuming prevailing densities for most igneous and metamorphic rock materials. As the results of geotechnical characterization become known, e.g., detailed knowledge about hydraulic jacking behavior in the rock mass, and variations of depth of overburden soils, the required minimum depth of a proposed tunnel may be somewhat reduced (or increased) locally.

Following establishment of minimum pressure tunnel depths at topographically critical locations, constructability considerations such as construction shaft location, invert gradients, tunnel heading lengths, and efforts to minimize shaft depth, are all incorporated to complete the preliminary vertical alignment.

Final Lining Design

The design of a systematically concrete lined pressure tunnel requires several important considerations during preconstruction design development, and continuous verification throughout the shafts and tunnels during excavation. Identification of any areas requiring augmentation of the systematic lining by installation of appropriate localized, special lining treatment is required as excavation proceeds. The final step of design may be considered to be verification of performance by pressure testing on completion of construction of the facility. The most significant considerations involved in design of the systematic lining, implementation of special lining treatment components, and performance testing are summarized in the following paragraphs.

Tunnel and shaft final linings may be broadly characterized as impermeable, semi-per-

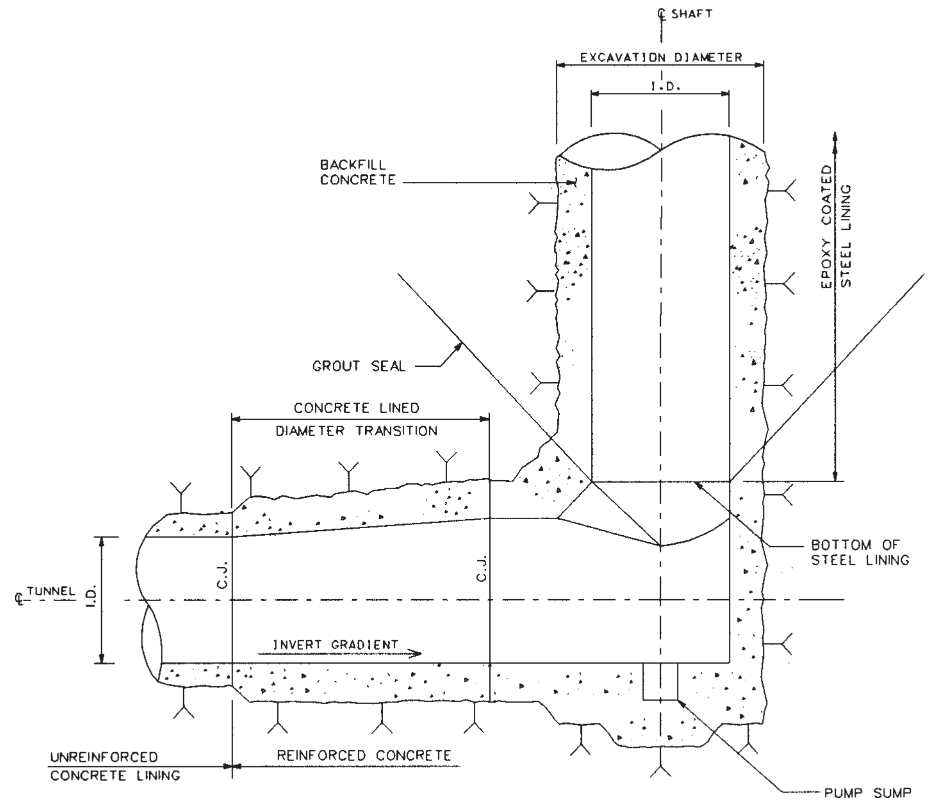

Figure 4. Typical Shaft.Tunnel Intersection

meable or permeable. Impermeable type pressure tunnel linings include jointed steel cylinder pipe linings and concrete cylinder pipe (PCCP and RCCP). These types of lining components are manufactured off -site in segments (sections or cans) and joined in the tunnel or shaft by welded or gasketed joints. Leakage from these types of linings is minimal and concentrated to the field joints. Extensively reinforced, cast-in-place concrete linings may be considered semi-permeable. Occasionally, nominally reinforced linings are installed to reduce infiltration in low pressure sewer tunnels, e.g., CSO tunnels. The distribution of circumferential (transverse) cracks is ameliorated by the introduction of longitudinal reinforcing steel. Nominal reinforcing is installed in pressure tunnels only to control and distribute cracking in areas of low modulus of deformation or in areas of stress concentration, e.g., at conduit intersections, to preserve structural integrity. An unreinforced or nominally reinforced concrete lining may be classified as a permeable lining. The latter types of linings are installed mainly to provide a smooth conduit surface, to protect and support the rock and to facilitate future inspection and maintenance work.

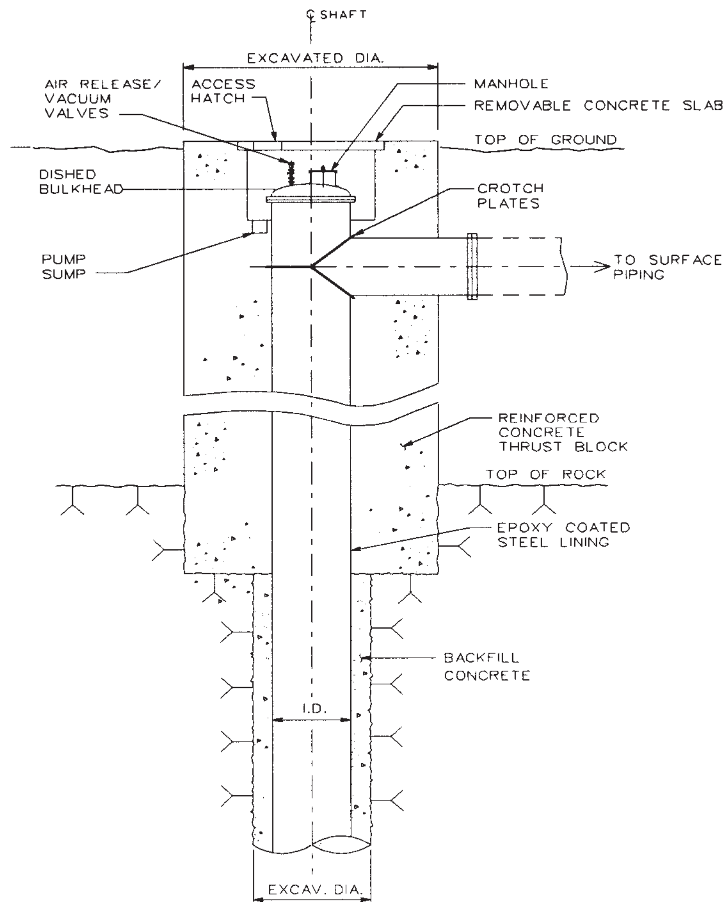

Figure 5. Typical Top of Shaft Structure

Because of the ongoing engineering evaluations during construction, the construction contract must contain mechanisms to deal fairly and efficiently with the required flexibility to install the various linings which are dictated by the actual ground conditions. This requirement is not unique to requirements for final linings in pressure tunnels, but is commonly used to handle tunnel primary support requirements. Accordingly, appropriate alternate final lining types are developed during preconstruction design with estimated quantities provided for bidding. Contractual provisions are employed to delay procurement of tunnel lining system components (e.g. reinforcing steel and steel linings), until the extent of the various lining types has been determined.

SHAFT STRUCTURES: Shaft linings must be of the impermeable type to at least the minimum depth required by the cover criterion. The extent of impermeable lining in a specific shaft also depends on other considerations, including in situ permeability as observed by testing and/or mapping information obtained during excavation. Impermeable linings in shafts are almost exclusively welded steel linings, with internal epoxy coating for corrosion protection. Reliance on ground/lining interaction is commonly not done in design of shaft steel linings. Shaft linings at lower elevations may consist of unreinforced or reinforced concrete as appropriate. Shaft/tunnel intersections may be lined with nominally reinforced concrete, with large diameter steel pipe bends, or constructed as a tee, as required for a specific shaft. All three types of intersections exist in various shafts along the MWWST facility. An example of a water tunnel/shaft intersection is shown in Figure 4.

The top-of-shaft structure must provide thrustblock functions to stabilize the structure vertically due to uplift, and horizontally due to the change of conduit direction on pressurization of the system. Accordingly, the top-of-shaft access manhole is structurally a “blockout” in an otherwise massive concrete thrust block, which is commonly reinforced in the vertical direction for continuity and to provide lateral bending strength. A large diameter, steel pipe tee is commonly embedded in this concrete at the top of the shaft with a horizontal branch for connection to surface level piping, a vertical branch facing upward and provided with a removable bulkhead for future access, and a second vertical branch facing downward and connected with the shaft conduit. A schematic illustration of a top-of-shaft structure is shown on Figure 5.

STEEL LINING: The thickness of steel linings for tunnels and shafts must satisfy three basic design criteria and would be the largest required by:

- Analysis based on internal pressure
- Analysis based on external pressure (buckling)
- Minimum thickness required for handling during construction

Earlier designs commonly added an allowance in thickness for corrosion considerations. Currently, corrosion protection systems are included where high risks for stray currents are present, as recommended in the AWWA M11 standards for steel pipes. Steel linings in tunnels are commonly coated internally with a field applied cement mortar lining in the order of 12.7 mm (1/2-in.) to 19.0 mm (3/4-in.) thick. The alkaline environment from the internal cement mortar lining and external backfill concrete provide the primary corrosion protection system for the steel lining.

The steel lining sections are installed and backfilled externally with concrete of strengths ranging from 10.3 MPa (1,500 psi) to 17.3 MPa (2,500 psi). This backfill concrete is required to have very low shrinkage characteristics and is installed to stringent construction quality requirements.

Analyses based on internal pressure and the common thin cylinder equation, is applied where load sharing (ground interaction) is not considered:

$$t_s = \frac{p_i d}{2 f_s}$$

where: t_s = thickness of the cylinder wall
p_i = internal pressure
d = internal diameter
f_s = allowable hoop stress

Thickness arrived from internal pressure analyses is required to be verified for both static pressure and total pressure including transient pressure.

Numerous closed form analysis approaches and formula have been presented during the last several decades to model load sharing between the steel lining, backfill concrete and surrounding rock to provide more economical designs of steel pressure tunnel linings. Available space prevents a detailed discussion of these methods of analysis in this paper. The various methods are summarized by Brekke and Ripley, 1987, and by Moore, 1989.

As in the case of most steel lining components designed for the MWWST facility, steel lining thickness requirements are generally governed by external pressures and the potential for buckling during future unwatering of a particular conduit. Two methods of analysis are currently available for plain cylinder steel linings, by Amstutz, 1970, and Jacobsen, 1974. Both methods are based on prevention of single lobe buckling of an embedded steel cylinder and apply a factor of safety of 1.5 against buckling. In this type of analysis a gap is assumed to exist between the outside surface of the steel lining and the surrounding backfill concrete. This assumed gap allows deformation of the steel lining in response to external pressure, forming a lobe whose stability is analyzed. The magnitude of this gap, which is assumed to account for shrinkage of the backfill concrete and thermal deformations in the steel lining itself, must be the minimum realistically achievable. This results in the requirement of a high quality, low shrink backfill concrete to fill the annular space between the steel lining and the excavated rock surfaces. An illustration of the assumed steel lining shape used in buckling analysis is shown in Figure 6.

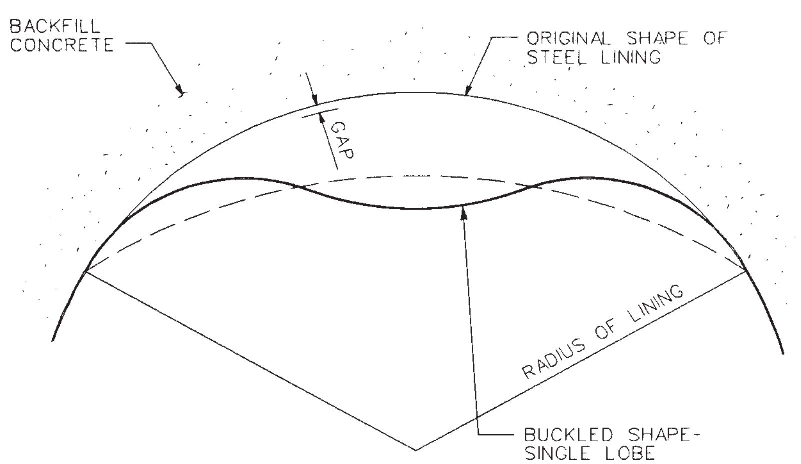

Figure 6. Single Lobe Buckling Principle

Amstutz's method was used almost exclusively during the past several decades, but has been superseded by the more conservative Jacobsen method with the advent of computerized analysis during recent years. The Jacobsen method of analysis was used for design of the relatively limited steel linings in the MWWST facility. Details of analysis for steel linings due to external pressures (buckling) are found in Moore, 1990, and Stutsman, 1993.

The use of external stiffeners to minimize steel lining thicknesses is attractive for economical design of large diameter steel linings subject to very high external pressures. However, the significant disadvantages of external stiffeners, e.g., underground transportation and clearance issues, and issues related to placing high quality backfill concrete, often preclude the use of external stiffeners, except where plain steel lining thickness exceed 38 mm (1.5 in.). Where thicker steel linings are required, space for manned access must be provided between the outside of the steel lining and the rock surface.

Steel linings up to 38 mm (1.5 in.) thick may be installed using external backing bars and internal, single sided, full penetration welding, without application of post weld heat treatment. Both post weld heat treatment and manned access for welding outside the steel lining are expensive requirements which may be avoided using external stiffeners to reduce lining thickness. Pressure relief valves have also been applied in several designs to reduce the required thickness of steel linings for hydropower developments. However, relief valves are not considered reliable in water supply tunnels due to potential deterioration after decades of inactivity. Only plain cylinder pipe steel linings are provided in the MWWST facility.

Steel linings for the small diameter community riser shafts along the MWWST alignment were analyzed as free standing cylinders exposed to external liquid grout pressure during installation. The excavated shaft openings for these structures are too small in diameter to allow manned entry. Installation and welding of these linings are made entirely from the ground surface.

A minimum steel lining wall thickness must be provided to allow practical handling of lining sections during manufacturing, transportation and installation. Two commonly applied rules to assess minimum plate thickness are:

$$t = \frac{D}{288} \quad \text{(PG\&E Rule, D} \leq \text{1.5 m.)} \qquad t = \frac{D + 0.5}{400} \quad \text{(Bu.Rec. Rule, D} \geq \text{1.5 m.)}$$

where: D = internal diameter (m)
t = minimum wall thickness (m)

Appropriate stulling and blocking must be applied during handling and installation of the steel linings to prevent damage. Design for minimum lining thickness was not applicable for the MWWST facility components, as external pressure governed the design.

Anchor rings to ensure positive anchorage of the steel linings to the concrete in the top-of-shaft structures, and seepage rings at the bottom of steel linings, were included in all shafts of the MWWST facility. General guidelines for design of these components are provided in Stutsman, 1993.

Economical, commonly available, fine grained, carbon steels and adequate material toughness are required for pressure tunnel steel linings. For the MWWST facility two alternate steels are allowed; A516 Grade 60 and A537 Class I. Either of these steels may be installed as long as the minimum thickness, shown on the drawings, is provided. Fabrication, installation and quality control specifications are required to be in accordance with applicable AWWA, AWS and ASME Boiler and Pressure Vessel Code requirements.

UNREINFORCED CONCRETE LINING: Unreinforced concrete linings are commonly designed for external water pressures, any applicable rock loads, and compressive strength requirements for an unwatered tunnel conduit. A minimum thickness of 0.3 m (12 in.) of 27.6 MPa (4,000 psi) concrete is required for the MWWST facility. Appropriate specification requirements govern mix shrinkage characteristics, protection of fresh concrete from groundwater during placement, and other construction quality issues. Contact grouting of all potential void areas between concrete and rock is also required.

In areas where permeability of the host rock, is of concern for the performance of the tunnel, as observed by mapping of the tunnel and monitoring of groundwater infiltration during excavation, the tunnel lining will be provided with reinforcing steel to control leakage. The amount of reinforcing steel will be determined by the assessed modulus of deformation and applicable internal pressure at the specific location. Discussions of design analysis applying ground/lining interaction for reinforced concrete linings are provided in Moore, 1989.

Locally, the completed, reinforced concrete lining may be further augmented with selective consolidation grouting, as determined to be required during the final lining determination process. At locations in the tunnel where performance of the final lining is determined to be of particular concern based on observations of ground conditions and applicable internal pressure, the concrete lining may be substituted with a limited length of steel lining.

On completion of the MWWST facility concrete lining, some infiltration of water through the lining is anticipated. This infiltration is generally associated with circumferential (transverse) shrinkage cracks and construction joints. The anticipated spacing of circumferential shrinkage cracks in a well constructed cast-in-place unreinforced concrete lining may range from 5 m (15 ft) to 8 m (25 ft). The spacing of construction joints in a lining may range from 46 m (150 ft) to 67 m (220 ft). Pressure grouting at relatively high pressures is prescribed at visible, leaking cracks and joints in the tunnel lining to meet stringent infiltration criteria.

TUNNEL MAPPING AND FINAL LINING DETERMINATION: Design of a systematically unreinforced concrete lined pressure tunnel continues through the excavation phase of construction. It is essential that the entire length of a pressure tunnel is mapped during tunnel excavation by a qualified engineering geologist who records all observations which are of importance for selection of appropriate final lining system components, including:

- Joint Spacings and Orientations
- Joint Aperture/Infillings
- Characteristics of Shears/Shear Zones/Fault Zones
- Rock Durability Information

- Weathering/Alteration Characteristics
- Groundwater Occurrence
- Rock Stability Information/Primary Support Installed
- Rock Type

The mapping information is used by the design engineer to verify design assumptions, and where required, to determine the appropriate lining system augmentation.

Groundwater occurrence information is of particular importance for pressure tunnel lining selection because of its value as an indirect measure of rock mass permeability and hence a measure of potential leakage from the tunnel. Groundwater occurrence requires observation and record keeping throughout the length of a tunnel and at specific locations, as it appears immediately following excavation and periodically as it dissipates with time following excavation.

For the MWWST facility a specific procedure has been established to select the final lining required at any location in the tunnel. The construction manager will map the tunnel. Following review of the tunnel mapping information by the design engineer, areas exhibiting characteristics of concern will be assessed in more detail. It is intended to complete determination of final lining requirements at selected points of progress of excavation in each heading. (e.g., at each one third point). After this initial information is reviewed, the need for in situ testing will be assessed. On completion of these assessments, appropriate final lining components/treatment will be selected for the portion of the heading under examination.

Performance Testing

The MWWST facility will consist of three separate, complete pressure tunnel loops which are inter-connected at the ground surface, where these are also connected to the parallel Hultman Aqueduct transmission system. On completion of the tunnels and shafts, these loops will be filled with water and pressure tested separately. Limited and controlled rates of pressure increase/decrease during filling, testing and unwatering of each complete tunnel loop is intended to limit the overall rates of change of pressure gradients across the final lining components, and thus promote distribution of cracking in concrete linings, and dissipation of external pressures behind steel linings.

The first phase of pressure testing is the filling of the tunnel and shaft conduits. This phase will be followed by “over-pressurization” of the system, i.e., pressure increase beyond complete filling of the loop conduits. For the MWWST facility, the rate of steady pressure change has been limited to pressure changes corresponding to 6 m (20 ft.) of head per hour during all stages of testing. A constant pressure holding period of 14 hours will be introduced at the end of each 60 m (200 ft.) increment of change of head. Also, a also limiting total head change of 60 m (200 ft.) in a 24 hour period is required.

At full test pressure, a holding period of 14 days will be maintained. The steady flow rate into the loop at the end of the 14 day test period may be considered close to the steady leakage rate out of the tunnel loop, although leakage from pressure tunnels has occasionally increased with time due to dissolution or erosion of joint infillings, or because of hydraulic jacking. If the observed rate of leakage is found to be unacceptable, additional final lining work, e.g., consolidation grouting, may be required within the specific tunnel loop. On completion of pressure testing, all conduits will be disinfected prior to commissioning and operation of the MWWST facility.

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