Appendix C2. Conceptual Tunnel Lining Evaluation (Final Draft)

1. Introduction and Purpose

This technical memorandum (TM) was initially prepared to document supported details for the Delta Conveyance Project (Project) Project Engineering Reports (EPR) (DCA, 2022a, 2022b). At that time of submittal in 2022, the Delta Conveyance Authority (DCA) prepared two Engineering Project Reports, one report with: the Central Corridor and Eastern Corridor and one report with the Bethany Reservoir Alternative. In December 2023, the Environmental Impact Report (EIR) (DWR, 2023) was released and stated that the Bethany Reservoir Alternative would be the selected Project and renamed the Bethany Reservoir Alignment.

In September 2024, this TM, as part of the Concept Engineering Report (CER), was updated to describe the Project selected by DWR, the Delta Conveyance Project Bethany Reservoir Alignment. No technical changes are presented since Final Draft Submittal in 2023. It should be noted that the term "Central Corridor" is no longer a part of the Project and the terms "Eastern Corridor "or "East Corridor" should be here on interpreted as part of the Bethany Reservoir Alignment Project only from Intake C-E-3 down to Lower Roberts Island Tunnel Launch Shaft. It also should be noted that some references to the Central and/or Eastern Corridors remain in the TM to provide a greater extent of background information for portions of the Delta between the intakes and Clifton Court Forebay which also influence design considerations for the Project.

The Bethany Reservoir Alignment tunnel upstream of the Bethany Reservoir Pumping Plant (BRPP) would be excavated at a depth ranging from 105 to 170 feet below ground surface (bgs) at the tunnel crown (top of the tunnel) for the 36-foot-inside-diameter tunnel, based as shown on the invert tunnel elevations shown in the Bethany Reservoir Alignment Engineering Concept Drawings (DCA, 2024a). Along the alignment, the groundwater level ranges from 5 to 10 feet bgs, and the groundwater at any location is assumed to be connected vertically within a single aquifer. The external hydrostatic pressure on the tunnel ranges between 3.0 bar (43.3 pounds per square inch [psi]) at the tunnel crown to 6.0 bar (86.6 psi) at the tunnel invert (bottom of the tunnel) for the 36-foot-inside-diameter tunnel.

Hydraulic models were utilized to assess the hydraulic grade line (HGL) throughout the entire system, encompassing the intakes, tunnel, and pumping plant, as detailed in the Concept Engineering Report (CER) Appendix A2 *Hydraulic Analysis of Delta Conveyance Options.* The results from the hydraulic models indicate a maximum surge pressure elevation of approximately 36 feet for a 36-foot inside diameter tunnel with a design flow capacity of 6,000 cubic feet per second (cfs) and a tunnel flow velocity of 6 feet per second (fps). The model results indicate that the maximum surge pressure occurs at the Union Island Maintenance Shaft.

When soil confinement is neglected, the external pressure is equivalent to the full hydrostatic head, resulting in a maximum differential water pressure of approximately 22 psi in tension. As part of a previous analysis, the maximum differential water pressure was calculated for the Central Corridor as 25 psi with the proposed tunnel lining assumptions. For the Project, given that the maximum differential water pressure is lower than the computed maximum of 25 psi for the Central Corridor; therefore, no additional analysis is needed. The proposed conceptual design outlined below for the Central Alignment could also be adapted for the Bethany Reservoir Alignment tunnel. More refined design analysis will be conducted for the tunnel lining during future Project design phases.

1.1 Purpose

The purpose of this technical memorandum (TM) is to summarize the results of a preliminary conceptual tunnel lining evaluation for the Project using precast concrete segmental tunnel liners for initial and final lining. The preliminary results contained in this memo will be used to establish the tunnel footprint, assist in logistical support, cost estimates and traffic impact studies.

This preliminary evaluation is performed on a 36-foot ID tunnel to reflect the 6,000 cfs capacity case that is proposed in the for the DCP. The segmental tunnel lining must be designed to handle several loading conditions, including:

- Handling and installation loads (or construction loads)
- Internal water pressures (net pressures in excess of groundwater pressures)
- External soil and groundwater pressures
- Seismic forces

This preliminary evaluation focuses on determining the feasibility of using segmental tunnel lining for this tunnel and identifying the possible loading conditions that could impact design based on a preliminary operating HGL and preliminary geotechnical condition assumptions.

It is not anticipated that the seismic criteria and demands would control the lining thickness due to high groundwater pressures. Therefore, the seismic evaluation of the tunnel lining would be evaluated in a separate TM.

1.2 Organization

This TM is organized as follows:

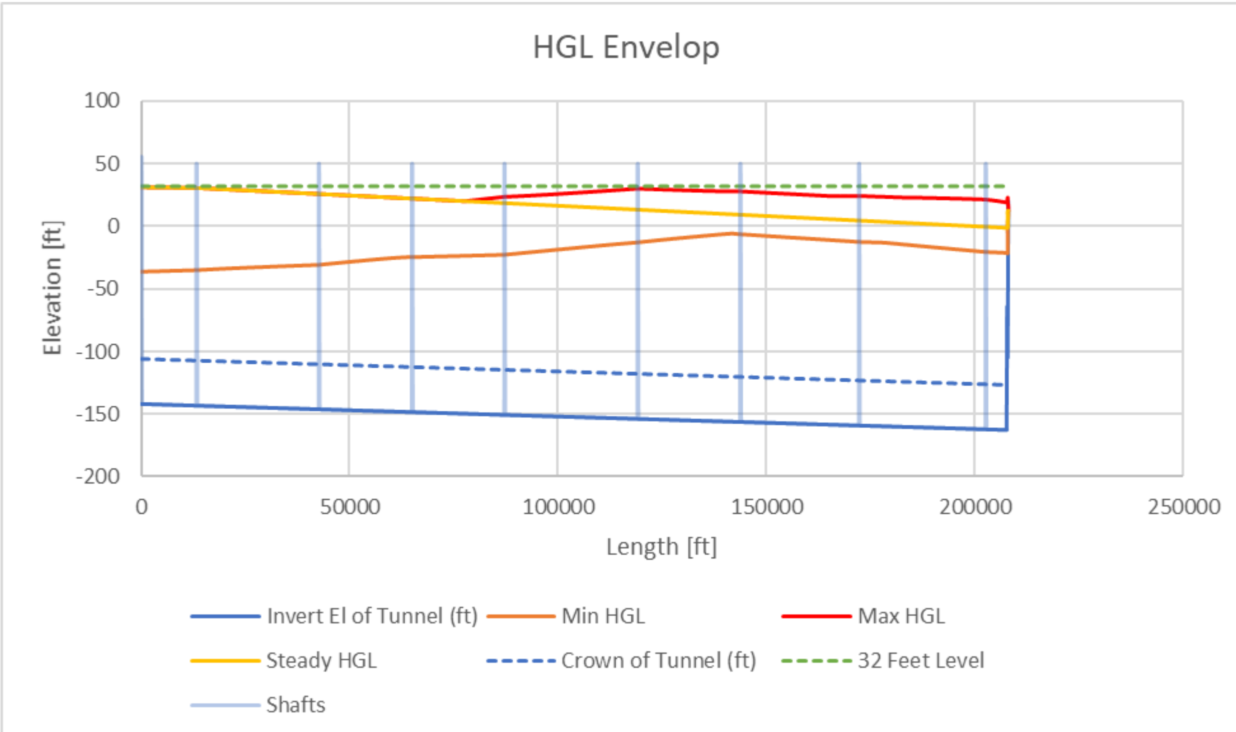
- Introduction and Purpose
- Tunnel Depth, Operation Pressures, and Geotechnical Conditions
- Feasibility of Precast Segmental Tunnel Lining
- Conclusions
- Recommendation for Additional Evaluations
- References
- Attachment 1 – Figures

2. Tunnel Depth, Operation Pressures, and Geotechnical Conditions

2.1 Tunnel Depth and Operating Pressure Assumptions

This evaluation assumes that the tunnel would be excavated at a depth of 98.6 to 119.2 feet below ground surface (bgs) at the tunnel crown (top of the tunnel) for the 36-foot ID tunnel based on the invert tunnel elevations as shown in the DCA Volume 2 – Drawings (2024a). Along the alignment, the groundwater level ranges from 5 to 10 feet bgs, and the groundwater at any location is connected vertically within a single aquifer. The external hydrostatic pressure on the tunnel ranges between 2.7 bar (39.psi) at the tunnel crown up to 4.2 bar (60.9 psi) at the tunnel invert (bottom of the tunnel) for the 36-foot ID tunnel.

Hydraulic models were run to evaluate the HGL in the entire system in the intakes, intermediate forebay, tunnel, pumping plant, and southern forebay, as CER Appendix A2. The hydraulic models indicate that a maximum surge pressure elevation of approximately 32 feet will be reached as shown on Figure 1. The HGLs presented on Figure 1 are based on a 36-foot ID tunnel with a design flow capacity equal to 6,000 cfs, a tunnel flow velocity of 6 feet per second (fps).



Source: (DCA, 2021)

Figure 1. Hydraulic Envelope, 36-foot Internal Diameter Tunnel

For the tunnel leakage analysis presented in Section 3.5, the system hydraulic model was run for the Project using the median diversion flow for each month of the year from the CALSIM 3 runs for the Project. The CALSIM 3 runs were provided to DCA by DWR on March 25, 2021. The resulting hydraulic grade line was established at each tunnel shaft and each month of the year and used to estimate the internal pressure in the tunnel for each reach of the tunnel system.

2.2 Geological Conditions Assumptions

Based upon information provided in the *2018 Conceptual Engineering Report* (CER) (DWR, 2018), it is anticipated that the tunnel and shafts would be excavated in saturated soft ground conditions. Based on the data previously collected within the potential tunnel alignments or corridors and the anticipated depth of the proposed tunnel, it is expected the soil deposits around the tunnel would consist of clays, silts, silty and clayey sands, and clean sands (DWR, 2018). According to Seed (1979) evidence of liquefaction has been observed at depths of less than 45 feet, with the groundwater table at depths less than 15 feet which is above the tunnel profile given in the CER. Additionally, some organic materials, primarily peat, could be encountered near the ground surface during shaft excavation. This information was based on a limited number of borings (DWR, 2018) and would need be confirmed by future field investigations. It is expected that the geology would vary over the very long tunnel alignments.

3. Feasibility of Precast Segmental Tunnel Lining

This feasibility analysis was based upon a review of the following parameters:

- Comparable tunnel projects
- Segmental lining geometry
- Ground and groundwater conditions
- Concept design-level methodology
- Tunnel boring machine (TBM) jacking loads
- Joint design considerations
- Durability and service life

3.1 Review of Comparable Tunnel Projects

Tunnel projects excavated in soft ground conditions with similar IDs to the subject tunnel were reviewed to evaluate construction feasibility. Since about 2010, Herrenknecht has manufactured approximately 48 similar TBMs with IDs equal to or greater than 36 feet, as summarized in Table A-1 of Attachment 1. Other manufacturers (for example, Robbins, NFM, Mitsubishi, Kawasaki, and Hitachi) have also produced TBMs of this size in the past 15 years.

3.2 Segmental Lining Geometry

This tunnel lining evaluation was based on the assumptions listed in Table 1 and for the geometric configuration, shown on Figure A-2 in Attachment 1.

Table 1. Geometric Configuration for 36-foot ID Tunnel

Description	36-foot ID
Segment Thickness (inches)	18
Segment Width (feet)	6
Number of Segments	7+ key
Weight of Each Typical Segment (tons)	10.9
Weight of Key Segment (tons)	5.8
Weight of Typical Segment Ring (tons)	82.1

3.3 Anticipated Loading Conditions

The precast concrete segmental tunnel lining acts as the initial and final tunnel support; therefore, critical loading conditions were assessed for several scenarios, including:

- Loading during construction
- Empty tunnel conditions (both for initial cases and during maintenance)
- Operating conditions (maximum internal pressure at the location of the least external pressure)

The principal load combinations considered in this evaluation include:

- Ground loads and external water pressure (during construction and after dewatering)
- Long-term ground loads and external water pressure combined with internal water pressure

During final design, these load cases and others would be evaluated in greater detail. However, for this conceptual evaluation, the focus is on developing a solution to support the differential internal water pressure using reinforcing steel and checking that the basic structural requirements could be accommodated.

3.3.1 Soil Confinement

While it is unlikely that full overburden would act on the lining, it is reasonable to expect that there will be some effective soil pressure acting on the lining. The TBM would control inward displacements to an extent to control settlements, and the soil would not be self-supporting in the long term, if at all. Based on previous analyses that determined the minimum pressure on the lining, including those made in the analysis of minimum soil loads on the Blue Plains Tunnel (Harding et al., 2014), it is likely that at the buoyant weight of soil at least half a tunnel diameter above the tunnel would act on the tunnel crown (which equates to roughly 8 psi at the tunnel crown).

AASHTO – LRFD Road Tunnel Design and Construction Guide Specifications (AASHTO, 2017) states for soft ground tunnels when the height of ground directly over the tunnel crown is greater than two times the excavated width of the tunnel, the minimum vertical pressure (EV) shall be the pressure resulting from a height of soil equal to two times the excavated width of the tunnel. Based on this reference the buoyant weight of soil would equate to 32 psi at the tunnel crown.

The tunnel lining evaluation contained in the CER considered the worst load case and ignored external ground load in the conceptual tunnel lining design analysis. The report further states the ground is expected to exert significant pressure on the lining long-term and that some loading contribution from the ground must be evaluated during final design.

Based on the above, the lining evaluation for the DCP tunnel includes both soil confinement and also discusses results when soil loads are neglected.

3.3.2 External Groundwater Pressure

The DCP tunnel would be located below the groundwater table and would experience external pressures caused by the head of groundwater above the tunnel. Along the alignment, the depth to groundwater level varies and ranges from 5 to 10 feet bgs as shown on Figure A-1e and the groundwater at any location is connected vertically within a single aquifer. Therefore, the external hydrostatic pressure varies along the tunnel and would range between 2.7 bar (39.2 psi) at the tunnel crown up to 4.2 bar (60.9 psi) at the tunnel invert (bottom of the tunnel) for the 36-foot ID tunnel.

3.3.3 Internal Operating Pressure

For this assessment, internal operating pressure has been determined based on the maximum HGL. Based on the HGL for a peak flow of 6,000 cfs, the maximum HGL would range from EL 21 to EL 32 distributed along the tunnel, as shown on Figure 1. The DCP main tunnel would be subject to a maximum internal operating hydrostatic head of 165 feet, equivalent to 71.5 psi internal operating pressure.

3.3.4 Net Pressure Assumption

Based on the confinement description in this TM and internal operating pressure assumptions, the lining design would vary along the tunnel alignment due to site-specific groundwater conditions. The net pressure theory was used to identify reaches with infiltration and exfiltration risk. This net pressure theory is based on the premise that the net groundwater pressure acting on the tunnel is the difference between the external groundwater pressure and the internal hydrostatic pressure. The net pressure could range for each of the three tunnel diameters evaluated, as shown on Figures A-1a to A-1d in Attachment 1.

3.3.5 Seismic Performance

The West Tracy Fault (WTF) is the is the only known active fault that crosses the tunnel alignments analyzed for the DCP; and seismicity, earthquake-induced ground shaking and fault rupture must be considered during the tunnel lining design. Ground shaking refers to the movement of the ground due to seismic waves traveling through the ground as a result of an earthquake. Ground shaking usually does not result in structural failure of modern and well-constructed tunnels, provided the lining is in continuous contact with the surrounding ground. A tunnel in continuous contact with the ground would typically experience the same strains as the surrounding ground during shaking. Earthquake-induced ground shaking can also cause liquefaction to occur in loosely compacted soils under the water table, which can result in excessive ground settlement and damage to structures. Liquefaction would not be a major issue for this tunnel, as it is expected to be located beneath the potentially liquefiable soils and will be verified in the future when site specific data becomes available.

Preliminary findings associated with fault movements have been addressed in the West Tracy Fault Displacement Hazard Analysis TM (DCA, 2021b). There are numerous proven designs and construction means and methods to safely build a tunnel through fault zones. During a seismic rupture, the tunnel would experience deformations and changes in external ground and external water pressures. In this case, the tunnel lining would be designed to be flexible to accommodate the fault rupture displacement and to be strong enough to withstand the external ground and water pressures. In order to accommodate the displacement and withstand the pressure changes, the following would be considered and potentially implemented for the lining design:

- Over-excavating the tunnel and backfilling the annular space outside the lining with compressible or collapsible material would provide a flexible tunnel lining in the zone of potential fault rupture.
- Steel segmental lining would provide ground support that is stiff enough to resist external (ground and water) pressures induced by earthquake but ductile enough to tolerate seismically imposed movements within the fault zones.
- Doweled or bolted joint connections for steel or concrete segmental linings in the zone of potential fault rupture deformation can be designed with special consideration for displacement/deformation capacity.

The location, width, and of style of faulting along the WTF at the tunnel crossing location are poorly known and additional investigations would be needed to more accurately define the extent and nature of the fault zones and their soil properties. Lining design to accommodate fault displacements such as those described above are not without precedent and practical solutions are available and would be further evaluated during future design phases.

3.4 Concept Level Design Methodology

3.4.1 Ground-Lining Interaction Analyses

At the conceptual stage, closed-form ground-lining interaction analyses were performed using the method developed by Peck (1969) and Ranken et al. (1978). In the Ranken method, moments and thrusts are a function of the following:

- The stiffness properties of the lining (including thickness, Young's modulus, Poisson's ratio, and number of joints)
- The stiffness properties of the surrounding medium (primarily Young's modulus and Poisson's ratio)

The Ranken method also has sets of equations for the condition of "full slippage" (also called no shear), when the shear stress around the perimeter of the lining is zero – and for "no slippage" (also called full shear), when the relative tangential deformation between the lining and the ground is set at zero. Both the full slippage and no slippage conditions were considered for the load cases described in Section 3.3 The calculations are provided in Attachment 1.

3.4.2 Concrete Strength and Reinforcement

The moment-thrust interaction diagrams developed for this analysis used a concrete strength of 6,000 psi and a total steel area of 4.71 square inches on each face of the lining which requires two layers of #8 bars at 12-inch spacing. The concrete section has been based on an 18-inch-thick segment shown in Table 1. Reinforcement details used in our evaluations have been based on the tunnel lining details provided in the Drawings (2024a).

3.4.3 External Loading Results

This section summarizes the results of the tunnel lining performance analyses when the lining is subjected to static loading conditions only (that is, no seismic loading). Due to in situ stress variability, two case studies were evaluated using K_o = 0.5 and K_o=0.75. The interaction diagram does not take into account internal water pressures. The expected combinations of thrust and moment (with load factors) were plotted on a moment-thrust diagram to confirm that the section can resist anticipated loads. Moment-thrust interaction diagrams were developed using American Concrete Institute (ACI) 318 code requirements.

Load combinations that plot to the left and within the capacity envelope are within acceptable limits; loads that plot to the right and are outside the capacity envelope indicate expected failure of the section per ACI code. Results from the analyses are summarized in Table A-2 and are also plotted on moment-thrust interaction diagram on Figure A-3.

3.4.4 Net Internal Pressures Results

Based on the external loading and internal operating pressures discussed in Section 3.3, the tunnel could be subjected to tension pressures, as shown on Figures A-1a to A-1d, in Attachment 1. The analysis was performed on a 36-foot ID tunnel assuming both soil confinement and neglecting any soil load contribution as discussed in Section 3.3.1.

When soil confinement is neglected the external pressure is equivalent to full hydrostatic head and this would result in a maximum differential water pressure of approximately 25 psi in tension based on the max HGL shown on Figure 1. The preliminary conceptual design for this case confirms that an 18-inch thick precast gasketed segmental lining (6,000 psi concrete) to serve as the initial support. The hoop tension requires two layers of # 8 bars at 12-inch spacing, and 1.375-inch diameter bolts at the radial joints similar to what is shown on concept drawings contained in the CER. The calculations are provided in Attachment 1.

When soil confinement is assumed the external pressure in our analysis includes full hydrostatic head and ground pressure equivalent to ½ tunnel diameter and this would result in a maximum differential water pressure of approximately 18 psi in tension based the max HGL shown on Figure 1. The preliminary conceptual design results in an 18-inch thick precast gasketed segmental lining (6,000 psi concrete) to serve as the initial support. The hoop tension requires two layers of # 8 bars at 12-inch spacing, and 1.375-inch diameter bolts at the radial joints.

We also evaluated the tunnel lining under normal operating conditions utilizing the design HGL (solid blue line) shown on Figure A-1e. The tunnel could be subjected to a maximum differential pressure of 3.9 psi in tension as shown on Figure A-1d, in Attachment 1. Based on the reinforcement used for the max HGL case this will result in a safety factor greater than 4 under normal operating conditions with no ground confinement.

Typically, precast concrete segmental liners are designed only for compression pressures. The precast concrete segmental lining also would have to resist the load from the internal pressure without a secondary lining. There are a few tunnel projects that have been designed in the past where internal pressures had to be accommodated using the same construction procedures anticipated for the DCP tunnel. For example, on the South Bay Ocean Outfall, the tunnel was constructed in 1995 with a one-pass precast gasketed concrete segmental liner and designed for 43 psi of net internal differential pressure head (Kaneshiro et al., 1996). Other examples where one-pass tunnel liners have been used include projects in Japan (e.g., Saitou et al, 1999).

Assuming no ground confinement to counterbalance the internal operating pressures results in a conservative design approach. As discussed above, AASHTO states the minimum vertical pressure (EV) shall be the pressure resulting from a height of soil equal to two times the excavated width of the tunnel when excavated in soft ground. In our analyses, we have assumed vertical pressures equal to ½ tunnel diameter which is 75% less than what is recommended by AASHTO. During final design the amount of load contribution from the ground must be evaluated so that costs can be minimized.

Based on review of similar projects, this analysis assumes a one-pass precast gasketed concrete segmental liner could be used for the expected loading conditions. On similar past projects, various structural systems have been employed to resist tensile stresses, including continuous steel reinforcement throughout the lining, special details at the joints, and load sharing. Given the anticipated range of net internal hydraulic head expected for the Bethany Reservoir Alignment tunnel, a precast concrete segmental lining would be feasible from a technical perspective.

3.5 Leakage Estimate

One of the critical design issues for the tunnel lining system is determining a feasible watertightness design for the tunnel that can withstand the external groundwater pressures on the tunnel, but also the internal water pressure. An assessment of the maximum expected leakage is summarized herein based upon external pressures and net internal pressures.

3.5.1 External Pressure

During construction or in the future when the tunnel is dewatered for inspection and/or maintenance the precast concrete segmental lining will only experience external pressures caused by the ground and groundwater above the tunnel. This, along with the tightening of the longitudinal and circumferential bolts, will maintain the segmental lining in compression and with the gaskets make the tunnel structure nearly watertight. Existing ethylene propylene diene monomer (EPDM) gaskets are available to provide a near watertight lining. EPDM is a synthetic rubber compound that is stiff, strong, inert stable and typically has a design life of 100+ years, and the durability and effectiveness of the gasket is proven in accelerated age testing that is performed by the manufacturer.

In theory, leakage should not occur at or below the design water pressure for a well-designed gasket. However, imperfections such as offsets between erected segments, post-installation damage, debris trapped between the gaskets, and other factors could result in minor leakage. A review of typical inflow specifications from other tunneling projects (e.g. City of Los Angeles Northeast Interceptor Sewer, County Sanitation Districts of Los Angeles County Effluent and Outfall Tunnel, San Francisco Central Subway, and D.C. Water Blue Plains Tunnel) indicates the following standards have been used for limiting water ingress into tunnels:

- Allowable Infiltration less than 0.025 gal/sf/day
- Local infiltration limited to 0.25 gallon per day for a 10-square foot area
- No water ingress that causes entry of soil particles.

Applying the infiltration rate above the water flow into the tunnel would range from approximately 40 to 60 gallons per minute (gpm) for a 4 to 6-mile-long tunnel, respectively, based upon external pressures. Tight quality control measures will need to be in place during construction to ensure a properly built tunnel lining such as dimensional checks on each ring assembled, to ensure that the tunnel diameter is within tolerance and that steps and lips between adjacent segments are minimized; and careful development and monitoring of the grout mix, pump pressure and injection volume to ensure complete filling of the annular void with a grout that provides stability to the lining and minimizes displacement after erection. Careful attention to ring build will result in a better alignment at gaskets and will avoid concrete damage, which will reduce leakage potential. If after the segmental lining has been installed, water inflow into the tunnel exceeds the specified inflow criteria remedial measures such as contact grouting would be used by the contractor to achieve the specified watertightness.

3.5.2 Net Internal Pressure

3.5.2.1 Leakage through Reinforced Concrete

This section summarizes the preliminary leakage estimates based on median diversion flows derived from the CALSIM3 model for each month over a 94-year period. As a result, the net internal pressures were computed utilizing the WSELs summarized in Tables A-3 to A-5 in Attachment 1. These pressures are an estimate of the normal pressures the tunnel system would experience over its life. In some cases, higher pressures would occur and in some cases lower pressures would occur; however leakage estimated based on the median value would estimate the typical leakage the system would experience over the long term.

Depending on the magnitude of the operating pressure, the tunnel can be subjected to net internal pressures that lead to leakage through cracks and through the body of the tunnel segments. As the pressure-induced tensile strain develops, radial (longitudinal) cracking of the lining would take place, causing permeability of the lining to increase. Because of the cracks, pressure tunnels lined with reinforced concrete are classified as semi-permeable linings. The combined effect of all the cracks in the lining determines its permeability characteristics.

Estimated leakage rates through reinforced concrete are based on the analytical method by Fernandez (1994), assuming the ground surrounding the tunnel has a modulus of elasticity and permeability of 600,000 pounds (600 kips) per square foot (ksf) and 1.6 x 10^{-6} feet per second, respectively. Water leakage computations were based on effective soil pressures equal to zero, 0.5, and 1 tunnel diameters. Assuming no ground confinement to counterbalance, the internal operating pressures results in an unrealistic conservative design approach since the ground is expected to exert significant pressure on the tunnel lining. Assuming an effective soil pressure equivalent to 0.5 tunnel diameters results in a total maximum leakage of 540 gpm for the Project, respectively. Assuming one tunnel diameter of ground confinement results in no leakage occurring in any of the tunnels since the segments will be under compression.

It is important to note that the Fernandez method, which was originally developed for cast-in-place concrete tunnel lining in rock, may overestimate the amount of leakage from a precast concrete segmental lining by ignoring the tensile strength of the concrete. Since the tensile strength of concrete is significantly less than that of the steel reinforcement, the assumption is reasonable when the internal pressure is high. However, when the internal pressure is low, like for some of the values that may be experienced during long term operations, the tensile strength of the concrete may be important in evaluating the potential for cracking of the lining. In other words, cracks may not form at low internal pressures, so the method likely overestimates the amount of leakage. Finally, the Delta Conveyance Project tunnel lining would consist of precast segments manufactured under higher quality control as compared with cast-in-place concrete lining in which the formation of shrinkage cracks is common. These assumptions would be reconsidered in later design phases.

3.5.2.2 Leakage through Gasketed Joints of the Segmental Lining

The maximum net internal water pressure during normal operating conditions excluding transient pressure for the tunnels on this Project is approximately 0.8 bar neglecting any contribution from soil loads, which is very conservative as explained above. The gasket details and the segment connections will need to be designed to ensure the gaskets remain adequately compressed when the internal water pressure is applied. The gaskets rely on being compressed together to provide a good seal, and a lining such as being considered for this Project that would have a net low loading will have to rely on a combination of a gasket that requires a low load to maintain closure. This would be accomplished by a combination of the initial tightening of the bolts and proper annulus grouting to ensure the gaskets remain compressed over the life of the Project.

In theory, leakage should not occur at or below the design water pressure for a well-designed gasket. However, imperfections such as offsets between erected segments, post-installation damage, debris trapped between the gaskets, and other factors could result in minor leakage. Because the net pressure is lower than the expected external pressure values, leakage estimates could be based on the values presented in Section 3.5.1. These values are very small compared to the leakage because of cracking of the concrete shown in Table A-3, and are therefore considered negligible for these calculations.

3.6 Other Considerations

3.6.1 Tunnel Boring Machine Jacking Loads

The total thrust generated from a TBM onto the tunnel lining segments must be considered in the design of the segments. The magnitude of the thrust required to advance the TBM is dependent on several factors, including:

- Type of ground
- Hydrostatic head of groundwater
- Diameter of the machine
- Size of overcut
- Rate of progress
- Alignment of the tunnel

For this evaluation, the TBM configuration used for the Port of Miami tunnel was assumed (Herrenknecht, 2011). Results from our analysis indicate the 36-foot ID reinforced concrete precast tunnel liner would be able to withstand the jacking loads generated by the Port of Miami TBM. Additional details about these analyses are presented in Attachment 1.

3.6.2 Tunnel Flotation Evaluation

The possibility of flotation exists when tunnels are constructed in an area with high groundwater. The buoyancy of the tunnel depends upon the weight of water for the volume displaced and is resisted by the weight of the precast segmental lining and the weight of the ground above the tunnel. The analysis was conducted as a "worst case" scenario with the tunnel assumed to be empty and not being used. A minimum ground cover of 98 feet was used (ground cover ranges from approximately 98 to 119 feet below ground surface (bgs) along the entire tunnel alignment) with the highest groundwater table at 10 feet below ground surface.

For the 6,000 cfs Project design capacity, an outside tunnel lining diameter of 39 feet was used having a wall thickness of 18 inches. The analysis used a Factor of Safety of 1.5 for soil conditions. The analysis indicated that at a minimum, 28 feet of ground cover would be needed to prevent floatation with a Factor of Safety of 1.5 which would result in a downward resisting force of the ground above the tunnel of approximately 171,700 pounds per linear feet (lbs/LF). The uplift of the empty tunnel would be 48,030 lbs/LF. Therefore, for the 6,000 cfs Project design capacity, if the depth of soil between ground surface and top of the tunnel (crown) is greater than 28 – feet, the tunnel would not become buoyant. For the 6,000 cfs Project design capacity, there would be a minimum of 98-feet of ground cover above the shallowest portion of the tunnel construction. Therefore, the flotation evaluation indicated there would be adequate forces from the ground above the tunnel to withstand flotation pressure for the Project design capacity of 6,000 cfs.

3.6.3 Durability and Service Life

At this stage of the Project, the durability of the tunnel and shaft structures have not been established because there have been no chemical analyses performed on soil samples taken from previous geotechnical investigations. Corrosion protection requirements for the tunnel would be prepared concurrently with future geotechnical investigations. The DCP tunnel would be designed for a 100-year design life. The key elements to be considered for design service life would include a minimum depth of concrete cover for conventional rebar, gasket design, and specific concrete mixes to account for corrosion design.

4. Conclusions

Conceptual static design analyses were performed for the main DCP tunnel. The tunnel lining was evaluated for the following situations:

- Anticipated variations in ground conditions
- Overburden depth
- Maximum anticipated hydrostatic pressures
- Operating conditions along the tunnel vertical alignment

The lining response analyses were carried out using closed-form solutions. Based on the results, the findings from the analyses indicated:

- The calculated moment-thrust interaction diagrams indicated that the static forces developed in the three concrete segmental lining diameters evaluated are within the capacity envelope for all load combinations considered.
- Operating conditions would be expected to generate net internal pressures, and a one-pass precast gasketed concrete segmental lining is expected to be feasible.
- The reinforced concrete precast lining would have adequate capacity to resist anticipated thrust loads generated from a TBM.
- Leakage rates based on long term operational conditions appear to be minimal when 0.5 tunnel diameter of effective soil pressure and water pressure is considered and no leakage is expected when 1.0 tunnel diameter of effective soil pressure is assumed.
- Additional geotechnical investigations, further studies, and prototype testing during final design are recommended for a thorough evaluation of the potential effects caused by exfiltration from the tunnel, and effect of ground load on reducing the net tension in the lining, and to verify that the proposed lining system with potential options, such as continuous hoop reinforcing and bolted connections, could meet all permanent loads cases. In addition, during the geotechnical investigation stage confirm if ground water pressures are under artesian pressures that can result in even higher hydrostatic pressures and potentially reducing the net tension even further.

5. Recommendation for Additional Evaluations

Additional actions and activities to help advance the tunnel lining design are described in this section. The following recommendations are not listed in any order of priority but should be considered when additional geotechnical information becomes available during the design phase.

- Re-evaluate the permanent soil loads that would be used to balance the internal pressure.
- Evaluate different joint and gasket configurations for items such as example double gaskets to protect against water exfiltration and gas intrusion, and infiltration piping of soils and groundwater.
- Advance the conceptual joint connection design details, and conduct laboratory performance testing to determine whether the leakage criteria can be achieved.

6. References

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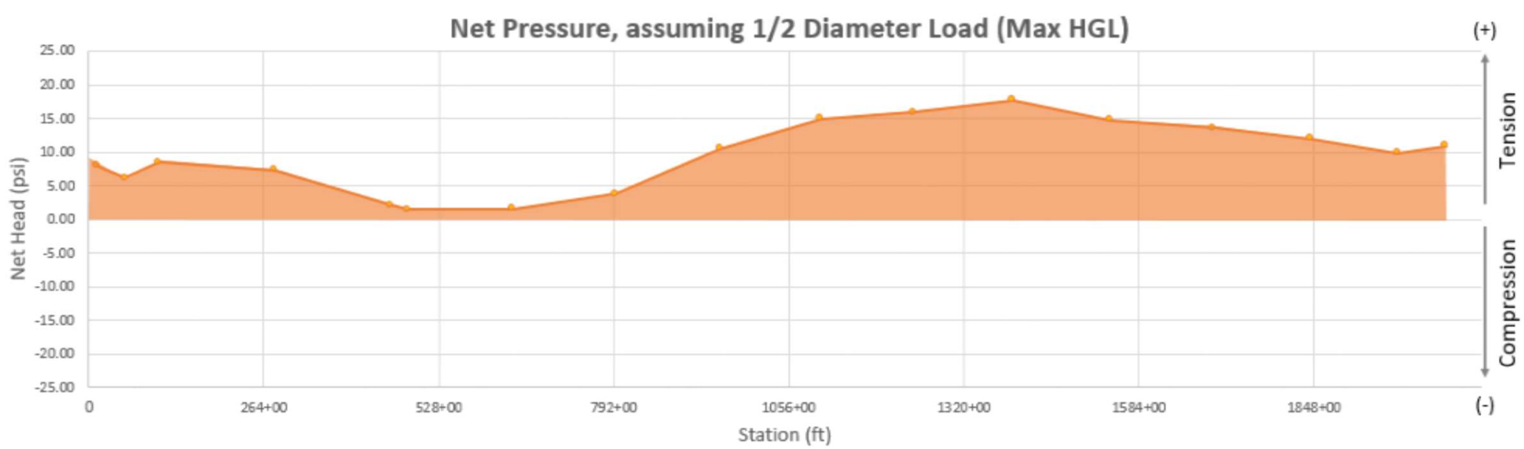
Attachment 1
Figures

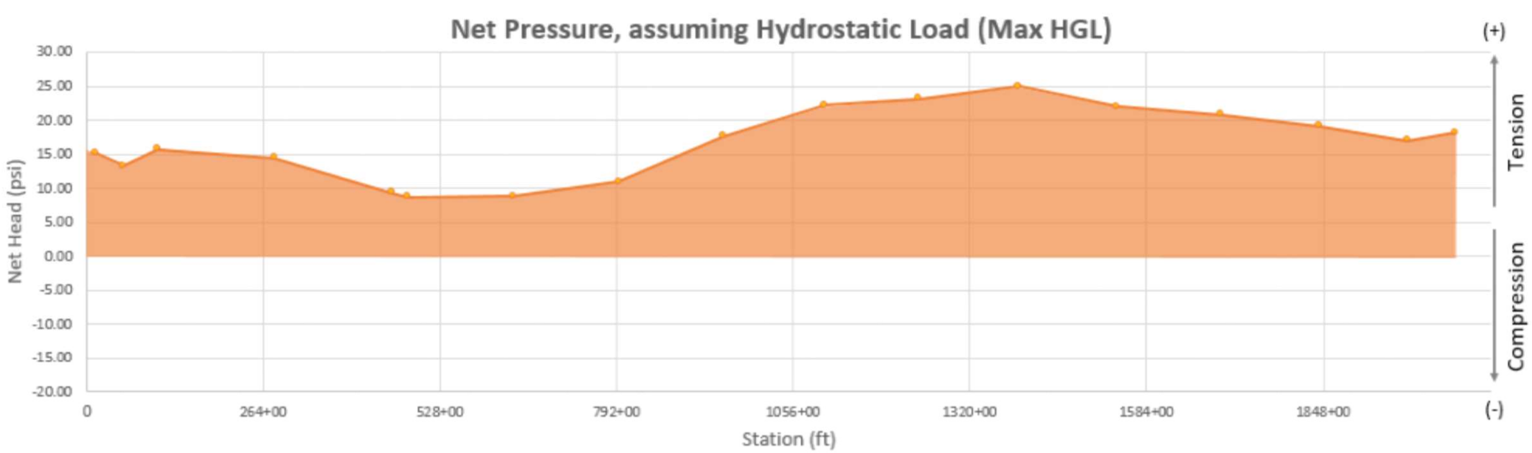
Figures A-1a to A-1d	Net Pressure Profiles
Figure A-1e	Groundwater Profile
Figure A-2	Tunnel Lining Sections
Figure A-3	Moment Thrust Interaction Diagrams
Table A-1	Soft Ground TBMs > 36’ Diameter
Table A-2	Calculated Maximum Thrust and Bending Moments
Calculations	Ground Lining Interaction Analyses
Table A-3	Leakage Rates, Zero Loads, All Alternatives
Table A-4	Leakage Rates, 0.5 Tunnel Dia. Ground Load
Table A-5	Leakage Rates, 1.0 Tunnel Dia. Ground Load

FIGURE A-1A:



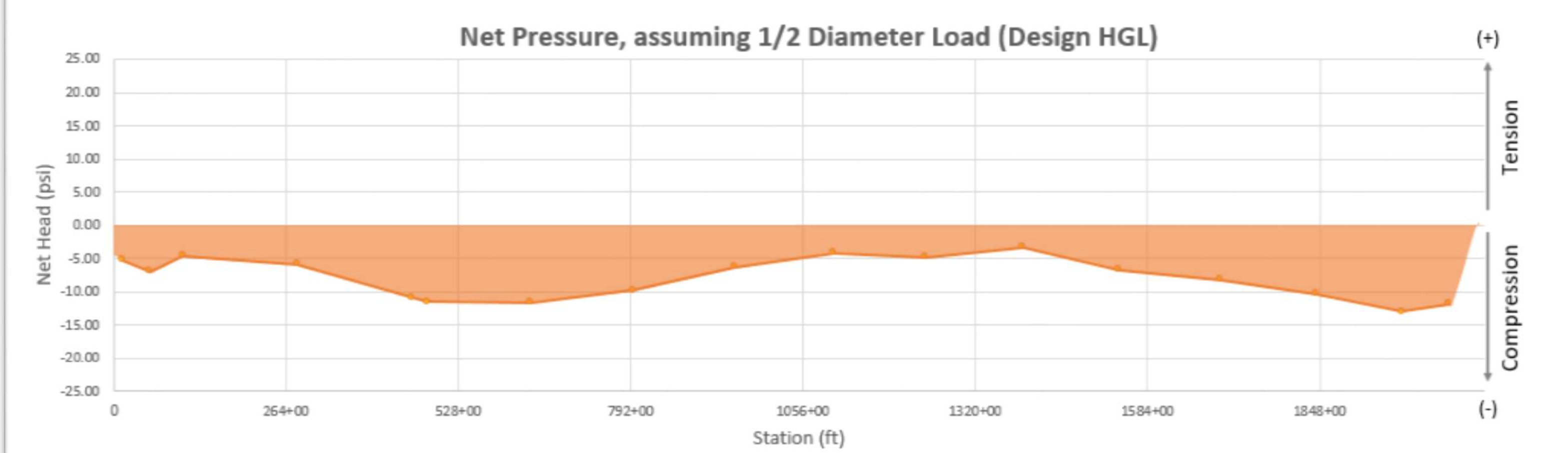
Net Pressure Along Delta Conveyance Tunnel (ϕ = 36')

FIGURE A-1B:



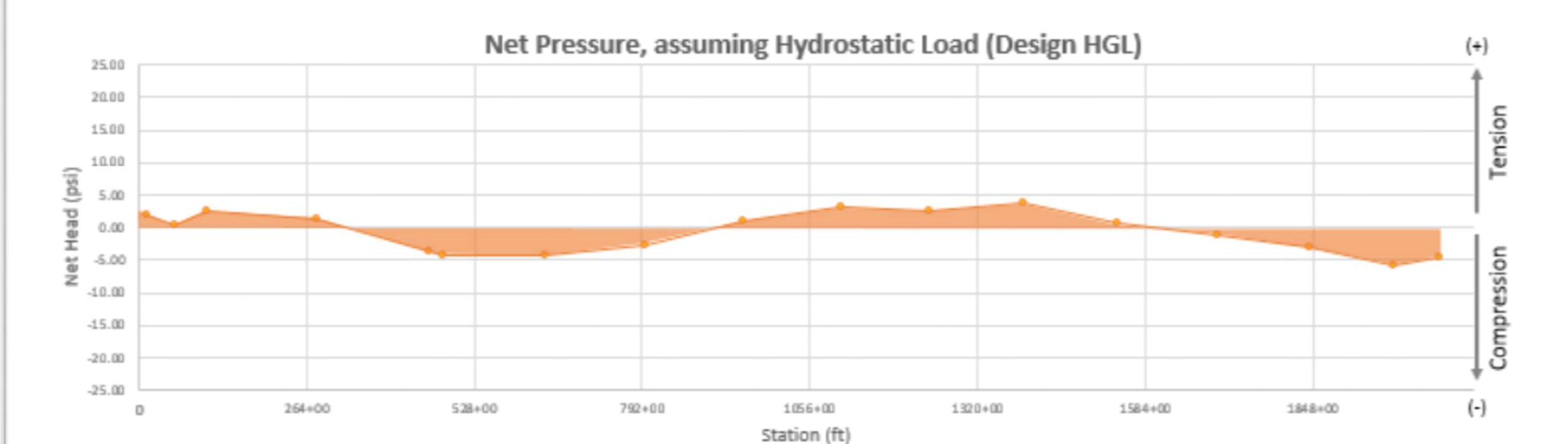
Net Pressure Along Delta Conveyance Tunnel (ϕ = 36')

FIGURE A-1C:

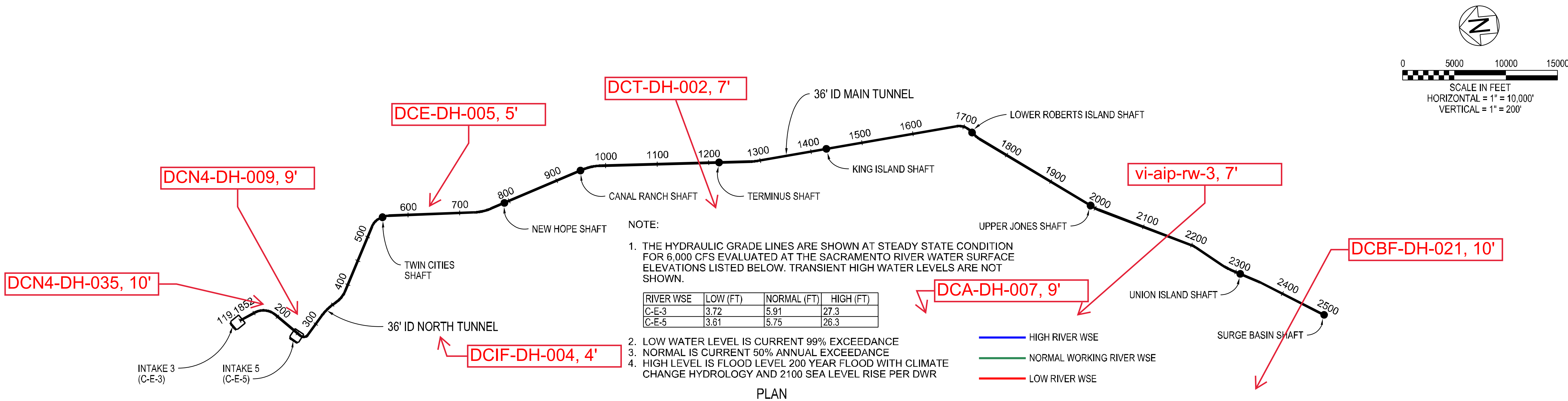


Net Pressure Along Delta Conveyance Tunnel (ϕ = 36')

FIGURE A-1D:



Net Pressure Along Delta Conveyance Tunnel (ϕ = 36')



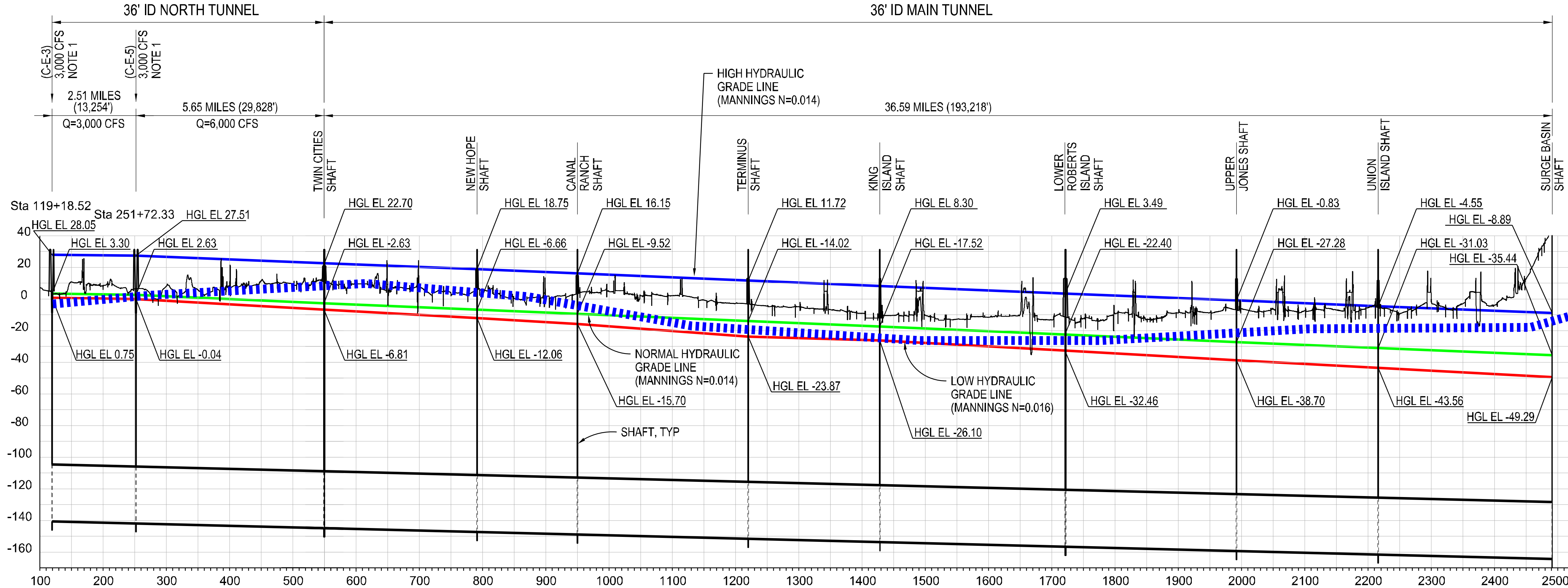
NOTE:

1. THE HYDRAULIC GRADE LINES ARE SHOWN AT STEADY STATE CONDITION FOR 6,000 CFS EVALUATED AT THE SACRAMENTO RIVER WATER SURFACE ELEVATIONS LISTED BELOW. TRANSIENT HIGH WATER LEVELS ARE NOT SHOWN.

RIVER WSE	LOW (FT)	NORMAL (FT)	HIGH (FT)
C-E-3	3.72	5.91	27.3
C-E-5	3.61	5.75	26.3

2. LOW WATER LEVEL IS CURRENT 99% EXCEEDANCE
3. NORMAL IS CURRENT 50% ANNUAL EXCEEDANCE
4. HIGH LEVEL IS FLOOD LEVEL 200 YEAR FLOOD WITH CLIMATE CHANGE HYDROLOGY AND 2100 SEA LEVEL RISE PER DWR

PLAN



PROFILE

Approximate Location of Groundwater (DWR, 2013)

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.

REV	DATE	DESCRIPTION	SUB.	APPD
	SEP 2024	DELTA CONVEYANCE PROJECT CONCEPT ENGINEERING REPORT DRAWINGS		

DESIGNED: K VAN DYK
DRAWN: J OYE
CHECKED: A NAIMEY
APPROVAL RECOMMENDED:
APPROVAL BY: A MURDOCK

CONCEPT PLANNING DOCUMENT
NOT FOR CONSTRUCTION

DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

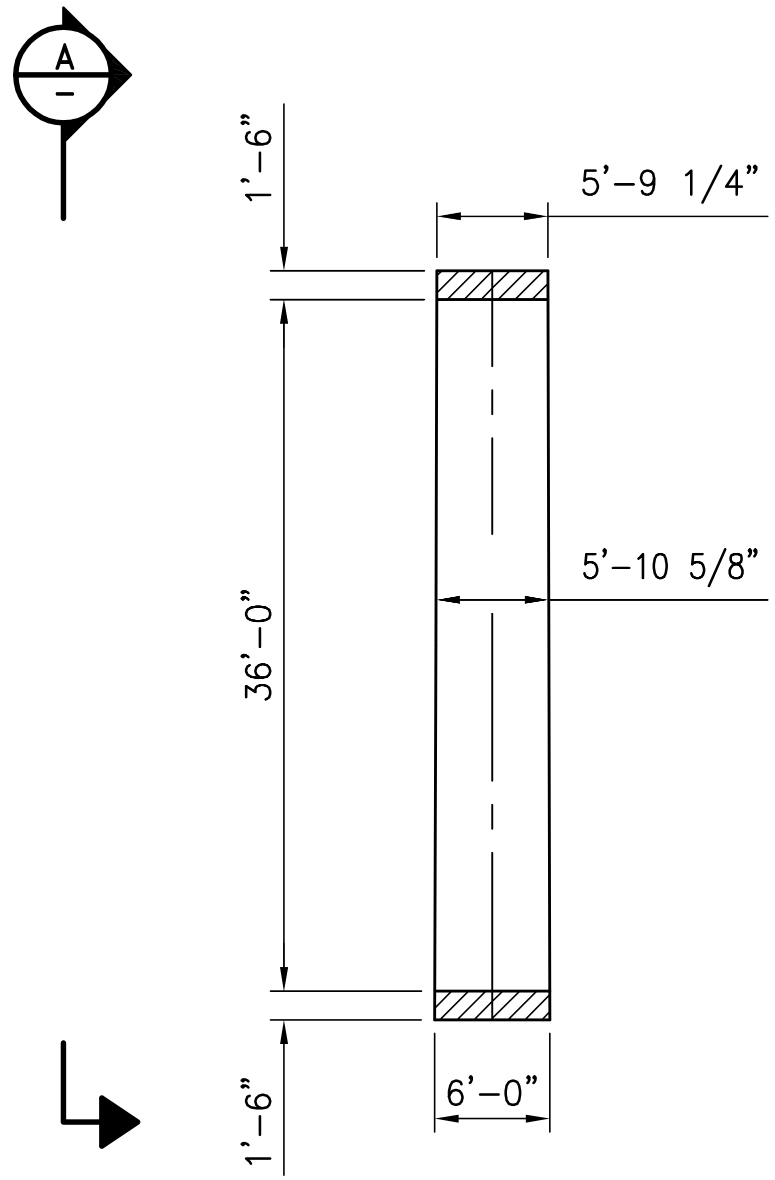
DELTA CONVEYANCE PROJECT
CONCEPT ENGINEERING REPORT

FIGURE A-1E: OVERALL HYDRAULIC PLAN AND PROFILE

PROJECT NO. W8X97000
SHEET NO. PPE-G-0080GN
REV | SEQUENCE NO.

PPE-G-0080GN_W8X97000.dgn

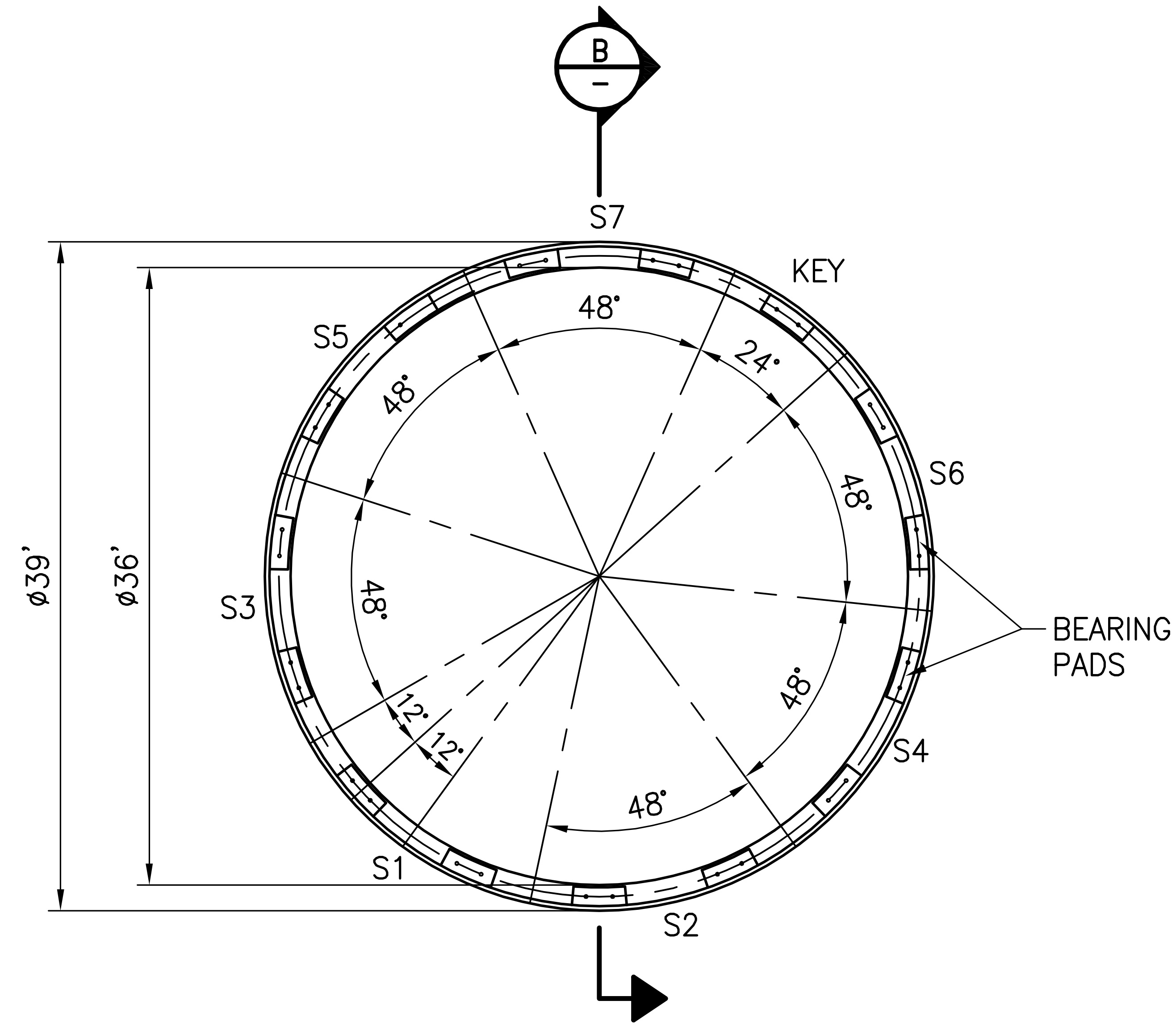
PLOT DATE:2024\09\29 PLOT TIME:9:47:49 AM



TAPER +/- 2.75" CORRESPONDING TO A MINIMUM CURVATURE RADIUS OF 1000 FEET

SECTION B-B

SCALE: 1"=10'



VIEW FROM TUNNEL FACE TOWARDS TUNNEL

ELEVATION A-A

SCALE: 1"=10'

FIGURE A - 2
36' DIA. MAIN TUNNEL LINING DETAILS

FIGURE A-3:

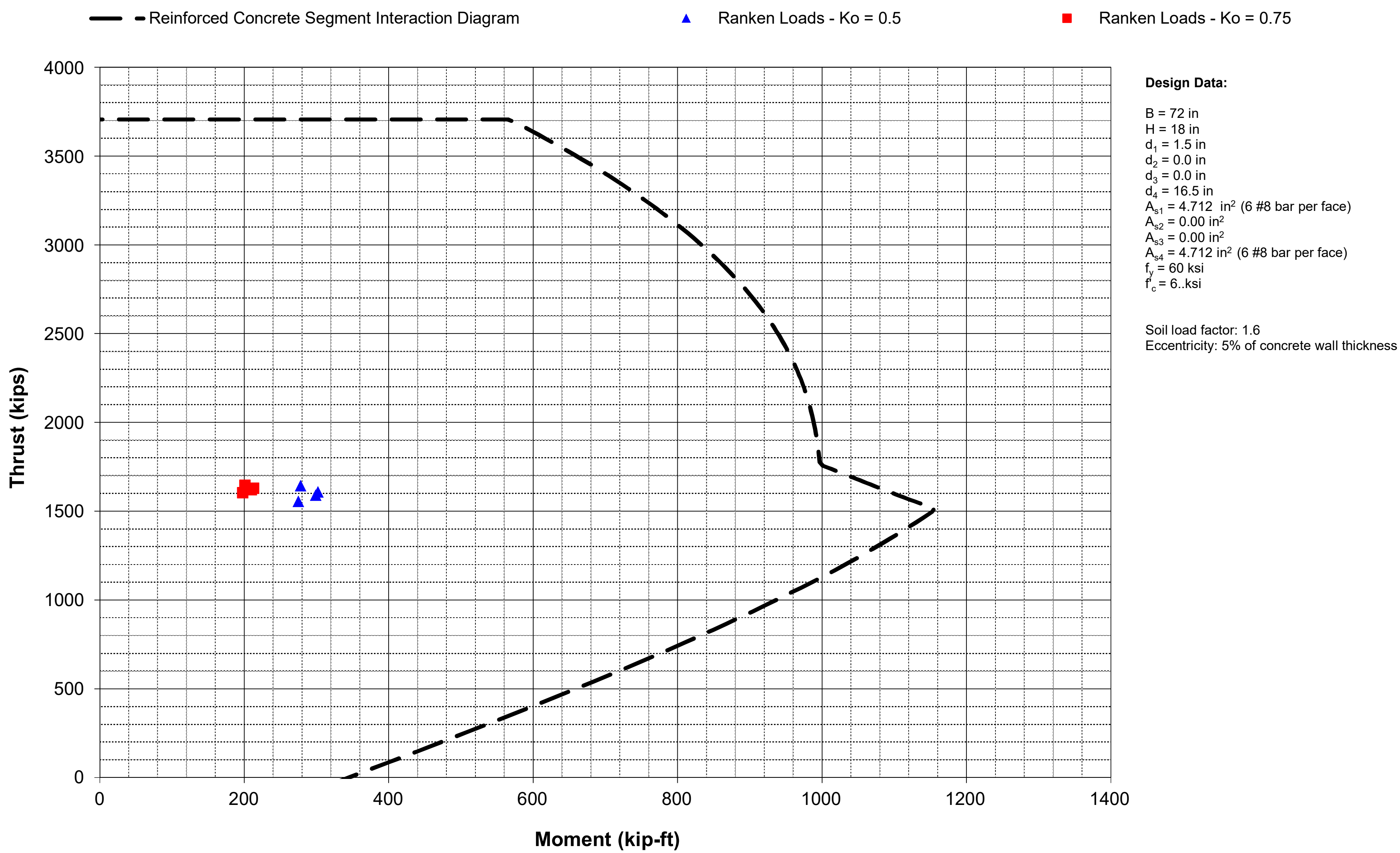


Figure A-3: 36' ID Reinforced Concrete Segment Moment-Thrust Interaction Diagram

EPB and Mixshield, 9-14m Diameter
Geology: clays, silts, sands, gravels, below groundwater table
Breakthroughs since 2010

Project No.	Construction Project	Country	Product	Diameter	Length	Geological Summary	Breakthrough
[-]	[-]	[-]	[-]	[mm]	[m]	[-]	[-]
S-000551	Taishan Water Tunnel	China	Mixshield	9000	3854	Completely to moderately decomposed granite, siltstone, sand, breccia, clay, silty clay	2019
S-000359	Yellow River Crossing	China	Mixshield	9000	4250	Pebbly sand, medium grained, coarse clay	2018
S-000358	Yellow River Crossing	China	Mixshield	9000	4250	Pebbly sand, medium grained, coarse clay	2019
S-000748	Paris Viroflay - Vélizy	France	EPB Shield	9150	1480	Alternation of calcarenite and sands, compact marl, fine clayey sands, clay and sands	2018
S-000620	Guangzhou - Shenzhen - Hong Kong XRL	Hong Kong, China	EPB Shield	9180	2344	Completely to medium weathered tuff, completely to highly decomposed metasedimentary rock (siltstone, sandstone)	2018
S-000630	Guangzhou - Shenzhen - Hong Kong XRL	Hong Kong, China	Mixshield	9250	4857	Granite, Alluvium, deposits	2018
S-000869	Karlsruhe Kombilösung	Germany	Mixshield	9290	2042	Gravel, sand	2018
S-001022	Quito Metro Line 1	Ecuador	EPB Shield	9330	3422	Mixture of sands, sandy gravel, silts, clays and tuffs boulder exist as well as andesite rock	2018
S-000516	Tunel de Quejigares	Spain	EPB Shield	9330	6539	Claystone, marlstone, flysch, gravel, conglomerate, sands, silt and clay, limestone	2019
S-001014	Sofia Metro Line 3	Bulgaria	EPB Shield	9360	5985	Clay, sand, silty sands, gravel	2019
S-000525	Sofia Metro Line 2	Bulgaria	EPB Shield	9360	3770	Silt, gravel, sand and clay	2017
S-001019	Quito Metro Line 1	Ecuador	EPB Shield	9365	9065	Mixture of sands, sandy gravel, silts, clays and tuffs boulder exist as well as andesite rock	2018
S-001018	Quito Metro Line 1	Ecuador	EPB Shield	9365	6866	Mixture of sands, sandy gravel, silts, clays and tuffs boulder exist as well as andesite rock	2018
S-000927	Palermo	Italy	EPB Shield	9370	1976	Weakly cemented limestone, calcareous sands	2020
S-000279	Barcelona Metro Line 9	Spain	EPB Shield	9370	9680	Clay, sand, loam	2017
S-000594	Cairo Metro Line 3 Phase 1	Egypt	Mixshield	9430	3096	Sand, clay	2017
S-000546	Lyon Metro	France	Mixshield	9430	1215	Molasse, sandy pebbly silt, granite	2018
S-000423	Cairo Metro Line 3 Phase 1	Egypt	Mixshield	9430	2937	Sand, gravel, partly cobbles	2017
S-000491	Wehrhahnlinie	Germany	Mixshield	9490	2253	Gravel, sand	2018
S-000955	Nice Tramway Line T2	France	Mixshield	9640	2914	Sand and silt, sandy-gravelly clays, limestone, breccia, marly clay and gypsum	2018
S-000554	Rome Metro	Italy	EPB Shield	9755	1381	Gravel in sandy matrix, silty clay	2018
S-000624	Guangzhou - Shenzhen - Hong Kong XRL	China	Mixshield	9900	3336	Highly to slightly weathered granite, meta-sedimentary rocks (stiff to very stiff sandy silt with silty sand), sandy clay, highly weathered marble	2019
S-000623	Guangzhou - Shenzhen - Hong Kong XRL	China	Mixshield	9900	3364	Highly to slightly weathered granite, meta-sedimentary rocks (stiff to very stiff sandy silt with silty sand), sandy clay, highly weathered marble	2015
S-000857	Koralmtunnel	Austria	Multi-mode TBM	9910	10057	Gravel, sand, silt, clay, rock, glimmer, slate, marble, fine-grained gneiss	2020
S-000733	Sao Paulo Metro Line 5	Brazil	EPB Shield	10540	4922	Clay, silty clay, sandy clay, clayey sand	2015
S-000485	Túnel Móstoles - Navalcarnero	Spain	EPB Shield	10550	1214	Silt and sand, clayey sand	2015
S-000485B	Barcelona (El Prat)	Spain	EPB Shield	10570	3086	Sandy silt, clayey silt and fine sand	2014
S-000782	St. Petersburg Metro	Russia	EPB Shield	10620	3978	Sand, clay, silt, gravel, sandstone, boulders	2014
S-000451	Weinbergtunnel	Switzerland	Multi-mode TBM	11240	4440	Molasse, unconsolidated material	2019
S-000727B	Rotterdamse Baan (Ex Trekvliet)	Netherlands	Mixshield	11340	3282	Loose to dense sand, mixed face sand with peat and/or clay	2013
S-000727	Sluiskil Tunnel	Netherlands	Mixshield	11340	2350	Sand	2014
S-000535	Calle Serrano	Spain	EPB Shield	11460	6800	Clay, clayey sand, sand	2013
S-000939	Tren Ligero de Guadalajara	Mexico	EPB Shield	11520	4003	Tuff SM (silty sand), weathered to massive basalt	2014
S-000450	Calle Mallorca	Spain	EPB Shield	11520	4849	Sand, gravel, hard clay	2015
S-000683	Nanjing Metro Line 3	China	Mixshield	11570	3356	Sand, clay	2015
S-000668	Nanjing Metro Line 10	China	Mixshield	11610	3600	Mudstone consisting silty sand and fine sand, silty clay, rounded gravel, gravelly sand and medium coarse sand with gravel and pebbles	2013
S-001065	Hangzhou Wangjiang Road Tunnel	China	Mixshield	11670	1836	Clay, sand and round gravel, partially very high permeability	2013
S-001067	Hangzhou Wangjiang Road Tunnel	China	Mixshield	11710	1836	Clay, sand and round gravel, partially very high permeability	2012

TABLE A-1: Soft Ground TBMs > 36' Diameter

Herrenknecht AG
Schlehenweg 2
D-77963 Schwanau

EPB and Mixshield, 9-14m Diameter
Geology: clays, silts, sands, gravels, below groundwater table
Breakthroughs since 2010

Project No.	Construction Project	Country	Product	Diameter	Length	Geological Summary	Breakthrough
[-]	[-]	[-]	[-]	[mm]	[m]	[-]	[-]
S-000977	Hangzhou Wenyi Road Highway Tunnel Project	China	Mixshield	11730	1748	Sludge Clay, Silty Clay, Grey Silty Clay with Sand	2012
S-001068	Sutong GIL Pipe Gallery Project	China	Mixshield	12010	5486	Clayey soil and silty soil including sludge silty clay, silty clay with silt, clayey and sandy soil including silty sand, fine sand	2011
S-000221	Barcelona Metro Line 9	Spain	EPB Shield	12060	8520	Grandiorite, sand, clay, gravel	2011
S-000362	Tùneles Urbanos de Girona	Spain	EPB Shield	12110	2901	Clayey silt / sandy clay, gravel and sand, marls & marly limestone, sandstone	2011
S-000978	Wuhan Metro Line 8	China	Mixshield	12510	3172	Silty clay, silty fine sand, round gravel, conglomerate	2011
S-000745	Gdansk Slowacki Tunnel (Ex Sucharski)	Poland	Mixshield	12560	2142	Sand, gravel	2011
S-000452	Biel Ostast	Switzerland	EPB Shield	12560	7164	Molasse, unconsolidated material	2011
S-001051	Beijing - Zhangjiakou High-Speed Railway Line	China	Mixshield	12610	2708	Silty clay, silt, silty sand, gravel, gravel with silt	2011
S-001050	Beijing - Zhangjiakou High-Speed Railway Line	China	Mixshield	12610	1742	Gravel, silty clay, silt, silty sand, gravel with silt	2018
S-000600	Miami Bay-Dodge Island	USA	EPB Shield	12860	2516	Cemented sand and shell, interbedded zones of weakly cemented sandstones, cemented limestone, sand with interbedded zones of limestone	2009
S-000961	Port Said Road Tunnels	Egypt	Mixshield	13020	2846	Soft silty clay, silty sand, hard silty clay	2012
S-000960B	Suez Canal Road Tunnel	Egypt	Mixshield	13020	3240	Hard silty clay/mudstone	2010
S-000960	Ismailia Road Tunnels	Egypt	Mixshield	13020	4820	Silty clay, medium to dense sand	2011
S-000959	Port Said Road Tunnels	Egypt	Mixshield	13020	2851	Soft silty clay, silty sand, hard silty clay	2010
S-000958	Ismailia Road Tunnels	Egypt	Mixshield	13020	4826	Silty clay, medium to dense sand	2011
S-000985	Foshan-Dongguan Intercity	China	Mixshield	13560	4900	Sludge,Sand, HW Mudstone, HW to MW Mudstone, MW Mudstone, Faults Weathered Mud Sandstone and Mudstone	2010
S-000762	Istanbul Strait Road Crossing	Turkey	Mixshield	13660	3350	Sandstone/mudstone, uncemented well-graded marine deposits	2010
S-000882	Hong Kong Tuen Mun	Hong Kong, China	Mixshield	13950	5009	Marine deposit, Alluvium, granite	2010
S-000881	Hong Kong Tuen Mun	Hong Kong, China	Mixshield	13950	4314	Marine deposit, Alluvium, granite	2013

TABLE A-1: Soft Ground TBMs > 36’ Diameter

Condition	Ko = 0.75		Ko = 0.5	
	Thrust (kips)	Moment (kip-ft)	Thrust (kip)	Moment (kip-ft)
Tunnel Springline – No Slippage	1,646	201	1,642	278
Tunnel Springline – Full Slippage	1,629	213	1,608	302
Tunnel Crown – No Slippage	1,603	198	1,555	275
Tunnel Crown – Full Slippage	1,620	210	1,590	299

TABLE A-2: Calculated Maximum Thrust and Bending Moments in 36' ID Concrete Segment Based on Static Loads

CALCULATIONS

GROUND LINING INTERACTION ANALYSES

36' Bolted and Gasketed Precast Concrete Segment
Design Ko = 0.5, Delta Conveyance Tunnel

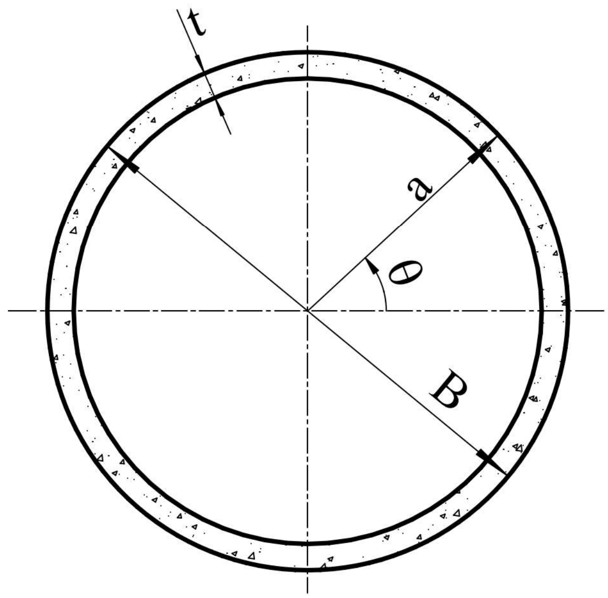
1. Input Data

1.1 Concrete Segment Input

$B := 36ft$		[Inside diameter of ring]
$B_e := 39.3ft$		[Excavation diameter]
$t := 18in$		[Ring thickness]
$f'_c := 6000psi$		[Concrete compression strength]
$E_c := 57000 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi$	$E_c = 4.415 \times 10^3 \cdot ksi$	[Concrete elastic modulus]
$E_1 := \frac{E_c}{2}$	$E_1 = 2.208 \times 10^3 \cdot ksi$	[Concrete elastic modulus taking into account for jointed segment]
$f_y := 60000psi$		[Rebar yield strength]
$b := 6ft$		[Ring length]
$\nu_1 := 0.2$		[Concrete segment Poisson ratio]

CALCULATIONS

The dimensions of the concrete are shown in the following figure:



1.2 Soil Properties

$\gamma := 120\text{pcf}$		[Average soil unit weight above tunnel springline]
$D := 133\text{ft}$		[Depth of soil to tunnel springline]
$H := 0.5 \cdot B_e$	$H = 19.65\,\text{ft}$	[soil load height, 1/2 tunnel diameter]
$H_w := 123.4\text{ft}$		[Depth of groundwater table to tunnel springline based on Sta 124+490 Table 1 (excel spreadsheet)]
$E_m := 350\text{ksf}$	$E_m = 2.431 \cdot \text{ksi}$	[Elastic modulus of ground around tunnel]
$\nu_m := 0.30$		[Poisson ratio of soil mass around tunnel]
$\gamma_w := 62.4\text{pcf}$		[Water unit weight]
$K_o := 0.5$		[Ratio of horizontal and vertical loads]

CALCULATIONS

$LF_E := 1.6$ [Load factor for soil load per ACI code]

$LF_w := 1.6$ [Load factor for for groundwater pressure per ACI code]

2. Calculation

This calculation was based on the undrained conditions. The full soil overburden pressure (caused by materials from ground surface to the tunnel springline) was used in the equations to calculate moment, thrust, and shear in lining segments. This overburden pressure is based on effective unit weight of the soil. Since the hydrostatic pressure causes no addtional moment and shear in the lining ring, it is only added to the thrust calculated from Ranken's method to obtain the total thrust in the rings.

2.1 Ranken, Ghabussi & Henddon (1978) method

2.1.1 No Slippage Condition

Radius to outer surface of concrete segment:

$$a := \frac{B}{2} + t \qquad a = 19.5\,\text{ft}$$

Check condition of thin liner:

$$\frac{t}{a} = 0.077$$

According to the Ranken's method, if $\frac{t}{a}$ ratio is less than 0.1 the segment can be considered thin liner. in this case, the ratio is slightly greater than 0.1. However, for simplicity the liner is still considered thin liner and all the equations applied to a thin liner were used in the later sections to calculate the internal forces in the concrete rings.

Flexibility coefficient:

$$F := \frac{E_m}{E_l}\cdot\left(\frac{a}{t}\right)^3\cdot\left[2\cdot\frac{\left(1-\nu_l^{\,2}\right)}{1+\nu_m}\right] \qquad F = 3.573$$

CALCULATIONS

Compressibility coefficient:

$$C := \frac{E_m}{E_l} \cdot \frac{a}{t} \cdot \left[\frac{\left(1 - \nu_l^{\,2}\right)}{\left(1 + \nu_m\right) \cdot \left(1 - 2\nu_m\right)} \right] \qquad C = 0.026$$

Effective soil load:

$$p'_E := H \cdot \left(\gamma - \gamma_w\right) \qquad p'_E = 1.132 \cdot ksf$$

Thrust from effective soil load:

$$T'_E := p'_E \cdot a \cdot b \qquad T'_E = 132.425 \cdot kip$$

Hydrostatic pressure:

$$p_w := H_w \cdot \gamma_w \qquad p_w = 7.7 \cdot ksf$$

Thrust from hydrostatic pressure:

$$T_w := p_w \cdot a \cdot b \qquad T_w = 900.919 \cdot kip$$

Calculate coefficients L_n, J_n, N_n as follows:

$$L_n := \frac{\left(1 - 2\nu_m\right) \cdot C}{1 + \left(1 - 2 \cdot \nu_m\right) \cdot C} \qquad L_n = 0.01$$

$$J_n := \frac{\left[2 \cdot \nu_m + \left(1 - 2\nu_m\right) \cdot C\right] \cdot F + \left(1 - \nu_m\right) \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C}{\left[\left(3 - 2\nu_m\right) + \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(5 - 6\nu_m\right) \cdot \left(1 - 2\nu_m\right) \cdot C + \left(6 - 8 \cdot \nu_m\right)} \qquad J_n = 0.179$$

$$N_n := \frac{\left[3 + 2 \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C}{\left[\left(3 - 2\nu_m\right) + \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(5 - 6\nu_m\right) \cdot \left(1 - 2\nu_m\right) \cdot C + \left(6 - 8 \cdot \nu_m\right)} \qquad N_n = 0.883$$

CALCULATIONS

The internal forces in the concrete ring caused by ground mass are calculated as follows, where θ is the counter-clockwise angle from horizontal direction as shown in the figure above.

$$\theta := 0deg, 1deg .. 360deg$$

$$T_N(\theta) := \left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[\left(1 + K_o\right) \cdot \left(1 - L_n\right) + \left(1 - K_o\right) \cdot \left(1 - J_n\right) \cdot \cos(2 \cdot \theta)\right]$$ [Thust]

$$M_N(\theta) := \left(\frac{p'_E \cdot a^2 \cdot b}{2}\right) \cdot \left[\left(1 + K_o\right) \cdot \left(\frac{L_n}{6 \cdot F}\right) + \frac{1}{2} \cdot \left(1 - K_o\right) \cdot \left(1 + J_n - N_n\right) \cdot \cos(2 \cdot \theta)\right]$$ [Moment]

$$V_N(\theta) := -\left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[\left(1 - K_o\right) \cdot \left(1 + J_n - N_n\right) \cdot \sin(2 \cdot \theta)\right]$$ [Shear]

Factored thrust at tunnel springline is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$$T_{N\_SL} := LF_E \cdot T_N(0deg) + LF_w \cdot T_w \qquad T_{N\_SL} = 1.642 \times 10^3 \cdot kip$$

Assume that the eccentricity of the ring is 5% of the segment thickness. The moment in the segment is calculated using the above calculated thrust.

$$e := 5\% \cdot t \qquad e = 0.9 \cdot in$$

Additional moment at the springline due to eccentricity (assumed that this moment is applied both at the springline and crown of tunnel):

$$M_{N\_ADD} := T_{N\_SL} \cdot e \qquad M_{N\_ADD} = 123.166 \cdot kip \cdot ft$$

Total factored moment at tunnel springline:

$$M_{N\_SL} := LF_E \cdot M_N(0deg) + M_{N\_ADD} \qquad M_{N\_SL} = 277.521\ ft \cdot kip$$

Factored thrust at tunnel crown is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$$T_{N\_CR} := LF_E \cdot T_N(90deg) + LF_w \cdot T_w \qquad T_{N\_CR} = 1.555 \times 10^3 \cdot kip$$

CALCULATIONS

Factored moment at tunnel crown:

$$M_{N\_CR} := LF_E \cdot M_N(90deg) - M_{N\_ADD} \qquad M_{N\_CR} = -274.497\ ft \cdot kip$$

Factored shear at 45 degree from tunnel vertical center line:

$$V_{N\_45} := LF_E \cdot V_N(45deg) \qquad V_{N\_45} = -15.676 \cdot kip$$

2.1.2 Full Slippage Condition

Calculate coefficients L_f, J_f, N_f as follows:

$$L_f := \frac{(1 - 2\nu_m) \cdot C}{1 + (1 - 2 \cdot \nu_m) \cdot C} \qquad L_f = 0.01$$

$$J_f := \frac{F + (1 - \nu_m)}{2 \cdot F + (5 - 6 \cdot \nu_m)} \qquad J_f = 0.413$$

$$T_F(\theta) := \left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[(1 + K_o) \cdot (1 - L_f) + (1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \cos(2 \cdot \theta)\right]$$ [Thust]

$$M_F(\theta) := \left(\frac{p'_E \cdot a^2 \cdot b}{2}\right) \cdot \left[(1 + K_o) \cdot \left(\frac{L_f}{6 \cdot F}\right) + (1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \cos(2 \cdot \theta)\right]$$ [Moment]

$$V_F(\theta) := -\left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[(1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \sin(2 \cdot \theta)\right]$$ [Shear]

Factored thrust at tunnel springline is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$$T_{F\_SL} := LF_E \cdot T_F(0deg) + LF_w \cdot T_w \qquad T_{F\_SL} = 1.608 \times 10^3 \cdot kip$$

Additional moment at the springline due to eccentricity (assumed that this moment is applied both at the springline and crown of tunnel):

$$M_{F\_ADD} := T_{F\_SL} \cdot e \qquad M_{F\_ADD} = 120.595 \cdot kip \cdot ft$$

CALCULATIONS

Total factored moment at tunnel springline:

$M_{F\_SL} := LF_E \cdot M_F(0deg) + M_{F\_ADD}$ $\qquad M_{F\_SL} = 301.831\ ft \cdot kip$

Factored thrust at tunnel crown is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$T_{F\_CR} := LF_E \cdot T_F(90deg) + LF_w \cdot T_w$ $\qquad T_{F\_CR} = 1.59 \times 10^3 \cdot kip$

Factored moment at tunnel crown:

$M_{F\_CR} := LF_E \cdot M_F(90deg) - M_{F\_ADD}$ $\qquad M_{F\_CR} = -298.807\ ft \cdot kip$

Factored shear at 45 degree from tunnel vertical center line:

$V_{F\_45} := LF_E \cdot V_F(45deg)$ $\qquad V_{F\_45} = -9.217 \cdot kip$

CALCULATIONS

GROUND LINING INTERACTION ANALYSES

36' Bolted and Gasketed Precast Concrete Segment Design Ko = 0.75, Delta Conveyance Tunnel

1. Input Data

1.1 Concrete Segment Input

$B := 36\text{ft}$ [Inside diameter of ring]

$B_e := 39\text{ft}$ [Excavation diameter]

$t := 18\text{in}$ [Ring thickness]

$f'_c := 6000\text{psi}$ [Concrete compression strength]

$E_c := 57000 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi}$ $\quad E_c = 4.415 \times 10^3 \cdot \text{ksi}$ [Concrete elastic modulus]

$E_l := \frac{E_c}{2}$ $\quad E_l = 2.208 \times 10^3 \cdot \text{ksi}$ [Concrete elastic modulus taking into account for jointed segment]

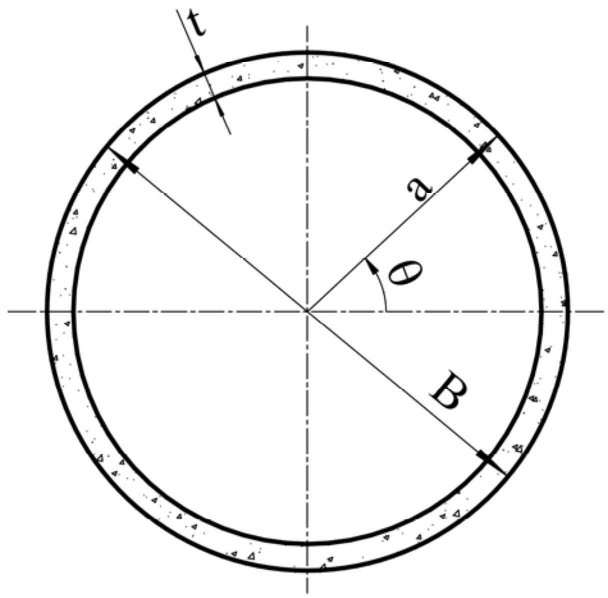
$f_y := 60000\text{psi}$ [Rebar yield strength]

$b := 6\text{ft}$ [Ring length]

$\nu_l := 0.2$ [Concrete segment Poisson ratio]

CALCULATIONS

The dimensions of the concrete are shown in the following figure:



1.2 Soil Properties

$\gamma := 120\text{pcf}$ [Average soil unit weight above tunnel springline]

$D := 133\text{ft}$ [Depth of soil to tunnel springline]

$H := 0.5 \cdot B_e$ $H = 19.5\text{ ft}$ [soil load height, 1/2 tunnel diameter]

$H_w := 123.5\text{ft}$ [Depth of groundwater table to tunnel springline based on Sta 124+490 Table 1 (excel spreadsheet)]

$E_m := 350\text{ksf}$ $E_m = 2.431 \cdot \text{ksi}$ [Elastic modulus of ground around tunnel]

$\nu_m := 0.30$ [Poisson ratio of soil mass around tunnel]

$\gamma_w := 62.4\text{pcf}$ [Water unit weight]

$K_o := 0.75$ [Ratio of horizontal and vertical loads]

$LF_E := 1.6$ [Load factor for soil load per ACI code]

$LF_w := 1.6$ [Load factor for for groundwater pressure per ACI code]

2. Calculation

This calculation was based on the undrained conditions. The full soil overburden pressure (caused by materials from ground surface to the tunnel springline) was used in the equations to calculate moment, thrust, and shear in lining segments. This overburden pressure is based on effective unit weight of the soil. Since the hydrostatic pressure causes no addtional moment and shear in the lining ring, it is only added to the thrust calculated from Ranken's method to obtain the total thrust in the rings.

2.1 Ranken, Ghabussi & Henddon (1978) method

2.1.1 No Slippage Condition

Radius to outer surface of concrete segment:

$$a := \frac{B}{2} + t \qquad a = 19.5\ \text{ft}$$

Check condition of thin liner:

$$\frac{t}{a} = 0.077$$

According to the Ranken's method, if $\frac{t}{a}$ ratio is less than 0.1 the segment can be considered thin liner. in this case, the ratio is slightly greater than 0.1. However, for simplicity the liner is still considered thin liner and all the equations applied to a thin liner were used in the later sections to calculate the internal forces in the concrete rings.

Flexibility coefficient:

$$F := \frac{E_m}{E_l} \cdot \left(\frac{a}{t}\right)^3 \cdot \left[2 \cdot \frac{\left(1 - \nu_l^{\,2}\right)}{1 + \nu_m}\right] \qquad F = 3.573$$

CALCULATIONS

Compressibility coefficient:

$$C := \frac{E_m}{E_l} \cdot \frac{a}{t} \cdot \left[\frac{\left(1 - \nu_l^2\right)}{\left(1 + \nu_m\right) \cdot \left(1 - 2\nu_m\right)} \right] \qquad C = 0.026$$

Effective soil load:

$$p'_E := H \cdot \left(\gamma - \gamma_w\right) \qquad p'_E = 1.123 \cdot ksf$$

Thrust from effective soil load:

$$T'_E := p'_E \cdot a \cdot b \qquad T'_E = 131.414 \cdot kip$$

Hydrostatic pressure:

$$p_w := H_w \cdot \gamma_w \qquad p_w = 7.706 \cdot ksf$$

Thrust from hydrostatic pressure:

$$T_w := p_w \cdot a \cdot b \qquad T_w = 901.649 \cdot kip$$

Calculate coefficients L_n, J_n, N_n as follows:

$$L_n := \frac{\left(1 - 2\nu_m\right) \cdot C}{1 + \left(1 - 2 \cdot \nu_m\right) \cdot C} \qquad L_n = 0.01$$

$$J_n := \frac{\left[2 \cdot \nu_m + \left(1 - 2\nu_m\right) \cdot C\right] \cdot F + \left(1 - \nu_m\right) \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C}{\left[\left(3 - 2\nu_m\right) + \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(5 - 6\nu_m\right) \cdot \left(1 - 2\nu_m\right) \cdot C + \left(6 - 8 \cdot \nu_m\right)} \qquad J_n = 0.179$$

$$N_n := \frac{\left[3 + 2 \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(1 - 2 \cdot \nu_m\right) \cdot C}{\left[\left(3 - 2\nu_m\right) + \left(1 - 2 \cdot \nu_m\right) \cdot C\right] \cdot F + \frac{1}{2} \cdot \left(5 - 6\nu_m\right) \cdot \left(1 - 2\nu_m\right) \cdot C + \left(6 - 8 \cdot \nu_m\right)} \qquad N_n = 0.883$$

CALCULATIONS

The internal forces in the concrete ring caused by ground mass are calculated as follows, where θ is the counter-clockwise angle from horizontal direction as shown in the figure above.

$$\theta := 0\text{deg}, 1\text{deg} .. 360\text{deg}$$

$$T_N(\theta) := \left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[\left(1 + K_o\right) \cdot \left(1 - L_n\right) + \left(1 - K_o\right) \cdot \left(1 - J_n\right) \cdot \cos(2 \cdot \theta)\right]$$ [Thust]

$$M_N(\theta) := \left(\frac{p'_E \cdot a^2 \cdot b}{2}\right) \cdot \left[\left(1 + K_o\right) \cdot \left(\frac{L_n}{6 \cdot F}\right) + \frac{1}{2} \cdot \left(1 - K_o\right) \cdot \left(1 + J_n - N_n\right) \cdot \cos(2 \cdot \theta)\right]$$ [Moment]

$$V_N(\theta) := -\left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[\left(1 - K_o\right) \cdot \left(1 + J_n - N_n\right) \cdot \sin(2 \cdot \theta)\right]$$ [Shear]

Factored thrust at tunnel springline is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$$T_{N\_SL} := LF_E \cdot T_N(0\text{deg}) + LF_w \cdot T_w \qquad T_{N\_SL} = 1.646 \times 10^3 \cdot \text{kip}$$

Assume that the eccentricity of the ring is 5% of the segment thickness. The moment in the segment is calculated using the above calculated thrust.

$$e := 5\% \cdot t \qquad e = 0.9 \cdot \text{in}$$

Additional moment at the springline due to eccentricity (assumed that this moment is applied both at the springline and crown of tunnel):

$$M_{N\_ADD} := T_{N\_SL} \cdot e \qquad M_{N\_ADD} = 123.47 \cdot \text{kip} \cdot \text{ft}$$

Total factored moment at tunnel springline:

$$M_{N\_SL} := LF_E \cdot M_N(0\text{deg}) + M_{N\_ADD} \qquad M_{N\_SL} = 201.059\ \text{ft} \cdot \text{kip}$$

Factored thrust at tunnel crown is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$$T_{N\_CR} := LF_E \cdot T_N(90\text{deg}) + LF_w \cdot T_w \qquad T_{N\_CR} = 1.603 \times 10^3 \cdot \text{kip}$$

Factored moment at tunnel crown:

$$M_{N\_CR} := LF_E \cdot M_N(90\text{deg}) - M_{N\_ADD} \qquad M_{N\_CR} = -197.558\ \text{ft} \cdot \text{kip}$$

Factored shear at 45 degree from tunnel vertical center line:

$V_{N\_45} := LF_E \cdot V_N(45deg)$ $\qquad V_{N\_45} = -7.778 \cdot kip$

2.1.2 Full Slippage Condition

Calculate coefficients L_f, J_f, N_f as follows:

$$L_f := \frac{(1 - 2\nu_m) \cdot C}{1 + (1 - 2 \cdot \nu_m) \cdot C} \qquad L_f = 0.01$$

$$J_f := \frac{F + (1 - \nu_m)}{2 \cdot F + (5 - 6 \cdot \nu_m)} \qquad J_f = 0.413$$

$$T_F(\theta) := \left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[(1 + K_o) \cdot (1 - L_f) + (1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \cos(2 \cdot \theta)\right] \qquad \text{[Thust]}$$

$$M_F(\theta) := \left(\frac{p'_E \cdot a^2 \cdot b}{2}\right) \cdot \left[(1 + K_o) \cdot \left(\frac{L_f}{6 \cdot F}\right) + (1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \cos(2 \cdot \theta)\right] \qquad \text{[Moment]}$$

$$V_F(\theta) := -\left(\frac{p'_E \cdot a \cdot b}{2}\right) \cdot \left[(1 - K_o) \cdot (1 - 2 \cdot J_f) \cdot \sin(2 \cdot \theta)\right] \qquad \text{[Shear]}$$

Factored thrust at tunnel springline is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$T_{F\_SL} := LF_E \cdot T_F(0deg) + LF_w \cdot T_w$ $\qquad T_{F\_SL} = 1.629 \times 10^3 \cdot kip$

Additional moment at the springline due to eccentricity (assumed that this moment is applied both at the springline and crown of tunnel):

$M_{F\_ADD} := T_{F\_SL} \cdot e$ $\qquad M_{F\_ADD} = 122.195 \cdot kip \cdot ft$

Total factored moment at tunnel springline:

$M_{F\_SL} := LF_E \cdot M_F(0deg) + M_{F\_ADD}$ $\qquad M_{F\_SL} = 213.122\ ft \cdot kip$

CALCULATIONS

Factored thrust at tunnel crown is the sum of factored thrusts caused by ground load and hydrostatic pressure.

$T_{F\_CR} := LF_E \cdot T_F(90deg) + LF_w \cdot T_w$ $\qquad T_{F\_CR} = 1.62 \times 10^3 \cdot kip$

Factored moment at tunnel crown:

$M_{F\_CR} := LF_E \cdot M_F(90deg) - M_{F\_ADD}$ $\qquad M_{F\_CR} = -209.621 \ ft \cdot kip$

Factored shear at 45 degree from tunnel vertical center line:

$V_{F\_45} := LF_E \cdot V_F(45deg)$ $\qquad V_{F\_45} = -4.573 \cdot kip$

Tunnel Diameter **36 feet ID** **18 inch Segment thickness**

Alternative A1: One-pass Lining with Rebar and Bolts at Radial Joints

	Hoop Tensile Force (kip)	Steel Yield Stress (ksi)	Allowable Stress (ksi)	Required Steel Area (sq inch)	Bar Size/ Bolt Dia	Bar Area (sq inch)	Bar Spacing s (inch)	Steel Area (sq inch)	Steel Area (%)	Calculated Steel Stress Fs (ksi)	
Max Diff Head (ft)	**57.5**	(zero soil confinment)									
6 ft segment	387.5	60	45.00	8.611	8	0.785	12	9.425	0.73%	41.12	< allowable
3 Bolts	387.5	130	97.50	3.974	1.375	1.485	20	4.455	0.34%	86.99	< allowable
Max Diff Head (ft)	**40.9**	(1/2 tunnel diameter soil confinement)									
6 ft segment (LF 1.35)	275.6	60	33.30	8.277	8	0.785	12	9.425	0.73%	29.25	< allowable
3 Bolts (LF 1.35)	275.6	130	72.20	3.818	1.375	1.485	20	4.455	0.34%	61.87	< allowable
Design Operating HGL Head (ft)	**8.9**	(zero soil confinment)									
6 ft segment	60.0	60	30.00	1.999	8	0.785	12	9.425	0.73%	6.36	< allowable
3 Bolts	60.0	130	65.00	0.923	1.375	1.485	20	4.455	0.34%	13.46	< allowable
Design HGL Head (ft)		(1/2 tunnel diameter soil confinement)									
6 ft segment	0.0	60	30.00	0.000	0	0.000	0	NA	NA	NA	liner is in compression
3 Bolts	0.0	130	65.00	0.000	0	0.000	0	NA	NA	NA	liner is in compression

ASTM A490 Type 1 - 1/2” to 1-1/2” nominal size, 130 ksi minimum yield strength.

Allowable Stress for transient pressures = 0.75 x min yield strength (AWWA Manual M11)

Allowable Stress for working pressures = 0.50 x min yield strength

CALCULATIONS

TBM Jacking Loads: Bearing Stresses on Joint Face

Tunnel inside diameter, ID =	36.00	ft
Tunnel liner thickness, t =	18	in
Tunnel outside diameter, OD =	39.0	ft
Tunnel diameter for analytical model, D =	37.5	ft

Normal Wright Concrete

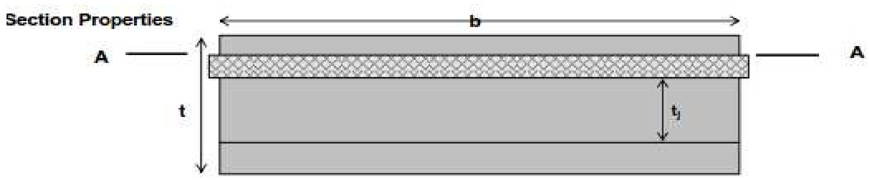
Compressive Design Strength, fc' (psi)=	6,000	
Unit Weight γ_c (pcf) =	150	
Modulus of elasticity, $E_c = 33^*\gamma_c^{1.5*}\sqrt{(f'_c)}$ =	4,696	(ksi), ACI318, 8.5.1
Poisson's Ratio, v_c =	0.2	

Reinforcement (ASTM A706)

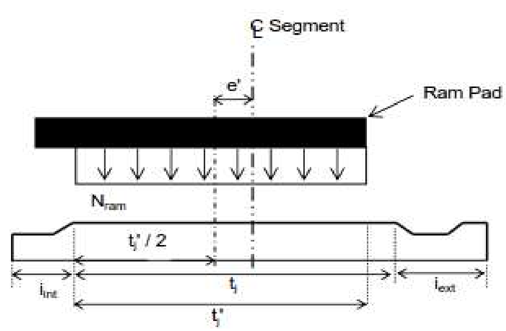
Yield Strength, f_y (ksi) =	60
Modulus of Elasticity, $E_{s\ (ksi)}$ =	29,000

Nominal force main thrust cylinder T_{main} (k) =	22,880	(Herrenknnecht, 2011)
Number of Jacks on TBM =	32	
Load Factor under Nominal Jacking Condition $\varphi_{jack,ave}$ =	1.4	
Load Factor under Maximum Jacking Condition $\varphi_{jack,max}$ =	1.2	
Force per Jack under Normal Operating condition $F_{jack,ave}$ (k)=	1,001	
Force per Jack under Maximum Operating condition $F_{jack,max}$ (k)=	1,201	

Assuming the shape of Sections:

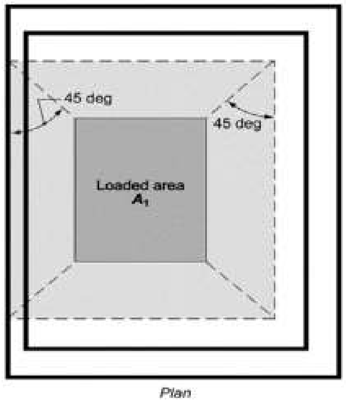


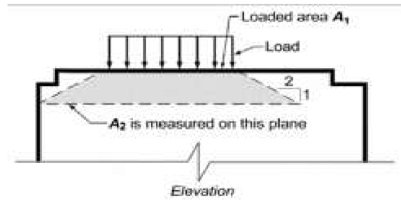
Determination of Bearing and Stresses on Joint Face under TBM Jacking Loads



Tunnel liner thickness, t (in)=	18
TBM Jacking Shoe Length L_{shoe} (in) =	30
Min TBM Jacking Shoe Bearing Width w_{shoe} (in) =	15
Depth of Caulking Groove on Internal Face i_{int} (in)=	2.00
Depth of Groove on External Face i_{ext} (in)=	3.00
Stepping Build Allowance $\Delta_{stepping}$ (in)=	1.00
Segment Bearing Width t_j' (in)= t- t_{int} -t_{ext} -$\Delta_{stepping}$ =	12.00

Check Bearing Stress under Jacking Shoe





Bearing Width, t_j' (in) =	12.00	
Loaded area A1 = t_j' L_{shoe} (in^2)=	360	
Width of distribution area, L (in) = t_j' + 2min(i_{int}, i_{ext}) =	16	
Max distribution area, A2 = L (L_{shoe} + 2min(i_{int}, i_{ext})) =	544	
Concrete Compressive Strength, fc' (ksi) =	6.00	
Factor of concrete in bearing, φ_b =	0.65	ACI 318; 9.3.2.4
Limit of $(A2/A1)^{0.5}$ =	1.23	≤ 2
Allowable bearing stress F_{br} (ksi) = φ_b 0.85fc' $(A2/A1)^{0.5}$ =	4.79	ACI 318; 10.14.1
Bearing stress across Segment Contact area,		
f_{br} (ksi) = $F_{jack,max}$/A1 =	3.34	
Factor of safety (Jack bearing) =	1.44	> 1.0 OK

Total Leakage (GPM) per reach														
			Q1			Q2-3						Q4		
Alternative 5 – 6000 cfs for Bethany Reservoir Alternative			**JAN**	**FEB**	**MAR**	**APR**	**MAY**	**JUNE**	**JULY**	**AUG**	**SEPT**	**OCT**	**NOV**	**DEC**
Flow (cfs)			**2295**	**2084**	**2197**	**311**	**483**	**744**	**812**	**202**	**673**	**457**	**1161**	**1533**
Reach	**Upstream Shaft**	**Downstream Shaft**												
1	C-E-3	C-E-5	146	146	146	129	129	129	129	129	129	120	120	120
1	C-E-5	Twin Cities Double Launch Shaft	54	54	54	43	43	43	43	43	43	38	38	38
1, 2	Twin Cities Double Launch Shaft	New Hope Tract Maintenance Shaft	43	46	45	48	48	47	47	48	47	43	41	38
2	New Hope Tract Maintenance Shaft	Canal Ranch Tract Maintenance Shaft	55	59	56	63	63	62	61	64	62	57	53	49
2	Canal Ranch Tract Maintenance Shaft	Terminus Tract Reception Shaft	249	267	257	302	300	294	293	303	296	283	265	248
2, 3	Terminus Tract Reception Shaft	King Island Maintenance Shaft	478	502	489	559	556	548	546	560	550	537	510	487
3	King Island Maintenance Shaft	Lower Roberts Island Launch Site	1031	1072	1050	1182	1176	1164	1160	1184	1168	1150	1105	1066
3, 4	Lower Roberts Island Launch Site	Upper Jones Tract Maintenance Shaft	954	1001	977	1138	1132	1118	1113	1141	1122	1107	1056	1011
4	Upper Jones Tract Maintenance Shaft	Union Island Maintenance Shaft	556	599	577	731	726	712	708	734	717	706	659	618
4	Union Island Maintenance Shaft	Surge Basin Reception Shaft	306	335	320	422	418	410	407	423	412	406	376	349
Total Leakage (GPM)			3871	4080	3971	4617	4590	4526	4506	4628	4547	4449	4223	4024

TABLE A-3: Leakage Rates, Zero Loads

Total Leakage (GPM) per reach														
			Q1			Q2-3						Q4		
Alternative 5 – 6000 cfs for Bethany Reservoir Alternative			**JAN**	**FEB**	**MAR**	**APR**	**MAY**	**JUNE**	**JULY**	**AUG**	**SEPT**	**OCT**	**NOV**	**DEC**
Flow (cfs)			**2295**	**2084**	**2197**	**311**	**483**	**744**	**812**	**202**	**673**	**457**	**1161**	**1533**
Reach	**Upstream Shaft**	**Downstream Shaft**												
1	C-E-3	C-E-5	0	0	0	0	0	0	0	0	0	0	0	0
1	C-E-5	Twin Cities Double Launch Shaft	0	0	0	0	0	0	0	0	0	0	0	0
1, 2	Twin Cities Double Launch Shaft	New Hope Tract Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2	New Hope Tract Maintenance Shaft	Canal Ranch Tract Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2	Canal Ranch Tract Maintenance Shaft	Terminus Tract Reception Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2, 3	Terminus Tract Reception Shaft	King Island Maintenance Shaft	0	1	0	13	13	11	11	14	12	9	3	0
3	King Island Maintenance Shaft	Lower Roberts Island Launch Site	57	66	61	250	246	237	234	252	240	228	195	66
3, 4	Lower Roberts Island Launch Site	Upper Jones Tract Maintenance Shaft	54	62	58	246	241	232	228	247	235	224	189	63
4	Upper Jones Tract Maintenance Shaft	Union Island Maintenance Shaft	0	0	0	22	20	16	15	23	17	15	6	0
4	Union Island Maintenance Shaft	Surge Basin Reception Shaft	0	0	0	3	2	0	0	4	0	0	0	0
Total Leakage (GPM)			110	129	119	535	523	496	488	540	503	476	394	128

TABLE A-4: Leakage Rates, 0.5 Tunnel Diameter Ground Load

Total Leakage (GPM) per reach														
			Q1			Q2-3						Q4		
Alternative 5 – 6000 cfs for Bethany Reservoir Alternative			JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
Flow (cfs)			2295	2084	2197	311	483	744	812	202	673	457	1161	1533
Reach	Upstream Shaft	Downstream Shaft												
1	C-E-3	C-E-5	0	0	0	0	0	0	0	0	0	0	0	0
1	C-E-5	Twin Cities Double Launch Shaft	0	0	0	0	0	0	0	0	0	0	0	0
1, 2	Twin Cities Double Launch Shaft	New Hope Tract Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2	New Hope Tract Maintenance Shaft	Canal Ranch Tract Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2	Canal Ranch Tract Maintenance Shaft	Terminus Tract Reception Shaft	0	0	0	0	0	0	0	0	0	0	0	0
2, 3	Terminus Tract Reception Shaft	King Island Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
3	King Island Maintenance Shaft	Lower Roberts Island Launch Site	0	0	0	0	0	0	0	0	0	0	0	0
3, 4	Lower Roberts Island Launch Site	Upper Jones Tract Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
4	Upper Jones Tract Maintenance Shaft	Union Island Maintenance Shaft	0	0	0	0	0	0	0	0	0	0	0	0
4	Union Island Maintenance Shaft	Surge Basin Reception Shaft	0	0	0	0	0	0	0	0	0	0	0	0
Total Leakage (GPM)			0	0	0	0	0	0	0	0	0	0	0	0

TABLE A-5: Leakage Rates, 1.0 Tunnel Diameter Ground Load