Design of Precast Segmental Tunnel Lining for Pawtucket CSO Tunnel Project

Irwan Halim and Seung Han Kim
AECOM

Vojtech E. Gall
Gall Zeidler Consultants

ABSTRACT: This paper will describe design of precast segmental tunnel lining for the Pawtucket CSO Tunnel Project in Providence, Rhode Island. The tunnel is 11,700 LF long with 30-foot finished diameter to be constructed in complex sedimentary rocks. Special features of the segments include a large keystone and no connector used on radial joints. The typical segments will be steel fiber reinforced, and hybrid segments with additional steel rebars and shear bicones will be used around the adit openings without use of any structural framing. The important design considerations will be discussed, including special 3D segment to adit connection analyses performed.

PROJECT OVERVIEW

The Pawtucket Tunnel Project is the first phase of the Narragansett Bay Commission (NBC) Phase III Combined Sewer Overflow (CSO) Program designed to reduce CSOs from the communities of Pawtucket and Central Falls in Rhode Island, USA. Phases I and II of the program were focused on the Providence area and were completed in 2008. The Pawtucket Tunnel is planned to have a finished inside diameter of 30 feet and a length of approximately 11,700 feet. The tunnel will be a rock tunnel with depth to invert ranging from 115 to 155 feet. The tunnel will be excavated using a TBM and lined concurrently with gasketed precast concrete segments. This project is being implemented using a design-build delivery process. The Design-Build Contractor consists of a joint venture of CBNA and Barletta (CBNA-Barletta JV); also known as CB3A. The prime designer is AECOM. GEI Consultants is assisting AECOM with geotechnical engineering and field support. Design subconsultants include Gall Zeidler Consultants, Mueser Rutledge Consulting Engineers and BETA Group, Inc.

The Pawtucket Tunnel Project construction will include a main conveyance and storage tunnel; a tunnel boring machine (TBM) launch shaft and receiving shaft; tunnel pumping station; drop and vent shafts and connecting adits. The tunnel construction will be performed with a hybrid TBM, capable of operating in a closed, pressurized-face earth pressure balance (EPB) mode if conditions warrant. As the TBM advances, the tunnel will be lined with precast steel fiber reinforced concrete segments. The main conveyance tunnel will be a deep rock tunnel with an invert generally located at El –87 feet or about 117 feet below ground surface (bgs) at its downstream end. The tunnel will rise in grade at a proposed slope of 0.1 percent to its upstream end with an invert at approximately El –76 feet or about 126 feet bgs at the TBM receiving shaft. The project location is shown on Figure 1. The tunnel boring machine Launch Shaft will be 60 feet in diameter and extend approximately 150 feet below existing ground. It is located 170 feet, shaft center-to-center, northwest of the Tunnel Pumping Station (TPS) Shaft in alignment with the tunnel. The Launch Shaft will be connected to the TPS Shaft with a 10-foot diameter Suction Header Tunnel. The TBM Tail Tunnel will extend 75 feet to the southeast of the Launch Shaft. The TBM Starter Tunnel will extend approximately 220 feet along the tunnel alignment. There are four drop shaft locations across the project site: DS-218, DS-213, DS-205, and Upper BVI DS. The TBM Receiving Shaft is in a parking lot north of the Blackstone River and west of Roosevelt Avenue. The Receiving Shaft will be 36-feet in diameter and extend approximately 130 feet below existing ground.

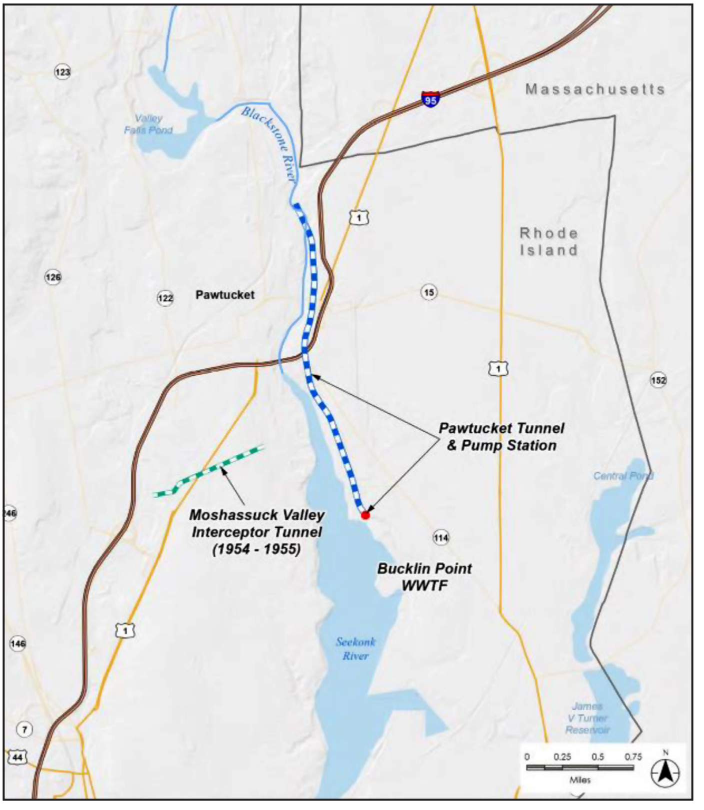


Figure 1. Project area map

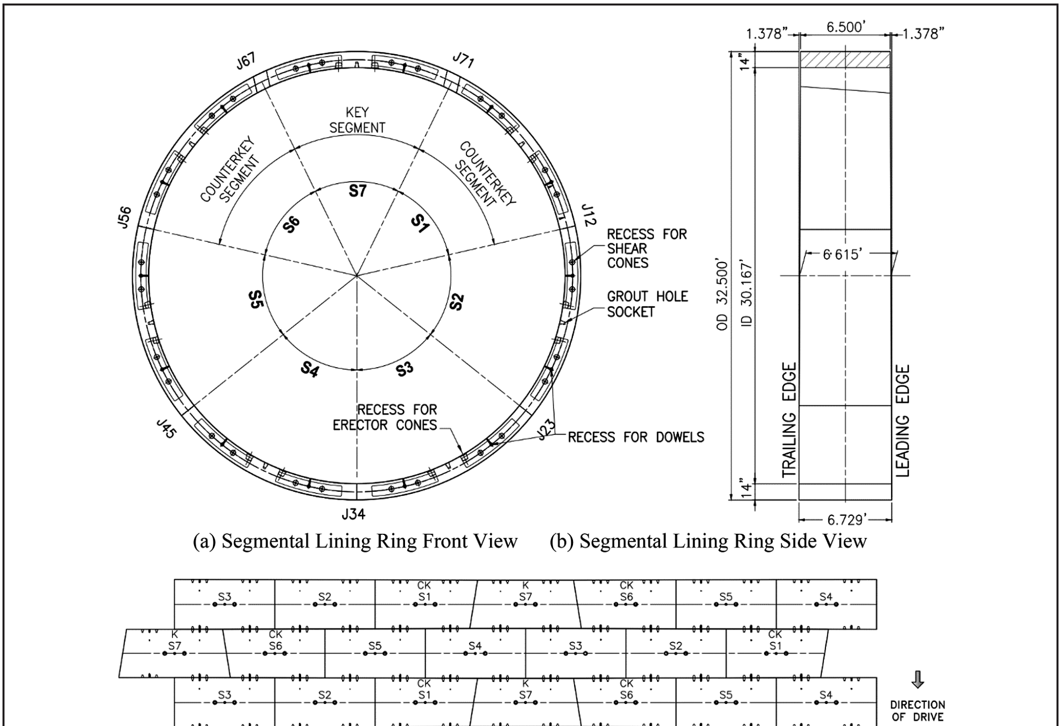
TYPICAL TUNNEL SEGMENTAL LINING DESIGN

The design of the precast concrete tunnel lining is an iterative process during which selected section details are assessed repeatedly until the design objectives for constructability, safety, and serviceability are satisfied. For brevity, the final details of selected design section and the highlights of numerical analyses are presented in the following paragraphs.

Tunnel Lining Design

The entire length of the tunnel will be constructed in the siliciclastic bedrock of Rhode Island formation overlain by glacial till deposits and other fill materials and will be located below the groundwater table. A seven-piece universal tapered ring system was adopted as shown in Figure 2. The tunnel lining ring is 14"-thick, 30'-2" in internal diameter, and 6'-7" in length, and consists of four (4) regular segments of rectangular geometry, two (2) counter-key segments of right trapezoidal geometry, and one (1) wedge-shaped key segment. All segments will be staggered to avoid creating cruciform joints which could cause leakage and structural distress due to stress concentration. As the key segment cannot always be installed at the tunnel crown, the TBM will need to be able to hold segments in place during ring assembly using erector and support roller system. The length of the ring was selected by balancing between the constructability factors (ease of transportation, assembly, and ability to negotiate curves) and the utility factors (limiting joint number to reduce leakage and production cost and to increase tunnel advancement rate). The selected ring length is close to the upper bound of those recommendation by the ITA (2019) for the size of the tunnel.

The wedge-shaped key block is often designed as a smaller size piece than other segments because smaller segments are easy to handle. Lately, a large key segment is becoming more accepted in the industry. Our design adopted similar-sized segments. The chord lengths at centerline are about 14.1 feet for the regular segments, 14.0 feet for the counter-key segments, and 14.3 feet for the key segment. It is advantageous to make all the segments as similar size as possible because of structural and constructability reasons. The longitudinal joints evenly spaced within the ring lead to increased load bearing capacity and reduce ring distortions. The larger key segment also



(a) Segmental Lining Ring Front View (b) Segmental Lining Ring Side View

(c) Schematic View of Developed Intrados

Figure 2. Adopted tunnel lining design

can reduce the size of the regular segments (and therefore the number of longitudinal joints). The segments will not be bolted to each other at the longitudinal joints because holding force is available from ring thrust. This will help increase tunnel advancement rate because bolting is eliminated which could be often time-consuming. The ring will be connected to each other at the ring joints using fourteen (14) equally spaced dowels (SOF FAST 110). In addition, fourteen (14) pairs of equally spaced shear connectors, i.e., shear bicones with a steel core (Optimas Sofrasar F500) were specified at the ring joints for centering and shear recovery purposes at the tunnel adit openings as described later.

For waterproofing purposes, all-around ethylene propylene diene monomer (EPDM) compression gasket (Datwyler M389 33 “Doha”) profile is specified which fits into an about 0.53" deep groove installed at ring and longitudinal joints. The compression gasket is anticipated to resist up to 25 bars of hydrostatic pressure under the design compression and allowable offset scenario, which was sufficient to withstand the maximum anticipated groundwater pressure of 5 bars. The typical segments will be reinforced by steel fibers (no less than 60 pounds per cubic yard of Dramix 4D 80/60). The minimum required strength at 28-day adopted for the design was f_c = 6,500 psi of the characteristic compressive strength and f'^{D}_{150} =700 psi of residual flexural strength at 3.5 mm crack mouth opening displacement (CMOD). The segment thickness was selected to withstand all short-term and long-term loading cases and service conditions. To achieve the desired 100-year service life, the selected segment thickness (14 inches) includes 2.35 inches of sacrificial concrete layer as a measure to protect the tunnel lining from concrete degradation due to hydrogen sulfide (H_2S) gas from the CSO water. The tunnel lining was designed in a way that the loss of a maximum 2.5-inch sacrificial layer does not impact the structural integrity of the tunnel lining system at the end of its design life. The large amount of sacrificial concrete thickness was due to the use of non-calcareous granitic aggregates, since limestone aggregates cannot be sourced easily for the project.

Tunnel Lining Analysis

To demonstrate the adequacy of the adopted tunnel lining design, the stress and deformation of the tunnel lining were evaluated using two-dimensional (2-D) numerical analysis. Total of five analysis sections were selected as representative of variations

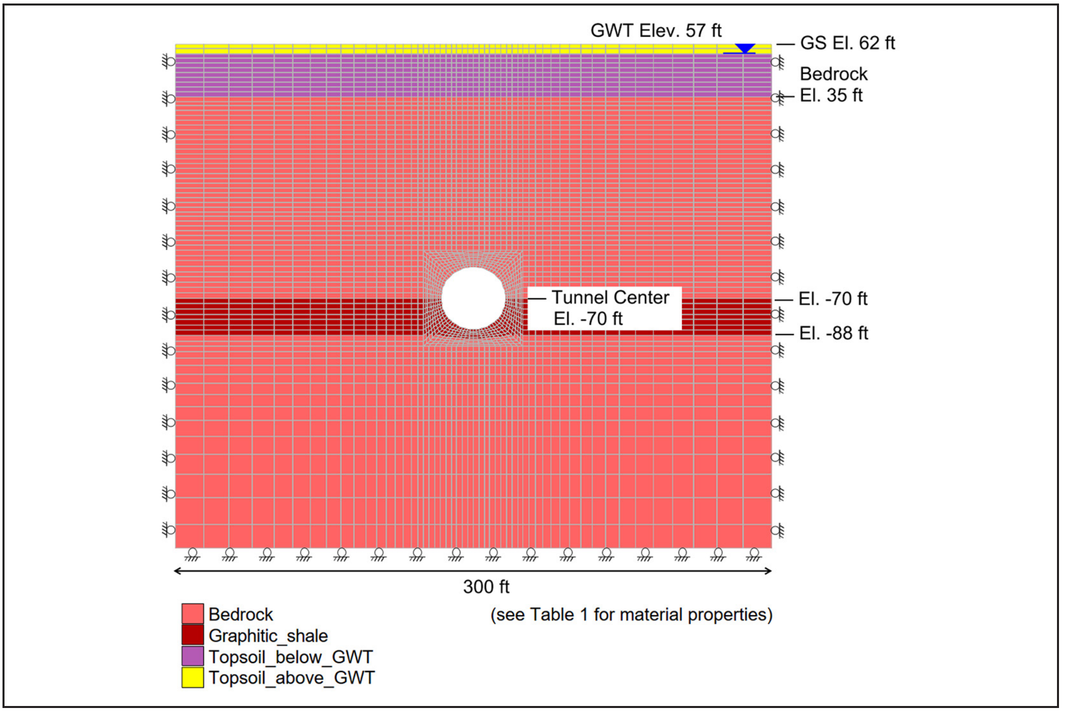


Figure 3. Model overview (analysis section 118+00)

in overburden depths and anticipated ground and groundwater conditions. Among the five analysis sections, the section that was cut at Station 118+00 was found to govern the design due to the presence of intensely fractured soft and weak graphitic shale layer that traverse the tunnel alignment. The graphitic shale layer is approximately 20-foot-thick sub-horizontal layer imbedded within bedrock. The overview of the model at Station 118+00 is presented in Figure 3.

The analysis was done in stages to evaluate the lining forces and the deformations under various loading conditions that are expected during the construction and throughout its design life as presented in Figure 4. Analysis stages 1 through 4 represent the conditions during or shortly after the construction, and therefore, the original tunnel lining thickness (14 inches) was used in the model. In analysis stage 1, the initial equilibrium before tunnel excavation was established according to the material properties presented in Table 1. In analysis stage 2, the tunnel was excavated with ground relaxation. The ground relaxation was modeled using traction control. Both 50% and 80% ground relaxation ratio were tried and the conservative result was used. In analysis stage 3, the tunnel lining was installed, ground was fully relaxed, and the secondary grouting pressure was applied. The secondary grouting pressure applied at the tunnel crown was in accordance with ACI 544.7R: a triangular pressure distribution that covers 36° area in the tunnel crown with 30 psi peak pressure above groundwater pressure. In analysis stage 4, the gantry train load of about 150 tons was applied at the rail sleeper locations. Analysis stage 5 represents the long-term operational conditions during the design life. In analysis stage 5, the tunnel lining thickness of 11.5 inches was used due to acid attack resulting from exposure to H_2S gas except for the bottom 60° area in the tunnel invert which is a submerged area due to minimum CSO storage and flow.

The tunnel lining was modeled as an elastic beam element with an effective (reduced) moment of inertia to account for the segment joints using Muir Wood's (1975) formula:

$$I_e = I_j + I_g\left(\frac{4}{n}\right)^2$$

where, I_e is the effective moment of inertia, I_j is the joint moment, and I_g is the moment of inertia of the gross lining section, and n is the number of joints. Based on the tunnel lining design, n=7 and I_j=0 was used conservatively. The elastic modulus of the concrete was calculated using the relation E_c=57,000$\sqrt{f_c'}$ and divided by $(1 - \nu^2)$ to account for the plane-strain conditions of the concrete lining that is continuous in the direction perpendicular to

Table 1. Geotechnical properties of rock and soil

Rock Properties	Bedrock	Graphitic Shale
Total unit weight (γ_t)	170 pcf	140 pcf
Uniaxial compressive strength (σ_{ci})	7,300 psi	4,600 psi
Geotechnical Strength Index (GSI)	40 to 50	30 to 40
Rock mass deformation modulus (E_{rm})	1,190 to 2,280 ksi	74 to 140 ksi
Poisson's Ratio (ν)	0.24	← *same*
Hoek-Brown parameter (m_b)	1.76 to 2.52	0.493 to 0.704
Hoek-Brown parameter (s)	0.0013 to 0.0039	0.0004 to 0.0013
Hoek-Brown parameter (a)	0.511 to 0.506	0.511 to 0.522
Lateral earth pressure coefficient (K_0)	0.5 to 5.0	1.0 to 2.4
Soil Properties	**Top-Soil**	
Total unit weight (γ_t)	130 pcf	
Elastic modulus (E)	3.5 ksi	
Poisson's Ratio (ν)	0.30	
Friction angle (ϕ)	35°	
Lateral earth pressure coefficient (K_0)	0.48	

Table 2. Mechanical properties of 14-inch-thick and 11.5-Inch-thick tunnel lining

	14-Inch-Thick Lining	11.5-Inch-Thick Lining
Unit weight (γ)	150 pcf	← *same*
Characteristic Compressive Strength (f_c')	6,500 psi	← *same*
Residual Flexural Strength ($f_{150}'^{D}$)	800 psi	← *same*
Poisson Ratio (ν)	0.2	← *same*
Elastic Modulus (E_c)	4,790 ksi	← *same*
Gross Moment of Inertia (I_g)	$I_g = 0.132\ \text{ft}^4$	$I_g = 0.073\ \text{ft}^4$
Effective Moment of Inertia (I_e)	$I_e = 0.043\ \text{ft}^4$	$I_e = 0.024\ \text{ft}^4$

the analysis plane. The mechanical properties of the tunnel lining are presented in Table 2.

Figure 5(a) shows the calculated axial force, shear force, and bending moment for the tunnel lining at Station 118+00. Figure 6 shows the thrust-moment (P-M) capacity curves for the 14-inch-thick tunnel lining and the 11.5-inch-thick tunnel lining after 2.5 inches of section loss. The P-M curves were developed in accordance with the procedure described in ACI 544.7R. The factored maximum axial forces and the bending moments calculated from the model at analysis stage 5 are plotted on the P-M space. The load factor of 1.4 was multiplied to all calculated axial forces, shear forces, and bending moments to account for the combined effects of ground loads, grout loads, and groundwater pressures. The calculated forces and bending moments were found to fall inside the P-M curve for all the sections and cases considered. Figure 5(b) shows the calculated tunnel lining deformation at Station 118+00 from which ring distortions in horizontal and vertical diametral directions were calculated. The maximum ring distortion was found to be about 0.2 percent from all cases considered, and this is lower than the maximum allowable ring distortion of 0.5 percent. The analysis showed that the performance of the adopted tunnel lining design would be satisfactory. The tunnel lining will be able to withstand the anticipated loads and meet the ring distortion requirement.

Verification of Tunnel Lining Design for Special Conditions

The adopted tunnel lining design was evaluated for various conditions using the similar approach described in the previous paragraphs. These include the variation of the anticipated groundwater levels, dewatering of groundwater during construction, and crossing of intensely fractured fault zone. The analysis showed that the tunnel lining will be able to withstand the anticipated loads and meet the ring distortion requirement for all these conditions. A seismic analysis was performed to evaluate the effects of seismic waves vertically propagating perpendicular to the tunnel axis. Owing to relatively low seismic hazard and for simplicity, a time history analysis was deemed unnecessary, and instead a transverse racking analysis was performed to quantify the racking deformations and their effects on the tunnel lining. The analysis followed the procedure described in Hashash et al. (2001). The analysis was done using the same FLAC model cropped around the tunnel

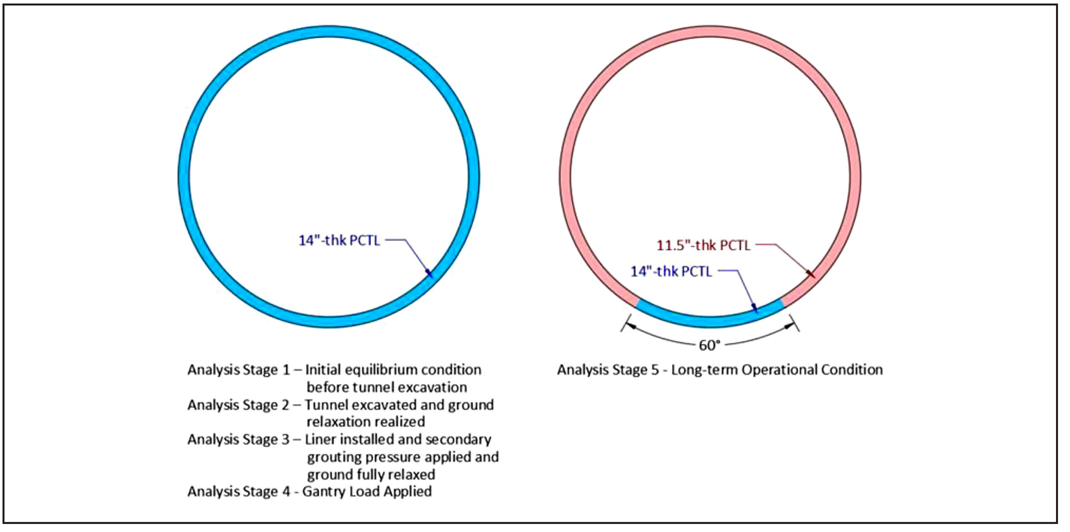
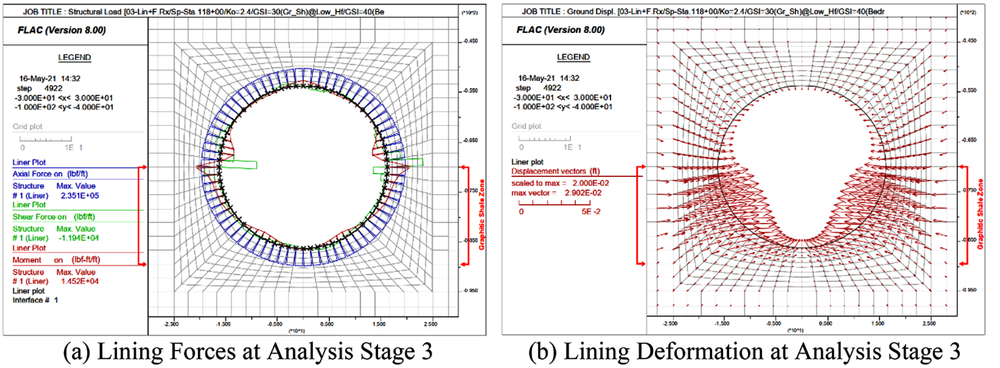


Figure 4. Analysis stages and tunnel lining thickness



(a) Lining Forces at Analysis Stage 3 (b) Lining Deformation at Analysis Stage 3

Figure 5. Calculated tunnel lining forces and deformation (analysis section 118+00)

opening to include the rock materials only without top-soil materials.

The prescribed boundary shear stress was calculated based on the peak ground velocity (PGV) associated with the shear wave velocity of the assumed design earthquake with estimated peak ground velocity of PGV=7.6 cm/s, PGV/PGA ratio of 66, and the apparent shear wave velocity of C_s=2,490 ft/s in bedrock. The design earthquake event can occur at any time during the 100-year design life of the tunnel. Therefore, the additional lining force calculated from the transverse racking analysis (Figure 7) was compared to the capacity of the 11.5-inch-thick tunnel lining after 2.5 inches of section loss. The calculated lining forces and bending moments including the effects of the earthquake ground motion were found to be within the capacity of the 11.5-inch-thick tunnel lining.

TUNNEL ADIT CONNECTIONS DESIGN

The Pawtucket tunnel is connected to four drop shafts by tunnel adits along its alignment. The tunnel adits are named after the drop shafts to which they connect. The adits, from south to north, are referred to as the Drop Shaft (DS) 218, DS 213, DS 205, and Upper Blackstone Valley Interceptor (UBVI) adits. The DS 218 adit connects to the SEM starter tunnel portion of the alignment and is therefore constructed as a typical SEM to SEM connection. The DS 213, DS 205 & UBVI adits, however, connect to the segmentally lined portion of the main tunnel alignment. The DS 205 & UBVI adits are planned as SEM adits.

The DS 213 adit will be constructed by pipe Jacking and microtunneling by MTBM. The SEM tunnel adits will be constructed by making a cut within the segmental lining and mining outwards towards the base of the drop shaft structures. The DS 213 adit will be constructed from within the drop shaft and be jacked towards the main TBM tunnel. Regardless of construction, however, it is foreseen that the adits be constructed after the TBM has passed the corresponding adit connection point.

To simplify construction, the connection design to all three adits has been standardized as much as possible. First, a two-segment wide rectangular cut will be made within the segmental lining. The cut

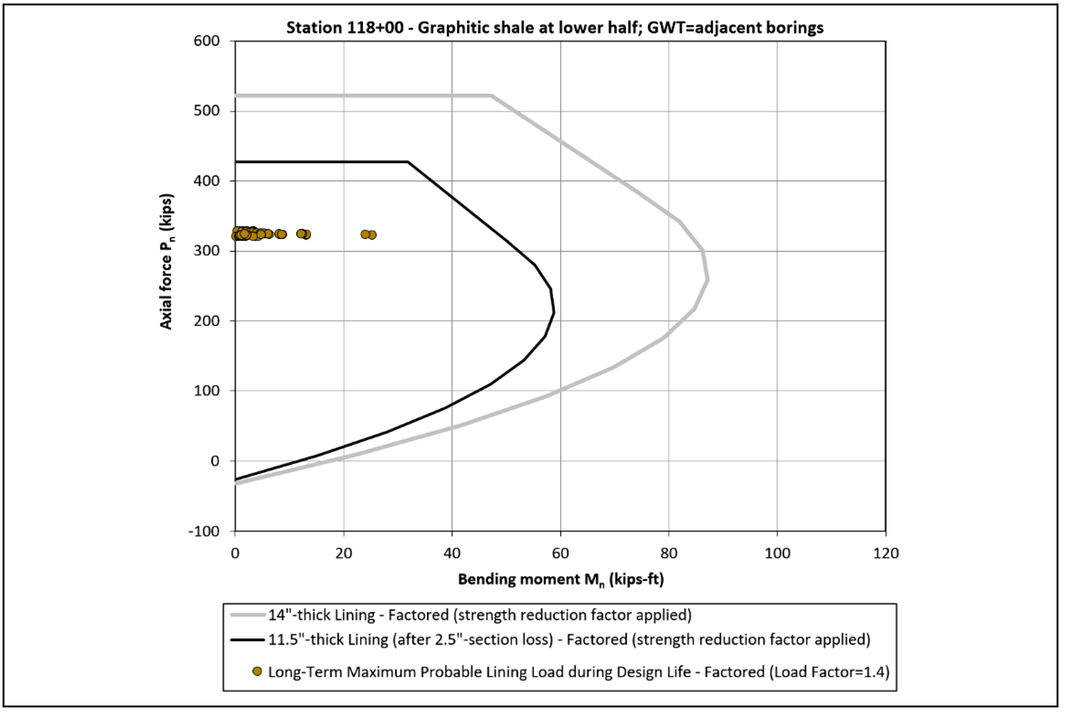
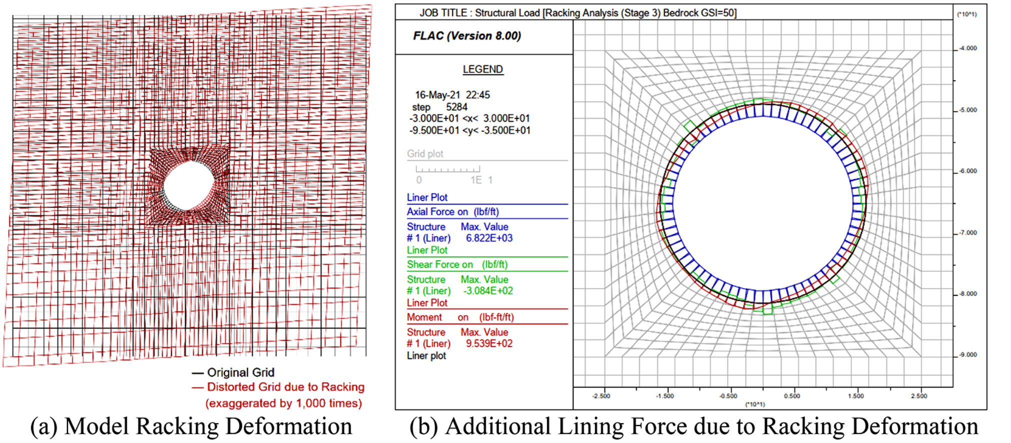


Figure 6. Tunnel lining load carrying capacity estimates using P-M diagram



(a) Model Racking Deformation (b) Additional Lining Force due to Racking Deformation

Figure 7. Seismic transverse racking analysis results

dimensions on the inside face of the segmental lining will be approximately 13 ft wide by 12 ft tall. The cut will be made along the longitudinal joints of the segmental lining on the sides of the cut, and through the segments along the top and bottom of the cut. The cut opening will be temporarily supported by additional reinforcement and shear elements included within the segments of the four TBM rings at and immediately adjacent to the opening. The specially reinforced segments around the opening have been designed such that no additional external bracing or framing is necessary for the temporary support of the opening. As described previously, the standard TBM lining design along the alignment is pure steel fiber reinforced concrete (SFRC) with two shear dowels installed at each jacking pad. Each segment features two jack pads, and each ring is composed of 7 segments, including an oversized keystone (7+0) arrangement. The lining has an inner diameter of 30 ft 2 in, and the segments are 14 in thick. The specially reinforced segments around the segment opening feature a heavy rebar cage as well as 2 additional high-capacity shear cones per segment ring joint installed in the middle of the jack pads between the shear dowels. The geometry of the specially reinforced segments is equivalent with that of the typical segments. A schematic of the cut and segment layout is shown in Figure 8.

After making the cut in the TBM lining, a short 5-ft long, SEM stub tunnel will be excavated. The excavation diameter will roughly follow the rectangular dimensions of the cut but will have slightly rounded sides and a crown to allow for some rock arching. The temporary support of excavation will consist of crown and sidewall bolts along with a protective shotcrete layer along the sidewalls, roof, and face of the excavation. Excavation of the DS 205 and UBVI adits will immediately follow the construction of the stub tunnel by drill & blast (D&B). Excavation will progress outwards from the stub towards the drop shafts. In the case of the DS 213 Adit, which will be constructed by pipe jacking & MTBM, the stub tunnel will act as a reception area for the MTBM. The MTBM will break through the face of the stub tunnel and be pulled out through the main TBM tunnel.

After excavation of the adits are complete, or, in the case of DS 213, the adit pipe has been jacked into the temporary stub tunnel opening, a monolithic cast-in-place (CIP) concrete collar will be cast around the opening and within the stub tunnel. A long section of the collar is shown in Figure 8. The collar will be constructed of SFRC with additional rebar reinforcement. The outer dimensions of the collar will follow those of the SEM excavation, whereas the inner portion of the collar will be cast to be flush with the

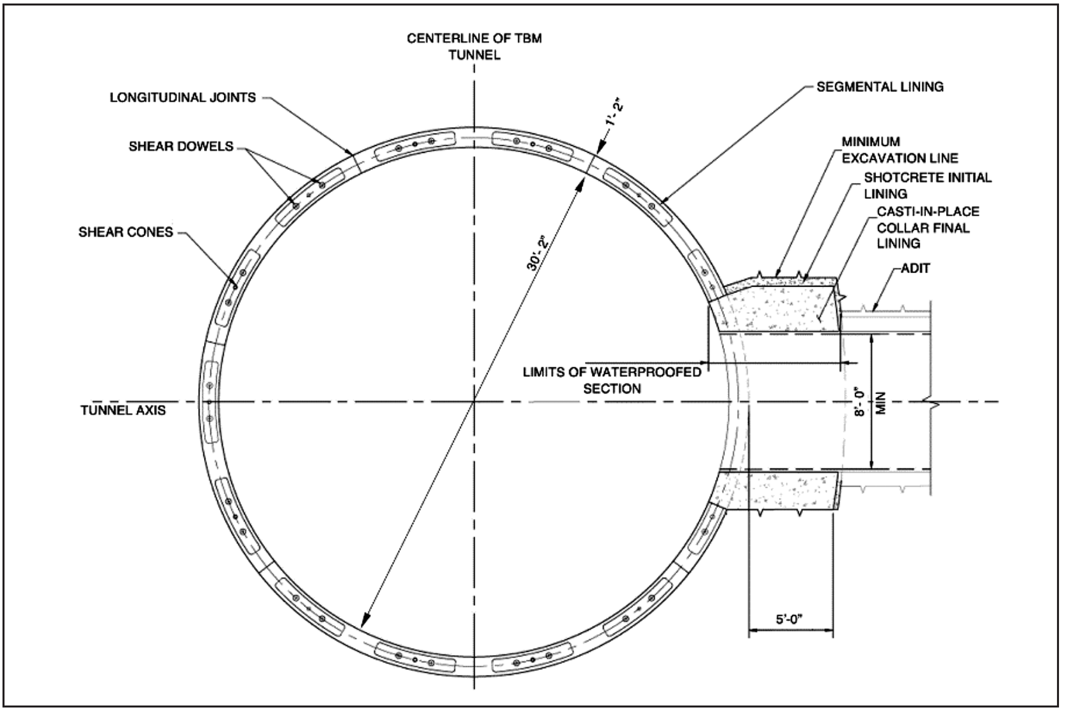


Figure 8. TBM adit opening ring cut with final cast-in-place concrete collar and adit

8-ft diameter adits (the jacked MTBM pipe inner diameter is slightly less than 8-ft). The CIP collar is designed to assume all long-term loadings to which the connection will be subject to.

Design Considerations

The in-situ ground pressures, the temporary and permanent ground water loads, the supporting action (or lack thereof) of friction between rings, and the durability of the lining concrete were determined to be the critical design factors. These were considered as follows:

- In-situ earth pressures: The Geotechnical Baseline Report (GBR) predicted horizontal in-situ earth pressure coefficient values, K_0, of up to 5. As such, a K_0 of 5 was assumed to be the base case for the design. In addition, as the tunnel alignment runs through competent rock, a case in which the ground opening is completely self-supported (i.e., the lining assumes no loads) was considered.
- Water pressures: In the permanent case, both high and low ground water loads based on 100-year levels were considered. In the temporary case, only dry conditions are considered. Although the permeability of the rock mass is expected to be low, dry conditions before cutting the segments will be ensured by drilling probe holes through the segments and into the rock and locally dewatered before making the cut.
- Ring Friction: A major stabilizing factor in the lining system is the residual ring friction present between successive rings due to ring compression resulting from TBM jacking. It is difficult to determine the magnitude of this force. As such, temporary load cases with and without an active frictional force due to pre-compression were investigated. Friction in the long-term case is only assumed to be engaged passively due to relative ring deformations.
- Durability: The greatest threat to durability of the TBM tunnel lining is expected to be concrete corrosion due to hydrogen sulfide attack. This is especially critical at the TBM adits connections as they feature rebar reinforcement in addition to the steel fibers, and a minimum rebar cover must be maintained. To avoid excessive section loss due to hydrogen sulfide attack, the specially reinforced segment concrete mix considered use of calcareous aggregates, whereas the standard segment concrete mix will use granitic aggregates. The alkalinity of the calcareous aggregates will mitigate concrete corrosion and it is expected that section loss in this case will only be less than half an inch.

Structural Analysis

Model Setup

To account for the design factors listed above, a staged Finite Element (FE) model was developed for the structural analysis of the lining and collar at the TBM tunnel-adit cut locations using the SOFiSTiK software package. An image of the model is shown in Figure 9.

The outer surface of the model is composed of volume elements representing the annular grout. The grout is modeled with a Mohr-Coulomb material behavior and is connected to the ground with tangential and radial springs. The radial bedding modulus is determined using the method described in the DAUB & ACI 544 (DAUB, 2013; ACI 544, 2016). The tangential bedding modulus is assumed to be a factor of $1/(2+2\nu)$ smaller than the radial bedding modulus in analogy to the relationship between the shear modulus and young's modulus.

The segmental lining is modeled using shell elements. The coupling of the segmental lining to the annular grout is modeled with high-stiffness frictional springs that are allowed to fail in tension. With these assumptions, the springs are effectively equivalent to a small-strain frictional contact condition. A total of nine lining rings are modeled. Two and a half rings of regular segments are modeled at the front and back of the model, and four rings composed of specially reinforced segments are modeled in the center of the model around the cut. Each successive ring is rotated by half a segment. The lining is modelled using a SOFiSTiK-supplied non-linear steel fiber reinforced concrete material law which considers the peak and residual strengths of the fibers. Two sets of springs are modeled in the longitudinal joint, one set simulates the transfer of normal and shear forces and the other set simulates the transfer of moments. The eccentricity of the longitudinal joint contact surface relative to the segment axis is explicitly considered by offsetting the spring contact point with a stiff plate. Circumferentially oriented springs with a very high stiffness mimicking a hard contact transfer the normal forces between segments. The springs also include a stiff perpendicular component to transfer shear forces in radial and longitudinal directions. To limit the size of the shear force in the contact zone, a friction coefficient of $\nu = 0.25$ is applied. The rotational behavior of the moment springs in the concrete hinge of a longitudinal joint is modeled according to the nonlinear behavior specified in the DAUB & ACI 544 Recommendations (DAUB, 2013; ACI 544, 2016). The relationship between the rotation, φ, and the transferable moment, M, is individually determined for each spring at the longitudinal joint

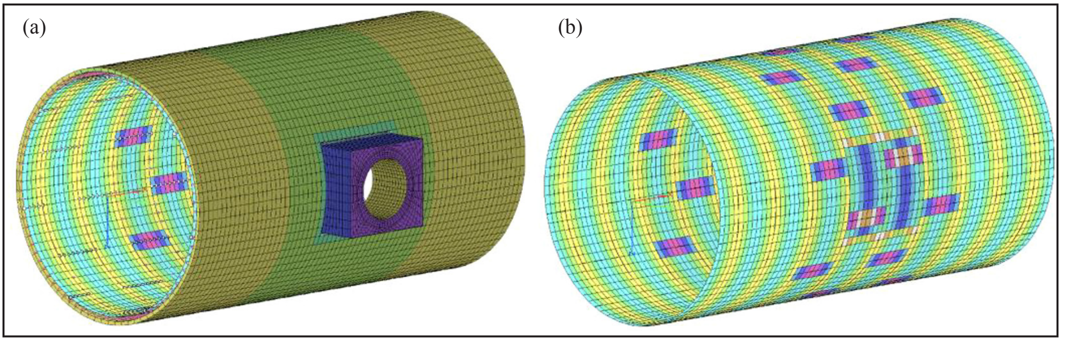


Figure 9. Model of adit connections: (a) adit collar; (b) isometric view of segmental lining ring

based on the magnitude of the normal force, N, in each calculation step.

The coupling between adjoining rings is simulated with springs pre-stressed in longitudinal direction. Coupling is induced through pre-stressing by the thrust forces of the TBM and exerted through friction. The friction coupling in the ring joint is implicitly calculated in analysis through a friction criterion and a transversal stiffness in the springs. The magnitude of the coupling force is proportional to the differential displacements of neighboring rings and to the respective normal force. As initial ram forces during operation will decrease due to the repeated interim release of the rams following ring erections and due to relaxation of the concrete, a reduced force of 50% of the operational thrust is applied as a prestressing force in the structural analyses.

In addition to the frictional coupling between rings, the shear connectors between rings (both bicones and dowels) are explicitly modelled by means of non-linear spring elements. This ensures that the local shear force concentration acting on the segments due to the stiff connector behavior is accurately accounted for in the model. The nonlinear behavior of the springs is based on the actual load displacement behavior of the shear elements, which is taken from manufacturer supplied loading diagrams as shown in Figure 10. As the dowels are made of a synthetic materials, their stiffness is reduced for all long-term load cases to account for relaxing of the synthetic over time.

The collar structure is explicitly modeled with volume elements. Due to the massive structure of the collar, it is assumed to behave linear elastically. On the outer surfaces of the collar, shell-elements provide the bedding for the collar with the same bedding stiffness as for the annular grout. The load transfer from the cut segmental lining and the annular grout onto the collar structure is modeled with high stiffness springs along the cut perimeter.

Loading

To ensure that the proper loading history is accounted for, the FE model is solved in the following staged loading sequence:

1. Apply ground loads. The ground loads are derived by calibrating the loading of the shell model to arrive at similar stress resultants as those derived in the tunnel lining using a plane strain pseudo-3D continuum model of the lining in the ground. The pseudo-3D Model involves assuming a ground relaxation before installation of the lining. To obtain conservative design loads, the pseudo-3D continuum model assumes the ground conditions of the tunnel at the maximum overburden along the alignment. Cases were investigated in which ground loadings derived from an 80% ground relaxation were applied to the tunnel lining as well as the case in which the tunnel lining assumes no rock load (i.e., 100% relaxation of the rock) this last case is considered a reasonable design choice due to the high rock quality along some stretches of the alignment.
2. Apply secondary grouting pressure.
3. Make the cut in the lining. This takes place under dry conditions as previously discussed due to local dewatering. Both cases in which ring joint friction/prestressing is actively and passively present were investigated. This represents the short-term loading.
4. Install collar and apply water loads. This represents a “middle-term” loading. Both high water table and low water tables were investigated.
5. Account for creep of shear elements (dowels and shear cones) and remove active prestressing/friction in ring joint (if accounted for). This represents the long-term loading.

As is evident from the above, several load combinations were investigated. The model used

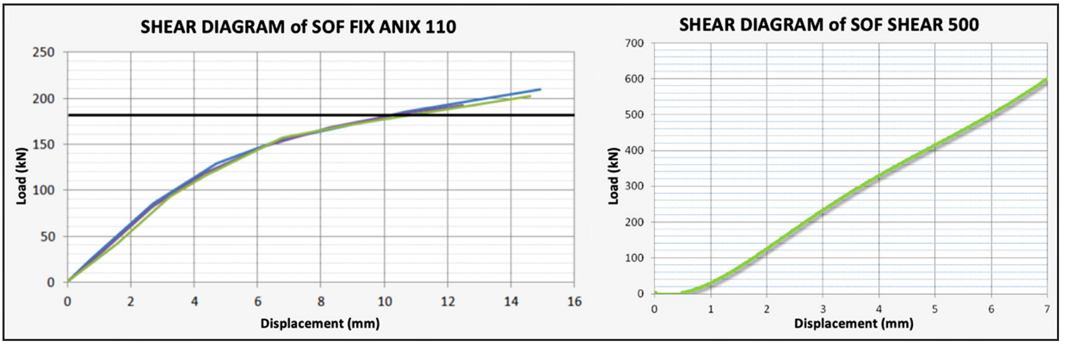


Figure 10. Shear connector behavior. Dowels on left and bicones on right.

accounts for non-linear material behavior, so only characteristic loads were used. All stress resultants were conservatively factored by 1.4 to arrive at design stress resultants for dimensioning.

Analysis Results

The structural analysis indicates that the most critical case for design is the temporary loading of the segments immediately following the cutting of the lining and before installation of the CIP collar. Specifically, the load case in which the ground load is considered is most critical. An image of the predicted displacements of the segments in this loading scenario is provided in Figure 11(a). The maximum displacement is 15.2 mm (0.6") and occurs above the opening at the ring joint. Large displacements only occur in the two middle rings which are completely opened. The uncut special rings, one on each side of the opening, exhibit much lower maximum displacements of 2.7 mm (0.11"). The corresponding principal membrane forces are provided in Figure 11(b). Red vectors represent compression, blue vectors represent tension, and the longer vectors in the crosses depict the direction of the maximum principal force. The load distribution around the opening is clearly visible from the inclination of the principal forces. Areas of high compression and tension occur in the segments directly above and below the opening as the compressive load in the ring is transferred around the segments. As such, high compression occurs in the corners of the opening. Similarly, the load transferred through the shear elements and through friction results in higher compressive forces in the uncut rings next to the opening, especially at points close to the ring joints. The tensile forces that can be observed at the ring joints immediately above and below the sides of cut develop as a counter-reaction to the transfer of the compressive forces around the opening.

The tensile forces as shown in Figure 11(b) are too high to be carried by steel fibers alone and result in a requirement for circumferential rebar reinforcement along the segments. In addition to these tensile forces, splitting forces resulting from increased normal force at longitudinal joints, shear resulting from increased bicone and connector loads, and longitudinal bending within the segments all contribute to the increased reinforcement requirements. In total, to carry the design loadings, the special segments are reinforced with a heavy rebar cage amounting to approx. 190 kg/m^3 (11.9 lb/ft^3) rebar reinforcement. The high reinforcement density is, however, partially a product of the slim dimensions of the segments (14 in thick) and the larger cover requirements on the inside face due to durability issues (2.25 in cover on the inner face vs. 1.5 in on the outer face). The collar, in contrast to the segments, experiences the largest loads in the long-term condition when the creep of the shear elements is considered. This is to be expected, as the creep of the shear elements and loss of ring friction results in more bearing against the collar over time. The loading of the collar is, however, not critical for design as the collar is sufficiently large enough to account for any needed reinforcement.

CONCLUDING REMARKS

This paper describes design of the tunnel segmental lining for the Pawtucket CSO Tunnel Project for both the typical standard segments as well as the special segments around the tunnel adit openings. The design considered stages or sequence in the segment loadings starting from the segment erection inside the TBM tunnel, grouting, tunnelling, and segment cutting for the adit construction. Short-term and long-term loadings on the segment were analyzed including estimated degradation or concrete loss over the project 100-year design life. Although not specifically covered here, the segment design also included other handling and construction related loadings such as lifting and stacking, transporting, and thrusting by the TBM rams inside the tunnel. Serviceability design also included evaluations of maximum crack widths to develop under the various

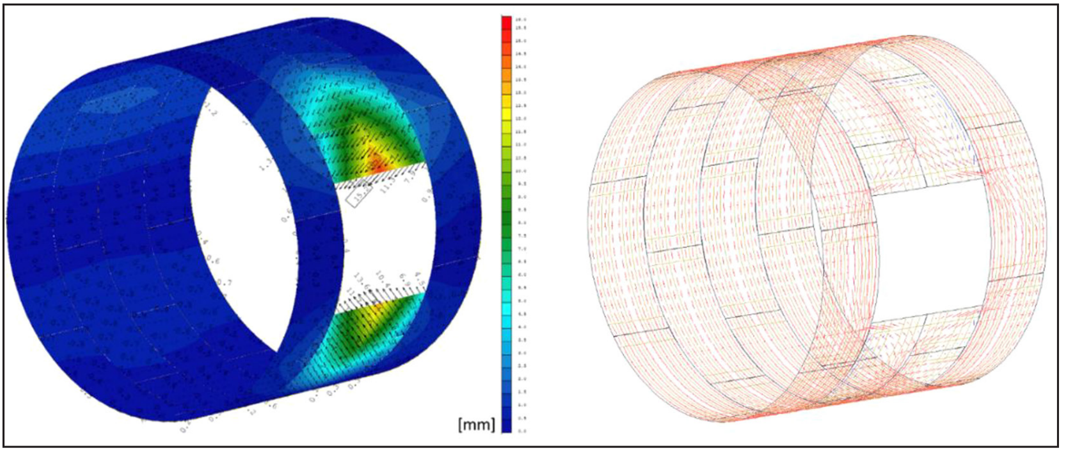


Figure 11. Deformations (a) and principal force orientations (b) immediately around segment cut

loading conditions to ensure compliance with the ACI code requirements.

The presented analysis and design show that it is possible to cut a TBM segmental lining without the necessity for any external supporting elements, provided that sufficient reinforcement and high-capacity shear connectors are designed for within the segments immediately surrounding the cut. Doing so provides several benefits to construction as it eliminates the need for tedious frame constructions to take place within the confined space of a TBM tunnel. The cut can be made immediately following the passing of the TBM and therefor simplifies construction logistics. Furthermore, casting reinforced segments ahead of time improves QA/QC processes, as the rings supporting the cut are produced in a factory environment and their construction can be better controlled. To ensure controlled behavior of the specially reinforced segments at the cuts, a ring at the cut as well as a ring ahead and a ring behind the cut will be monitored (3 rings in total at each cut). This monitoring plan ensures that rings of the same segmentation, i.e., same longitudinal joint locations, are monitored. The rings ahead and behind the cut ring will act as control deformations against which the deformations of the cut ring can be compared.

ACKNOWLEDGMENTS

The authors would like to thank the construction joint venture of CBNA (Bouygues Construction) and Barletta (CBNA-Barletta JV); also known as CB3A for their support in the design work described in this paper. The authors would also like to acknowledge the Narragansett Bay Commission (NBC) as the owner of this great project. The authors also thank Benno Ring for his contribution to this paper.

REFERENCES

ACI Committee 318 (ACI 318). 2019. ACI 318: *Building Code Requirements for Structural Concrete*. Farmington Hills, MI.

ACI Committee 533 (ACI 533). 2020. ACI 533.5R: *Guide for Precast Concrete Tunnel Segments*. Farmington Hills, MI.

ACI Committee 544 (ACI 544). 2016. ACI 544.7R: *Report on Design and Construction of Steel Fiber-Reinforced Precast Concrete Tunnel Segments*. Farmington Hills, MI.

German Tunneling Committee (DAUB). 2013. *Recommendations for the design, production and installation of segmental rings*. Cologne, Germany.

Hashash, M.A., Hook, J.J., Schmidt, B., and Yao, I.C. 2001. *Seismic design and analysis of underground structures*, Tunnelling and Underground Space Technology 16: 247–293.

International Tunnelling Association (ITA). 2019. *Guidelines for the Design of Segmental Tunnel Linings*. ITA Working Group 2. Geneva, Switzerland.

Itasca Consulting Group. 2018. *FLAC User's Manual*, Version 8.0.