San Diego Courthouse Commons Tunnel: Challenges and Design Solutions

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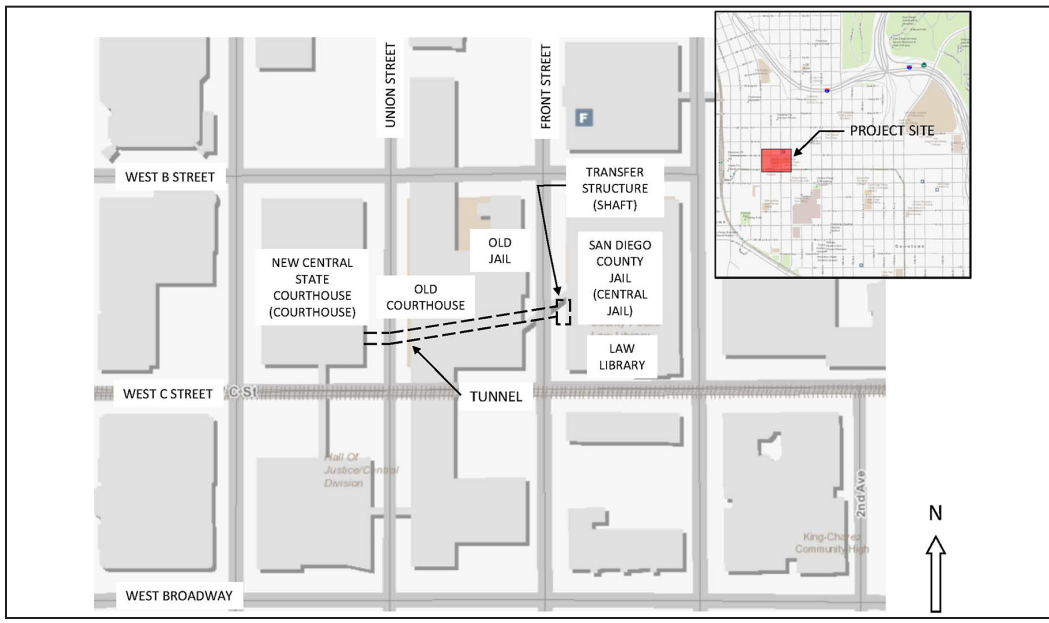
ABSTRACT: The Courthouse Commons Tunnel facilitates the secure transfer of inmates between the County Jail and State Courthouse across a city block in downtown San Diego, California. This shallow tunnel was excavated using the Sequential Excavation Method in soft ground. It passes under 8- and 10-story buildings with a foundation cover of 6 feet, and crosses the active San Diego Fault with a design offset displacement of 18 inches. In this paper, key design challenges of tunnel construction are presented: limiting building settlements without using underpinning of building foundations and incorporating an oversized seismic section with seismic joints and crushable backfill.

INTRODUCTION

The Courthouse Commons Tunnel Project (Project) is located in downtown San Diego, California, and comprises the construction of a pedestrian inmate tunnel (Tunnel) and other associated improvements connecting San Diego County's Central Jail (County Jail), located at 1137 Front Street, with the State-owned New Courthouse (State Courthouse) at 1140 Union Street. The Tunnel will facilitate the secure transfer of inmates between the County Jail and the Courthouse. The Project also provides improvements inside and adjacent to the County Jail to accommodate the Tunnel and improvements inside the State Courthouse to receive inmates who are to be transferred between the two facilities. A new Inmate Transfer Facilities Shaft (Shaft) has been constructed adjacent to the County Jail as part of the connection between the Tunnel and the County Jail. See Figure 1 for a map of the project site and vicinity.



Source: San Diego Geographic Information Source Interactive Map, accessed 1/17/2019

Figure 1. Vicinity map of the project site

The Tunnel is approximately 328 feet long, connecting to the State Courthouse B3 level on the west side of Union Street with the Shaft on the east side of Front Street. The finished Tunnel has a 1% slope to the east with an invert elevation varying from approximately 4.75 feet Mean Sea Level (MSL) on the west to –8.25 feet MSL on the east. The Tunnel has a horseshoe shape with a finished width of 15 feet and a suspended ceiling height of 10 feet with a 6-foot plenum above. An approximately 50-foot-long seismic section is located within the San Diego Fault Zone in the eastern reach of the Tunnel under Front Street. The Tunnel also includes three storage niches and a wider niche to accommodate electric vehicle parking and battery charging station. The locations and dimensions of these niches are shown in Figure 2.

The Tunnel passes below existing basement foundations for the Old Courthouse and Old Jail buildings in the block bounded by Front and Union Streets. The 10-story Old Jail building including a two-level basement is occupied, whereas the 8-story old courthouse building including a two-level basement is unoccupied but remains in place during Tunnel construction. At the west end adjacent to Union Street the old courthouse basement includes an electrical room, which remains in service during Tunnel construction. These buildings will be demolished at some point following Tunnel construction.

The Shaft is approximately 58 feet deep with finished dimensions of 15 feet by 52 feet and provides passage between the Tunnel and the Central Jail B1 level using a stairwell and two elevators. Figure 2 shows the Tunnel and Shaft plan and profile.

The Tunnel and Shaft are part of a larger development of three blocks by a developer, Holland Partner Group. The Engineer of Record for the Tunnel civil and structural is McMillen Jacobs

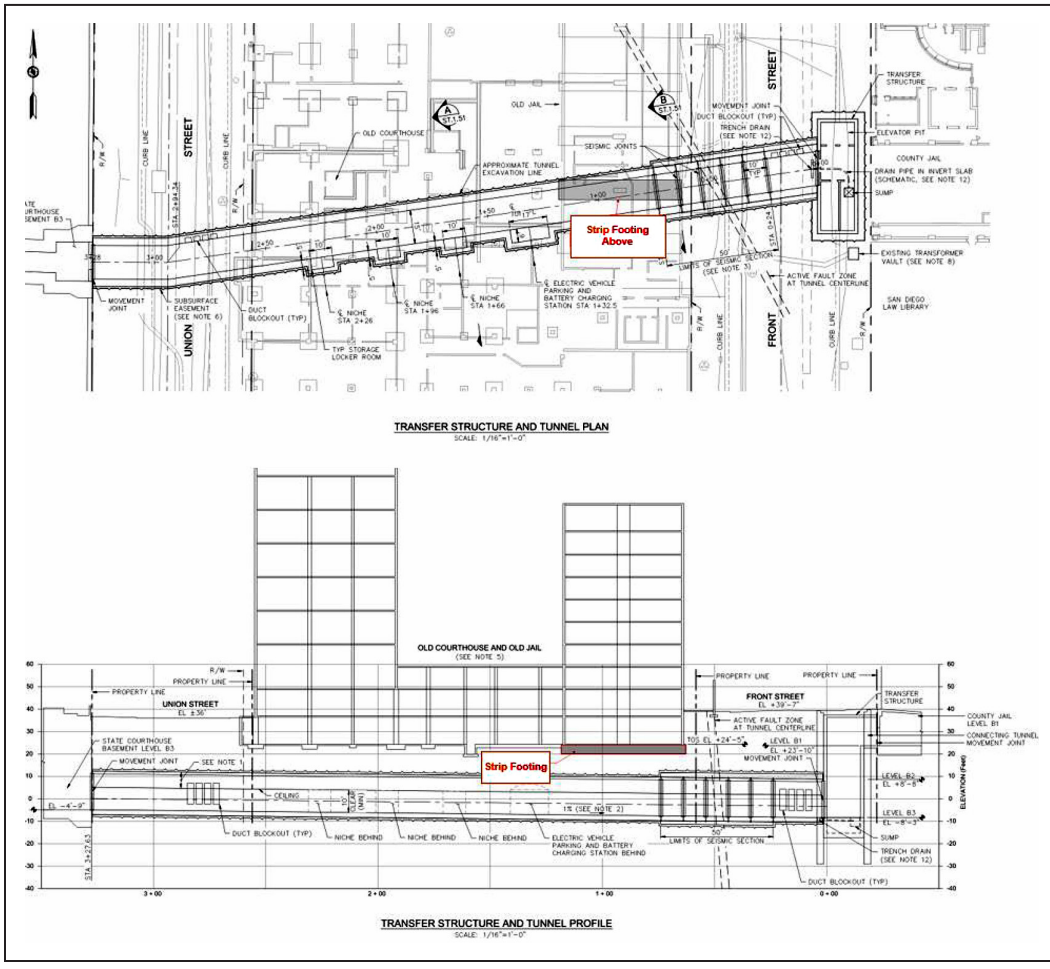


Figure 2. Courthouse Commons Tunnel plan and profile

Associates. The Structural Engineer for Buildings is KPFF Consulting Engineers. The Geotechnical Engineer is Kleinfelder. The Contractor is Atkinson Construction. Once completed, the Tunnel and Shaft will be owned and operated by San Diego County.

GEOLOGIC CONDITIONS ALONG TUNNEL ALIGNMENT AND AT SHAFT LOCATION

The site is underlain by approximately 5 feet of fill over approximately 8 to 13 feet of Pleistocene- to Holocene-age alluvial deposits over the middle to late Pleistocene-age old paralic deposits. The alluvium generally consists of silty to clayey sand and sandy clay. Based on standard penetration testing, the relative density of these materials ranges from medium dense to dense or medium-stiff (Kleinfelder, 2019b).

The Tunnel was excavated in the middle to late Pleistocene-age old paralic deposits (the Bay Point Formation). It is generally interpreted that old paralic deposits formed within a near-shore environment and were deposited across a marine/nonmarine environment. Old paralic deposits consist of poorly sorted, moderately permeable deposits of siltstone, sandstone, and pebbly conglomerates, with dense to very dense, poorly graded sand, poorly graded sand with silt, silty sand, and clayey sand. Gravels and cobbles in a sand matrix are present. Figure 3 shows the geologic profile along the Tunnel alignment and at the Shaft location.

The design groundwater table is at Elevation +3 feet MSL (Kleinfelder, 2019b). The typical variation of the groundwater table is at Elevation +1 to –7 feet MSL. Ground surface elevation is approximately at Elevation +35 feet MSL at Union Street and +40 feet MSL at Front Street. Tunnel invert elevation is at –8 feet MSL at Front Street and –5 feet MSL at Union Street. Therefore, part of the Tunnel and Shaft is below the water table (see Figure 3). The groundwater was specified to be lowered to a minimum of 3 feet below tunnel and shaft invert elevation during construction.

The Tunnel crosses the active San Diego Fault beneath the Project site at the west curb line on Front Street (see Figure 3). The fault strike is at a high acute angle to the Tunnel alignment with a dip of 85 degrees to the northeast. Its width varies from approximately 5 feet on the south to approximately 7 feet on the north across the Tunnel alignment at Tunnel springline. Based on deterministic analysis (Kleinfelder, 2019a), the fault is expected to exhibit up to 15 inches of right lateral displacement in combination with 10 inches of vertical displacement during a design earthquake event. This resolves to a total of 18.3 inches of right lateral, down to the east, vector displacement during the event. Primary displacement is expected to be distributed within the defined fault zone. Refer to Figure 3 for the fault zone location and geometry at the Tunnel horizon.

KEY CHALLENGES AND SOLUTIONS FOR TUNNEL DESIGN AND CONSTRUCTION

The Tunnel was excavated using the Sequential Excavation Method (SEM). Existing conditions vary along the Tunnel alignment as follows (see Figures 2 and 3):

- Station (Sta.) 0+00 to 0+24: Front Street crossing with a ground cover of approximately 25 feet. Existing utilities are located 12 to 18 feet above the Tunnel.
- Sta. 0+24 to 0+74: San Diego Fault Zone. Front Street is located between Sta. 0+24 and 0+60 followed by the Old Jail with strip footings located as close as 6 feet above the Tunnel crown.
- Sta. 0+74 to 1+90: Old Jail where the combination of strip and spread footings are located as close as 6 feet above the Tunnel crown.
- Sta. 1+90 to 2+60: Old courthouse with spread footings located as close as 7 feet above the Tunnel crown.
- Reach from Sta. 2+60 to 3+28: Union Street with a ground cover of approximately 25 feet before intercepting the State Courthouse. Some utilities are located 18 to 20 feet above the Tunnel.

These existing conditions pose challenges to the tunnel design and construction. Key challenges include:

- Limiting ground movements caused by tunnel excavation is critical to protecting the Old Jail since any settlement or damage could have significant security complications.
- Preventing potential damage to the tunnel structure by fault offset displacements is crucial from the designer and owner's perspective for tunnel serviceability following a design earthquake.

Solution to Limiting Building Footing Settlement

As indicated above, the tunnel is excavated entirely in sandy soil (beach sand–like material) with an excavated width of about 28 feet and a height of about 25 feet. A strip footing 9 feet wide and 56 feet long with high surcharge loads is located about 6 feet above the tunnel crown centerline (see Figure 2). The maximum settlement limit for the Old Jail foundation is 1 inch as specified by the Structural Engineer of the project. It should be noted that this settlement limit was higher than a typical limit of 0.5 inch given the fact that the building will eventually be demolished in the future, so cosmetic damage to this

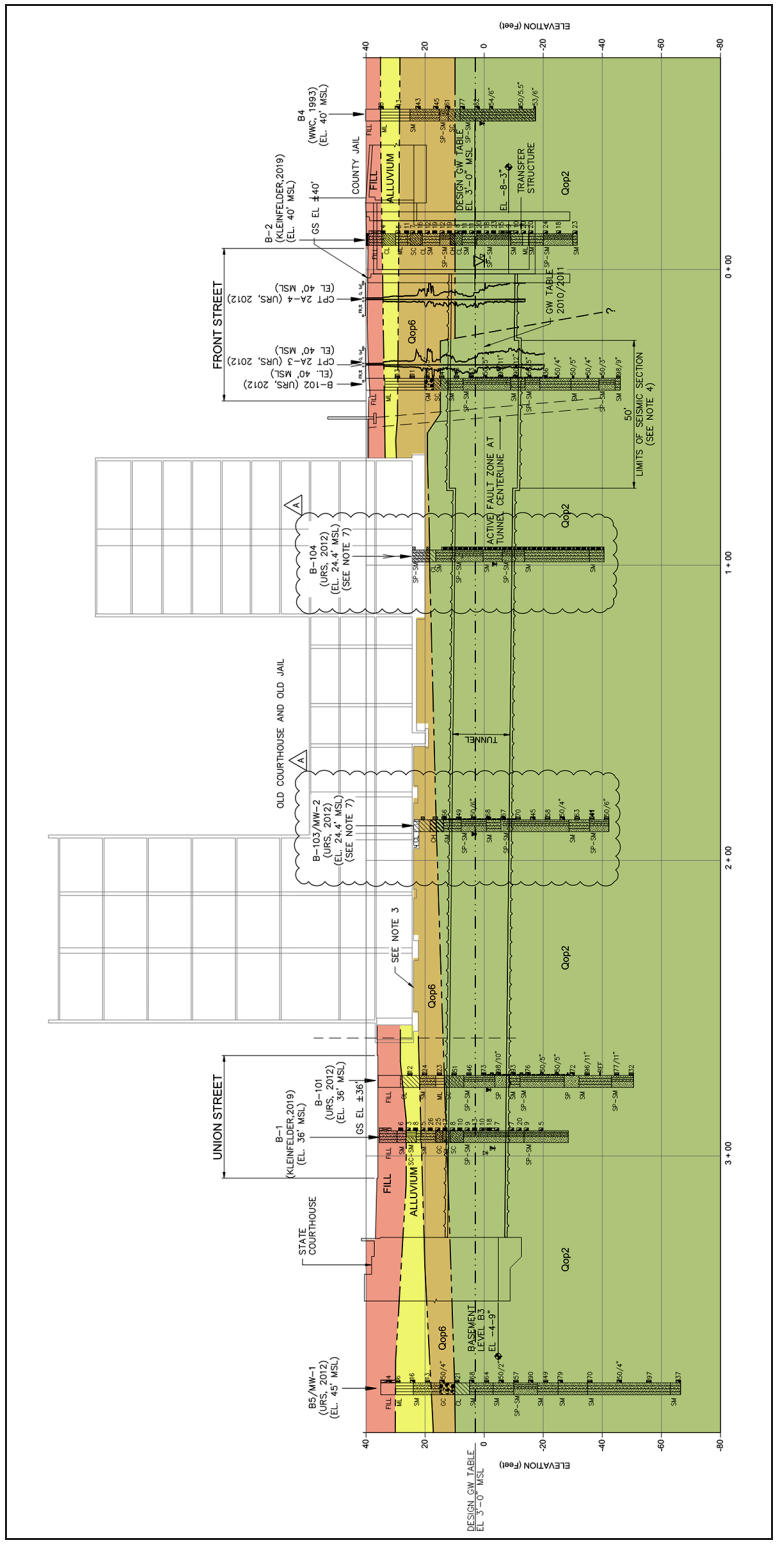


Figure 3. Geologic profile along tunnel alignment and at shaft location

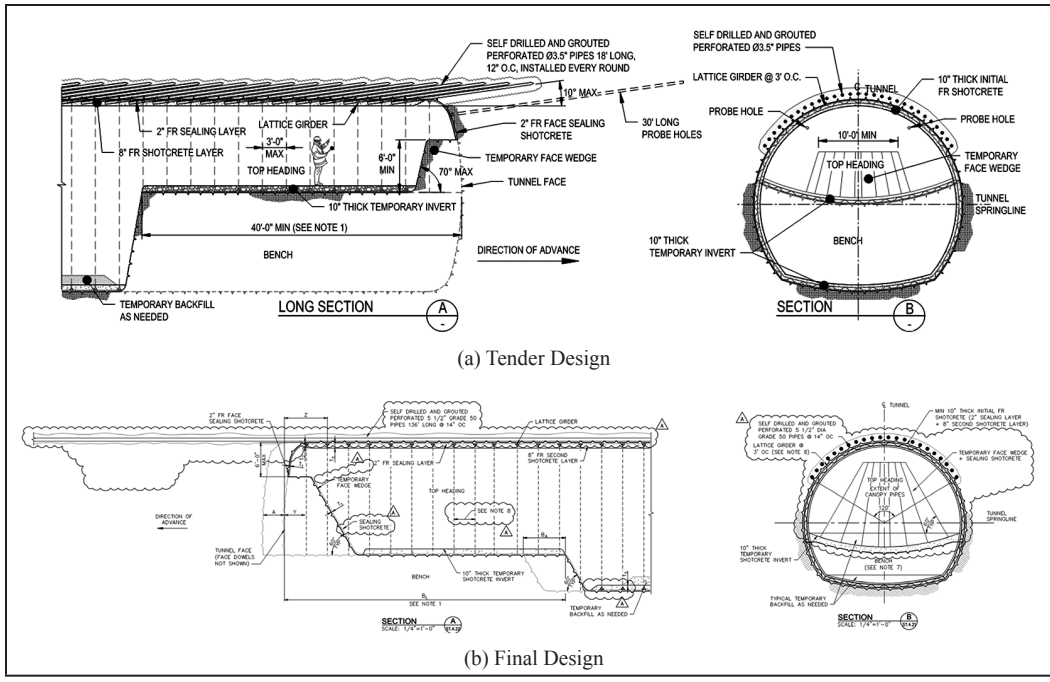


Figure 4. Excavation sequence and initial support measures proposed in tender and final designs

building is allowed as long as it does not affect structural integrity.

During the tender design, the tunnel initial support consisting of 3½-inch-diameter canopy pipes as presupport and 10-inch-thick fiber-reinforced shotcrete lining installed through a top heading and bench sequence was proposed. This design was developed based on an assumption that both the Old Jail and Old Courthouse would be demolished prior to the tunnel excavation. At the start of the detailed design, however, the development plan was modified to keep the Old Jail occupied and Old Courthouse unoccupied but in place during construction. This revised development plan called for a reassessment of the tunnel excavation sequence and initial support requirements since the protection of the Old Jail building became a critical design issue.

The originally proposed combination of Tunnel presupport and support was now called into question and proposed solutions were exacerbated by the parallel orientation of the strip footing with respect to Tunnel alignment and the location of the footing along the Tunnel centerline. One potential solution involved an underpinning design for the strip footing. This was quickly discarded because of the logistics of working under jail security protocols in a basement level with difficult access and low headroom where the underpinning micropiles themselves would be founded below the Tunnel horizon, and thereby conflict with tunnel construction. Thus, during further evaluation of the adequacy of the tender design approach, consideration focused on the viability of relying on the stiffness of the presupport itself to control ground settlement.

Because of the complexity of this design issue, detailed assessment based on two- and three-dimensional (2D and 3D) numerical analyses using FLAC and FLAC3D were undertaken to simulate the sequence of tunnel excavation and initial support installation. The effect of using canopy pipes as presupport and a temporary face wedge as face support was also explicitly analyzed. Initial findings identified two key factors controlling building settlement: the height of the top heading excavation and the stiffness of canopy pipes used for presupport.

Height of Top Heading Excavation

As shown in Figure 4(a), the tunnel excavation was sequenced into a top heading and bench configuration with approximately the same height, resulting in a width-to-height ratio of about 2.2:1.0 for the top heading advance. Preliminary results from the analyses based on this ratio indicated that the maximum building settlements would likely exceed the specified limit of 1 inch, especially during the top heading advance. This was because this high width-to-height ratio would not yield an efficient geometry for stress redistribution around the tunnel opening and the effect of

vertical stresses above the tunnel would dominate and push the arch downward, resulting in a large deflection. In the process, the sidewalls of the top heading opening would be pushed outward, further increasing the arch deflection and resulting in a deformed shape of horizontal ovaling. The ideal circular geometry of the opening maximizing stress redistribution around the opening and limiting tunnel deformation would require a full-face excavation. This would introduce concern of face stability for a large opening in sandy soil, including the ability to service the Tunnel with the size of excavation equipment needed via the relatively narrow and congested Shaft.

Further analyses were undertaken based on a reduced width-to-height ratio of about 1.5:1.0 by increasing the height of the top heading while maintaining the same width; see Figure 4(b). Results of these analyses indicated that the maximum building settlements could be reduced to less than 1 inch, which confirmed that lowering the width-to-height ratio approaching 1.0 could enhance the effect of in situ horizontal stresses by counteracting the effect of vertical stresses and building surcharge loads. As a result, overall settlements could be reduced. For this reason, the design specified a top heading height at the centerline equal to about three-fourths of the total height of the excavated tunnel. However, the main drawback was an increased face area, requiring enhancement of face support measures.

Size (Stiffness) of Canopy Pipes

Another adjustment of the design was to specify the use of 5½-inch-diameter canopy pipes for presupport. The use of 3½-inch-diameter canopy pipes as proposed in the tender design was judged as inadequate when the building surcharge loads were considered. The increased size of canopy pipes provides a stiffer pipe roof with increased load-carrying capacity to support the building surcharge loads and reduce the ground movements ahead of the face. This change also enhanced face stability as the surcharge loads on the face wedge ahead of the face could be supported by the stronger canopy pipes. Condon Johnson, Atkinson's subcontractor for canopy pipe installation, proposed the reduced number of larger 7⅝-inch-diameter pipes while maintaining the originally specified 14 inches between adjacent pipes. This change increased the overall stiffness of the pipe canopy support system, contributing to minimizing observed building settlement to about 0.5 inch (refer to discussions below). Figure 5 shows a completed array of installed canopy pipes in the Tunnel.

Tunnel Excavation and Initial Support Measures

To address the variations of the existing conditions, four Tunnel Excavation and (Initial) Support Measures (TESMs) were proposed in the final design. These TESMs define different excavation and initial support requirements for these four reaches as follows:

- TESM1 (Sta. 0+00 to 0+24). Canopy pipes (Stage 1: one set, 99 feet long, 5½ inches in diameter, later modified to 7⅝ inches as discussed above, at 14-inch clear spacing) for presupport; lattice girders and 10-inch-thick shotcrete with closed temporary invert, temporary face wedge, and face dowels as required.

Figure 5. Photo of a completed array of installed canopy pipes in tunnel

- TESM2 (Sta. 0+24 to 0+74). Continuation of canopy pipes installed under TESM1 as presupport; lattice girder and 10-inch-thick shotcrete with closed temporary invert, temporary face wedge, and face dowels as required.
- TESM3 (Sta. 0+74 to 1+90). Canopy pipes (Stage 2: one set, 136 feet long, otherwise identical to Stage 1) as presupport; lattice girder and 10-inch-thick shotcrete with closed temporary invert, temporary face wedge, and face dowels as required.
- TESM4 (Sta. 1+90 to 3+28). Pipe spiles (46 sets: 12 feet long, 2 inches in diameter at 12-inch OC installed every advance for 9-foot overlap) as presupport; lattice girder and 10-inch-thick shotcrete with closed temporary invert, temporary face wedge, and face dowels as required.

Figure 4b shows a typical tunnel cross-section and associated initial support components for TESM1 and TESM2.

Grouting Plan for Old Jail Strip Footing and Grade Beams

To address the risk of excessive settlement even with the design revisions discussed above, a compensation grouting plan was established. Concerns were raised that, during tunnel excavation before the top heading reached underneath the strip footing of interest, the footing settlements would exceed the action trigger level of 0.5 inch as specified on the contract drawings. KPFF indicated that the Old Jail structural frame supported by the strip footing of interest is stiff and subsidence would cause a stress redistribution through adjacent structural members on either side of the Tunnel while allowing the Tunnel-induced subsidence to generate a void under the immobilized strip footing. Without compensation grouting to fill such voids, the adjacent members could be subjected to higher loads. Based on KPFF's structural analysis, the maximum allowable gap would be 0.5 inch with a differential gap of 0.5 inch between any two adjacent monitoring instruments.

As shown in Figure 2, the strip footing is oriented along the tunnel centerline. During the tunnel advance, a differential ground settlement, especially in the tunnel longitudinal direction, would theoretically occur and create a differential gap underneath the strip footing, adversely affecting its load-carrying capacity. To ensure the structural integrity of the strip footing and its supporting structural frame, a grouting plan was developed for settlement mitigation (see Figure 6). This grouting plan specified the layout of

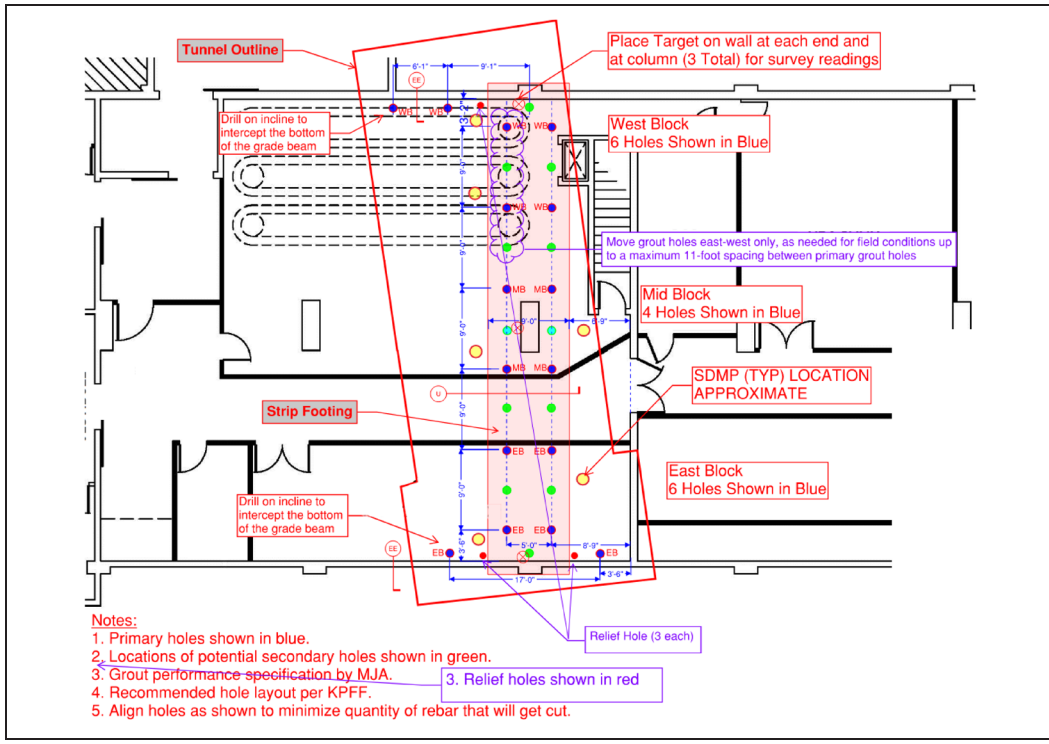


Figure 6. Grouting plan for settlement mitigation

grout holes, grouting procedure, performance criteria, and monitoring requirements. As part of this settlement mitigation grouting plan, additional soil deformation monitoring points (SDMPs) and wall and column displacement monitoring targets were installed around the strip footing. These SDMPs were able to measure the absolute soil vertical displacements below the strip footing at various stages (top heading and bench advances and local enlargement for niche construction) of tunnel excavation.

Though the grouting operations were not completely decoupled from the tunnel excavation, they were operated separately without interfering with each other. These two operations were coupled in the sense that starting a grouting campaign was triggered by the location of tunnel top heading or bench face relatively to the strip footing regardless of the measured settlements from the SDMPs. Subsequent grouting campaigns would be required at any time when differential settlements of 0.5 inch or greater relative to those measured at the end of prior grouting campaign were measured by any adjacent SDMPs and/or wall and column displacement monitoring targets.

Solution to Accommodating Fault Offset for Final Lining in Seismic Section

As discussed above, the Tunnel crosses the San Diego Fault, as shown in Figure 3. Based on the Fault Study Report (Kleinfelder, 2019a), a design earthquake event with a 2,475-year return period would result in an offset displacement of up to 18 inches within the fault zone. This fault offset is assumed to occur over the fault width ranging from approximately 5 to 7 feet along the Tunnel.

To accommodate this offset displacement and at the same time maintain the Tunnel final lining structural integrity following the design earthquake event, a special seismic section is constructed over a length of 50 feet in the reach of Tunnel crossing the fault zone. This seismic section consists of an enlarged cross-section formed by a series of structural segments (rings) with circumferential seismic joints between these rings designed to allow slippage and deformation in the fault zone, dubbed a "slinky system" (see Figure 7). These structural segments are designed to accommodate the anticipated asymmetric ground loads without collapse while the

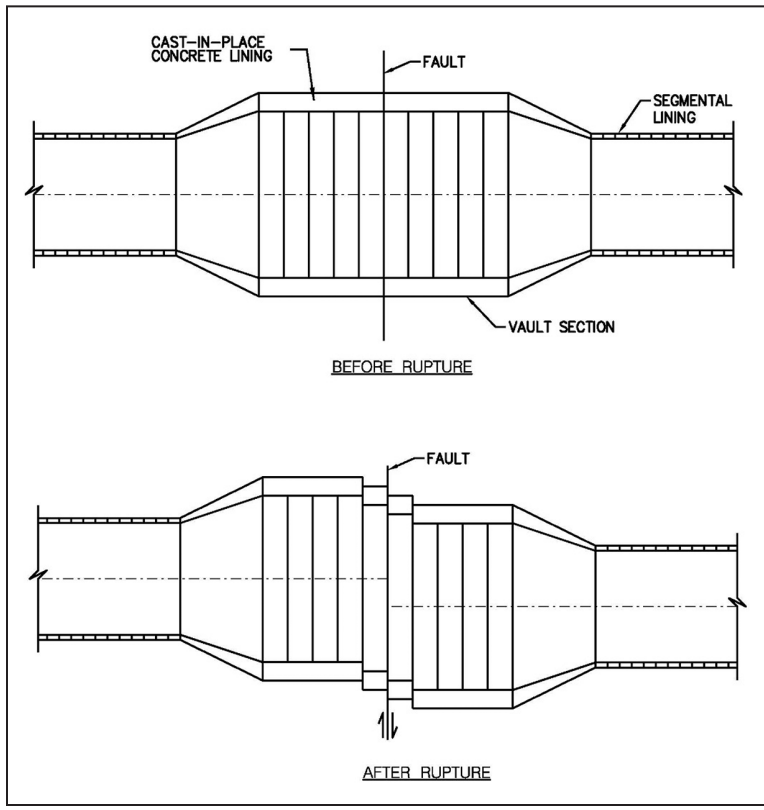


Figure 7. Plan layout of seismic section, demonstrating slinky articulation concept

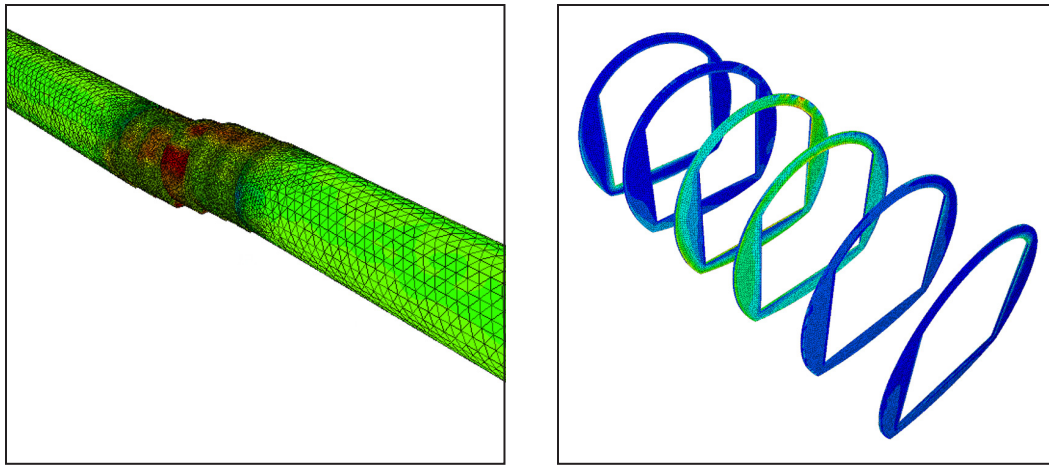


Figure 8. Contours of calculated displacements of seismic section (left) and seismic joints (right)

circumferential seismic joints are designed to allow the lateral, vertical, and longitudinal components of the distributed fault offset to take place at the joints between the segments. The advantage of this concept is that the resulting Tunnel structure can accommodate differential offset while maintaining the Tunnel functionality without extensive damage to the Tunnel final lining in the fault offset zone.

Fault offset analyses were performed based on 3D numerical method using FLAC3D. These analyses assumed various spacings and widths of seismic joints as well as thickness of the annulus or crushable material between the initial shotcrete lining and the final concrete lining so that these design parameters can be defined to achieve the seismic performance objective for the seismic section of the Tunnel. In these 3D analyses, both the ground (soil mass and deformable fault zone) and the Tunnel final lining were modeled with conventional solid, continuum elements in FLAC3D. To simulate the fault offset, the two blocks of ground on either side of the 18-inch fault slip zone were modeled to move with the same amount but in the opposite directions along the fault plane. The resulting total relative offset between these two blocks of soil mass is equal to the design fault offset. Both the horizontal and vertical components of estimated fault offset were considered in the analyses.

Initial fault offset analyses indicated that in order to make the seismic section work and prevent potential damage to the segmented tunnel final lining in this zone, four design parameters are crucial and have significant effect. These four parameters are:

- Spacing of the seismic joints
- Width of the seismic joints
- Size or width of the annular gap between the initial shotcrete lining and the final lining
- Stiffness of the crushable backfill material within the annular gap

Among these parameters, the annular gap backfilled with crushable material is considered the most important to ensure that a significant portion of the fault offset displacement is absorbed by high deformations of the crushable material during a fault offset event without causing large deformations and strains in the segmented final lining so that structural integrity of the segmented final lining is maintained.

Sensitivity analyses were performed by varying these four parameters to optimize their combination, while meeting seismic performance requirements and practical aspects of constructability and cost effectiveness.

Figure 8 shows contours of displacements of the tunnel final lining over the seismic section and along the seismic joints. As indicated, the displacements are expected to be highly nonuniform because of the width and orientation of the fault zone relative to the tunnel alignment. The seismic joints located in the middle of seismic sections would experience high offset displacements, while the rest would have much smaller displacements. Nevertheless, the design of seismic joints was based on the predicted maximum displacements of all seismic joints over the seismic section. Results of the analyses also indicated that by using a deformable backfill material, a majority of the fault offset displacements could be absorbed, resulting in a maximum displacement of less than 4.0 inches in shear direction and less than 0.5 inch in axial or normal direction.

Figure 9(a) shows the final design of the 50-foot-long seismic section, consisting of six seismic joints, which are 4 inches wide and spaced 10 feet apart. The spacing of seismic joints allows distribution of fault offset displacements over the 50-foot-long

seismic section and limits the magnitude of relative displacement between adjacent final lining segments. The enlarged cross-section, shown in Figure 9(b), has an annular gap of 18 inches between the initial support and the final lining. This gap is filled with a crushable or deformable low-density styrofoam/geofoam material to limit the potential impact of fault rupture displacements on the final lining in terms of displacements and stress concentrations. The seismic joints installed at adjacent segmented final lining sections provide lining longitudinal flexibility. Each of the 4-inch-wide seismic joints (gaps) is filled with deformable low-density styrofoam/geofoam material. With the crushable/deformable annulus backfill material and flexible lining in the seismic section, each seismic joint is designed to accommodate a maximum of 4 inches of shear displacement and 0.5 inch of normal displacement while remaining watertight during and after a design earthquake event. For comparison, a regular tunnel cross-section is shown in Figure 9(c). Note that the above analysis is based on a minimum 14-inch-thick Tunnel final lining. Figure 10 shows a completed seismic section with seismic joints in the Tunnel.

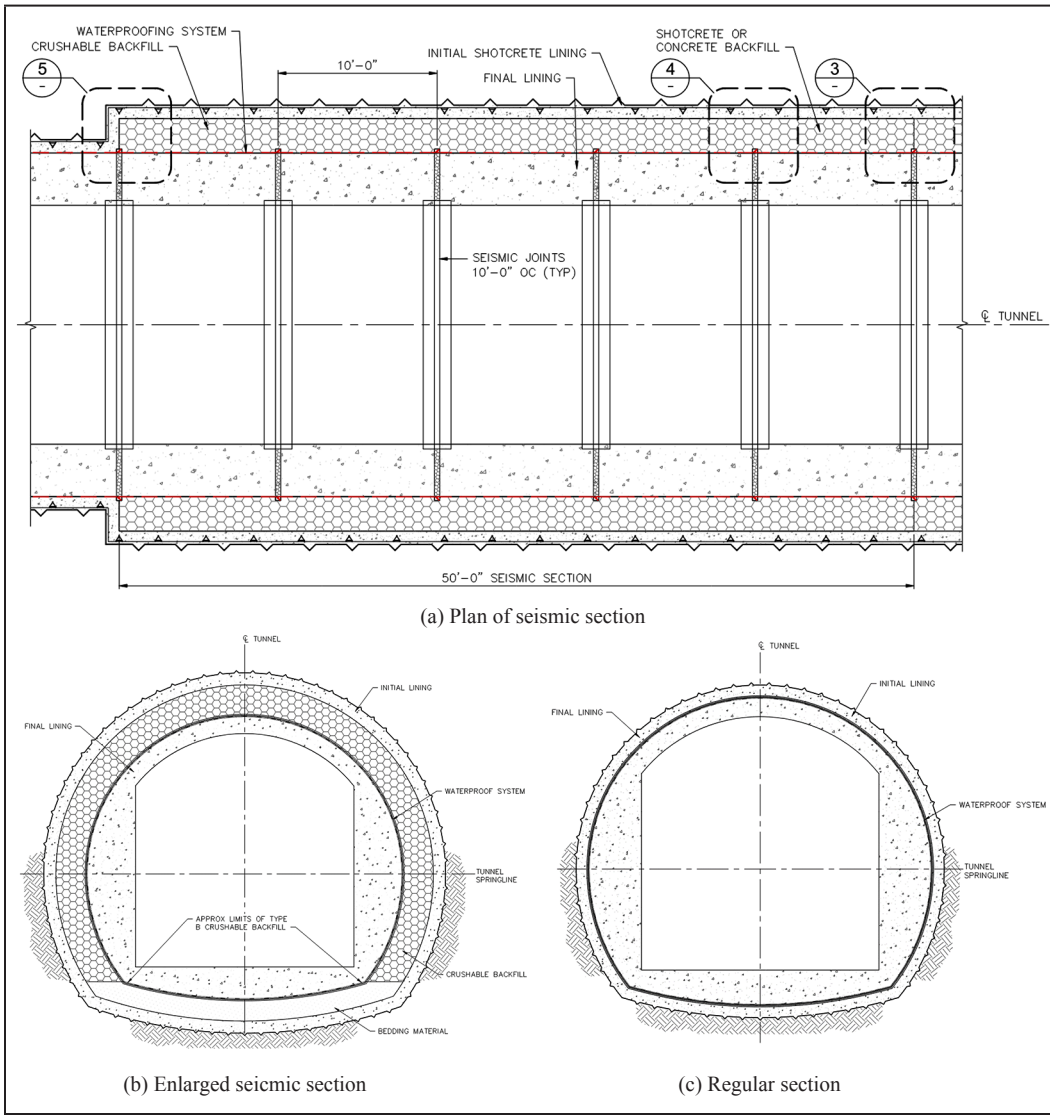


Figure 9. Plan and section of the seismic section

Figure 10. Photo of a completed seismic section with seismic joints

OBSERVATIONS DURING TUNNEL CONSTRUCTION

Detailed discussion on the tunnel excavation, issues encountered, and lessons learned is provided in a separate paper presented in these conference proceedings. This section presents some observations from tunnel excavation relevant to the settlements of the Old Jail building.

A comprehensive geotechnical instrumentation and monitoring program was developed and implemented to verify that tunnel convergence and surface settlements are within the allowable movements to prevent damage to existing structures, streets, and utilities. The instrumentation program includes the following:

- In-tunnel convergence monitoring targets (CMT)
- Multipoint borehole extensometers (MPBX)
- Inclinometers (IN)
- Surface settlement points (SSP)
- Soil deformation monitoring points (SDMP)
- Utility monitoring points (UMP)
- High sensitivity building settlement sensors (HSSS)
- Liquid level sensors (LL)
- Vibrating wire piezometers (PZM) and observation wells (OW)

Of particular importance was the ability to monitor the existing Old Courthouse and Old Jail for movements for their protection during the tunnel construction. Response values, also known as trigger values, are established in the design. Response values consist of two levels: action levels and maximum levels. Action levels are set to provide advance notification of ground movements that are trending toward damaging levels so that appropriate mitigation measures can be implemented to control movements below the maximum allowed. When the maximum level is imminent or reached, an immediate suspension of excavation activities is required until movements are controlled and corrective measures implemented to prevent further movement. Automated instrumentation readings were continuously monitored by Sixense and reported on a project-dedicated website in real time. Manually read instruments such as inclinometers and SDMPs were located in basements with no line of sight to automatic motorized total station (AMTS) units. Figure 11 shows the instrumentation and monitoring plan.

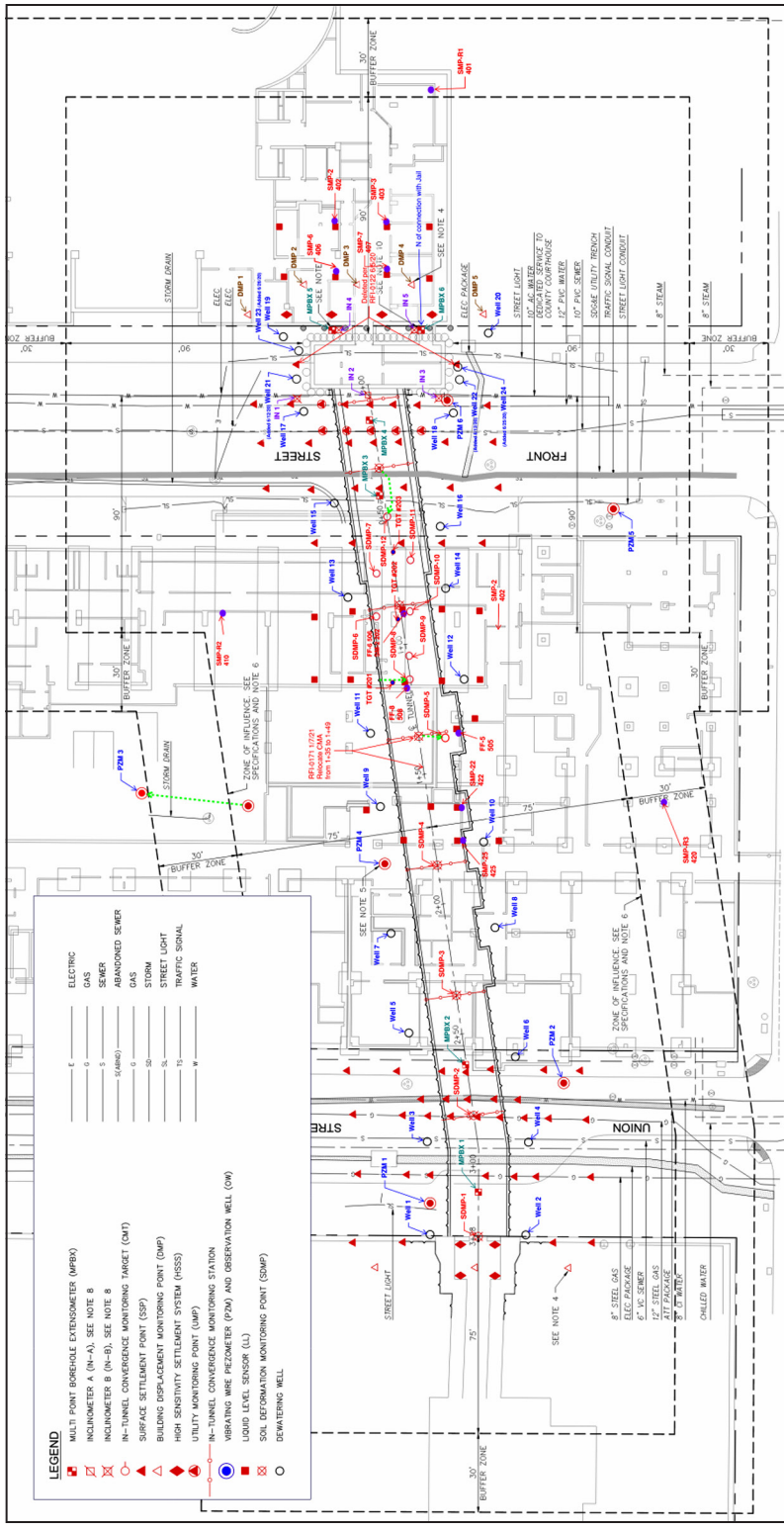


Figure 11. Geotechnical instrumentation and monitoring plan

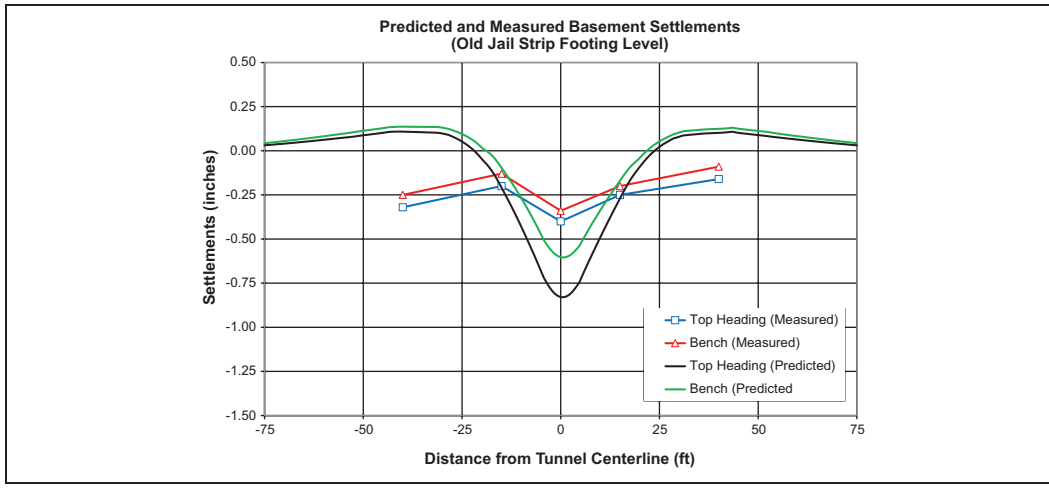
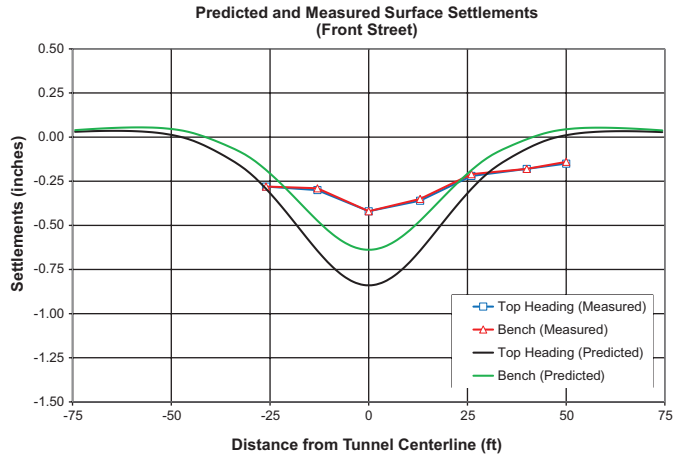


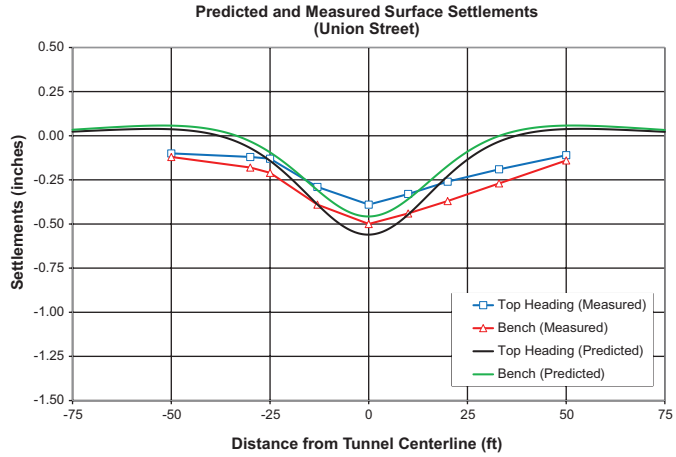
Figure 12. Predicted and measured settlements at Old Jail strip footing elevation

The following summarizes key observations:

- 7⅝-inch-diameter canopy pipes as presupport provided a stiff longitudinal arch for minimizing the Old Jail building footing settlements induced by tunnel excavation. As shown in Figure 12, the predicted (calculated) maximum settlement at the strip footing elevation is about 0.83 inch, which is two times higher than the measured of 0.40 inch. This result offers clear validation of the tunnel excavation and initial support measures. For tunneling in soft ground with limited ground cover underneath a building like the Old Jail, use of a stiff canopy pipe system can be a cost-effective alternative to a traditional approach using underpinning as long as the quality installation of the canopy pipes can be ensured. Note that the staged grouting campaign performed during the time when tunnel advance occurred underneath the strip footing also helped minimize ground loss during tunneling and contributed to the small footing settlements.
- One interesting finding is the consistent trend in the strip footing settlements. Between the predicted and the measured settlements, the maximum settlement after completion of both the top heading and bench excavations is smaller than that caused by the top heading excavation prior to the bench excavation (see Figure 12). The bench excavation pushed the sidewalls inward, which in turn reduced or offset some of the deflections of the crown, though the latter was not observed from in-tunnel convergence monitoring. However, this phenomenon was not observed from the surface settlements along both Front and Union Streets (see Figure 13). In these locations, the maximum settlements occurred following the bench excavation, even though additional settlements induced by the bench excavation were relatively small: nearly zero on Front Street and less than 0.15 inch on Union Street.
- As shown in Figure 13, the measured settlements on Front Street were smaller than those on Union Street. This was because in Reaches TESM 1 and TESM 2 underneath Front Street, 7⅝-inch-diameter canopy pipes were used instead of the less stiff 2-inch-diameter pipe spiles used in Reach 4 underneath Union Street, even though the former had an enlarged cross section. Measured surface settlements at these two locations further confirm that stiffer canopy pipes were able to limit ground loss and reduce ground movements and settlements. Larger settlements predicted on Front Street were associated with the enlarged cross section in Reaches TESM 1 and TESM 2. The same observations were seen from the MPBX measurements (see Figure 14), which captured the vertical displacements at depths (10 to 20 feet from ground surface). The maximum vertical displacements at depths below Front Street were much smaller (about 0.4 inch) than those below Union Street (about 0.7 inch).
- Figure 14 also shows the vertical displacements as a function of time and the sequential excavation of the top heading followed by the bench. With a stiffer canopy pipe presupport, the majority of displacements as recorded by



(a) Front Street



(b) Union Street

Figure 13. Predicted and measured surface settlements at Front and Union Streets

MPBX #3 at Front Street occurred following the top heading excavation, with minimal additional displacements after the bench excavation. With a less stiff pipe spile presupport, about a third of the total displacements occurred due to the bench excavation, as recorded by MPBX #2 at Union Street. It is noted that the MPBX measurements demonstrate the time-dependent effect of the soils since the total displacements occurred after the tunnel face was at a distance of more than two times the tunnel diameter ahead of the measurement location.

CONCLUSIONS

Large-diameter stiff canopy pipes as presupport were effective and successful in limiting ground loss and settlements without causing any observed damage to the Old Jail building. Compared to an alternative approach using underpinning, the specified and implemented method achieved savings in construction schedule and costs. With a carefully specified tunnel excavation sequence and initial support measures, tunneling in beach sand–like soft ground with low cohesion and underneath tall buildings with only a few feet of ground cover was carried out successfully using SEM.

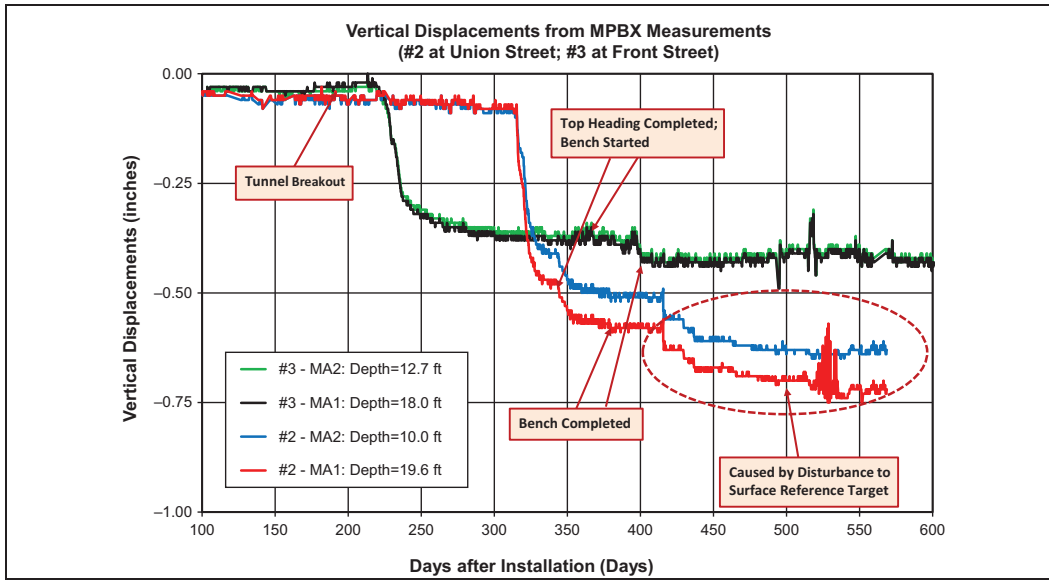


Figure 14. Vertical displacements measured from MPBXs

A unique seismic section has been incorporated into the tunnel final lining. Though not yet (and hopefully never) validated by actual conditions, the design of this seismic section will accommodate the anticipated offset displacements of the San Diego active fault during the design earthquake without compromising the structural integrity of the tunnel final lining.

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REFERENCES

Itasca Consulting Group, Inc. 2012. *FLAC3D—Fast Lagrangian Analysis of Continua in Three-Dimensions*, Ver. 5.0. Minneapolis. http://docs.itascacg.com/flac3d700/contents.html.

Kleinfelder. 2019a. *Fault Hazard Investigation*. Prepared for the County of San Diego for the Courthouse Commons Tunnel Project. Revision dated August 13.

Kleinfelder. 2019b. *Geotechnical Design Summary Report*. Prepared for the County of San Diego for the Courthouse Commons Tunnel Project. Dated April 12.