Courthouse Commons Tunnel: A Case Study of Modern SEM Tunneling

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ABSTRACT: The Courthouse Commons SEM Tunnel was excavated within 6 feet of cover beneath two buildings in downtown San Diego utilizing high-precision geotechnical instrumentation. The excavated dimensions are approximately 25 feet in diameter and 328 feet in length connecting two in-service detention-level security buildings. Presupport includes large-diameter grouted canopy pipes and smaller pipe spiles. Initial support consists of fiber-reinforced shotcrete with lattice girders. This paper presents observed ground and structure behavior and lessons learned. During construction an unanticipated brick structure was encountered, and methods were developed to improve adhesion of shotcrete to sandy soil and to reduce rebound.

INTRODUCTION TO SEM

The Sequential Excavation Method (SEM) of tunneling is an observational method that involves evaluating the type and behavior of ground during excavation and allows for variation of support throughout the tunneling process. The observational method plays a significant role in optimizing the underlying tunnel design by determining the best combination of ground support components and excavation sequence in real-time.

During the tunnel design phase, the ground support is prescribed based on what is known from advance geotechnical explorations. Predetermined additional measures and other support elements—commonly referred to as "toolbox items"—are also formulated during the design phase and are developed based on anticipated variability and behavior of the ground, thus making them more efficient to implement during construction. The evaluation of ground behavior during excavation of an SEM tunnel can result in either implementation of these additional measures and toolbox items or reduction in the level of ground support set forth in the design where ground is found to be more favorable than anticipated. These ground behavioral observations are made both from within the tunnel excavation by highly trained and experienced SEM engineers on site and externally, through various structural and geotechnical instrumentation devices.

COURTHOUSE COMMONS TUNNEL DESIGN ELEMENTS

SEM tunneling was successfully used for the construction of the Courthouse Commons (CoCo) Tunnel in Southern California. The CoCo Tunnel is one of multiple components of the Courthouse Commons Development Project, located in downtown San Diego. Once completed, the tunnel will provide secure pedestrian and electric vehicle access between the Central Jail and Superior Courthouse of California.

The CoCo Tunnel was designed with four discrete zones of tunnel excavation and initial support measures (TESMs) across the 328-foot length and consists of three different excavated cross sections based on anticipated ground conditions and ground support type. The largest of the TESM cross sections was excavated to allow for placement of crushable backfill and joints in the final lining to accommodate movement in the event of fault rupture where the alignment passes through the San Diego Fault. The tunnel cross section throughout the entire alignment generally takes the shape of a modified horseshoe with curved invert. From east to west, the cross-sectional diameter at springline measures 25'10" at TESMs 1 and 2, and 21'6" at TESMs 3 and 4. The typical excavated cross section for TESMs 3 and 4, which made up the western 254 feet of the tunnel, is shown in Figure 1. The difference between these two

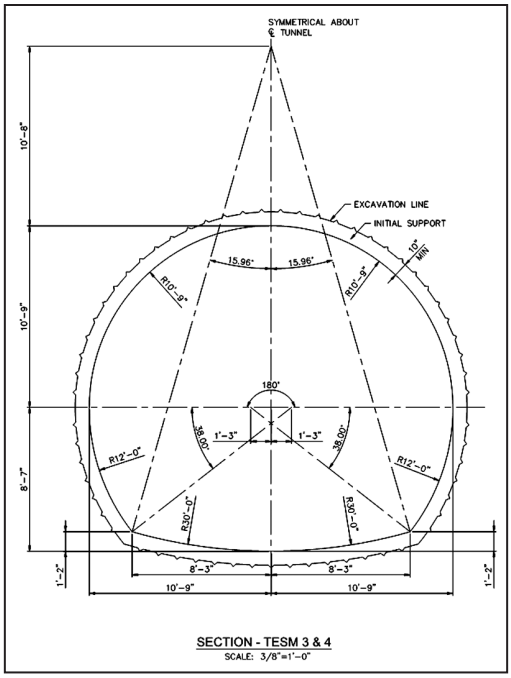

Figure 1. Excavated cross-sectional geometry of TESMs 3 and 4

TESMs is the type of presupport utilized—canopy pipes for TESM 3 and pipe spiles for TESM 4.

Four niches were located along the southern sidewall of the tunnel alignment. The niches varied from 13'8" to 22'0" wide, 14'2" to 17'5" tall, and 4'5" to 8'10" deep. The niches provide storage for required equipment, including an electrical vehicle charging station, for operation in the tunnel.

The main tunnel drive was excavated in two separate runs consisting of the top heading and lower bench. The top heading excavation consisted of the top 18 feet of the tunnel cross section at maximum, while the lower bench removed the remaining ground of the cross section. Niches were excavated after the full top and bottom headings of the tunnel had been excavated and supported.

Ground Conditions

The ground at the project site consisted of cohesionless soils ranging from coarse, poorly graded sand to fine, well-graded sand with silt. Intermixed in the soil matrix were intermittent bands of rounded gravels and clayey layers typically less than 12 inches wide. The natural groundwater table before dewatering efforts rested approximately at the springline elevation of the tunnel but was lowered to below the invert level of the excavated cross section prior to completion of the starter shaft and tunnel advancement. The soil type at the project site resulted in rapid groundwater movement, and rising piezometer levels could be observed within a few hours when a dewatering well was inadvertently shut down or taken offline for maintenance.

Presupport Elements

Across TESMs 1, 2, and 3 the presupport installed in advance of excavation consisted of 7-⅝ inch outside diameter (OD) steel canopy pipes with ½-inch-thick walls. Clearance between adjacent canopy pipes is a nominal 8 inches. Canopy pipes were placed approximately 2 feet above the excavation line across a 120° subtended angle along the tunnel crown symmetrical about the tunnel centerline. These pipe arrays were installed from three locations at tunnel STA 00+00, 00+77, and 01+37 to lengths of 100 feet for the first array and 80 feet subsequently.

For the remainder of the tunnel, beginning at STA 01+97, a similar array of 2-inch OD, 12-foot-long grouted pipe spiles was installed utilizing a smaller robotic percussion drill jumbo with each round of excavation advance, and pipe spiles were spaced 12 inches center to center along the same 120° angle across the crown. Canopy pipes and pipe spiles were manufactured with a pair of ½-inch-diameter holes opposite one another every 20 inches to operate as grout ports. After drilling and installation of the hollow pipes, grouting was performed through the free end inside the excavated tunnel to fill both the pipe and any surrounding annulus void developed during drilling through the grout ports utilizing a grout pot and a positive displacement pump. The 2-inch OD pipe spiles were grouted using mechanical packers, while custom caps with grout injection and air vent valves were manufactured for the larger canopy pipes.

Initial Support Elements

Initial support of an SEM tunnel refers to the ground support installed immediately after an excavation advance and is designed to support loads during construction. However, the tunnel final structural lining was designed to accommodate permanent loading conditions, meaning there was no load sharing assumed between the initial support and final lining. Initial support utilized for the CoCo Tunnel consisted of pneumatically applied concrete (shotcrete) and steel lattice girders.

After each excavation advance, a thin layer of shotcrete of approximately 2 to 3 inches was immediately applied to all open and exposed ground. This first layer of shotcrete is referred to as the flash coat or flashcrete. After flashcrete placement, and initial shotcrete strength achieved, a lattice girder was erected with the aid of reflector-less survey to act as a shotcrete profile template. The lattice girders

took the cross-sectional shape of an isosceles triangle with one number 10 and two number 6 rebars (bent to the curvature of the 2-inch offset from the initial lining intrados) at each point of the triangle and intermediate bracing along the section. They were manufactured in six separate pieces with butt plates for bolted connections to one another, and when combined would ultimately take the shape of the full tunnel cross section. To complete the initial support of the tunnel, more shotcrete was placed to both encase the lattice girder and build a lining throughout all sections of tunnel crown, invert, and sidewalls to a minimum thickness of 10 inches. During the top heading advance, a 10-inch shotcrete temporary curved invert forming a closed ring of support was placed to ensure the stability of excavation and control tunnel convergence. This temporary invert was later removed during the bench excavation. All flashcrete/shotcrete used on the project was synthetic fiber reinforced with 12 pounds of fiber for each cubic yard and was delivered to the site in prebatched supersacks.

A final element used for ground support during excavation was the face wedge, used to provide stability to the face of the advance. The face wedge always stood a minimum of 8 feet tall with 6 feet of clear space on each side relative to the excavated sidewall. The wedge was approximately 2 feet deep before sloping down to the invert at 45° to 60°. These dimensions allowed for a mass of soil to act as a gravity wall in front of the excavated face, but also provided adequate clear access for shotcrete installation. Figure 2 shows a typical excavated top heading with face wedge in place prior to application of shotcrete.

EXCAVATION AND INSTALLATION OF SUPPORT

The CoCo Tunnel was excavated from the starter shaft on the east end of the alignment from which the first array of canopy pipes was installed to the west. The top heading advance of the tunnel was excavated in this direction and traversed beneath Front Street, the Old Jail, the Old Courthouse, and Union Street. The alignment ended once reaching the basement of the Superior Courthouse, where the tunnel would ultimately connect. After reaching the basement, the remaining cross section comprising a single bench was excavated from west to east. A three-dimensional schematic of the tunnel alignment is shown in Figure 3. Advances of the top heading ranged from 3 feet to 4 feet based on ground behavior observations and consisted of a single lattice girder bay. The bench heading was prescribed to be up to twice the corresponding top heading advance length but was generally relaxed to 12- to 16-foot rounds, except where changes in geometry were more conducive to smaller rounds.

Figure 2. Typical excavated top heading prior to flashcrete with face wedge in place

After the completion of the main drive excavation and initial support, the four niches in the southern sidewall of the tunnel were excavated. The niches included the same 2-inch OD pipe spiles as portions of the tunnel main drive for presupport and were constructed using pocket excavation techniques to not cause excessive disturbance during removal of the existing lining.

The first array of canopy pipe installation commenced on August 17, 2020, and the final niche excavation was completed on April 30, 2021. The site typically operated on a 24 hours per day, 5 days per week schedule with a few exceptions for weekend work and holidays. In total, the construction phase from the start of presupport installation to the completion of all excavation and initial support lasted for 181 working days. Table 1 provides a detailed breakdown of the total number of working days specific to each indicated activity. It should be noted that the total for this table does not match 181, and this discrepancy accounts for days where other nonproduction activities were performed. Such activities included preparatory work for canopy pipe installation, remedial action for a discovered brick structure, movement of equipment, preparation of shotcrete lining for waterproofing acceptance, limited shotcrete supply, dedicated spoils removal days, and other similar ancillary work.

Installation of Presupport

The canopy pipes were installed in 5-foot segments as an outer casing using a hydraulic, rotary-percussive drill rig with a fully articulating mast and directional drilling capability. The leading 5-foot pipe casing had carbide tip teeth welded flush to cut through the soil during installation with no more than a ⅛-inch overcut. An inner drill rod was used with a drag bit that led the pipe casing by 6 inches to guide the drilling and serve as a conduit for air and water to flush spoils out between the inner rod and outer casing. After the full length of canopy pipe installation

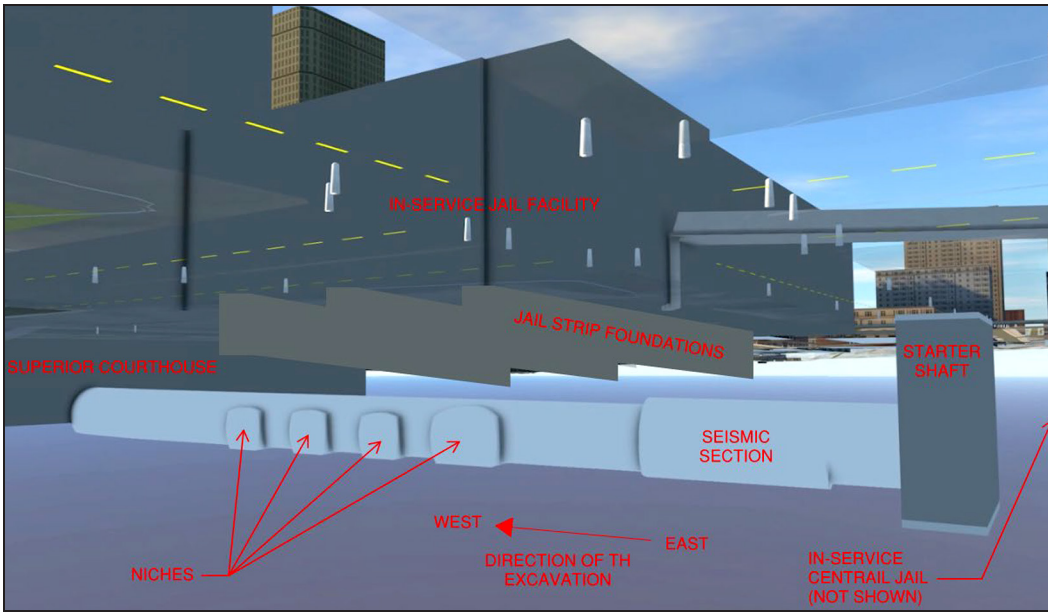

Figure 3. Three-dimensional schematic of tunnel alignment from starter shaft to superior courthouse

Table 1. Working days per excavation and support activity

Activity	STA Location Start	STA Location Finish	No. Working Days
Canopy Pipe Array #1	00+00	01+00	14
Top Heading Advance	00+00	00+77	25
Canopy Pipe Array #2	00+77	01+57	10
Top Heading Advance	00+77	01+37	17
Canopy Pipe Array #3	01+37	02+17	9
Top Heading Advance	01+37	03+28	42
Lower Bench Advance	03+28	00+00	16
Niche Excavation (qty 4)	—	—	19

was completed, the inner rods were removed, and a gyroscopic survey was performed to evaluate the alignment of the installed canopy pipe using a rod-mounted Gyro with centralizers. A drilling accuracy of less than 1% overall deviation (horizontal and vertical) was achieved across all pipes. The rig setup for canopy pipe drilling within the tunnel is shown in Figure 4, where the hammer has been retracted for placement of another section of drill steel and casing by a mini excavator and two-person crew.

Once a canopy pipe was verified to have correct alignment, the pipe was grouted prior to any adjacent pipes being installed. The grouting process required a ½-inch-diameter PVC pipe to be placed within the casing through the full length with a foam float on the far end corresponding to the highest elevation of the casing. This PVC pipe terminated at a valve in the top of the pipe cap, and a second valve on the pipe cap was used as a grouting port. Grout was injected until return was observed through the PVC vent pipe, which was subsequently closed, and grouting then continued until refusal. Refusal was defined in this case as a grouting flow rate of less than 1 gallon per minute (gpm) for at least 2 minutes at the maximum injection pressure of 50 psi. Grout take never exceeded 170% of theoretical volume (inside volume of pipe plus ⅛-inch assumed overcut) and generally remained within 20% of theoretical. Low grout take beyond the volume of pipe to be filled can be attributed to the tight beach-like sands, with two high-take exceptions being (1) the brick anomaly encountered (discussed below), and (2) one instance where canopy pipe tips reached one another during drilling, causing soil erosion at the ends.

The three arrays of canopy pipes that were installed as presupport in the eastern portion of the tunnel alignment at STA 00+00, 00+77, and 01+37 consisted of 24, 21, and 21 pipes, respectively. The installation rate of canopy pipes increased over time, and the three arrays required 14, 10, and 9 full working days, respectively, to drill and grout in place.

Pipe spiles were installed using a small robotic drill jumbo, which was also utilized for probe hole drilling at predetermined intervals. The hollow pipe spiles measured 12 feet long and were placed by advancing with an internal drill steel rotating a sacrificial drill bit in advance of the hollow pipe and a percussive hammer on the end of the spile. The set

of spiles was driven/drilled through the recently installed lattice girder for the round of advance, which was placed approximately 1 foot from the excavation face, with less than 12 inches of stick-out from the lattice girder. The configuration resulted in a 10-foot embedded length of pipe in advance of the face under which the next round (3 to 4 feet) of advance could be made. After the full set was placed, the pipe spiles were grouted using a mechanical packer on the protruding end of the pipe to the same refusal criteria as the canopy pipes. In some locations along the western alignment of the tunnel, when excavating beneath Union Street, ground conditions were found to be more favorable because of the presence of a hard clay layer in the arch, and the quantity of pipe spile was reduced to as few as five per round.

Installation of Initial Support

The first step of initial support installation, the flashcrete, was performed as soon as possible after excavation was completed. The flashcrete and subsequent shotcrete layer were placed using a small shotcrete robot, which was critical to prevent personnel from standing beneath open ground during the early stages of initial support construction. Once the flashcrete reached an initial set and strength of 75 psi, as determined by use of a punch penetrometer device, personnel entry was allowed at the face, and the lattice girder was set at a spacing equal to the previous excavation length.

Lattice girders generally trailed the excavated face by approximately 1 foot, and where the excavation round length was increased from 3 feet to 4 feet because of favorable ground conditions, so was the lattice girder spacing. During placement of the upper three sections of lattice girder, the crown segment was held in place using a cradle attachment on an inverted bucket of an excavator. Miners were then able to bolt the rib sections of the lattice girder on each side and make final adjustments with total station survey guidance prior to locking in place. Hooked spacer bars were used to connect adjacent lattice girders prior to placement of initial lining shotcrete after survey verification had been completed.

With the flashcrete and lattice girder in place for a given advance, the final step for initial support was to encase the lattice girder and fill the excavated bay to full thickness (shotcrete by use of the shotcrete robot, which is shown in Figure 5). Despite the thickness of shotcrete to be applied in this stage being greater than the flashcrete, the initial lining could be placed relatively quickly since it was sprayed on hardened shotcrete as opposed to looser sands and gravels, resulting in significantly less rebound. The robot was fed by a 3-inch-diameter slickline that ran along the tunnel, up the starter shaft, and to a concrete pump that was loaded by a concrete volumetric mixer truck operated by construction crew personnel. Volumetric mixer trucks holding tanks were loaded at a nearby staging site with premixed shotcrete supersacks, hydration stabilizer, high range water reducer, and water; the trucks were brought to the excavation site for on-demand shotcrete batching.

Figure 4. Setup of canopy pipe installation drill rig at STA 00+77

Figure 5. Placement of shotcrete initial lining after lattice girder installation

CONSTRUCTION CHALLENGES

Multiple factors primarily related to the urban setting and ground conditions challenged the construction team. The leading difficulties can be summarized as limited operational space, loose or disturbed sand in various locations, and an unanticipated brick well

structure within the tunnel alignment. Each of the three are discussed in detail in the following sections.

Difficulties of an Urban Setting

The starter shaft of the CoCo Tunnel was excavated within the boundaries of a single lane of vehicular traffic and adjacent sidewalk in downtown San Diego. The immediate construction area at the shaft spanned the length of less than one city block and thus provided space in plan view for only three Connex Boxes, a 23'9" by 19'1" track-mounted hydraulic crane, a concrete pump, and parking for one full-sized passenger vehicle. Space for all other equipment, material storage, shop facilities, muck handling, volumetric mixer loading, and employee parking was provided in a lot one block away from the shaft site. With the majority of streets in the project vicinity being one way, and the perpendicular street to the south of the shaft containing a streetcar line, transporting equipment and hauling materials was a slow process.

In addition to the narrow width, the space allocated at the ground surface for the starter shaft—the location of which was also constrained by where the basement tie-in could be placed—was immediately in front of the refuse truck loading dock of the Central Jail. Since the off-haul of waste is a necessary service of the jail facility that could not be disrupted, a sliding deck equipped with Hillman rollers on the cap beams was installed on top of the starter shaft during construction so that the deck could be parked to one side of the shaft when not in use. When in place for refuse truck ingress and egress, the temporary deck blocked nearly all crane access to the shaft and prevented activities such as mucking out spoils, despite the use of relatively small 2.5 cubic yard muck boxes. While this solution allowed for continued operation of the Central Jail, there was no more feasible or economical system that would allow crane access to the shaft during refuse pickup.

Loose and Disturbed Sandy Soil

While much of the ground could stand near vertical for short periods of time immediately after excavation and prior to initial support installation, various locations along the tunnel alignment consisted of loose sand. When this condition was encountered, the minimum air pressure used for flashcrete application was able to erode the sand and prevent the flashcrete material from remaining in place for sufficient time to reach initial set, and at some locations, air currents alone would cause sloughage of the face. These conditions notwithstanding, the ground never raveled above the installed presupport. Shotcrete robot operators improved their abilities to build off previously hardened shotcrete throughout the construction process, but where sand was exceptionally loose, it was pretreated with a sodium silicate spray to create a more competent crust.

The sodium silicate was applied to the ground by use of an airless paint sprayer, which could cover the entire exposed face and tunnel sidewalls in a safe and timely manner with the silicate solution. After approximately 20 to 30 minutes, the sodium silicate would react with the sand and generally form a hardened layer of up to 2 inches and yielded punch penetrometer readings in the range of 15 to 25 psi. This moderate improvement in the surficial stiffness of the sandy soil often reduced the time required to apply the flashcrete layer by half.

Sodium silicate was used heavily in conjunction with pocket excavation for the niches in the southern sidewall of the tunnel. In these areas, the initial lining had to be broken out by use of line drilling, rock splitting, and light chipping with the excavator-mounted hoe ram. These activities caused significant disturbance to the surrounding soil and, at times, ground loss that required contact grouting behind the lining to fill generated voids. In hindsight, modifications such as block-outs around the niche perimeter or construction sequencing to reduce or eliminate the effort needed to break into the initial lining for these relatively shallow niches would have preserved days in the schedule which were lost to the slow pocket excavation and ground treatment required.

Abandoned Brick Well

During the drilling of the second canopy pipe array at STA 00+77, brittle red fragments were discovered in the drill spoils of certain pipes when the end of the pipes were located at approximately STA 01+15, which is below the in-service Old Jail facility. The pipes that encountered this material also lost air circulation while drilling and were paused while other pipes were drilled and grouted. With multiple pipes paused at tip locations around STA 01+15, soil samples were extracted via the canopy pipe casings for material and environmental testing and a plumber's camera was inserted to the far end of the pipes to visually identify the material encountered. Through evaluation of hand samples from the drill spoils, video from within the casings, and literature review of excavation history in downtown San Diego, it was determined that the red fragments were brick and that an abandoned brick well trash structure, commonly encountered throughout downtown Dan Diego, was likely to be encountered in the future face of excavation. Ultimately, all canopy pipes in array 2 were advanced to the full 80-foot length and excavation resumed beneath them as normal.

When the top heading excavation face reached within 8 feet of the anomaly, additional probe hole drilling was performed using the drill jumbo and a 2.5-inch-diameter bit to determine more precise

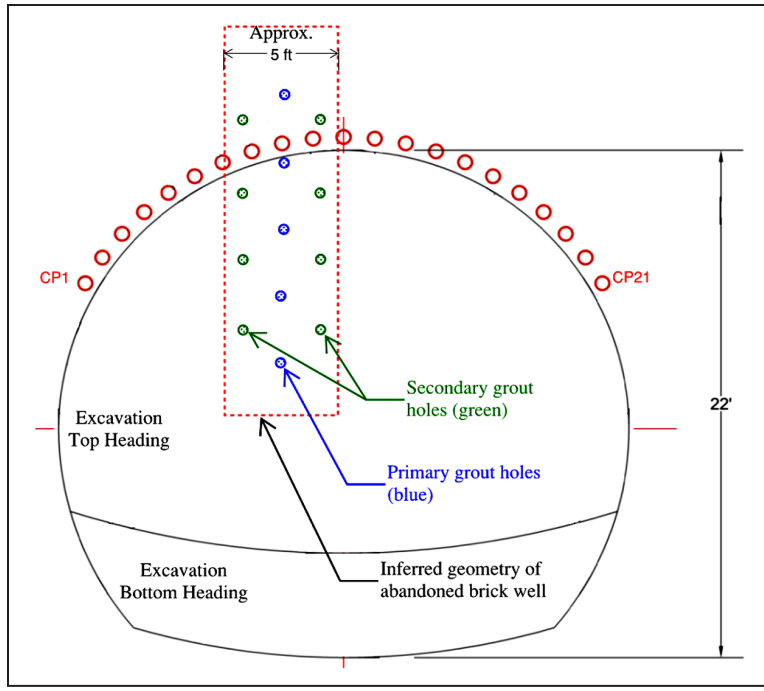

Figure 6. Simplified layout of abandoned brick well grouting campaign with canopy pipe (CP) layout included

extents of the brick well structure soon to be encountered. This activity resulted in an inferred geometry of the structure of 5 feet in diameter and extending from near the bottom of the top heading to above the tunnel crown. The probe holes consistently found that air circulation was lost when the structure was penetrated, which the construction team interpreted as the well being filled with loose material having a significant amount of void space. Probe holes that penetrated the structure were thus cased with PVC pipes that were perforated in the 5-foot section passing through the brick well structure.

Five primary grouting holes were established along the centerline of the vertical brick well structure from top to bottom at approximately 2- to-3-foot intervals as shown in Figure 6. The PVC casings were grouted using a ball valve and grout port and an ordinary Portland cement, lean grout mix of 1:1 by weight. All five primary holes were set up with valves in the open position and grouting was performed starting at the lowermost hole. Working from the bottom up allowed for grout to be pumped until return was observed at the next highest hole, at which point the current valve through which grout was being injected was shut off and the operation would begin grouting through the next highest hole. In the first day of grouting these five primary holes, a total of 2,092 gallons of grout was injected into the loose material ahead of the face.

On the second day of the grouting campaign a total of eight additional secondary grout holes were set up with packers and grouted. These holes were placed in pairs at approximately 3 feet horizontal spacing and splitting the primary holes vertically (Figure 6). Grouting of all secondary holes again took a bottom to top approach and significantly less grout take was observed through these holes—a total of only 165 gallons. Once grouting of primary and secondary holes was completed, a total of 302 cubic feet of cement grout had been injected into the brick well, equivalent to a completely open cylindrical void of 5 feet in diameter and 15 feet tall. Since multiple rounds of excavation were to be completed prior to reaching the brick well structure, the top heading advance proceeded as soon as the crew was ready after breaking down grouting operations on the second day.

Before excavating the material in and around the abandoned brick well, additional safety precautions were put in place because of the results of sample testing from within the structure. It was determined that material to be excavated from the structure contained RCRA hazardous waste levels of lead and potentially harmful amounts of volatile organic compounds. Accordingly, all material from

Figure 7. Brick structure excavation during removal of (a) upper portion and (b) lower portion

within the brick structure would need to be segregated for disposal and additional personal protective equipment was required of all personnel when handling the material or within the tunnel during excavation. Once the expected advance in which the brick well structure would be encountered was reached, excavation proceeded with removal of ground at the face and face wedge around the well that could be mucked out as usual. Flashcrete was then applied on all portions of open ground surrounding the then exposed brick well, and plastic sheeting was set to contain any contaminated material. This process was required for two separate rounds of excavation, one where the brick well was excavated from the upper portion of the advancing face, and a second when the abandoned brick well was excavated out of the face wedge as shown in Figure 7.

The excavation of the abandoned brick structure was performed without any measurable disturbance beyond typical movements from tunnel advancement to the in-service Old Jail immediately above the structure. The removed material was consolidated adequately by the grouting campaign but was still removable by use of an excavator-mounted hoe ram and standard toothed bucket. The majority of the material removed from the well structure can be summarized as early 20th century waste consisting of porcelain, metal, glass, and burnt organics.

INSTRUMENTATION AND MONITORING PROGRAM

Monitoring for the project involved a suite of geotechnical and structural instruments from which real time data could be viewed on an online platform as well as various manual surveys within the tunnel and on critical aboveground structures as the active heading made passage below. Table 2 provides an overview of the types of instrumentation installed and used for monitoring across the project site. Site features are listed from east to west—the same direction as top heading tunnel advance. Except for the instruments that were monitored manually (indicated in Table 2), all instruments were read at minimum on a daily basis and automatically transferred to an online platform where the full history of data points were available. When the active tunnel heading was within two diameters of a given set of instruments, the reading frequency was adjusted to every four hours. Manual surveys were also increased when in this range to a minimum of once every advance or once per day, whichever resulted in more readings. For the purposes of this exercise, one tunnel diameter was conservatively considered to be 25 feet in all locations.

Maximum permissible movements of any instruments ranged from ¾ inch to 1-½ inches depending on the location and criticality of the structure. The new Superior Courthouse and Central Jail structures corresponded to the most stringent of displacement requirements, while MPBX anchors set immediately above the tunnel crown at Front and Union Streets had the largest range of permissible movements. For all instruments, half of the maximum movement value was considered as the action level, which was the point at which further action to investigate the precise cause of movements was required and mitigation measures had to be employed to prevent further movement per the contract. Nearly all instances of

Table 2. Summary of instrumentation monitored throughout tunnel excavation

Site Feature	Location Relative to Tunnel	Types of Instrumentation Utilized
Central Jail	East of Shaft	High Sensitivity Settlement Sensors (HSSS)
		Liquid Level Sensors (LL)
		Building Monitoring Points (BMP)
		Intermittent Manual Leveling Surveys*
Starter Shaft	Eastern Terminus	Inclinometers*
		Multiple Point Borehole Extensometers (MPBX)
		Intermittent Manual Leveling Surveys*
Front Street	STA 00+00 to 00+49	Surface Settlement Points (SSP)
		Utility Monitoring Points (UMP)
		Multiple Point Borehole Extensometers (MPBX)
Old Jail	STA 00+49 to 01+54	Liquid Level Sensors (LL)
		Building Monitoring Points (BMP)
		Soil Displacement Monitoring Points (SDMP)*
		Intermittent Manual Leveling Surveys*
Old Courthouse	STA 01+54 to 02+58	Liquid Level Sensors (LL)
		Soil Displacement Monitoring Points (SDMP)*
Union Street	STA 02+58 to 03+28	Surface Settlement Points (SSP)
		Multiple Point Borehole Extensometers (MPBX)
Superior Courthouse	Western Terminus	High Sensitivity Settlement Sensors (HSSS)
		Building Monitoring Points (BMP)
		Intermittent Manual Leveling Surveys*
In-Tunnel	STA 00+00 to 03+28	Convergence Monitoring Arrays (CMA)*
Site	Throughout	Piezometers (PZM)

*These instruments were monitored manually by a surveyor or technician.

movement exceeding the predetermined action levels were determined to be the result of an erroneous reading (e.g., a traffic cone in front of an SSP prism).

Arguably the most simply applied instrument readings that directly corresponded to the excavation and ground performance were the SSP prisms on the street surfaces. SSP and BMP prism targets were shot constantly by use of multiple robotic Automatic Total Stations (AMTS), and a colored gradation of surface settlement was updated online every four hours. Figure 8 depicts these surface settlement views and displays a final condition of ground movement on the respective street surfaces at the end of all tunnel excavation and support including the final lining. SSP prisms were placed in north–south trending arrays, as can be seen in Figure 8, which established useful checkpoints along the tunnel alignment in which settlement troughs could be compared to settlements predicted during the design phase.

Six different settlement troughs corresponding to the array of prisms set in a north–south trending direction down the center of Union Street are shown in Figure 9. The six plotted lines show increasing settlement and formation of a parabolic settlement trough when viewed in the following order:

1. Excavated top heading face 2 diameters before the SSP array
2. Excavated top heading face 1 diameter before the SSP array
3. Excavated top heading face directly beneath the SSP array
4. Excavated top heading face 1 diameter past the SSP array
5. Excavated top heading face 2 diameters past the SSP array
6. All tunnel and niche excavation and initial support complete

Also noteworthy is the –0.48 inch of settlement that occurred at the center point of this array by the time all excavation on the project was completed, and the –0.19 inch (40%) that occurred during the period where the excavation top heading face ranged from directly beneath the array to 1 diameter distance past.

While LL sensors provided unreliable data at many times throughout the project and necessitated maintenance at unpredictable intervals, they typically provided reasonable approximations of building settlement when manual surveys were not performed within the basements. Figure 10 depicts a graph of all LL sensors on basement columns in the Old Jail from a period ranging from October 3, 2020 to January 21, 2021 (at 350 and at 460 days from the commencement of monitoring, respectively). The array of LL sensors in this building ranged from STA

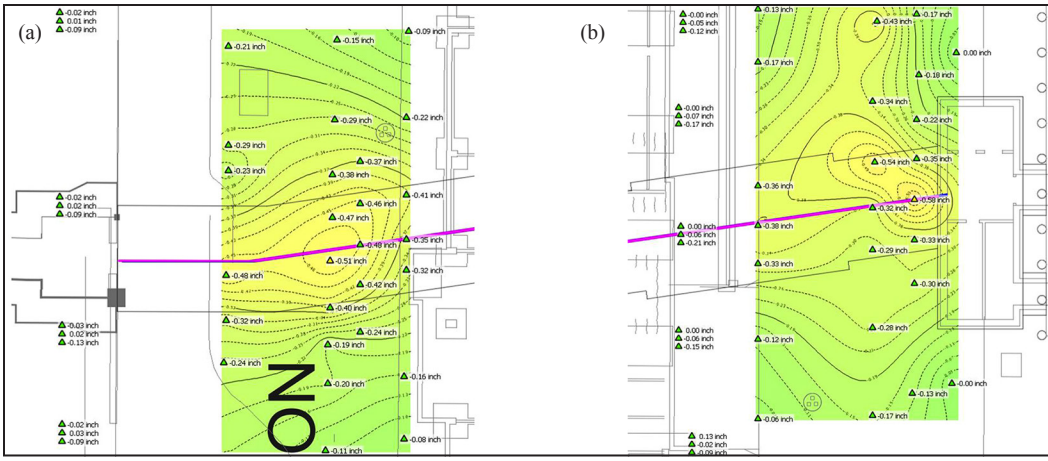

Figure 8. Settlement gradation view at (a) Union Street and (b) Front Street taken on 10/31/2021

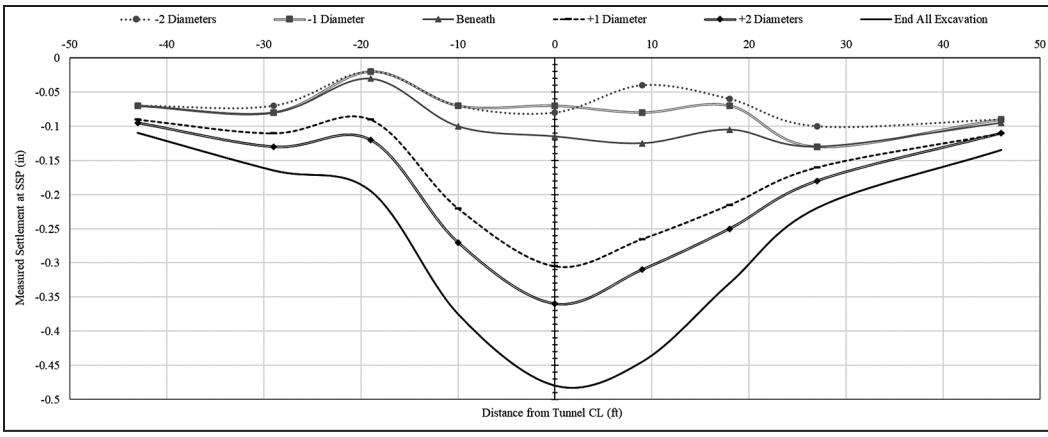

Figure 9. Settlement troughs measured along the center of Union Street during top heading excavation

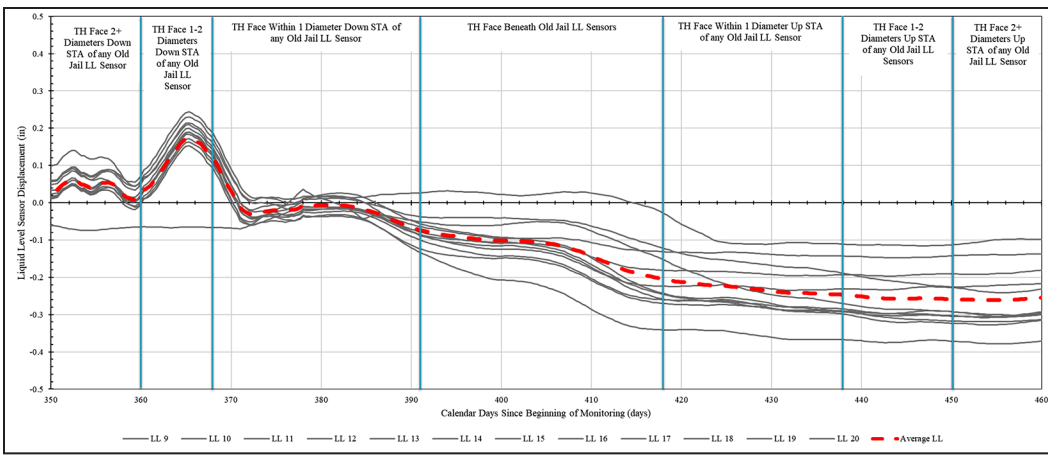

Figure 10. Liquid level sensor output within the Old Jail from 10/03/20 (at 350 days) to 01/21/21 (at 460 days)

00+87 to 01+35 along the tunnel alignment. Figure 9 is notated with the periods in which the top heading of the tunnel was mined beneath these sensors, and when the top heading face was within one or two tunnel diameters' distance from the sensors. The precise value outputs from the LL sensors in this case were too small relative to the accuracy of the system, but the general trend of most of the settlement occurring while mining beneath and beyond for one diameter's distance from the sensors can again be observed. The divergence of LL sensor values from one another during this period and throughout the future of the project also exemplifies the settlement trough formation as discussed earlier.

CONCLUSIONS

The CoCo Tunnel team utilized conventional methods for SEM construction and developed new techniques for handling loose sandy ground and unanticipated conditions. While deformations at the surface followed traditionally accepted predictions and never reached close to maximum allowable displacements, the path forward underground at times required creativity and quick decision-making. The encounter with the brick well reminds tunnel design and construction teams that conditions that significantly deviate from what is known of the ground are possible, even in areas with a plethora of subsurface data. While some schedule was lost to this surprise, the excavation was generally able to proceed in a timely manner and the Sequential Excavation Method proved to be an excellent technique for the challenges presented by the project.