Shallow SEM Tunneling with Limited Clearance to Existing Structures: Design, Construction and Observations

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ABSTRACT

The excavation and initial lining has just been successfully completed for the Downtown Bellevue Tunnel using Sequential Excavation Methods (SEM). The soft ground tunnel, approximately 2,000 feet long and 38 feet wide with very shallow cover, faced several unique challenges and design refinements that were managed and implemented during final design and construction. This paper provides an overview of the tunnel construction, then focuses on three design refinements; tunneling under an existing utility trench with 4 feet of vertical clearance, tunneling within 4 feet of an existing building basement, and eliminating pipe canopy and replacing 12-foot thick overburden soil with controlled low strength material (CLSM) at the tunnel's north portal. Details of the analysis, design, and construction of these design refinements, together with the comparisons between the predicted and observed settlement results are presented.

INTRODUCTION

East Link is a 14-mile extension of the existing Sound Transit light rail transit system from downtown Seattle, across Lake Washington, to the cities of Mercer Island, Bellevue and Redmond. The Downtown Bellevue Tunnel (DBT) is the only tunnel on the extension and, as its name suggests, extends through the downtown of Bellevue with relatively low cover between the at-grade East Main and Bellevue Downtown stations (Figure 1). The DBT consists of 250-foot-long south cut-and-cover portal structure, the 1,983-foot-long SEM tunnel, the mid-tunnel access shaft and connecting adit, and the 200-foot-long north cut-and-cover structure. The DBT was originally planned

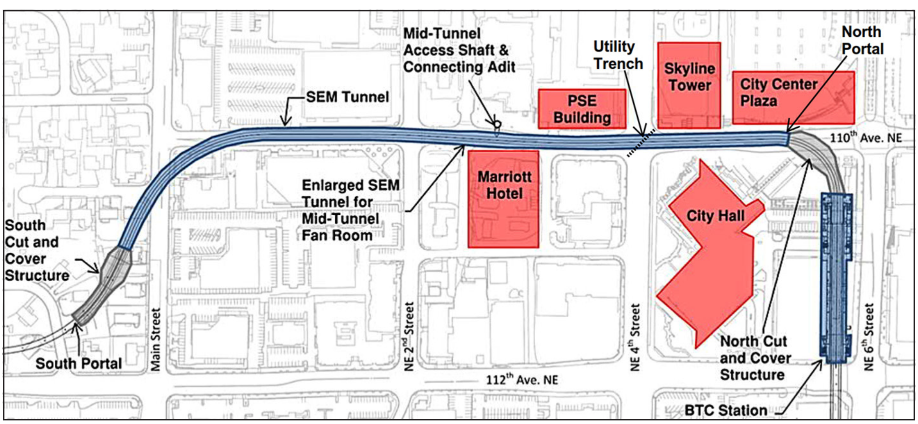

Figure 1. Plan view of project alignment

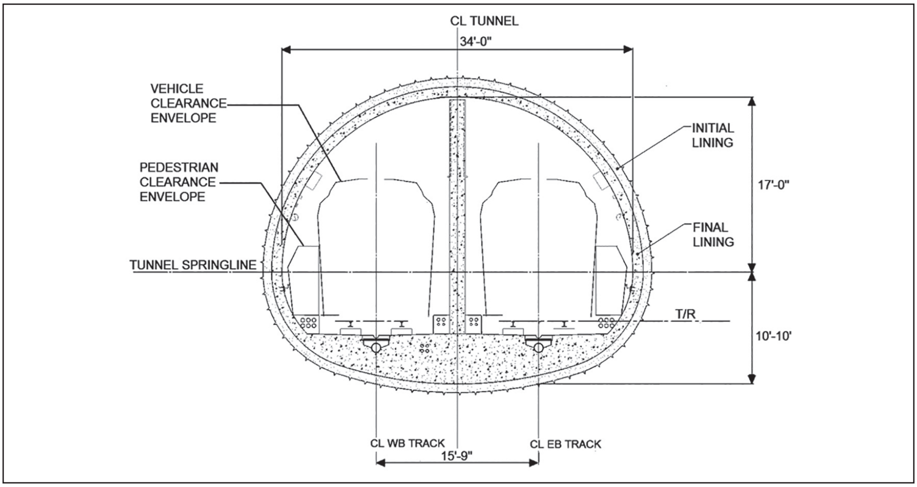

Figure 2. Tunnel geometry for typical tunnel section

as a cut-and-cover tunnel, but this was changed to reduce surface disruption and community impacts (Penrice etc, 2017).

The excavated cross-section for the typical SEM tunnel is a large 38-foot-wide by 30.5-foot-high ovoid, sized for twin rail tracks (Figure 2). The cross section is enlarged near the tunnel's mid-point to provide additional space for an emergency ventilation fan room over the tracks. An adit and shaft, also constructed using SEM provide maintenance access to the fan plant from the ground surface.

SITE GEOLOGY

The geologic profile along the tunnel indicates glacial deposits consisting of glacially overconsolidated stratigraphic sequence that includes Vashon till, Vashon advance outwash deposits, and pre-Vashon glacio-lacustrine deposits (Figure 3). North of the enlarged tunnel section the profile indicates an "anomaly zone." During the design an extensive ground investigation program was executed but no conclusive geological model could be established for this zone. During excavation of the tunnel no change of the ground behavior was observed in this zone, however offsets in the stratigraphy were encountered. The design groundwater table generally follows the top of the advance outwash, and was expected to be encountered in the tunnel face in the northern half of the tunnel. During excavation of the tunnel the ground showed more favorable conditions than anticipated. In particular, the groundwater table was much lower than expected and the planned dewatering measures which included dewatering with surface wells and vacuum dewatering from within the tunnel were not required. However perched ground water with a water inflow rate of approximately 0.2 gallon per minute was periodically encountered from within sand layers in the till.

CONSTRUCTION OVERVIEW

The excavation of the DBT started on February 3, 2017 and was completed on July 17, 2018, several months ahead of schedule. Prior to the start of the tunnel excavation a pipe arch canopy with a length of 70 feet was installed at the south portal. At the north portal, in lieu of a pipe arch canopy, the area was excavated from the surface to approximate springline elevation and the in-situ ground was replaced with CLSM

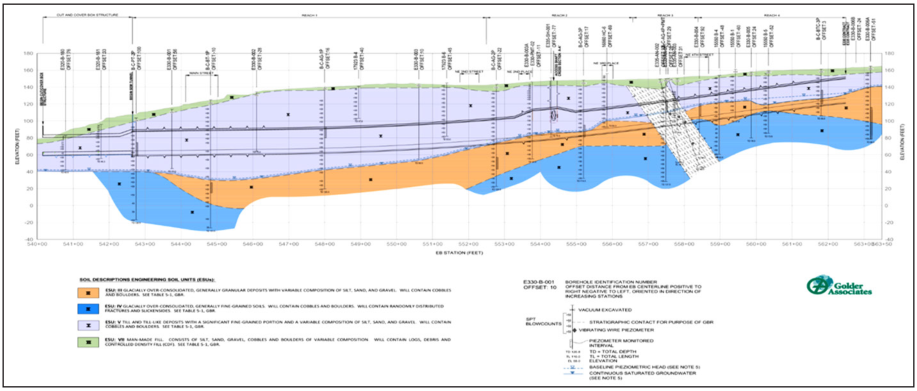

Figure 3. Geologic profile with tunnel alignment

and concrete. This significantly reduced the interference with the adjacent contractor for the cut-and-cover tunnel at the north portal. This design change is described in greater detail later.

The DBT was designed as a single side-drift excavation with a 6-heading sequence and with five Ground Support Classes, and with round lengths from 3 feet to 4 feet and with systematic spiling. To help mitigate delays caused by delayed right of way acquisition, the Contractor proposed changing the excavation from the 6-heading sequence to a 3-heading sequence from the south portal to the start of the enlarged tunnel section (approximately 50% of the tunnel length). The 3-heading sequence included a top heading, bench, and invert. This change was implemented for construction after concept validation by performing detailed analyses, and agreement on criteria that would result in a return to the as-designed 6-heading sequence (see Brodbaek etc. 2018).

From the enlarged tunnel section to the north portal, a single side-drift excavation with the 6-heading sequence was implemented, which consisted of a temporary centerwall, and each drift consisted of a top heading, bench and invert. Figure 4 shows the transition for the 3-heading to the 6-heading sequence.

For the excavation using the 6-heading sequence, two Liebherr 950 tunnel excavators were utilized. The majority of the tunnel was excavated with a bucket. Only in very short stretches of the tunnel, e.g., where lean concrete was present, a roadheader attachment or a hoe ram was used. Mucking and limited invert excavation was performed using a John Deere 135D. Figure 5 shows some of the typical activities during tunneling using the 6-heading sequence.

Figure 4. Transition from 3-heading to 6-heading sequence

It was anticipated that the tunneling would encounter tiebacks and soil nails, left in place from adjacent building basement excavations. In areas with tiebacks the utilization of a bucket made it possible to facilitate a more precise excavation around the tiebacks. This was especially important in the vicinity of the Skyline

Tower, where it was imperative that the small pillar between the tunnel and the existing building not be disturbed. The tiebacks and soil nails were included in the project Geotechnical Baseline Report as obstructions. However, their removal was relatively straightforward, with most of the elements being cut out with a torch such that the Contractor did not maintain a count of their number. For the removal of the centerwall a concrete shear was utilized, and the last foot was ground down to the required clearance with the Liebherr 950 and a roadheader attachment.

Figure 5. Excavation in left drift and preparation for setting girder in right drift

Due to favorable ground conditions encountered, only a small number of the designed spiles were installed. However, pre- and during-construction investigations of utilities and basements of buildings showed that pre-treatment of the ground was required in the proximity of the Skyline Tower and at the intersection of 110th Ave and NE 4th Street, as also described herein.

In order to have shotcrete available as required over the 24-hour work day, a batch plant was setup on site. Wet-mix shotcrete with designed 5,000-psi 28-day compressive strength was applied using a Normet shotcrete robot. As a contingency a second setup with a Reed pump and a shotcrete manipulator mounted on a tractor was used. Since the Reed pump is not an integrated system like the Normet and requires a separate accelerator pump, the dosing of the accelerator had to be carefully monitored.

Several changes to the design were made to adapt to the favorable ground conditions encountered. In addition to the 3-heading excavation sequence and the deepened CLSM at the north portal, modifications to the design included the following:

- Use of macrosynthetic fiber in lieu of steel fiber in the initial lining shotcrete
- Elimination of prescriptive spiling
- Elimination of 2 probe holes per cycle
- Elimination of bench probe holes in last 300 feet of tunnel approximately
- Elimination of continuous core holes over tunnel extent
- Increased round length (typical) from 4'-0" to 4'-3"
- Use of 4'-0" advance length in anomaly zone
- Elimination of invert excavation face shotcrete, provided invert was backfilled after shotcrete placement
- Elimination of bench sidewall flashcrete, allowed placement of 10" at once
- Elimination of convergence monitoring points in tunnel invert
- Reduced frequency of shotcrete testing based upon favorable results of previous testing (time dependent)
- Elimination of 28-day shotcrete strength testing should 7-day strength test results exceed 28-day strength requirement
- Elimination of the surface-based dewatering system

During daily SEM meetings in early mornings, the ground conditions, monitoring data and other relevant data of the previous 24 hours were reviewed and discussed. In conjunction with designer's representative, construction management and Contractor, ground support classes for the next 24 hours were agreed on. The change from a robust prescriptive design to a more adaptable design required experienced personnel from all parties attending the daily SEM meetings. Effective communication and collaboration between the designer, construction management, Contractor and owner's representatives was essential for the successful implementation of this more adaptable design.

A number of challenges and refinements were managed and implemented during the final design and the construction, in particular in the northern section of the project. Three such refinements, from south to north of the alignment, are tunneling under an existing utility trench with 4 feet of vertical separation at NE 4th Street; tunneling within 4 feet of the existing Skyline Tower basement; and eliminating pipe canopy and replacing 12-foot thick overburden soil with CLSM at the tunnel's north portal, discussed hereafter in greater details. The locations of these three refinements are shown in Figure 1.

TUNNELING UNDER NE 4TH STREET UTILITY TRENCH

At the intersection of NE 4th Street and 110th Ave (Figure 6), the Contractor's preconstruction investigation indicated that an existing storm drain utility was in poorer condition than anticipated during the design. Inspection of video survey and still photographs revealed holes in the invert and offsets at the pipe joints of a 12-inch diameter concrete drainage pipe which had likely resulted in leakage. The duration and extent of the leakage was unknown. It was postulated that the leakage may travel along the bedding of the pipes and accumulated at low points, resulting in saturation and degradation of the surrounding soils. The soil cover underlying the utility trench above the tunnel excavation was limited, with a minimum estimated thickness of approximately 4-foot of glacial till. The potential degradation of the soil between the tunnel and utility trench and the risk of the presence of sand lenses in the till between the tunnel and trench, coupled with the small soil cover, created significant risk to the tunneling. It was further discovered during the Contractor's exploratory work comprising field investigations and potholing that the utility trench was up to 6-foot wide, significantly wider than the 2-foot trench originally anticipated. Furthermore, the trench had been backfilled with pea gravel, which would have exhibited running behavior if encountered during tunneling. Correspondingly ground stability was a principal risk.

The original design of the SEM tunnel at the 4th Street crossing was based on assumptions that the utilities were in relatively good condition, that the groundwater table was located much deeper (close to tunnel springline), and that groundwater would be removed by surface dewatering measures applied in advance of tunnel excavation. In addition, the existing utility trench was assumed to be 2-foot wide. A trench of this width was considered in the design and was shown to not impact or be impacted by the tunneling.

Because of the new information provided during the construction, prior to tunneling reaching the intersection of 4th Street and 110th Ave, a number of mitigation measures were proposed and implemented, including local excavation of the trench and removal of the pea gravel by vacuum extraction; permeation grouting of the pipe bedding using 150-psi grout; then backfilling the trench with CLSM. As part of the design service during construction, 2-dimensional numerical modelling was performed to determine the minimum thickness of till that would support the utility trench without CLSM. Where

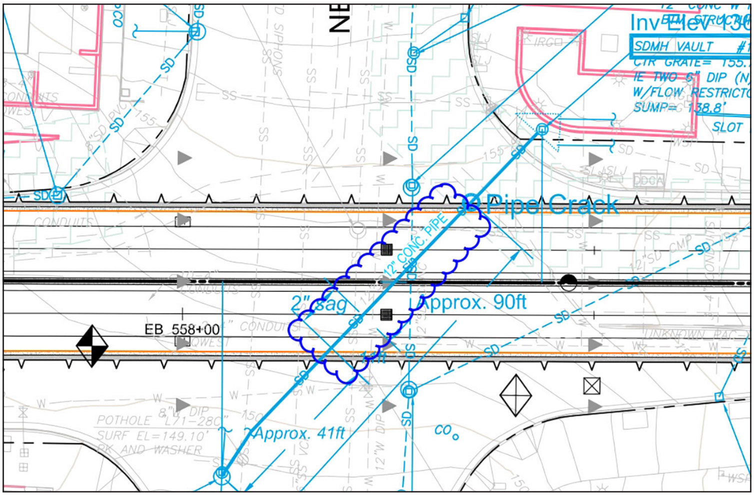

Figure 6. Plan view of utility trench crossing tunnel alignment

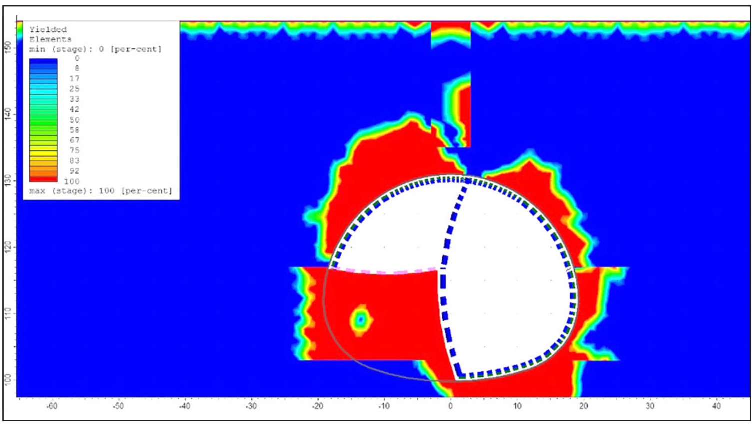

Figure 7. Numerical modeling result for utility trench overlying tunnel

the till thickness was less than this minimum, the trench would be filled with CLSM to the ground surface. The locations where sufficient till cap existed served as the limits of the CLSM replacement. The modeling steps and methods involved the use of the ground relaxation method for SEM (FHWA 2010) and was performed using the Rocscience RS2 software. An example of the model result is shown in Figure 7. Parametric studies were performed by varying the till cap thickness, soil strength and stiffness parameters, Ko values, and ground relaxation factor to envelope the potential conditions which could be encountered at the crossing location.

The modeling results showed that the various conditions modelled resulted in ground volume loss ranging from 0.16 to 0.4%, and estimated ground surface settlement ranging from 0.3 to 0.7 inches. The effectiveness of the CLSM mitigation was also demonstrated by the modelling results. The tunnel shotcrete lining capacity showed the as-designed lining would have adequate capacity under the various conditions modelled. Based on the modeling results, it was concluded that tunneling under the utility trench was feasible with the implementation of the proposed pre-tunneling mitigation measures.

A number of additional mitigation measures were implemented during tunneling at the crossing, including: reducing round length and lattice girder spacing from 4 feet to 3 feet; excavating the top headings utilizing pocket excavation with a minimum of two pockets of similar sizes; installing 9-foot long spiles with a maximum of 18-inch spacing under the utility pipe, with the spiles extending to 45 degrees to either side of the tunnel centerline; and maintaining a face wedge (Figure 8) in each of the top headings. These agreed measures further reduced the risks associated with the existing utility trench during tunnelling. During construction, water inflow of maximum 0.5 gallon per minute was observed inside the tunnel when crossing the trench, and this water inflow continued for several months. Water encountered drained off inside the tunnel, and no surface dewatering was performed based on the Contractor's engineer's understanding of the groundwater regime in the project area. No significant instabilities or overbreak were observed during excavation under the utility trench.

Figure 8. Face wedge formed during tunnel excavation under the utility trench

Instrumentation and monitoring results verified that the ground and tunnel were stable during the tunneling. An example of monitoring results is shown in Figure 9, which shows a maximum ground surface settlement of 0.30 inches and also shows good correlation with the lower end of the range of the predicted settlement.

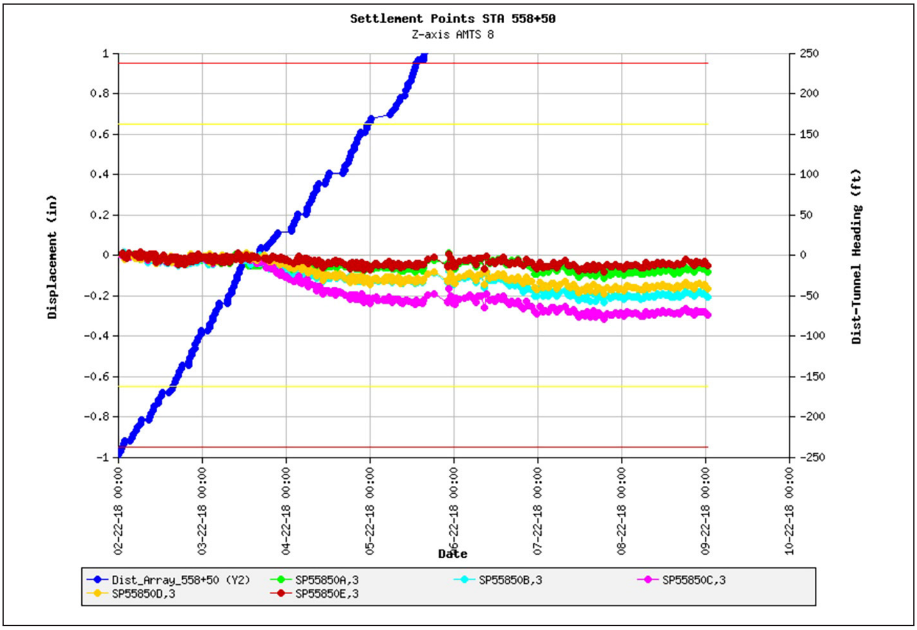

Figure 9. Ground surface settlements measured at utility trench crossing

TUNNELING NEXT TO EXISTING BASEMENTS

Several existing building basements are located in close proximity to the tunnel. In particular at the Skyline Tower basement, the minimum separation between tunnel extrados and outside of basement excavation support was limited to approximately 4 feet (Figure 10). The existing permanent basement wall construction comprised a composite system of concrete encased steel soldier piles at 7-foot spacing with 7-inch thick cast-in-place concrete facing that was lightly reinforced.

The interaction between the tunnel, ground, and the existing basement was studied in depth during the final design. The key objectives were to confirm the stability of the relatively narrow soil pillar between the tunnel and the basement, determine the movement of the ground and basement structure in response to tunnel excavation, and assess the change of ground stresses and soil pressure on the basement walls as a result of SEM tunnel construction sequence, among others.

Two-dimensional finite element analyses were performed using the Rocscience RS2 software. The effects of ground relaxation ratio were used to simulate three-dimensional effect ahead of the heading. Soil moduli, Ko and soil cohesion were parametrically studied. The basement wall(s) and floor slabs were added to the 2D numerical models. The basement walls were modeled as vertical beam elements. The basement floor slabs were modeled as spring supports for the basement wall. To replicate the existing conditions of stress in the ground prior to tunneling, modeling and analysis steps simulated the support of excavation (SOE) installation, basement excavation sequence, and the application of building loads to the foundation soil. An example of the modeling results in terms of ground displacement is graphically shown in Figure 11.

Parametric studies evaluated the sensitivity to the modeling techniques and input parameters, such as the mathematical representation of the floor slab (as beam elements vs spring supports), depth of the first lift of basement excavation, the level of prestressing force in the temporary tiebacks, the interface between the basement wall and soil (slip, no-slip, or with friction angle), Ko value, and the stiffness relaxation of

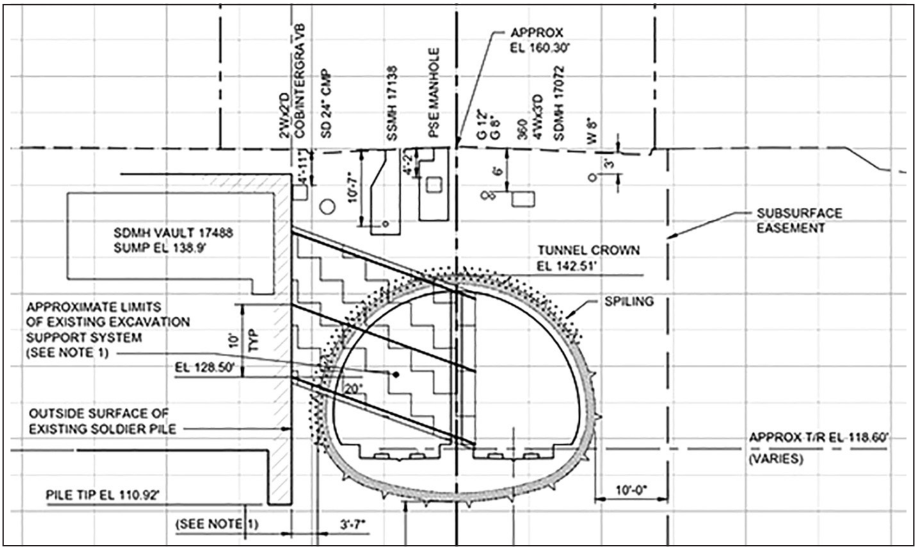

Figure 10. Cross section at Skyline basement showing small clearance from tunnel

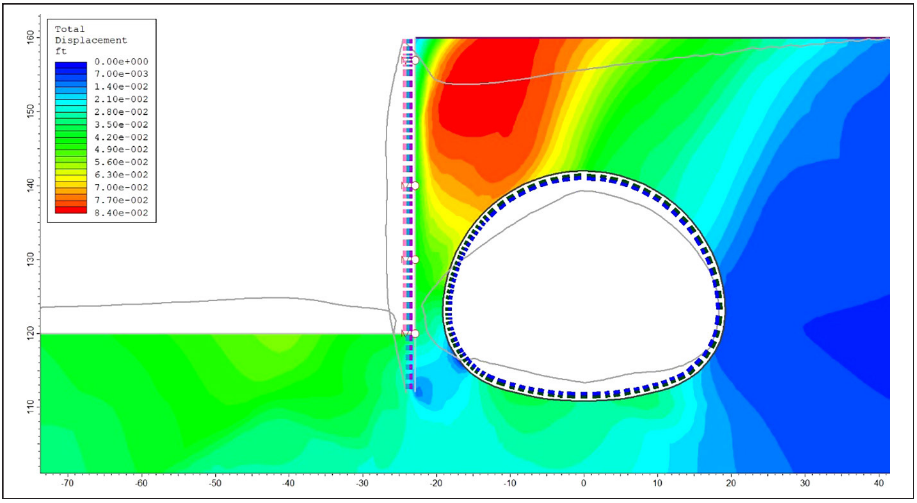

Figure 11. Numerical modeling result for post-tunneling condition at Skyline basement

the soil pillar between the basement and the tunnel. For these parametric studies, the change in lateral soil pressures on the basement wall after SEM tunneling were found to be sensitive to the assumed stiffness relaxation of the soil pillar, Ko value, and the soil-wall interface friction angle. The changes in soil pressures on basement wall after SEM tunneling did not appear to be sensitive to the variation in the tieback prestressing level or the depth of the first lift of basement excavation.

The modeling results showed that lateral soil pressures on the basement wall would increase significantly after tunneling. The structural capacity of the adjacent basement was evaluated for the change in lateral soil pressures and was found to be inadequate. Given the unusual basement design, and limited capacity to absorb any changes in stress arising from the tunneling, structural strengthening of the basement wall, comprising an additional 6-inch of bar reinforced shotcrete was performed in advance of tunneling. During this work a water filled void behind the basement wall was encountered. Due to the close proximity to the tunnel and the risk of water causing further deterioration, the thin pillar and void was grouted extensively. Additional monitoring points were installed in the garage to measure the performance of the building structure during tunneling, including tiltmeters, strain gages and structural monitoring points.

Tunneling adjacent to the Skyline Tower basement was successfully completed based on the design that was supported by the extensive analyses, and 4-foot round length excavation was adopted. The as-designed excavation sequence was modified to include pocket excavation during bench excavation for the left tunnel drift next to the basement due to the close proximity to the basement wall. During this excavation, one particular challenge was the removal of the existing tiebacks within the tunnel excavation limit. Figure 12 shows a tieback

Figure 12. A tieback encountered during excavation next to Skyline basement

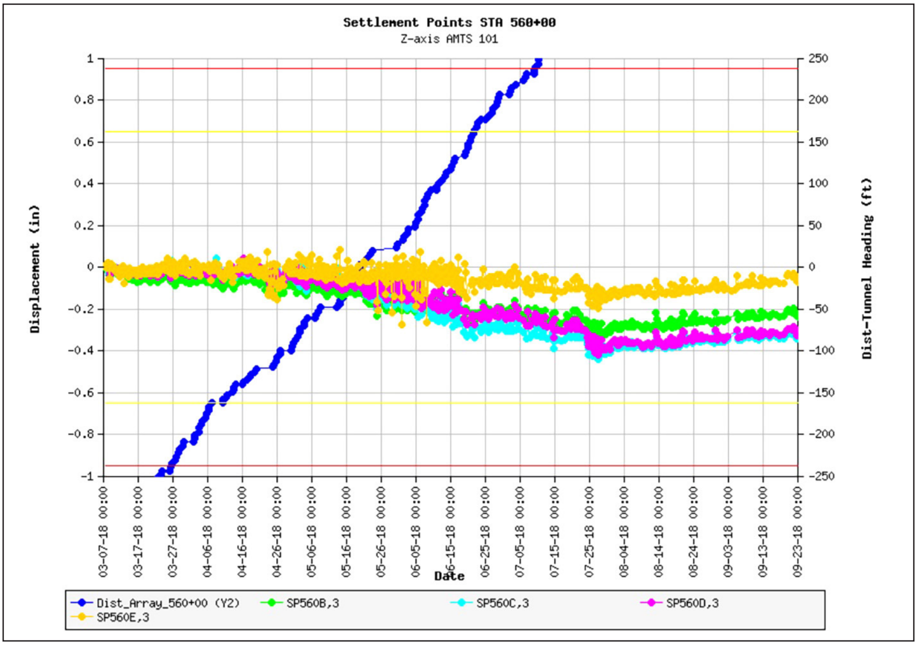

Figure 13. Ground surface settlements measured at Skyline

encountered during tunnel excavation. Another challenge was to remove boulders encountered. Tieback cutoff and boulder removal had to be performed with precision to minimize the disturbance to the ground.

The stability and safety of the ground, tunnel and basement were monitored and confirmed by the results of the extensive instrumentation and monitoring. The largest ground surface settlement observed was 0.45 inch (Figure 13). The strengthened basement wall was observed to move only 0.07 inch laterally. The observed maximum ground surface settlement value compared well with the predicted settlement that was estimated ranging from 0.5 to 1.0 inch.

TUNNELING AFTER GROUND REPLACEMENT AT NORTH PORTAL

At the northern section of the SEM tunnel, the soil cover over the tunnel crown was approximately 12 feet, which was shallow relative to the tunnel excavation size. Originally it was perceived that the material overlying the tunnel crown was principally Vashon Till, overlain by a limited depth of fill and roadway pavement. A pipe canopy, comprising a double row of 6-inch diameter pipes was to be installed over the tunnel over a length of approximately 70 feet to help support the ground in the area of limited cover. These soil conditions, with the inclusion of the canopy, were demonstrated to be sufficient to maintain a stable excavation. However, greater scrutiny of existing borings during the final design phase and removal of borings more remote from the tunnel alignment suggested that the fill was far deeper than originally anticipated, presenting significant risk to the tunneling.

After a series of mitigation options were studied during the final design, the as-bid design required that the fill material be removed over the width of the tunnel and over a length of 125 feet from the north SEM tunnel section and that the pipe canopy be

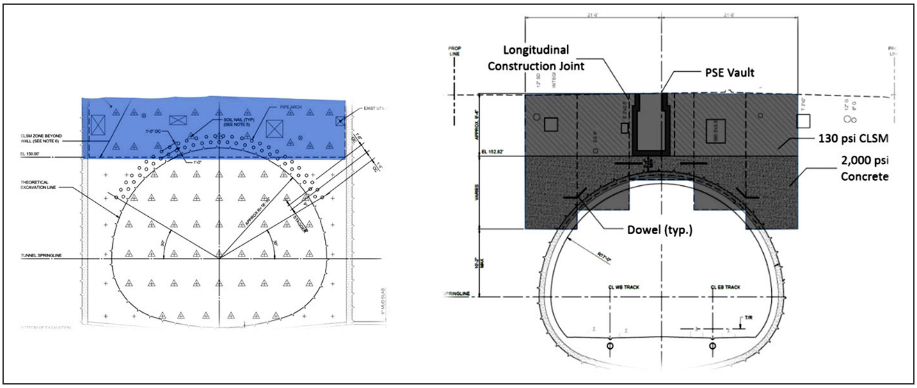

Figure 14. North portal as-bid design (left) and proposed scheme (right)

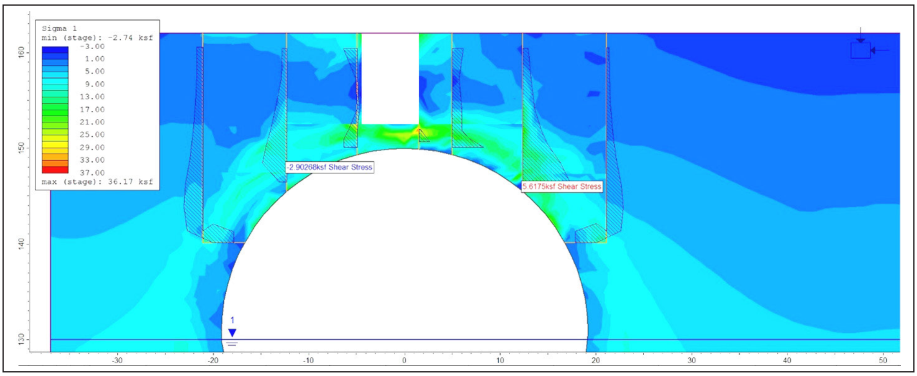

Figure 15. Numerical modeling result for north portal area with vault based on the full ground replacement scheme

installed (Figure 14). The installation of the pipe canopy at the north SEM limit would require the Contractor to coordinate possession of the north cut-and-cover excavation site with an adjacent contract, introducing risk of delays and conflicts for both contracts. As a mitigation, the Contractor proposed deepening the existing CLSM to create a 'structural' arch over the crown of the tunnel (Figure 14), and thereby avoided the need to install the pipe canopy.

A further complication in the north portal area was that a large electrical utility vault was found to be deeper than previously identified and could conflict with the pipe canopy installation. As part of the evaluation of the deepened CLSM, a separate structural concrete arch capable of spanning the tunnel excavation was developed for the vault.

To maintain surface traffic, the Contractor further proposed to install the structural concrete and CLSM in a series of 5 longitudinal trenches, as indicated in Figure 14 and the concept was evaluated. The details of the extended CLSM replacement, construction joints, material properties, and required analyses and modeling inputs and parameters were agreed by all parties in advance. The strengths of the CLSM and concrete arches were 130 psi and 2,000 psi, respectively. The design was confirmed by a series of 2D numerical models using the Rocscience RS2 software, as shown in Figure 15 for result, for example. For the structural concrete arch, dowels were required at each longitudinal joint to accommodate shear transfer. For the CLSM, only roughening and cleaning of the vertical cold joints was necessary.

Since the numerical modeling demonstrated the feasibility of the deepened CLSM solution, and as the solution eliminated a contract interface and reduced costs on two separate construction contracts, this change was implemented for construction with the agreement of all parties.

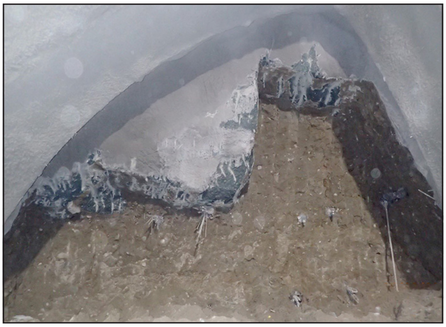

Figure 16. Concrete of 2000 psi from ground replacement exposed during tunneling

One challenge during tunneling under the CLSM replacement area was to remove a limited amount of the 2000-psi structural concrete (Figure 16) that was installed to protect the electrical vault; the concrete that protruded into the tunnel excavation line had to be carefully trimmed off to form the tunnel profile, while the disturbance to the underlying soil had to be minimized. Tunnel excavation and initial lining construction for the tunnel under the CLSM replacement area was completed safely in July 2018, with hole-through occurring in mid-July.

During and after the tunneling, the ground movements observed were small; the largest ground surface settlement observed was 0.33 inch (Figure 17), which was comparable with the predicted settlement of 0.5 inch.

SUMMARY AND CONCLUSIONS

Three cases involving unique challenges and design refinements facing the design and construction of the DBT have been presented and discussed. The mitigation measures implemented prior to or during tunnelling and the monitoring results have also

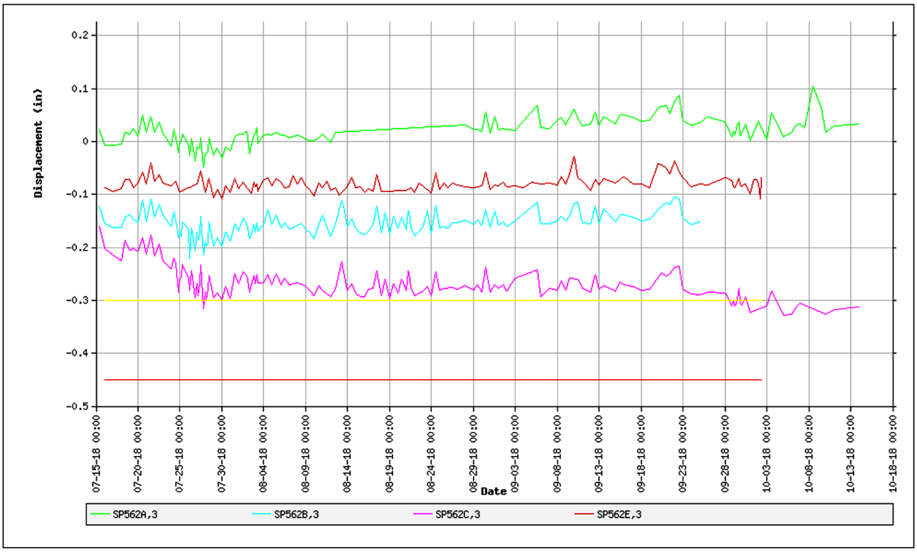

Figure 17. Measured ground surface settlements at north portal after tunneling

been discussed. These cases show that numerical modeling has proven to be efficient as part of the analyses, designs and design refinements, even during the construction. Instrumentation and monitoring results confirmed the validity of the designs and analyses and ensured the safe proceeding of construction. The cases presented also demonstrate the flexibility of the SEM tunnelling method to adapt to the changes in field conditions, provided the issues are identified sufficiently in advance. The cooperation of the project parties, including the design engineer, the construction management team, the Contractor, and Sound Transit, was also the key in achieving a successful outcome in each case.

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