# CONSIDERATIONS FOR VERY SHALLOW CONVENTIONAL (SEM) TUNNELING IN URBAN SETTINGS

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## ABSTRACT

Construction of shallow mined tunnels is frequently undertaken by use of conventional tunneling known as the Sequential Excavation Method (SEM) and sometimes called NATM. In soft ground at shallow cover the method relies on the use of presupport methods and ground improvement to satisfy permissible deformations of affected facilities. A recent project involved tunneling for twin single-track Metrorail guideway tunnels at Tysons Corner, Virginia at depths as shallow as 7–8 feet overburden over the crown. The paper addresses the range of considerations, including ground conditions, insurer's requirements, utilities, and existing structures, that prompted design and construction refinements. These led to a robust design and successful execution nowadays called for in urban environments.

## INTRODUCTION

Construction of Phase I of the Dulles Corridor Metrorail Project (DCMP) by the Metropolitan Washington Airports Authority is well under way and is scheduled to be operational by 2013. The 11.8 miles that constitute Phase I are being engineered and constructed by Dulles Transit Partners, LLC (DTP) a Bechtel led JV with URS in a Design-Build arrangement. Together, Phase I and Phase II will add a total of 23 miles to the Washington D.C. Metro system, with the new extension designated as the Silver Line.

In addition to the 11.8 miles of track, Phase I will include construction of 5 new stations (two at grade and three elevated) and two 1,700 foot long mined tunnels (Figure 1) at Tysons Corner excavated using the Sequential Excavation Method (SEM), also referred to as the Conventional Method of Tunneling. Phase II will extend the final 11.2 miles of the Silver Line to its terminus station in Ashburn, Virginia and will include a station at Dulles International Airport.

## REGIONAL GEOLOGY

Tysons Corner is located in the Piedmont Province and is predominately underlain by schist, phyllonite, gneiss, and to a lesser extent, igneous intrusive rocks. The project site is located just west of the Fall Line, which is the contact between the metamorphosed bedrock of the Piedmont Physiographic Province and the un-lithified sediments of the Coastal Plain Physiographic Province.

Resting un-conformably atop the Piedmont at Tysons Corner are ancient unlithified Coastal Plain sediments. These sediments act as a protective cap, preventing
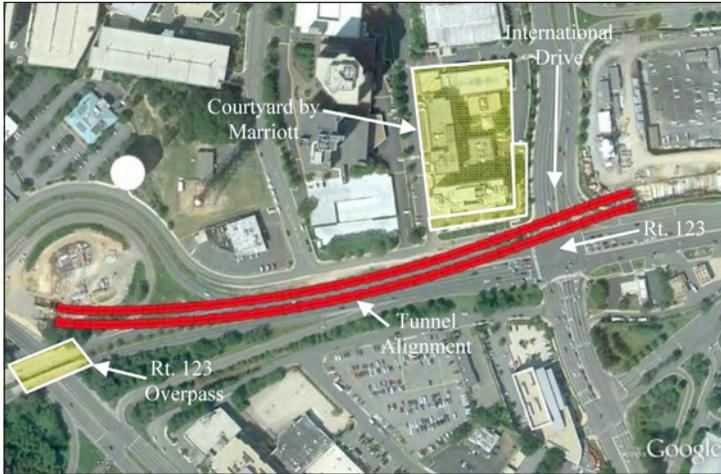
Figure 1. Relation of tunnel alignment to critical structures at Tysons Corner

erosion of the Piedmont residual soils on which they rest. The result is the hill through which the tunnels must pass, which is also the highest point in Fairfax County.

The Piedmont residual soils are the result of in-place weathering of the underlying bedrock and are typically fine sandy silts, clays and silty fine sands. The project soil classification identifies the residual soils as Stratum S, which is divided into two substrata (S1 and S2) based on the consistency and degree of weathering. S1 Substratum produced an average N-value of 12 bpf while S2 Substratum produced an average N-value of 30 bpf. Only to a limited extent where the tunnel is deepest tunneling encountered decomposed rock referred to as D1 in the relict structures of the bedrock material, and produces a range of N-values from 60 to 100 bpf. Groundwater is generally at invert elevation at portal locations and rises up to just above the tunnel spring line at the mid-point of the tunnel alignment.

### Supplemental Geotechnical Ground Investigation

Prior to the start of excavation of the Outbound tunnel (OB), concern was raised regarding the quality of the Coastal Plain materials and the possibility of raveling sand and gravel. An additional ground evaluation of the shallowest points of the tunnels was conducted using the Cone Penetrometer Test (CPT). This test provided details of the stratigraphy of the Coastal Plain material including the dynamic pore pressure, tip resistance, and sleeve friction. The latter three are used to interpret the type(s) of soil encountered. A total of 20 CTP tests were conducted above the tunnels and one additional test was conducted within ~20 feet of the East Portal, centered between the two tunnels (Figure 2). A photo mosaic of the East Portal geology was created using images of ground exposed during excavation of the East Portal SOE and was compared side-by-side with the final CTP test output to assist in confirming material interpretations.

### RISK CONSIDERATIONS

During the Preliminary Engineering of the tunnels a risk assessment was conducted. This assessment established the considerations that would influence the overall design of the tunnels (Rudolf 2007 and 2008). Items considered included potential for excessive surface settlements or heave, tunneling safety, potential for uncontrollable ground inflow, adaptability to geologic uncertainty and buried obstructions, severity

**Figure 2. View of International Drive ramp showing outline of tunnel sawteeth and location of CPT tests in the shallowest area over the tunnels**

of required surface disruption, and tunnel construction duration. All considerations were addressed in the design or during construction as additional obstructions or unforeseen ground conditions were encountered. The flexibility allowed by the conventional tunneling method ensured a rapid response to any unforeseen risks. The main risk mitigation elements that were called for by the design and specifications included the following:

- Positive pipe arch pre-support by steel grouting pipes above the tunnel crown throughout the entire alignment combined with short excavation rounds with early ring closure. With these in place a robust design was established.
- Contingency support measures were developed and formally addressed in a Contingency Plan document reviewed by all parties prior to start of construction.
- Very skilled conventional tunneling personnel assigned to the key roles of SEM Senior Tunnel Engineers, SEM Superintendents and operators were assigned to the site to lead the tunnel construction during all shifts.

This well rounded tunnel package found approval by third parties including external review agencies as well as project insurers and can be seen as a model for the use of conventional tunneling in urban, shallow settings.

## TUNNEL DESIGN CONSIDERATIONS

The original geometry for the SEM tunnels was based on the Washington Metropolitan Area Transportation Authority's (WMATA) standard tunnel design drawings. However DTP augmented and enlarged the geometry in compliance with revised National Fire Protection Association (NFPA) standards (Rudolf 2010). The result was the geometry shown in Figure 3. The tunnel design consists of a dual lining with a fully tanked PVC waterproofing system. The initial lining consists of 10" thick steel fiber reinforced shotcrete and steel lattice girders, while the final lining will be 12" thick cast-in-place generally plain concrete. Approximately 600 feet of the concrete final lining of each of the two tunnels will be reinforced. A PVC waterproofing membrane, installed between the initial and final linings, will exclude groundwater and promote dry tunnels.
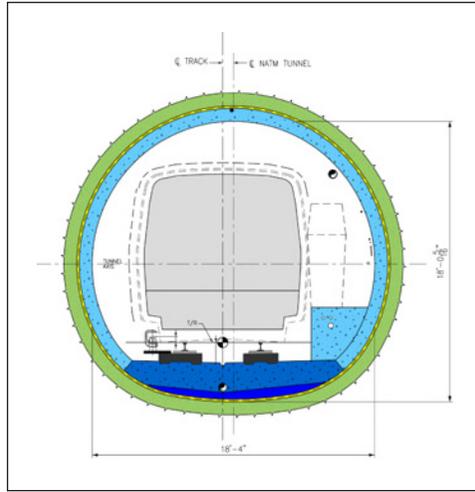
**Figure 3. Typical tunnel cross section**
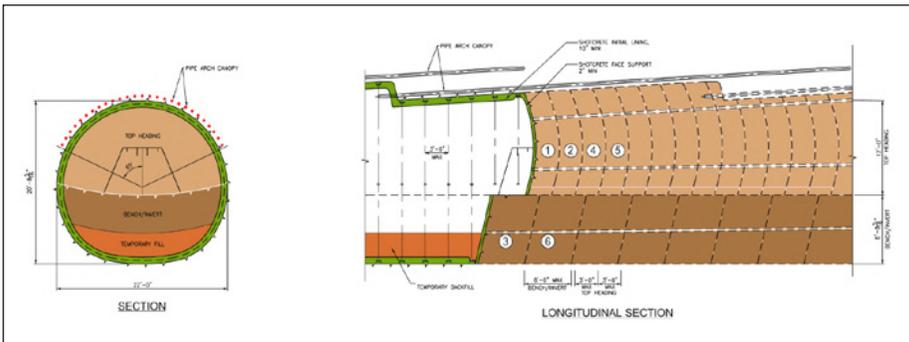


**Figure 4. Pre-support and excavation and support sequence of Tysons Corner tunnels**

With soft ground conditions as described above and very shallow cover in particular at International Drive, implementation of a proper pre-support system was vital to a successful tunnel design and construction. A steel tube pipe arch canopy was incorporated into the design to support the ground q1uring excavation and mitigate the effect of tunneling on surface facilities and utilities (Figure 4). Installation of the pipe arch canopy required a gradual increase in the tunnel cross section to allow installation of the subsequent canopy resulting in a "sawtooth effect" (Figure 5). The shallow topography of the first 300 feet of construction in both tunnels necessitated a double row steel pipe arch canopy, while the remaining length of both tunnels implemented a single row pipe arch canopy.

## Excavation and Support

Numerous elements, discussed in greater detail below, contributed to the robust design of the tunnels. A steel grouted pipe arch canopy was implemented along with steel fiber reinforced shotcrete and a quick ring closure. The excavation followed a typical SEM sequence with two 3 foot top heading rounds, followed by a single 6 foot bench/invert round as shown in Figure 4.

**Figure 5. View of inbound tunnel displaying sawteeth**

Contingency measures were planned in the event of excessive convergence or surface settlement. These included implementation of face support measures by dowels or a support wedge, installation of grouted spiles between pipe arch canopy pipes, earlier closure of each ring, grouting of non-cohesive receptive ground, and pocket excavation.

## Utilities

Utilities were a major concern for the first ~400 feet of excavation due to the shallow cover and, thus, close proximity of the utilities to the pipe arch pre-support (Figure 6). Eight major utilities had to be abandoned and relocated prior to construction of the East Portal Support of Excavation (SOE) and the tunnels. These abandoned utilities included a 34.5 KV electric line, five communication lines, and two gas lines. Some were abandoned due to construction of the East Portal SOE, while others were either within the pipe arch canopy envelope or were pressurized and too dangerous to risk compromising during construction (i.e., the gas lines). To account for the utilities and shallow ground cover, the sawtooth and pipe arch canopy arrangement was modified at the shallowest point in the Inbound (IB) tunnel. The design allowed for such adjustments, which in this case changed the first sawteeth in the IB tunnel from 42 feet with double row pipe arch canopies to 21 foot and 33 foot sawteeth with single row pipe arch canopies. This adjustment provided additional ground cover by reducing the size of the sawteeth and increased overlapping of the steel pipes to minimize surface settlements.

A majority of the utilities remained active during construction and were fitted with Utility Settlement Indicators to monitor settlement during construction. None of the utilities surpassed threshold settlement values during construction.

An additional concern during construction was the removal of an abandoned Verizon manhole and the resulting 11'×19' and 16' deep hole. The hole was backfilled with lean concrete, but concern remained regarding the extent of potentially disturbed soil or fill placed during installation of the manhole since part of the manhole was within the shoulder of the OB tunnel. To secure any loose soil or fill surrounding the back filled hole prior to excavation, the construction team grouted using cementitious grouts in select locations. Ultimately construction proceeded through the backfilled trench without any issue.
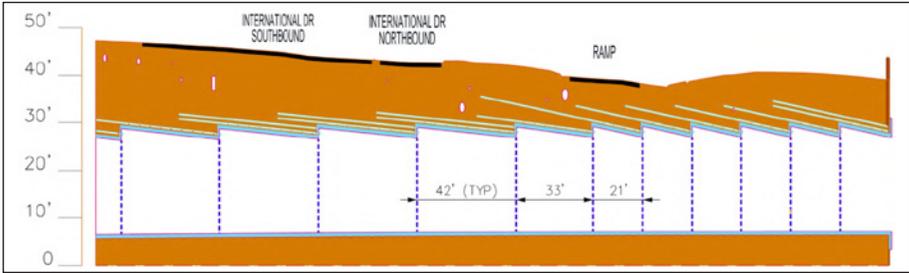
**Figure 6. Profile of tunnel depicting adjusted sawteeth and pipe arch canopies in relation to utilities**



**Figure 7. Coastal Plain sediments sitting unconformably atop Piedmont residual soil**

### Existing Structures

The urban environment of Tysons Corner presented a number of sensitive structures along the tunnel alignment that had to be taken into consideration in both the design and during construction. These structures included an underground Marriott Hotel parking garage, Route 123 overpass bridge piers, Route 123, and International Drive. Figure 1 shows the structures and roads in relation to the tunnel alignment. The Route 123 overpass bridge piers and Marriott underground parking garage are ~50' and ~25' respectively from the tunnels. The ramp onto International Drive and International Drive itself represented the shallowest points of the tunnels. The ramp onto International drive is ~7 feet above the IB tunnel crown, while International Drive was only 15 feet above the IB tunnel crown.

### Site Conditions

The first few hundred feet of tunneling for both tunnels involved excavation through the above referenced Coastal Plain sediments and Piedmont residual soil. As excavation progressed deeper into the side of the hill the Coastal Plain sediments slowly rose out of the tunnel face until the only material encountered was Piedmont residual soil

**Figure 8. Piedmont residual soil displaying blocky nature**

(Figure 7). Within the Coastal Plain sediments construction encountered a layer of iron-oxide cemented basal conglomerate that spanned both tunnels, but only persisted a relatively short distance along the alignment. This cemented layer was as much as 36" thick at some locations. The primary material encountered during excavation was the Piedmont residual soil and soil-like decomposed rock. During excavation the Piedmont often displayed relict foliations and joints, which were responsible for the occasional inconsequential block fall-out from the face (Figure 8).

## Groundwater Control During Construction

Being a soft ground tunnel, groundwater control was particularly important to ensure a stable face during excavation. Numerous groundwater control measures were implemented including probe drilling at the beginning of each sawtooth, maintaining small movable electric pumps at the excavation face, and installation of pump sumps throughout the tunnel as needed (see Figure 9). The pump sumps were installed in the tunnel invert and had a perforated drain pipe that stretched across the tunnel invert to divert any groundwater collected into the sump, where it would be pumped out of the tunnel. The implemented measures were successful in controlling groundwater infiltration and ensuring face stability during excavation.

## 2-D FINITE ELEMENT ANALYSIS

To assess the tunnel design with regard to ground and tunnel lining behavior, a two-dimensional (2-D) finite element analysis was performed. The analysis was conducted using Phase 2, v7.005 by RocScience, Inc and formed the basis for the tunnel structural design and assessment of tunneling induced deformation. In the analysis the lining was modeled as beam elements, while the ground was modeled using triangular solid elements. Input values for the physical properties of the ground were taken from the project geotechnical investigation programs.

Modeling consisted of the sequential excavation of top heading and bench/invert with installation of the shotcrete initial lining. This approach resulted in ground relaxation and subsequent loading of the shotcrete initial lining. The results of this analysis were used to verify the shotcrete lining design thickness and its reinforcement.
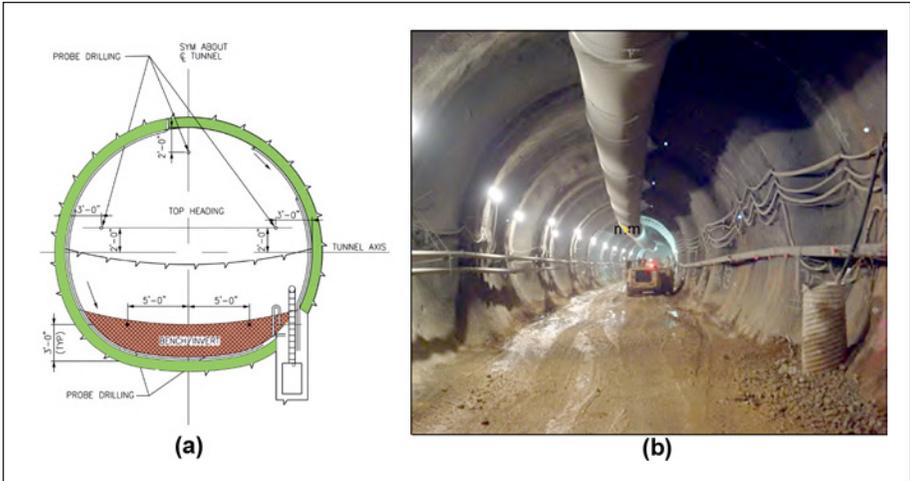
**Figure 9. (a) Typical tunnel pump sump design and (b) implementation in Tysons Corner tunnels**
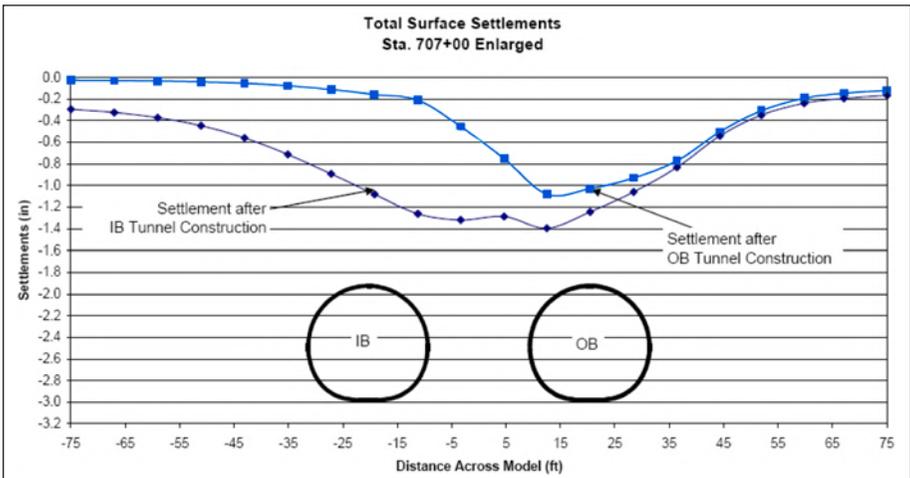


**Figure 10. Typical surface settlement output from 2-D analysis**

Additionally, deformations associated with the excavation were used to assess the ground and surface deformations for both tunnels.

The analysis results indicated a maximum vertical cumulative surface settlement of 1.40 inches and tunnel convergence of 1.18 inches. Figure 10 displays an example of the anticipated surface settlements after Outbound and Inbound tunnel construction at a depth of about 25 feet.

## 3-D FINITE ELEMENT ANALYSIS

A three-dimensional (3-D) finite element analysis was conducted to supplement the 2-D analysis and assess the Pipe Arch Canopy performance at the shallow over-burden at International Drive. The 3-D analysis was performed with ABAQUS v6.8-1,
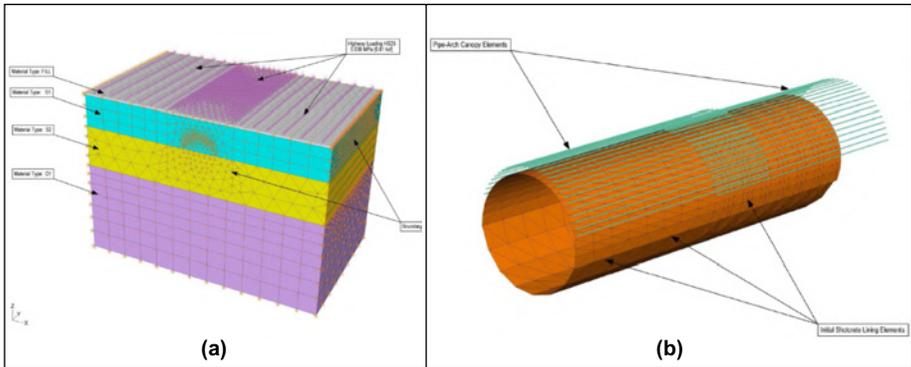
**Figure 11. (a) 3-D Model of tunnel with soil layers, (b) 3-D model of pipe arch canopy**

by Simulia Corp, using continuum 3-D shell elements to model the shotcrete initial lining, and beam elements to model the grouted steel pipes forming the pipe arch canopy. A view of the 3-D model showing a single tunnel with pipe arch canopy and soil strata is shown in Figure 11a.

The complexity of the model and the required computation time necessitated that the analysis be performed on a single tunnel only. This single tunnel was modeled with the shallowest overburden encountered by the tunnels (~7 ft) and as a constant cross section equal to the largest cross section achieved by the sawtooth excavation. Two single row Pipe Arch Canopies were implemented as pre-support and excavation followed the designed arch and bench/invert sequencing with a 10 inch shotcrete initial lining. A view of the tunnel section and pipe arch canopies can be seen in Figure 11b.

## INSTRUMENTATION AND MONITORING

Determining the threshold deformation values for in tunnel convergence and surface settlements in the instrumentation plan was a critical element that required careful consideration. With International Drive and Route 123 crossing the tunnel alignment at its shallowest point and remaining operational throughout construction, surface settlements and slopes were critical. DTP evaluated criteria from both the construction and operational portion of a pavement's lifespan to establish the threshold values. This evaluation included numerical analysis which ultimately led to the establishment of the threshold values in Table 1 as accepted by the client. Additional analysis of utilities, structures and the finite element analysis of the tunnel initial lining produced the remaining threshold values for the tunnel structure. The next step was to have an instrumentation system in place that would confidently assess the deformations and settlements.

One of the most critical sections of the tunnel construction consisted of the approximately 300 feet in which the tunnels passed beneath International Drive at shallow depths; as little as ~7 feet of overburden at one point. This shallow overburden between the tunnels and road structures concerned the Virginia Department of Transportation (VDOT), the owner of the public traffic facilities at Tysons Corner. A "Real-time" monitoring of the surface for the first 300 feet of tunneling was implemented using dense arrays of monitoring points on the ground surface. The area was designated the "Intensified Monitoring Zone" or "IMZ" (Figure 12a). The "Real-time" monitoring entailed taking measurements of the surface arrays every hour. The recorded data was then automatically processed into graphs and loaded onto a website accessible by VDOT and other permitting agencies. To accomplish the "Real-time" monitoring, DTP decided to use

**Table 1. Tunnel construction threshold limit values**

| Description | Level 1 | Level 2 |
|---|---|---|
| Ground surface settlements (1st tunnel excavation) | ¾-inch | 1¼-inch |
| Ground surface settlements (1st + 2nd tunnel excavation) | 1-0-inch | 1½-inch |
| Horizontal ground movement at tunnel elevation (at 25 feet distance from tunnel) | ⅛-inch | ⅛-inch |
| Tunnel roof settlement | ½-inch | ¾-inch |
| Horizontal movement of tunnel sidewalls | ⅓-inch | ½-inch |
| Maximum utility settlement and slope of settlement trough | 1-0-inch 1/250 | 1½-inch 1/200 |
| Maximum bridge foundation settlement | ¼-inch | ½-inch |
| Maximum surface settlement trough of road surfaces | 1-0-inch | 1½-inch |



(a)  (b)
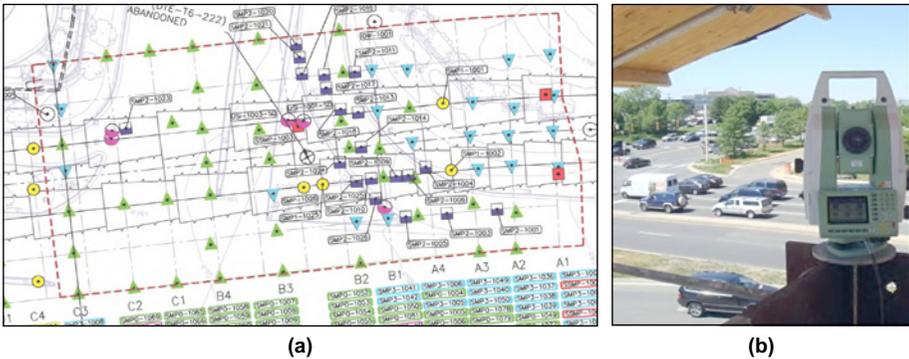
Figure 12. (a) View of the "IMZ" (dashed line) and the surface settlement monitoring points including the Virtual Points (green triangles) and (b) Total Station Theodolite at Tysons Corner

the Total Station Method which involves the use of a robotic theodolite equipped with a Direct Reflection (DR) Electronic Distance Meter (EDM) (Figure 12b). The theodolite is able to locate "virtual points" on the road surface, which are x- and y-coordinates defined in the system, and measure the z-coordinate. The recorded z-coordinates are then compared with the pre-construction baseline z-coordinates to determine settlement. Using the Total Station method allows the input of as many virtual points needed as is shown by the high density of virtual points on International Drive (Figure 12a).

In addition to the Total Station Method, the monitoring program also employed monitoring points requiring physical measurements using measuring rods and conventional optical methods. These additional monitoring points included Shallow Subsurface Monitoring Points (SSMP) for vertical deformation at a depth of approximately 8 feet, Utility Settlement Indicators (USI) for vertical deformations directly above utilities, Inclinometers (IC) near sensitive structures such as the Marriott Parking Garage and the Route 123 overpass bridge piers, and crack gages in the Marriott Parking Garage and Route 123 overpass bridge piers. Nine observation wells (OW) were also installed along the tunnel alignment to monitor groundwater elevation.

Deformation monitoring within the tunnels involved the installation of convergence bolt arrays (Figure 13) every 30 feet for a total of 40 convergence monitoring cross
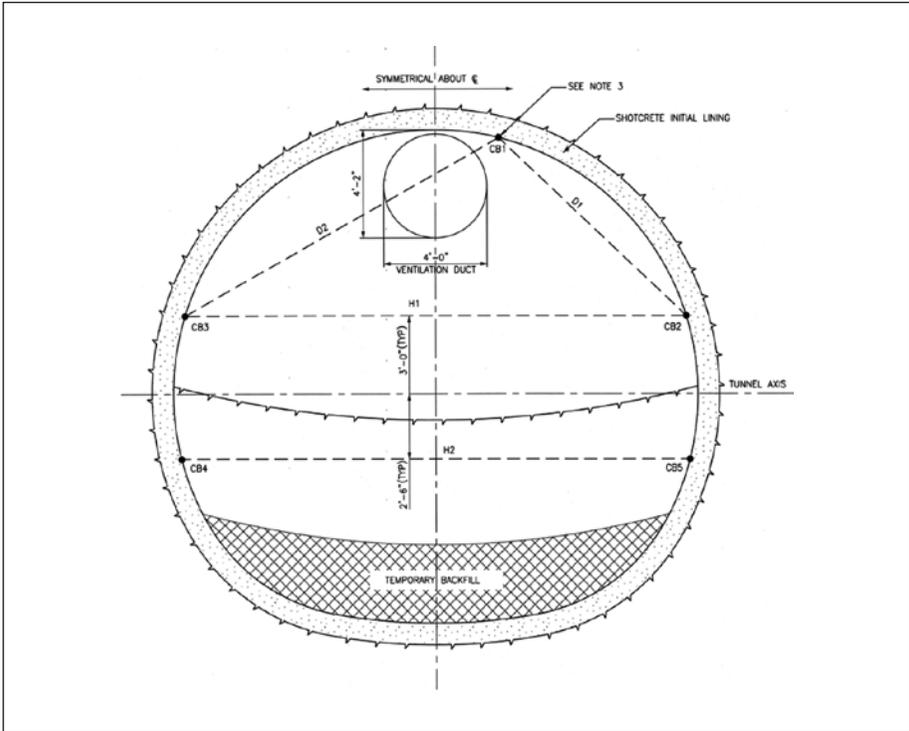
**Figure 13. Typical convergence monitoring cross section**

sections per tunnel. Each array consisted of 5 convergence bolts (CB) with each CB consisting of a rod embedded in the shotcrete initial lining and a target (Figure 13). The arrays were monitored once a day.

## AS-BUILT SURFACE SETTLEMENTS

"Real Time" monitoring of the IMZ using the Total Station method produced an extensive amount of surface settlement data. This data is invaluable in producing as-built surface settlement plots to evaluate the affect of tunneling on the sensitive road facilities and utilities. In this instance the surface settlement data has been plotted in relation to the completion of sawteeth within and slightly beyond the approximately 300 foot IMZ. Figure 14 displays the final surface settlement results of the IMZ.

Evaluation of the final data reveals no surface settlements surpassing the maximum threshold values outlined in Table 1. During tunneling the construction team held daily RESS meetings to evaluate the day's convergence and surface settlement values among others. The owner/client and permitting agencies received an open invitation to attend these meetings and the monitoring data was consistently posted in an online web portal accessible by the owner and permitting agencies.

A critical element not clearly reflected in the surface settlement contour plots is the maximum slopes resulting from the settlements. With a maximum allowable slope of 1/200, it was necessary to verify that the as-built slope values did not surpass this limiting value. A series of cross sections depicting the surface settlements were created to
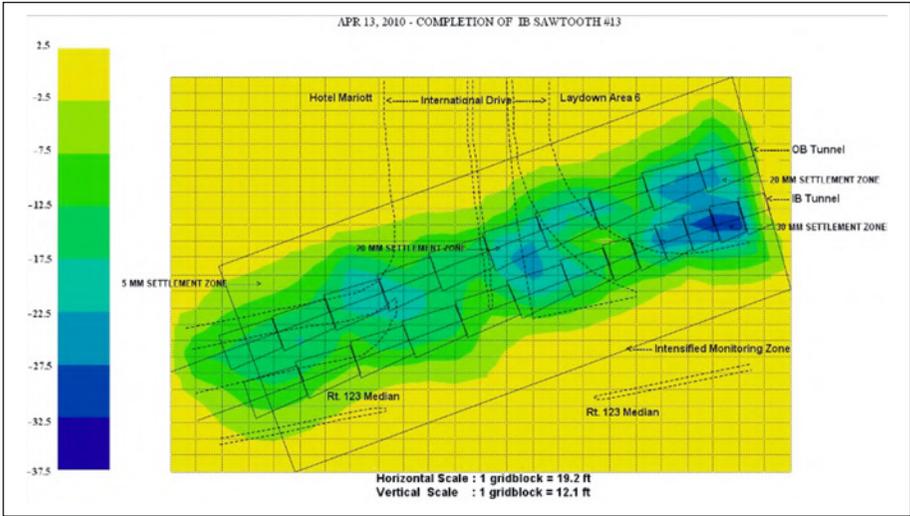
**Figure 14. Final surface settlement contour plot (values in mm) for intensified monitoring zone and beyond**
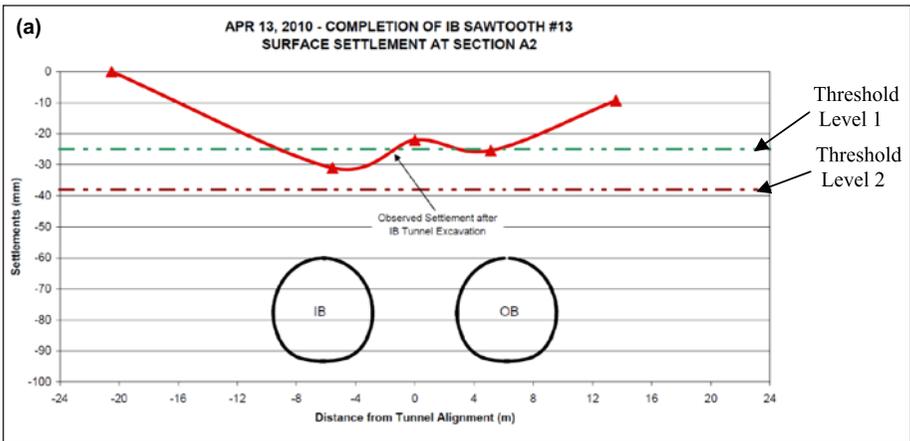


**Figure 15. (a–d) As-built surface settlements and slopes along tunnel alignment. See Figure 12a for location of arrays A2, B1, B3, and C3. (Figure continues)**

allow a determination of the as-built settlement slopes. Evaluation of the cross sections revealed a maximum observed slope of 1/300, below the "Level 1" threshold value of 1/250. As such, excavation proceeded successfully at shallow depths without negatively impacting the existing road structures and utilities. Development of the surface settlement troughs along the tunnel alignment can be seen in Figure 15a–d.

## FINAL CONCRETE LINING STRIPPING STRENGTH

With excavation completed for both tunnels, construction has begun to shift into waterproofing and installation of the concrete final lining as of January 2011. During planning for the concrete final lining and evaluation of the construction schedule, the construction team began to evaluate methods to optimize the installation process. This
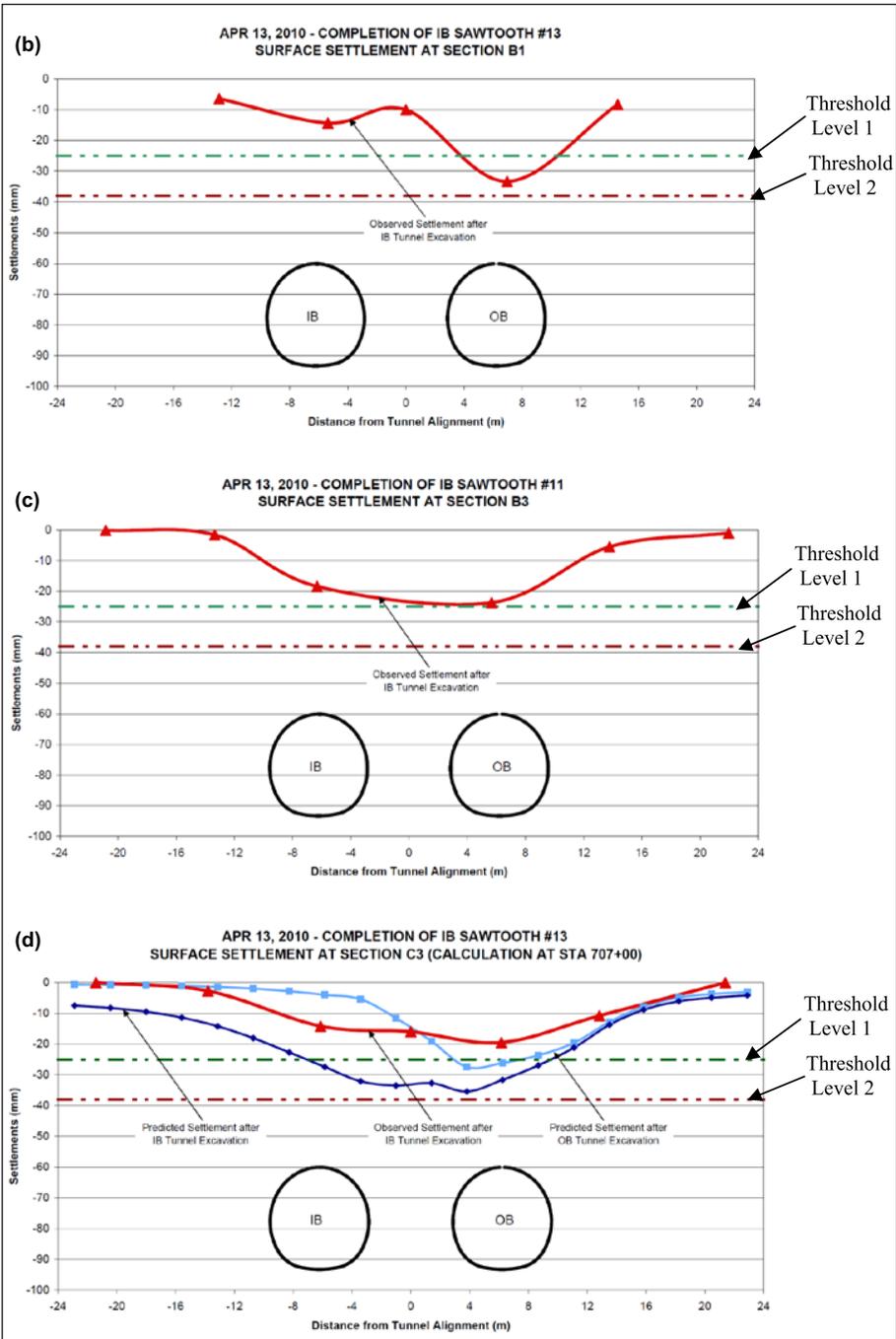
Figure 15. (a-d) As-built surface settlements and slopes along tunnel alignment. See Figure 12a for location of arrays A2, B1, B3, and C3. (continued)
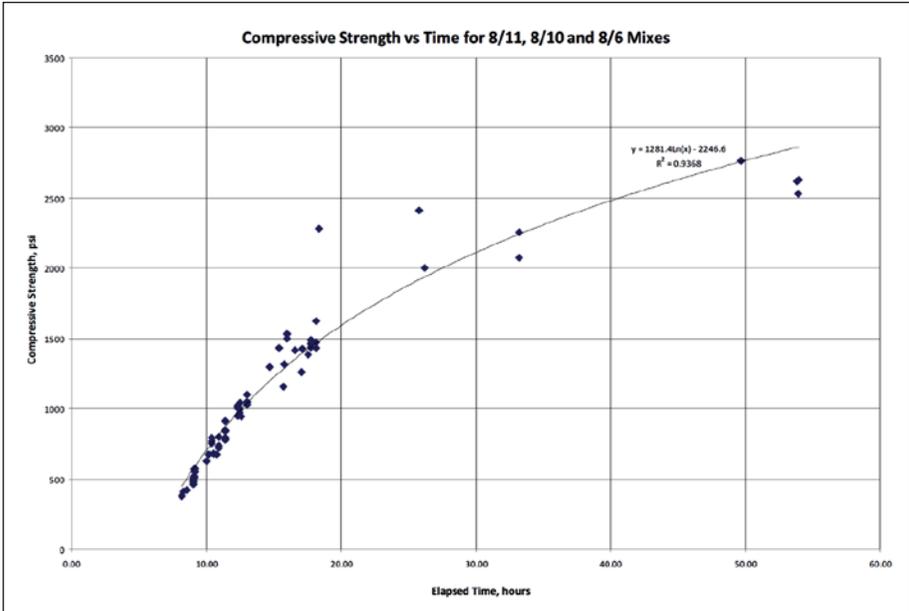
**Figure 16. Compressive strength development of concrete mix design**

resulted in a re-evaluation of the concrete formwork stripping criteria as established in the project specifications.

The DCMP specifications for the tunnels were developed based upon WMATA standard specifications. The minimum strength of concrete for stripping of formwork was established as 35% of the specified design strength which is 4,000 psi for the cast-in-place concrete final tunnel lining. For the tunnels this meant the concrete had to attain a strength of 1,400 psi before the formwork could be stripped. In order to maintain the project construction schedule, the Design-Build team proposed stripping at an earlier strength; first 500 psi and then 750 psi. This proposal was based on experience at similar European projects. MWAA and WMATA were generally open to DTP's proposal but required sufficient technical supporting data.

The DTP Construction and Engineering teams worked to collect case history data for domestic and European projects that stripped at such strengths and evaluated the concrete stripping standards for Austria, Germany, and Japan. The standards for all three countries supported the DTP proposed stripping criteria with stripping strengths generally ranging from 290 to about 450 psi.

In addition to researching supporting case histories and standards, DTP Engineering developed a 2D structural analysis to analyze the final lining forces and deflections. The concrete mix design was developed to achieve a minimum strength of 750 psi after 12 hours (Figure 16). The models were built using Modulus of Elasticity and Poisson's Ratio data from the project concrete mix design developed specifically for the tunnel CIP lining and assumed concrete thickness's of 36" (maximum thickness in sawteeth backfill areas—most conservative load) and 12 inches (minimum design thickness).

Analysis results revealed a maximum displacement of 0.17 mm for 750 psi concrete and 0.13 mm for 1,400 psi concrete (Figure 17). The resulting lining forces for the 750 psi concrete were also checked per ACI 318—M-N Interaction Chart. The results from both the 2D analysis and M-N Interaction Chart supported the DTP proposal and
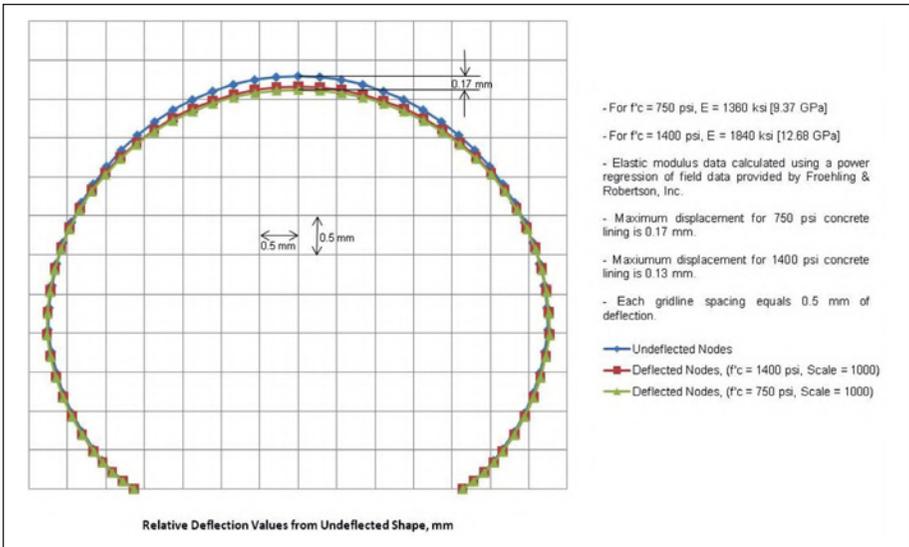
**Figure 17. Relative deflection of 1,400 psi and 750 psi concrete from an undeflected shape**

were assembled with the previously collected case history and concrete standard research to present to MWAA and WMATA. The technical argument presented by DTP convinced MWAA and WMATA to reconsider and allow a reduction in stripping strength. WMATA and MWAA agreed to reduced stripping requirements and allow use of 900 psi after a minimum of 12 hours.

## CONCLUSIONS

Successful tunneling through very shallow soft ground in an urban environment can be achieved efficiently and safely using the Conventional Tunneling Method. The success will depend on careful consideration of all conditions such as structures, utilities, ground conditions, and groundwater, and ensure that a robust design properly reflects these conditions. A properly designed pre-support system was the key to mitigating potential risks during excavation and is critical to ensure minimal surface settlements that could damage structures. Careful consideration of site conditions and implementation of a steel grouted pipe arch canopy pre-support system at Tysons Corner allowed excavation to proceed with minimal surface settlements and no damage to roads, structures, or utilities.

## ACKNOWLEDGMENTS

## BIBLIOGRAPHY

Darmody, J. 1991. *Geology and Construction of Fort Totten Station Washington D.C.—A Case History of NATM in Soft Ground Mining*. Department of Design and Construction—Geotechnical Branch Report. Washington, D.C.: Washington Metropolitan Area Transit Authority.

Federal Highway Administration (FHWA). 2009. *Technical Manual for Design and Construction of Road Tunnels—Civil Element*, FHWA Publication No. FHWA-NHI-09-010. Washington, D.C.: FHWA.

Gall, V. and N. Munfah. 2010. Design Guidelines for Sequential Excavations Method (SEM) Practices for Road Tunnels in the United States. In *Proceedings of North American Tunneling 2010,* Portland, OR, June 20–23. Littleton, CO: SME.

Rein, L. 2009. 18 Feet Done, Many More to Go. *Washington Post*. October 18.

Rudolf, J., and V. Gall. 2007. The Dulles Corridor Metrorail Project—Extension to Dulles International Airport and its Tunneling Aspects. In *Rapid Excavation and Tunneling Conference Proceedings,* Toronto, ON, Canada, June 10–13. Littleton, Colorado: SME.

Rudolf, J., V. Gall, and A. Nitschke. 2008. Selection of Alignment and Tunneling Methods in Urban Settings. In *Proceedings of the World Tunnel Congress 2008—Underground Facilities for Better Environment and Safety,* Agra, India, September 22–24. New Delhi, India: Central Board of Irrigation & Power.

Rudolf, J., V. Gall, and T. O'Brien. 2010. Past and Present Soft Ground NATM for Tunnel and Shaft Construction for the Washington, D.C. Metro. In *Proceedings of North American Tunneling 2010,* Portland, OR, June 20–23. Littleton, CO: SME.