Quality and Risk Management Considerations for Very Shallow Soft Ground Conventional Tunnelling in Urban Settings

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Summary

Construction of shallow mined tunnels in urban environments is frequently undertaken by use of conventional tunnelling methods. A successful shallow urban tunnel will rely largely on the use of pre-support measures and ground improvement to limit the deformations of sensitive facilities and underground utilities. Accurate information regarding utilities, sensitive structures, and ground conditions is essential to successful risk management on a project. It is important to assess all risks during early stages of the project so that they may be properly addressed in the design, instrumentation and monitoring plans and Quality Assurance/Quality Control program. An equally important key element of risk mitigation is assembling a skilled and experienced tunnelling crew that will be able to execute the project meticulously according to design and effectively manage any unanticipated events.

The currently ongoing Dulles Corridor Metrorail Project (DCMP) involves tunnelling for twin single-track, 520 meter long metrorail tunnels in Tysons Corner, Virginia at depths as shallow as 2.5 meters. The twin tunnels have been successfully excavated in an urban environment with the assistance of a robust design that included pre-support measures in the form of grouted steel pipe arch canopies. Strict quality control measures during construction as well as having experienced tunnelling personnel have allowed successful excavation of both tunnels. This paper addresses quality and risk management considerations implemented on the Dulles Corridor Metrorail Project. It reports on the experiences of installing pre-support pipe arch canopies when manoeuvring very close to utilities under shallow cover, quality assurance methods for the shotcrete initial lining, and additional risk control measures such as an extensive instrumentation and monitoring plan.

Keywords: Risk management, quality control, quality assurance, Conventional tunnelling, pipe arch canopy, urban tunnelling, shallow cover

1. Project Information

1.1 Project Description and Approach

Dulles Transit Partners (DTP), a joint venture of Bechtel and URS, the design-build contractor for the Dulles Corridor Metrorail Project Phase 1, commenced with the construction activities of two mined tunnels in Tysons Corner, Virginia in October 2009. The 520 meter long, 6.7 meter diameter twin tunnels are an integral part of the Metropolitan Washington Airports Authority's (MWAA) 37 km metrorail extension into Loudoun County, Virginia, which will include a station at Dulles International Airport (IAD). DTP selected the Conventional Tunnelling Method to construct the tunnels beneath the congested urban business corridor of Tysons Corner (see Fig. 1).

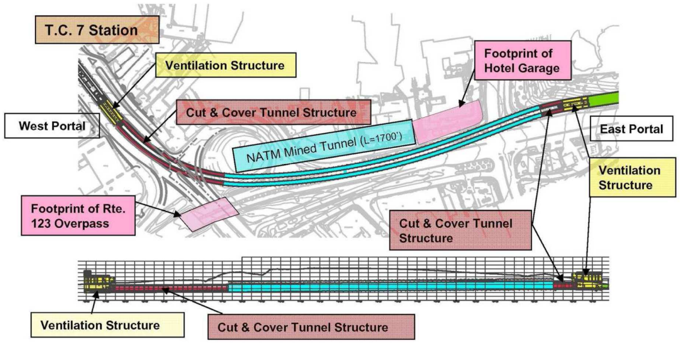


Fig. 1 Conventional Tunnel Alignment ("NATM Mined Tunnel")

The tunnels are being constructed in soft ground and are located adjacent to existing structures and utilities that are sensitive to ground movements as well as a six-lane divided highway which is located generally about 4.6 meters above the tunnel crowns. Due to these surface and sub-surface installations, ground displacements must be kept at strict pre-defined maximum levels.

In order to achieve a consistently high level of quality and mitigate tunnelling associated risks during all phases of the construction process, the DTP tunnel team established and implemented a project quality and risk management system. This system, in accordance with which all tunnel construction activities are performed, includes Inspection and Test Plans (ITPs) as well as an elaborate instrumentation and monitoring program customized to meet the quality requirements specified in the design and aimed at reducing risk during construction.

At the end of October 2010 the outbound tunnel holed through at the west portal ahead of schedule and the inbound tunnel is scheduled to hole through in November 2010 thus completing excavation and initial support of both tunnels.

1.2 Geologic Conditions

The project is situated within the Piedmont Physiographic Province, just west of the boundary with the Coastal Plain Physiographic Province, which is known as the "Fall Line". The Piedmont is characterized by deep residual soils, which overlay metamorphic bedrock. The soils are typically fine sandy silts, clays and silty fine sands and often displayed the preservation of relict foliation and joint structures during excavation. Decomposed rock, a soil-like material with higher strength, was encountered only to a limited extent and bedrock was not encountered at all.

The hill through which the tunnels were excavated happens to be the highest point in Tysons Corner and is capped by ancient Coastal Plain material; mainly unconsolidated layers of sandy silt, cobbles and gravels, and thick bands of clay, which sit unconformably on Piedmont residual soil. The Coastal Plain material was encountered in the top heading of both tunnels over the course of the first about one hundred meters of excavation, eventually rising above the tunnel crown as the tunnel overburden increased. The Piedmont residual soil was the primary material encountered during excavation thereafter. The ground water was generally at invert level at portal locations and rose to just above tunnel spring line at the mid-point of the tunnel alignment.

1.3 Tunnel Design and Construction

1.3.1 Pre-Support

Because of the shallow depth, the prevailing soft ground conditions, the need to control settlements, and risk mitigation issues, the tunnel design implemented a steel grouted pipe arch canopy for the entire length of the tunnels. The 114 mm diameter steel grouted pipes are installed at 30 cm center-to-center around the tunnel crown. A double row of steel pipes was installed for the first 100 meters of both tunnels at the east portal where tunneling is at its shallowest depth. Sensitive road structures (International Drive, International Drive Ramp) remained active and completely unobstructed during construction. Where the overburden is greater and surface structures are less sensitive a single row pipe arch canopy was employed.

1.3.2 Excavation and Support

Tunnel excavation proceeded according to a typical excavation sequence in soft ground conventional tunnelling with two top heading rounds of 0.9 m followed by a 1.8 m bench/invert round as shown in Fig. 2. Initial support consists of steel lattice girders installed after every excavation round and 250 millimetres thick steel fibre reinforced shotcrete (SFRS).

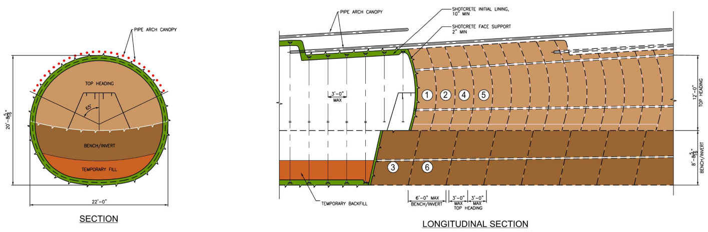


Fig. 2 Typical Excavation and Support Sequence

Upon completion of excavation a smoothing shotcrete layer will be sprayed followed by installation of the flexible PVC, “full-round” waterproofing membrane. The final lining is 300 millimetre thick, generally unreinforced cast-in-place concrete. Steel reinforcement is only employed in the arch concrete for approximately 152 m from the East Portal and 33 m from the West Portal due to sensitive road structures and the shallow nature of the tunnels at these locations.

2. Risk Management

Risk management began during preliminary engineering with the identification of risks. Identified risks were addressed at an early stage in the design with proper pre-support, a staged excavation, and robust initial support. This was supplemented during construction with extensive geotechnical instrumentation and monitoring as well as a strong Quality Assurance/Quality Control (QA/QC) plan.

Just as important as the identification of risks early in the design are the personnel chosen to implement the design, monitoring, and QA/QC plan. The design-builder specified minimum requirements for the tunnelling and QA/QC personnel, requiring extensive experience with conventional soft ground tunnelling. It is critical to have experienced personnel in the tunnels during construction to recognize any potential risks during excavation and act quickly to mitigate those issues by mainly implementing contingency measures specified in the contingency plan.

2.1 Identifying Risks at the Preliminary Engineering Stage

The tunnels pass beneath the intersection of International Drive and Chain Bridge Road with shallow cover ranging between 1.5 and 4.2 metres as well as in close proximity to a high concentration of existing utility lines and infrastructure including a Marriott hotel underground parking garage and Route 123 Overpass bridge piers (see Fig.1 and Fig. 3). A number of utilities were abandoned prior to the start of construction including gas lines, power lines, and communication lines; however, an equal number remained active. These active utilities made the first 100 meters of tunnelling even more critical, as the shallow overburden placed utilities in close proximity to the tunnel pre-support. Knowledge of the location and depths of active utilities was essential when designing the pre-support to avoid any conflict.

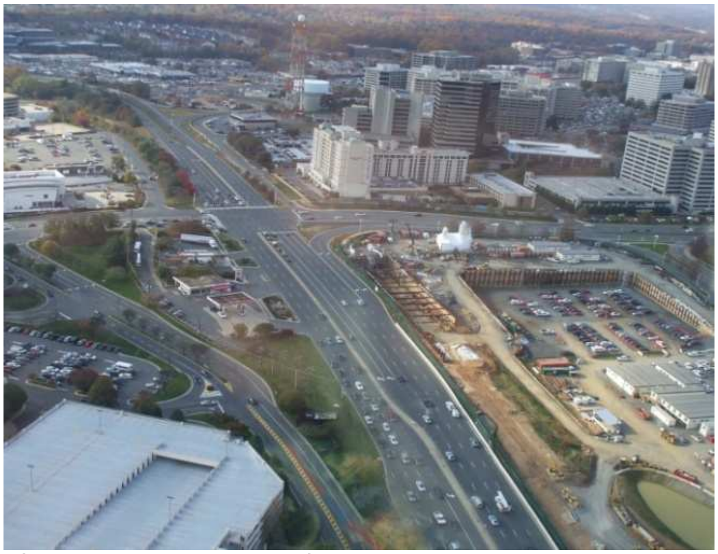


Fig. 3 Tysons Corner project area

DTP conducted an extensive geotechnical investigation of the project site during the preliminary engineering stage to accurately assess the ground conditions and potential risks they may have posed during tunnel construction. Geotechnical risks that were identified include:

- Excessive ground surface settlement induced by underground excavation on the active roadways, buildings and utilities
- Tunnel wall and crown instability during excavation due to unforeseen ground conditions and sudden changes in strata
- Influence of groundwater on face stability
- Seasonal ground water fluctuation delaying the excavation process, in particular the invert ring closure

2.2 Risk Management

2.2.1 Pre-Support

Due to prevailing soft ground conditions, pipe arch canopies were selected as an effective pre-support measure for the tunnels to assist with face stability and minimize ground deformations from the tunnel excavation.

The pre-support consists of a series of 114 millimetre diameter, 18 metre long grouted pipes, installed at a radial spacing of 300 millimetres centre-to-centre around the tunnel crown and shoulder. The canopy pipes are installed by conventional drilling and subsequently grouted to fill the annular void between the perforated steel pipes and surrounding ground. This method is designed to create an arching effect around the tunnel opening during excavation.

There are two basic types of canopies used in this project; the 27-pipe single row canopy, and a 55-pipe double row canopy. Double row canopies are installed for the first 100 metres from the east portal in the section of tunnel with the shallowest cover (see Fig. 4), and single row canopies over the remaining length of both tunnels (see Fig. 5).

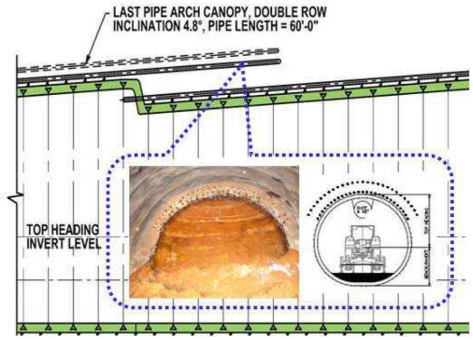


Fig. 4 Double Pipe Arch Canopy

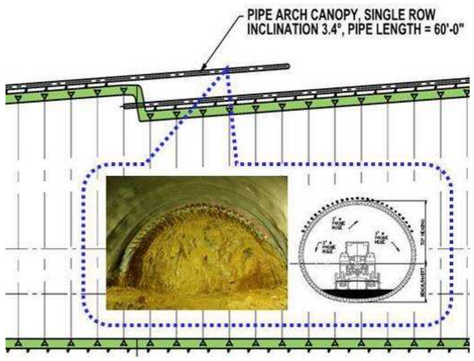


Fig. 5 Single Row Pipe Arch Canopy

Using the appropriate drilling rig can achieve precise direction control and installation accuracy. To ensure an efficient installation of the pre-support system, the following quality assurance measures are implemented during the installation of the pipe arch canopy:

1. Setting-out drilling points prior to drilling operation
2. Drilling the canopy pipes using laser alignment controls mounted on the drilling boom to ensure the proper alignment of the canopy pipes
3. As-built alignment check on the installed pipes before injecting grout
4. Canopy pipe grouting using two termination criteria: achieving a maximum cement-grout volume and/or a maximum applied pressure

In some instances, steel reinforcing bars were installed as pre-spiles (25 millimetre diameter) to compensate for any misaligned canopy pipes that led to too large spacing between the pipes. This measure helped to prevent ground ravelling and also limit over-excavation. Additionally, a standby dry shotcrete spray unit is kept on hand as a backup to the primary shotcreting equipment.

2.2.2 Groundwater Control

Due to the location of the groundwater table, local face instability problems have been experienced immediately after sealing the excavated face, in some cases due to pre-existing slickensided features and water seepage ingress. Drainage measures quickly followed to ensure face stabilization. Per design, local sump pumps were installed along the tunnels in order to release the pore water pressure ahead of the tunnel face (see Fig. 6). Drawing down the groundwater was beneficial to tunnel face stability, particularly during the bench/invert excavation.

Fig. 6 Sump pumps installed in invert for dewatering

In addition to pump sumps, five probe holes measuring 19 metres long are drilled through the shotcrete sealing layer prior to the beginning of excavation for each new 13 metres of tunnel assuring knowledge of ground and ground water conditions a minimum of 6 meters ahead of the tunnel face. The probe holes aid in draining the groundwater.

2.2.3 Staged Excavation and Initial Support

To ensure the proper excavation sequence and shotcrete lining installation Required Excavation & Support Sheets (RESS) are jointly prepared by DTP construction management and the design team on a daily basis. These forms guide all tunnelling activities and are the result of daily discussions and frequent evaluations of the ground behaviour observed during construction.

In the event that surface settlements or tunnel convergence should reach and subsequently exceed pre-defined limits a contingency plan was created prior to the start of construction. The contingency plan presented measures to be implemented in the event that established in-tunnel convergence and/or surface deformation thresholds were surpassed. Additionally, a phone tree of critical personnel to contact in the event of excessive deformations or other problems was included. However, the contingency plan also presented alternative methods of excavation and additional support measures that could be implemented at the request of the Senior Tunnel Engineer. An example, shown in Fig. 7, is a half-face excavation, implemented in particularly weak or jointed ground to limit the size of exposed soil.

Fig. 7 Face Partitioning to limit exposure of weaker ground material

2.3 Construction Inspection and Material Quality Checks

Quality checks generally include ITPs, which describe the inspection points, inspection methods, extent of inspection, criteria for acceptance, and specific construction stage. There are quality control engineers assigned to each 12-hour shift to perform around-the-clock construction inspections.

Routine inspection checks are performed in the following order to ensure quality control:

1. Excavation profile check: during top heading and bench/invert excavations
2. Exposed pipe-arch canopy check: during top heading excavation
3. Steel lattice girder positioning: both heading & bench/invert
4. Girder bracing reinforcement: during installation of lattice girders
5. Shotcrete spread and temperature checks: before and during installation
6. Sprayed shotcrete thickness check by survey instrument: after spraying shotcrete

Prior to installation of the steel lattice girders, the excavation profile is checked to identify areas of under/over excavation. The fabricated shape of the lattice girders and the location of the pipe arch canopy aid in maintaining an accurate tunnel profile during excavation and the precise installation of these elements is critical (see Fig. 8). Surveyors guide the lattice girder installation crew with lasers to pin point the placement and maintain the proper profile. After the bench/invert excavations are complete, the final excavation profile is checked by installing the two invert girders. Limiting over-excavation saves time spent on each round as well as construction material.


Fig. 8 Positioning the steel lattice girders in top heading and invert

Shotcrete spreads are checked to be within the acceptable range of 550 millimetres to 600 millimetres with temperatures between 50° F and 90° F. Finally, the shotcrete lining profile is checked and evaluated relative to the specifications and the associated ITPs. A series of strength tests are conducted to guarantee minimum shotcrete strength per specifications.

Early shotcrete strength is determined by the Penetration Needle Test (Fig. 9) which is conducted between 3 and 30 minutes after spraying. The early strength is then tested again using the Hilti Stud Driving Test between 3 and 9 hours after spraying (see Fig. 10).

Fig. 9 Penetration Needle Test

Fig. 10 Hilti Stud Driving Test

Cores are taken from the initial lining, as shown in Fig. 11, for determination of the compressive strength of the in-situ shotcrete. The cores are used to determine compressive strength of the shotcrete after: 24 hours, 7 Days and 28 days.

Fig. 11 Shotcrete strength test cores

The shotcrete strength results are plotted using a Shotcrete Strength Development Curve. The shotcrete strength development is carefully monitored from the early shotcrete strength to 28 days final strength. The test results of the shotcrete strength development curve shown in Fig. 12 where the shotcrete strength development is related to the J-curves per Austrian shotcrete standards [1].

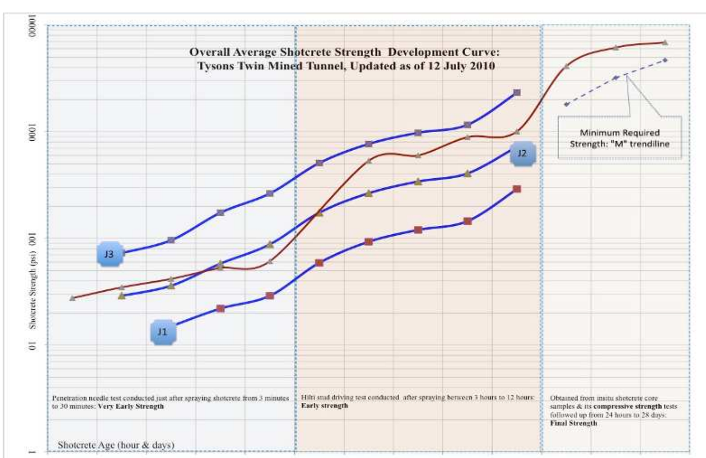


Fig. 12 Shotcrete Strength Gain Curve

2.4 Geotechnical Instrumentation and Monitoring

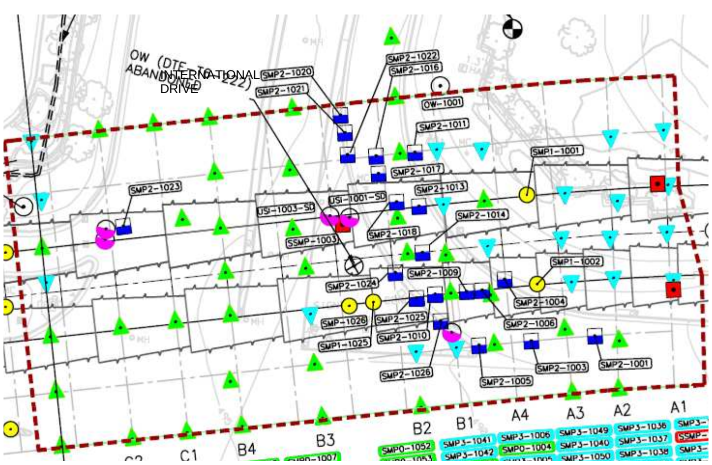
One of the areas identified during the preliminary engineering risk assessment was the 100 meters in which the tunnels pass beneath International Drive at shallow depths; as little as 1.5 meters of overburden at one point. The shallow overburden concerned Virginia Department of Transportation (VDOT), the owner of the public traffic facilities at Tysons Corner, which requested “Real-time” monitoring of the surface when tunneling the 90 m under the road, which was subsequently labeled the “Intensified Monitoring Zone” or “IMZ” (Figure 13a). An agreement was reached that the “Real-time” monitoring will entail taking measurements of surface points every hour, which were then plotted automatically in graphs and placed onto a website accessible to the Owner and VDOT.

To accomplish the “Real-time” monitoring, DTP employed the Total Station Method which involves the use of a robotic theodolite equipped with a Direct Reflection (DR) Electronic Distance Meter (EDM) (Figure 13b). The theodolite is able to measure “virtual points” on the road surface which are x- and y-coordinates defined in the system. The theodolite locates the defined points automatically and measures the z-coordinate or vertical deformation. The measured z-coordinates are then compared with the initial 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 13a).

In addition to the Total Station Method, the monitoring program also employed monitoring points that require physical measurements using measuring rods and conventional optical methods. The monitoring plan also included the use of Shallow Subsurface Monitoring Points (SSMP) for vertical deformation at a depth of about 2.5 m, 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 installed along the tunnel alignment to monitor groundwater elevation.

Deformation monitoring within the tunnels included the installation of convergence bolt arrays every 10 m totaling 40 convergence monitoring cross sections per tunnel, IB and OB. Each array consisted of 5 convergence bolts (CB) with each CB consisting of a rod embedded in the shotcrete initial lining and a target. Measurement of the arrays provides horizontal (x), vertical (y) and longitudinal (z) movements.

This extensive monitoring including the “Real time” monitoring of International Drive was critical, not only to mitigating any risks before they evolved, but also in alleviating concerns by the Owner and other entities such as VDOT.



a)

 b)

Fig. 13 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

3. Conclusion

The Tysons Corner tunnels have demonstrated the successful use of conventional tunnelling in combination with a systematic pre-support system in limiting ground settlements in urban soft ground tunnelling. These achievements would not be possible without understanding and addressing the geotechnical conditions observed and the implications of tunnelling under shallow cover that this project has presented. Additionally, various complex geotechnical instrumentation and monitoring methods can be used to accurately track construction-induced ground movements on existing surface structures and underground utilities, allowing for immediate mitigation of risks. Anticipating and managing the potential project risks from the onset of the project has been imperative in limiting risks during the construction process.

4. References

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