HISTORY AND RECENT DEVELOPMENTS IN SOFT GROUND NATM TUNNELING FOR THE WASHINGTON, DC METRO

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INTRODUCTION

The first application of soft ground NATM in the US was on Washington, DC Metro's Section E5 Fort Totten Station and tunnels. Due to its success it was subsequently applied to the tunneling under Rock Creek cemetery, Section E4b, and a few years later on Section F6b on the Outer Branch Avenue south of Congress Heights Station. The application and its successes laid the ground work for the method's increased use on many urban soft ground tunnels in US cities. Soft ground NATM tunneling will be used again for the extension of Washington's Metro to Washington Dulles International Airport.

In this paper the authors provide detailed case history summaries and describe developments and refinements of the NATM in Washington ranging from technical to contractual to new risk management aspects. Many innovations that led to the method's refinements and its widespread use in the US were pioneered on the Washington, D.C. Metropolitan Area Transit Authority (WMATA) Metro system and during its continued use on Metro, namely for the Dulles Corridor Metrorail project at Tysons Corner, Virginia, where soft ground NATM construction will start in mid 2009.

FORT TOTTEN STATION AND RUNNING TUNNELS, SECTION E5

Project Description

Section E5 of the Washington Metro at Fort Totten Station in Northeast Washington, D.C. on the Green Line represents the first Washington Metro station and running tunnels to be constructed using the soft ground NATM, the first soft ground NATM in the US (Heflin et al., 1991). The underground structures of the project consisted of a station chamber, a vent shaft, twin tunnels, and a fan shaft. The station chamber measures approximately 19.7 m in width by 9.3 m in height by 91.5 m in length (Donde and Wagner, 1992). The twin NATM tunnels are approximately 305 m in length (Jenny et al., 1989). Overburden ranges from approximately 6 m at the least over the station to as much as 30.5 m over the tunnels. The section designer was Sheladia Associates of Maryland and the tunnel designers were Jenny Engineering of New Jersey and Mayreder-Consult of Austria. The construction contract was awarded in August 1988 to a joint venture between Mergentime Corporation, Flemington, New Jersey, and HT Construction, the U.S. subsidiary of Hochtief AG, Essen, Germany. The contractors had the option to submit bids based upon conventional construction methods, NATM, or a combination of both. The contractor chose the NATM option with Hochtief providing specialized equipment as well as management personnel with prior NATM experience.

Geologic and Hydrologic Conditions

Washington, D.C. is located on the boundary between Virginia's Piedmont and Coastal Plain physiographic provinces. This boundary is commonly referred to as the "Fall Line." Section E5 is located east of the "Fall Line" within the Coast Plain province. The station and tunnels were constructed within the lowest of the Coastal Plain sediments identified collectively as the Potomac Group. More precisely, the Section E5 structures were constructed within the Patuxent Formation, the lowest member of the Potomac Group. The Patuxent Formation is predominately unconsolidated sand and gravel with lesser amounts of clay, and its sediments are overconsolidated and contain some irregular beds of iron-oxide cemented sand or gravel. The Potomac group sediments at section E5 were categorized into two primary groups: "P1" clays and "P2" sands following the general and system-wide nomenclature established by WMATA's General Soils Consultant.

The ventilation and fan shafts were excavated within interlayered sands and clays while the two single track tunnels were excavated primarily through cross-bedded, silty or clayey, fine to medium sands. The station was excavated within loose to compact interlayered sand and gravels, stiff clay, and clean to silty sand in the eastern half and interlayered clay and sandy clay in the western half. See Figure 1 for the geologic profile longitudinal section and cross section from the center drift of the station as obtained from borings and field mapping.

Piezometer information obtained prior to construction indicated that the static groundwater table was generally below invert elevation of the NATM structures and during construction was only intercepted when excavating the fan shaft invert. Due to the lenticular nature of the Patuxent sediments subsurface water inflows and perched ground water were more of a concern during construction. The presence of perched ground water made it complicated to simply drain the face, as it is difficult to ensure that drain holes penetrate perched basins. Prior to the start of excavation, the contractor drilled two 24 m long, 152 mm horizontal drains at the portal wall on either side of the center drift. During the following month of excavation, over 750,000 liters of water were drained from the two holes. The initial high flow rates were attributed to the drainage of a large perched groundwater basin. The subsequent flow rate of 2.8 liters per minute from one of the holes for the remainder of the station excavation was attributed to groundwater recharge for the basin (Darmody, 1991).

NATM Station Tunneling

The original design foresaw the use of horizontal jet grouting above the entire crown of the station and for a length of about 20 meters starting from the portal wall. The purpose was to pre-stabilize the ground and therefore its stand-up time and control deformations in this first very shallow tunnel section at the portal. However, the contractor proposed changing this approach to eliminate the jet grouting described in the contract documents. After many iterations and discussions with WMATA the jet grout approach was changed to the use of a combination of interlocking metal sheeting forepoling installed by pneumatic hammers with micro fine cement grouting in that first section of the station tunnels.

The station excavation sequence was broken down into multiple drifts. The center drift was the first to be excavated, with excavation proceeding in 0.9 m increments westward from the portal wall. The top heading was excavated in its entirety, followed by the bench. Upon completion of the center drift, station excavation activities paused for approximately 4 months while a cast-in-place concrete center drift support frame consisting of an invert beam, nine columns, and a roof beam was constructed (Figure 2).
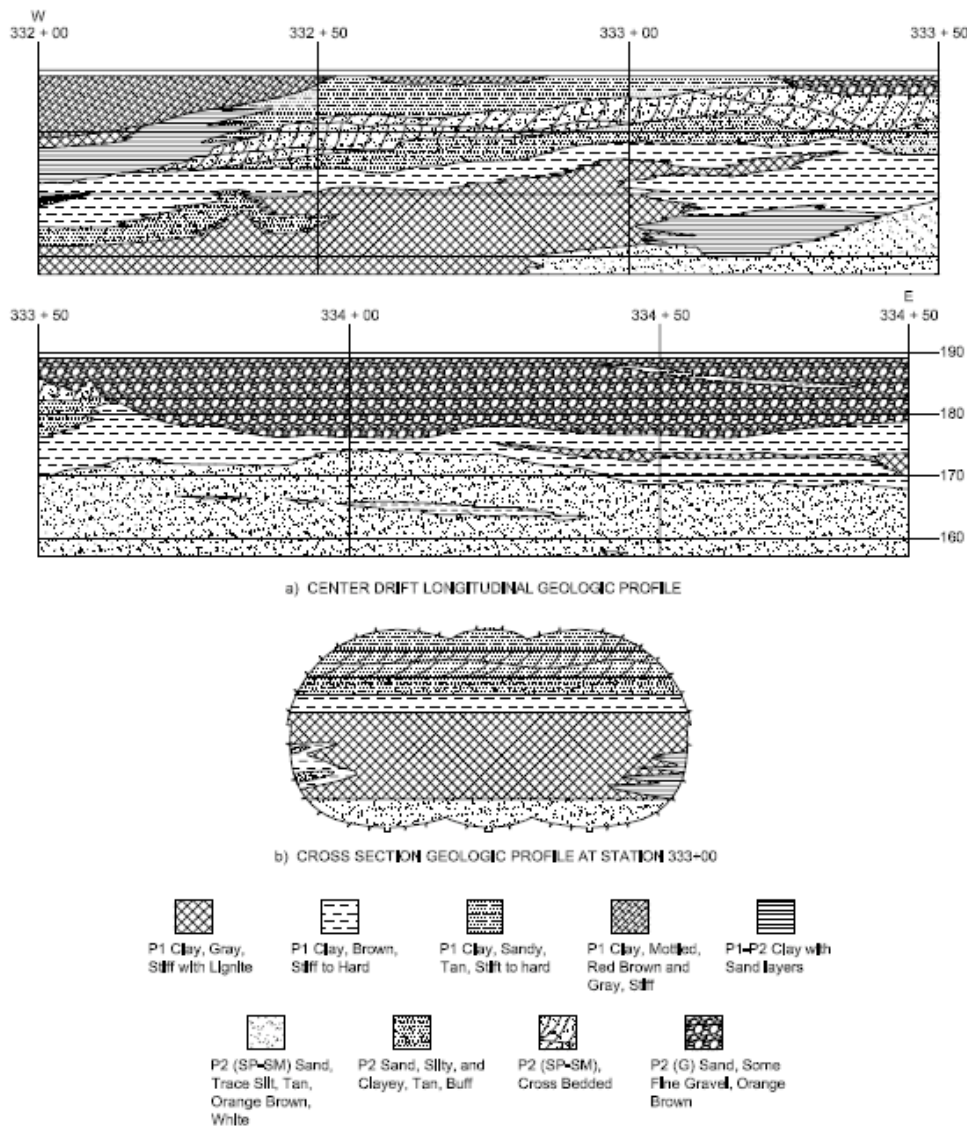
a) CENTER DRIFT LONGITUDINAL GEOLOGIC PROFILE

b) CROSS SECTION GEOLOGIC PROFILE AT STATION 333+00

P1 Clay, Gray, Stiff with Lignite

P1 Clay, Brown, Stiff to Hard

P1 Clay, Sandy, Tan, Stiff to hard

P1 Clay, Mottled, Red Brown and Gray, Stiff

P1-P2 Clay with Sand layers

P2 (SP-SM) Sand, Trace Silt, Tan, Orange Brown, White

P2 Sand, Silty, and Clayey, Tan, Buff

P2 (SP-SM), Cross Bedded

P2 (G) Sand, Some Fine Gravel, Orange Brown

Figure 1 – Longitudinal Geologic Profile and Cross Section Geologic profile from Station Center Drift

Once the center drift and its support frame were completed, the contractor proceeded with simultaneous excavation of the IB and OB side drifts in 0.9 m increments, alternating between the top heading and bench. Once the side drifts had advanced 15 m, the contractor proceeded with excavation of the (inbound) IB and (outbound) OB middle drifts. However, ultimately, the contractor elected to complete the side drifts before proceeding further with the middle drift excavation. See Figure 2 for the excavation sequence of the NATM Station. Excavation of the twin running tunnels proceeded in 0.9 m increments as well, with excavation alternating between top heading, bench, and invert (Wagner et al., 1991).

The standard NATM initial support for each excavation round consisted of the application of a sealing layer of shotcrete to the excavated ground, followed by installation of a lattice girder, installation of the first layer of welded wire fabric (WWF), shotcrete to the outer edge of the lattice girder, excavation and shotcrete a temporary invert, installation of a second layer of WWF, and finally the application of a second layer of shotcrete.
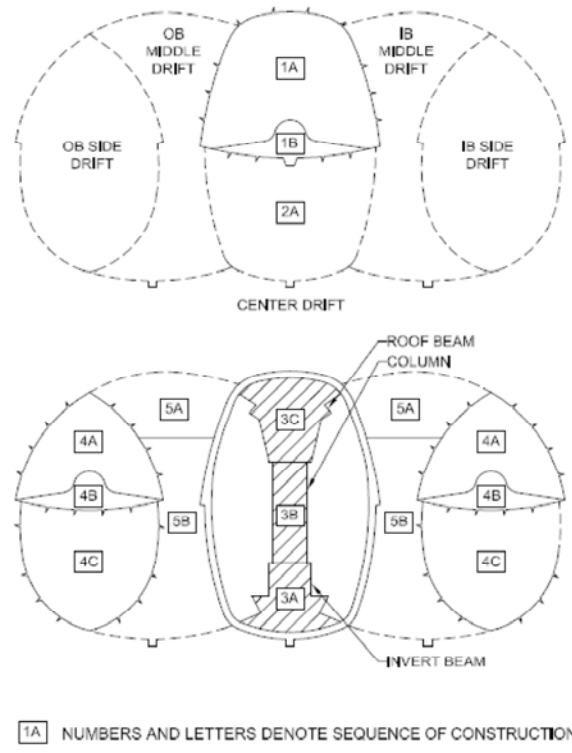
Figure 2 – Excavation and Support of NATM Station

The occasional running or raveling ground encountered during excavation was successfully controlled using pre-support means as local measures, however, only with one significant instance of ground loss. In this instance some 38 cubic meters of sand fell from a small hole in the IB tunnel crown. The resulting surface settlement was abrupt, but minor, with deformations of less than 0.5 mm. The ground loss area was stabilized with additional shotcrete and positively drained by installing drain pipes. Once excavation had proceeded 4.5 m, the contractor grouted the area of ground loss with water-cement grout to stabilize the ground further in the immediate area. All other instances of ground loss were the result of small lenses of running or raveling ground being encountered within the P2 material. Overlapping forepoling sheets were effectively used to control raveling ground during excavation. Perched ground water, while occasionally encountered when excavation breached thin layers of water bearing sands and gravels, was never a major issue and could be controlled by local dewatering and drainage means.

Upon completion of the initial lining for all structures, the station, twin tunnels, and shafts were waterproofed using a full round or "tanked" PVC waterproofing membrane. Subsequently the cast-in-place concrete final lining was placed.

Instrumentation and Monitoring
The instrumentation and monitoring program for the project consisted of 168 pressure cells, 6 deflectometers, 276 convergence points, 15 micrometers, 10 borehole extensometers, 3 inclinometers, and 129 settlement/level points. With this number of instruments, it was estimated that data collection would require approximately 2,000 measurements per week. Ultimately, many of the instruments were read far less often than specified in the contract, but overall, data collection was performed with sufficient frequency to monitor lining stresses and displacements such as to obtain accurate and adequate real-time information.

An evaluation of the pressure cell readings showed the soil pressure values against the initial lining and within the shotcrete were well below its designed capacity. Shotcrete stress values were less than a third of what was anticipated.

The overall range of convergence was 10-30 mm for the station and 5-10 mm for the tunnels. These convergence values translated to surface settlements ranging from 10-20 mm according to the sliding micrometers.

TUNNELING UNDER ROCK CREEK CEMETERY, SECTION E4B

Project Description

Section E4b was constructed as part of WMATA's $642 million Mid City E-Route that provided a connection between the U Street/Shaw Station and the Fort Totten Station in northeast Washington, D.C. Construction began in the mid 1990's and included two cut-and-cover stations, 4.7 kilometers of single-track tunnels constructed mainly by TBMs and a series of ventilation and emergency access and egress shafts. Section E4b adjoined the running tunnels of Section E5, which were constructed several years earlier by NATM. The Farragut shaft, part of Section E4b, was the only shaft constructed by NATM. The excavation of the shaft encountered serious water flows through the walls at lower depth and required immediate on site engineering to adapt NATM support measures to conditions encountered. These consisted mainly of metal sheeting driven under an angle into the wall followed by immediate application of shotcrete. From the Farragut shaft, which adjoined Section E5 and the Buchanan shaft located further south, the tunnels had to underpass the historic Rock Creek Cemetery, the oldest cemetery in Washington, D.C. The tunnel alignment passed beneath the cemetery compound at a depth of about 15-25 meters and early on the cemetery stated that any tunneling approach under the cemetery would have to be carried out without any disruption to the cemetery property and tunneling would have to be without any "discernible settlements." To maintain these requirements WMATA led an investigation into a series of tunneling and ground conditioning options that were investigated by WMATA's Board of Engineering Consultants (BOEC, a selected group of US tunneling/geotechnical experts, including Professor Peck, D. Deere, A. Chase, and J. Gould) and Design Engineers. With initial hesitation mainly due to the fact that soft ground NATM was still a relatively new technique in the US at that time, an engineering recommendation was made to apply NATM with the aid of ground improvement and pre-support measures to suit local conditions. At the end it was concluded that only the NATM with such additional measures would be able to meet the requirement for close to negligible surface settlements, thus satisfying the restrictions imposed by the cemetery. Tunneling surface settlements were estimated to be less that 1 cm and were projected to spread over a wide surface settlement trough.

Tunneling

The two single-track tunnels, separated by about one tunnel diameter pillar width were each approximately 940 meters in total length. A classical soft ground NATM excavation and support sequence was prescribed with top heading and bench excavation sequencing in which the top heading advance was limited to a maximum of 1 meter per round. The bench followed at a distance of 2 meters (Figure 3). The initial shotcrete lining was 18 cm thick and reinforced with welded wire fabric. The tunnel was waterproofed using a PVC continuous membrane and an unreinforced cast-in-place 30 cm thick concrete was installed as the final lining (Irshad, 1995).

Depending on geologic conditions, pre-support in the crown involved pipe spiling, forepoling sheets, and chemical grouting. Forepoling sheets were used to control soils involving running ground conditions, possibly including soft silts and silty sands. Vacuum lances and ultimately local dewatering sumps were used to control water when encountered in the invert and bench. Pipe spiling was used as needed in stiffer silts. Chemical-grout was specified in portions of the alignment where silty sands with occasional pockets or zones of clean sand and perched water were anticipated within the P2 materials. Installation of a chemically improved soil arch above the tunnel crown was carried out using the horizontal directional drilling technique, a novelty for tunnel pre-support installation at that time.

Figure 3 - Section E4b – NATM Tunneling under Rock Creek Cemetery (Courtesy Paul Madsen, Kiewit Construction)

Grouted Pre-Support Arch – Horizontal Directional Drilling and Chemical Grouting
Chemical grouting was required above both the inbound and outbound NATM tunnel crowns for about 100 meters starting from the Farragut shafts towards Ft. Totten Station, from Farragut shaft for about 250 meters towards the Buchanan shaft and from the Buchanan shaft toward the Farragut shaft for about 160 meters, for a total length of about 2 x 510 meters. The original design foresaw installation of the improved ground by chemical grouting using 24 m long tube-a-manchette (TAM) pipes installed by conventional drilling methods. The grouted arch would be accomplished by 9 pipes installed around the tunnel crown in a two-row array covering an arc of 120 degrees. Pipes would be installed every 12 meters to assure a sufficient overlap between pipe installation sections. The grout injection would have been carried out after pipe installation causing a standstill at the tunnel heading for a significant time period. The grouting subcontractor, Hayward Baker, Inc., proposed a horizontal directional drilling (HDD) program for the installation of steel TAM pipes to allow unimpeded access to the heading for continuous mining, better control of the grout injections by minimizing the number of faces exposed to the grouting process. HDD is a technique that uses guiding or steerable drill bits to install a drill pipe along a horizontal trajectory combined with in-hole navigation. The disadvantage of the HDD program is its high drilling costs that may be up to 12 times higher than conventional drilling (Blakita, 1995). Also, grouting of longer TAMs may involve a more difficult grouting process causing a potential delay in the start-up of tunneling. However after weighing schedule and cost advantages/disadvantages of HDD vs. conventional drilling the general contractor, a joint venture of Kiewit/Kenny and Hayward Baker, Inc., proposed and WMATA, led by the main author who was the reviewing engineer, accepted the HDD installation of TAMs for chemical grouting.

The installation of the TAMs involved the steering of an 88.9 mm diameter "inner string" to total depth, following the inner string with a 114 mm inside-diameter wash over casing and extracting the inner string. Subsequently the sleeved port grout pipe was placed inside the wash over casing, and the wash over casing was withdrawn while a weak cement grout was pumped through the wash over casing into the annulus and soil. The inner string was steered with a slant-face steering shoe as commonly used in directional drilling applications, such as utility river crossings. The borehole gyroscope verified the trajectory of the drill string at 20-30 m intervals, thus the location of the drilled hole was known prior to installing the TAM pipe. A gyroscope uses the inertial reference plane created by a rotating sphere to measure azimuth change. The measurement of azimuth is used to calculate left/right offset from the initial baseline. In addition, the gyroscope uses a pendulum to measure inclination changes to calculate the elevation from the starting point. Using HDD and period in-hole surveys the PVC pipes could be placed to follow the tunnel alignment on a radial curve of up to 250 meters. As per contract requirements, the grout holes were directionally drilled horizontally above the tunnel in a nine-pipe, two-row array to achieve a canopy of stabilized soil above the tunnel crown as per contract intent. The lower row of five pipes was placed 0.6 m outside the tunnel cutline. The upper row of four pipes was placed 1.4 m outside the tunnel theoretical excavation line at radial intermediate points to the lower row. All pipes were installed parallel to the tunnel centerline following both the horizontal and vertical planned curvature of the tunnel. Placement of the pipes targeted to maintain a 0.6 m placement window for each pipe along the alignment.

Chemical grout consisted of a sodium-silicate-type grout mixture of 50% liquid sodium silicate, 6% organic reactant, and 44% water was used. For reference this material provided a minimum unconfined compressive strength of 0.69 MPa at 28 days in Ottawa 20-30 sand. Typical pumping rates were in the range of 25-55 L/min. Grout injection occurred through the individual sleeve ports until the quantity of grout reached a predetermined value or when the in-hole flow rate decreased to 1 L/min at the maximum allowable injection pressure. The grout manifold allowed simultaneous and independent injection of all nine grout pipes.

To isolate the injection ports double-ended pneumatic packers were used in the TAM pipes. These provided the ability to re-inject sleeves that quality-assurance testing indicated had been insufficiently grouted. Such testing involved both pre- and post treatment permeability testing at intervals throughout the process.

A total of more than 8,000 m of TAM pipes was installed either from within the shafts or from grouting chambers created in the tunnel crown near the shaft (Figure 4). HDD use enabled NATM tunneling in silty and clean sands on Section E4b without the need for stoppages during tunneling. The HDD process is not limited to chemical grouting, but can also be used for compensation grouting, installation of drainage for dewatering or hydrostatic pressure relieve prior to tunneling.

In summary, use of the HDD pipe arch for chemical grouting and standard pre-support measures, including spiling and sheeting in combination with NATM sequencing and early support installation achieved project objectives and guaranteed the tunneling performance with "no discernible" surface settlements measured within the cemetery.

PRE-SUPPORT OPTIMIZATION, SECTION F6B
Project Description
Section F6b included the construction of Congress Heights Station using cut-and-cover techniques and two 460 m long, single track tunnels. This section is part of the Branch Avenue Route and is situated in southeastern Washington, D.C. (Meyers Böhlke et al., 1996) NATM was selected for tunnel construction because of (1) the short length of tunnels, (2) short mobilization duration, and (3) the ability to mine from multiple headings. A few years earlier NATM had been successfully applied at Fort Totten Station (Section E5a) and the Metro tunnels under the Rock Creek Cemetery (Section E4b). It was anticipated that with rapid mobilization and the use of multiple headings to

advance through a relative short length and comparatively homogeneous geology, the NATM tunnels could be completed during the early part of the overall project construction. The Contract for Section F6b was awarded to a joint venture of Clark/Shea with J.F. Shea being responsible for NATM tunneling which started in October 1996 and the "hole through" in the IB tunnel completing both tunnel drives was made in September 1997.

Figure 4 - Section E4b – Horizontal Directional Drilling (HDD) from within Drilling Chamber (Courtesy Paul Madsen, Kiewit Construction)

A number of pre-support measures were used to stabilize the tunnel crown in areas where saturated sand layers were known to occur and in adjacent sections where the thickness, extent and stability of firm clays over the tunnel were uncertain. These measures included pre-support grouting using directional drilling, grouted pipe spiling, and rebar spiling, in combination with dewatering in advance of the face. These measures were required to increase the stand up time and cut off groundwater. Pre-support chemical grouting of the northernmost 70 m was required to consolidate a saturated sand layer in the crown to enable successful NATM tunneling.

Per WMATA design criteria the tunnel cross section was designed as a dual lining structure comprising a shotcrete lining and a cast-in-place unreinforced concrete lining. The shotcrete lining was reinforced with welded wire fabric, lattice girders, splice bars, and splice clips. This tunnel was waterproofed by a flexible plastic membrane around its entire circumference.

The tunneling of the outbound tunnel occurred as specified in the contract documents. During mining of the inbound tunnel, however, interception of 4 large unmapped sand lenses in the crown resulted in unexpected inflow of groundwater and work stoppages. With chemical grout injection

primarily from inside the tunnel and the installation of drain pipes ahead of the face, the contractor was able to resume safe mining below a hydrostatic pressure of up to 8 m (Gall et al., 1998).

Geologic Conditions and Pre-Support Methods

Based on exploratory borings along the NATM tunnels, the first approximately 70 meters of the tunnel beginning at the station in the north were mapped as a thick sequence of firm over consolidated (P1) clays with up to 1.5 m of saturated sands (P2) in and above the tunnel crown (Figure 5). The presence of the P2 sand layer dictated the use of pre-support methods to stabilize the crown and prevent the inflow of flowing sands and uncontrolled loss of ground. Directional drilling, pipe spiling, and grout injection were specified to assist in stabilizing the crown of the tunnel through this particular reach. Beyond the first 70 m south of Congress Heights Station, the predominant soil type at the face and above the tunnel was anticipated as a thick sequence of over-consolidated Cretaceous P1 clays with occasional sand lenses.
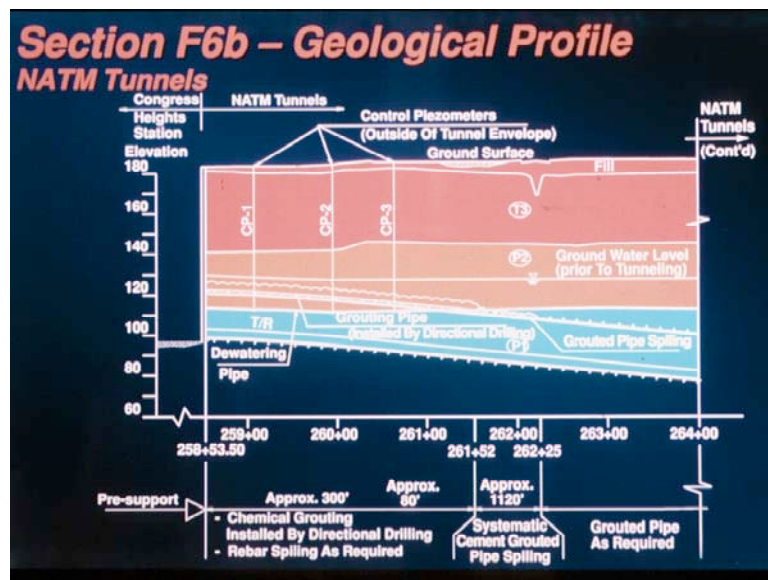

Figure 5 - Section F6b – Geological Profile (Schematic)

Based on ground conditions the main types of pre-support anticipated to be used in construction of F6b included rebar spiling, grouted pipe spiling, and installation of a chemical grouting canopy.

Rebar spiling consists of steel rebar rammed into the ground in soils where a bridging effect can be achieved between closely spaced bars. Rebar spiling was also specified in the area of the chemical grout canopy to limit overbreak in the tunnel arch. Experience during construction of past projects has shown that even when the P2 material is improved by chemical grouting, overbreak in the crown and tunnel shoulders may occur because of inhomogeneities in the ground. To limit such over break No. 8 (25 mm) rebar spiling installed at 0.3 m distances was specified on an as required basis.

Grouted pipe spiling consisted of perforated steel pipes through which grout was injected to stabilize the ground. The pipes were 4.5 m long and had a 3.8 cm inner diameter. They were installed in pre-drilled holes at a look-out angle of 5 degrees and cement grouted in place. These pipe spiles were located above the tunnel crown from approximately 10 to 2 o'clock at a centerline spacing of approximately 0.3 m. These were installed systematically between the ground improved by chemical grouting and to a location where the P1 Clays would extend a minimum of 1.5 m above the tunnel crown.

Installation of the grout canopy was specified to ensure that groundwater is prevented from drainage into the tunnel section, the ground properties are improved, and the steel grouting pipes provide longitudinal reinforcement by forming a supporting arch extending into the P1 clays above the tunnel crown. The chemical grouting canopy was to consist of steel pipes installed into the ground ahead of the tunnel face for a distance of up to 100 m using horizontal directional drilling (HDD). Following placement of the pipes chemical grout was to be injected to stabilize the ground in sandy P2 materials along the perimeter of the tunnel crown.

Directional drilling in the mid to late 1990's was still considered an advanced drilling technique and was employed for installation of facilities or excavation supports where high accuracy of the drilling direction was required. Precise steering of equipment and a continuous check of the drill direction by sophisticated navigation tools maintained the accuracy of up to 0.01%. Directional drilling for the installation of steel tube-a-manchette (TAM) grouting pipes had been successfully applied at WMATA's Section E4b for NATM tunneling beneath Rock Creek Cemetery as previously described. On that section the grouting pipes had been successfully installed up to lengths of approximately 250 m through layers of P2 materials.

Although the directional drilling technique is often more costly compared to conventional drilling it has many other advantages. Installation of TAMs may be carried out in one operation with subsequent grouting from the tunnel portal, thereby eliminating surface disruptions typical of when grouting is injected from the ground surface. It also eliminates stoppages for drilling and grouting operations during tunnel excavation. In particular at F6b the use of directional drilling was specified to avoid any drilling operation from within the tunnel heading that could have potentially penetrated the P2 layer and conduct groundwater into the tunnel.

However, shortly afterward the contractor proposed to install roughly 45 m long drills from within the Congress Heights Station excavation into the arch (and saturated sand layer) above the future tunnel using conventional methods. The remaining length of this canopy was then grouted from within the tunnel during advancing northward from the south using shorter, overlapping 21 m long TAMs. The grouting pattern using 9 TAMs around the tunnel crown perimeter remained unchanged from that specified. Using the staggered pipe installation approach required a minimum 7 m overlap of each length of longitudinally installed TAMs.

Dewatering/Groundwater Control

To dewater the saturated P2 sands in advance of the tunnel heading after installation of the chemical grout arch the contract foresaw drilling of horizontal dewatering pipes to lower the groundwater table for the tunnel construction in the P2 material. The groundwater table was to be lowered to the interface between the P2 and P1 material. To achieve a continuous dewatering system drilling for the horizontal dewatering pipes was also to be carried out using directional drilling. Excavation of the station to the north of the tunnels, also, would help in drawing down some groundwater close to the excavation. To verify groundwater levels prior to and during construction three control piezometers were to be installed along the alignment center line between the two bores (Figure 5).

For the exploration of ground and groundwater conditions ahead of the tunnel face three probe drills were specified to be carried out from within the top heading and at a length of 15 m. A lap of 3 m between probe drills was to be maintained to ensure sufficient length of investigated ground ahead of the face.

Tunneling

The tunnel excavation and support sequence followed basic considerations for soft ground NATM tunneling. The excavation face was subdivided in top heading and bench with excavation rounds of up to 1 m length in the top heading. The typical excavation sequence for tunneling in P1 clays called for a ring closure distance of maximum 5 m. The actual support installation in the top heading is shown in Figure 6. For tunneling beneath the chemically grouted sand canopy the

excavation procedure was modified, calling for an earlier shotcrete ring closure, allowing a maximum distance between advancing top heading and closed shotcrete ring in the invert of only 4 m. Tunnel excavation began in October 1996 from a southern shaft at Mississippi Avenue with two headings being driven simultaneously.

Geologic conditions observed in the outbound tunnel (OB) were in overall agreement with, or better than those established in the geotechnical documents and tunneling proceeded with pre-support as specified or less. During tunneling through the P1 clays peak advance rates were achieved with up to 6 m/24 hours. Peak rates of 5 m/24 hours were achieved when tunneling beneath the chemical grout canopy. Average advance rates achieved for a 24-hr day was 5 m, reaching a maximum of 7 m for a day.

In contrast to the outbound tunnel, the inbound tunnel excavation was affected by occasional, large inflow of water at the face when the heading intercepted unmapped saturated sand layers. Tunnel advance was stopped four different times with groundwater inflow rates of 3.2 to 5.0 liters per second. In those instances tunnel excavation was halted until the water bearing P2 material affecting the tunnel was sufficiently grouted to enable tunneling beneath and through a soil improved ground with additional dewatering through drainage pipes as required. Due to the required use of probe drilling to check for presence of water saturated sand lenses known to be present in the P1 clay material were detected early on. Upon encountering the water, probe drills would be used as drainage pipes and the sand lenses dewatered by gravity flow. Drainage would then be continued until all the water pressure was relieved and additional probe holes verified that the surrounding ground was free of isolated or interconnected saturated sand layers. This exploratory measure provided knowledge of ground conditions ahead of the face, and repeatedly, proved essential to assessing tunnel face stability and tunneling safety.
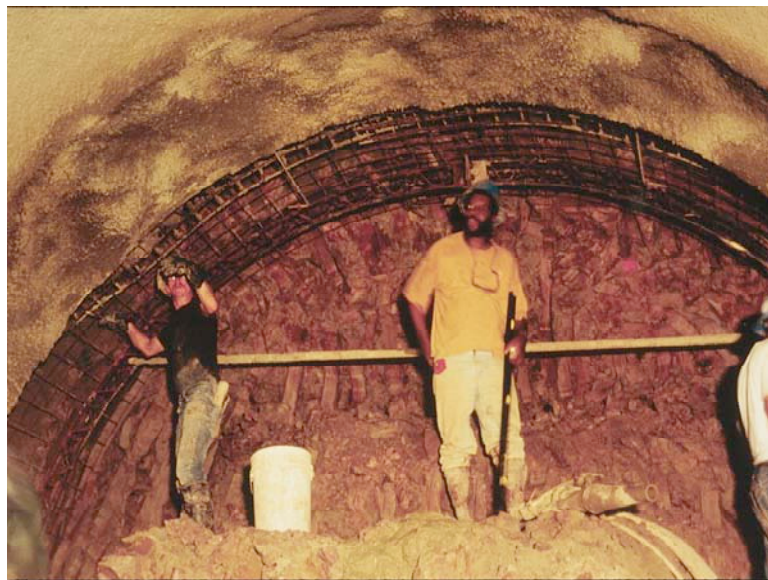
Figure 6 - Section F6b – Top Heading in P1 Clays

In one instance inflows were of such magnitude resulting in ground loss estimated at 50-100 cubic meters which required the use of cement grouting to fill the void created by the loss of ground ahead of the tunnel face. This was achieved by a surface grouting program which led to an interruption of tunneling for about 2 months. Four work stoppages occurred during inbound tunneling and in each case, tunneling was able to resume once the crown was adequately stabilized with the use of pre-support chemical grouting in advance of the face. Average advance rates for a

24 hour day and week were similar to those achieved in the OB tunnel. The maximum one week advance was 29 m advance.

In summary, NATM tunneling was carried successfully out for the first time under a hydrostatic head of up to about 8 m. Through the prescribed systematic and continuous probe drilling ahead of the face, knowledge of ground conditions was gained sufficiently early to halt tunneling within stable conditions and prepare additional ground pre-support measures using chemical and cementitious grouts to improve stand up time of the P2 sands and for ground water cut-off.

TUNNELING AT TYSONS CORNER FOR DULLES METRORAIL
Project Description
The Metropolitan Washington Airports Authority (MWAA) is currently undertaking the first major expansion of the WMATA system that reached a total rail length of 107 miles with the completion of the Largo Line in 2004 (Rudolf and Gall, 2007 and 2008). This current expansion is known as the Dulles Corridor Metrorail Extension Project (DCMP) and will significantly improve the service of WMATA's metrorail system in the Capitol Region in Northern Virginia and will connect the Washington Dulles International Airport (IAD) with Washington D.C. The expansion will add a total length of 37 kilometers of rail. Its first phase is currently being implemented in a design build contract by Dulles Transit Partners (DTP) a joint Venture of Bechtel, Inc. and URS. This Phase 1 will involve a short NATM tunnel as the most feasible tunneling option at Tysons Corner.

The NATM tunnel segment includes twin single-track tunnels at a length of approximately 520 m each. A short cut-and-cover section adjoins the NATM tunnels at the east portal and a longer cut-and-cover section exists at the west portal. These tunnels will be constructed in soft ground and will be located adjacent to existing structures and utilities that are sensitive to ground movements. The alignment and elements of the short tunnels at Tysons Corner are shown in Figure 7. For this very shallow overburden alignment, a busy 4-lane thoroughfare at International Drive is located about only 4.6 m above the tunnel crown. The deepest overburden cover exists at about mid-point of the alignment with nearly 11.6 m. At the west portal and the transition to the cut-and-cover box the overburden is about 6 m.
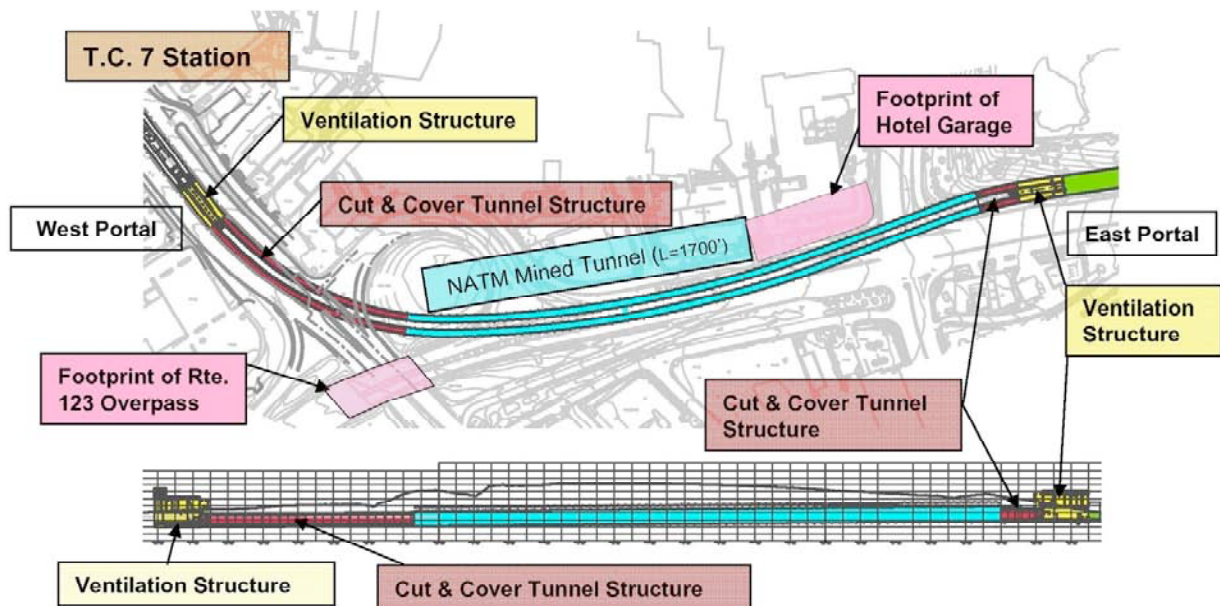

Figure 7 - NATM Tunnel and Tysons Corner Tunnel Alignment

Geologic Conditions

The soils along the tunnel alignment include mainly residual soils and soil-like completely decomposed rock. The residual soils are the result of in-place weathering of the underlying bedrock and are typically fine sandy silts, clays and silty fine sands. According to the project classification, the residual soils are identified as Stratum S, which can be divided into two substrata (S1 and S2) based on the consistency and degree of weathering.

Within the tunnel alignment for low overburden alternatives, the thickness of substratum S1 varies considerably, from 0 – 0.6 m to almost 10 m. The lower substratum, S2, is similar to S1, but typically exhibits higher strength and is made up of more granular particles. Its thickness within the tunnel alignment ranges from 1.2 m to 18 m. Substrata S1 and S2 will be the predominant soil types encountered during tunnel construction, with tunneling mainly near the portals and stratum S2, where the tunnel is located deeper in the mid portion of the alignment. Only to a limited extent where the tunnel is deepest will tunneling encounter decomposed rock referred to as "D1" in bench and invert. The decomposed rock is a soil-like material but has higher strength. Ground water at portal locations is generally at invert elevation, at the mid-point of the tunnel alignment it rises up to the tunnel spring line.

Tunneling

Due to WMATA's long history of tunneling, the agency has developed a comprehensive and detailed set of design criteria that often impose "mandatory" design requirements on the tunnel engineers. The NATM design at Tysons Corner will use a newly developed NATM tunnel cross section that is wider than the classical and previously mandatory WMATA NATM regular cross section. This adaptation was implemented during the final design and review for permitting by the State of Virginia's Department of General Services (DGS) to fully satisfy latest NFPA 130 requirements. DGS is the MWAA assigned Authority Having Jurisdiction (AHJ). As the previous section for soft ground NATM, this section is of dual lining character and features a waterproofing wraparound system but is wider to accommodate fire-life-safety considerations called for by NFPA 130 requirements for walkway width. This new section is displayed in Figure 8 in comparison to the original WMATA Design Standard cross section.
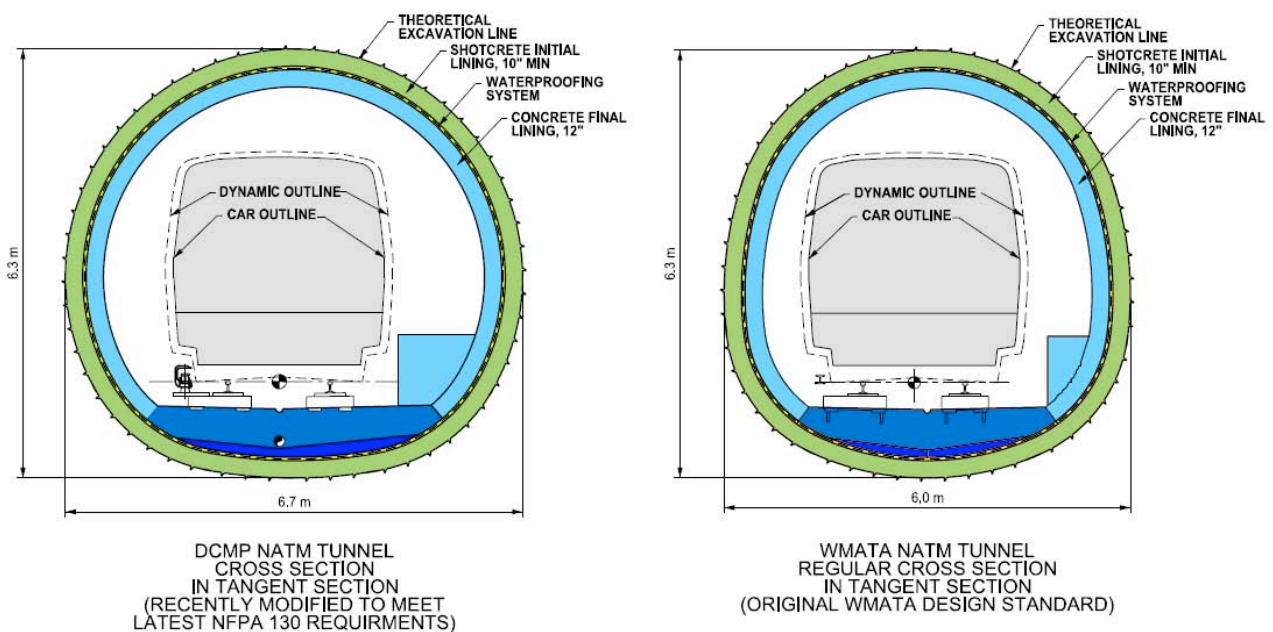

Figure 8 – WMATA NATM Regular Cross Section vs. DCMP Modified Cross Section

Because of the shallow depth, the prevailing soft ground conditions, the need to control settlements, and risk mitigation issues the NATM is supplemented by a grouted pipe arch canopy for the entire length of the tunnels. This will be sufficient for pre-support where the overburden is greater and surface structures are less sensitive. An additional row of pipe arch umbrellas, using closely spaced, approximately 114 mm diameter grouted steel pipes will be used on the first 90 m length at the east portal where tunneling is shallow with 4.6 m overburden. The pipes will be installed at 30 cm center-to-center distances around the tunnel crown.

Figure 9 displays a single row pipe arch umbrella above the tunnel section along with a typical NATM tunnel excavation and support sequence.
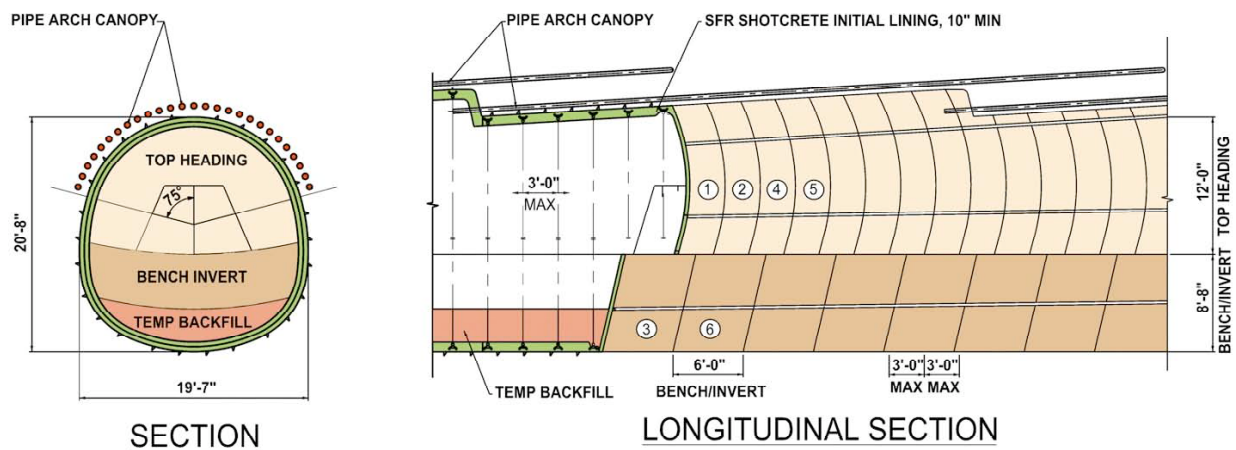

Figure 9 - NATM Tunnel with Single Pipe Arch Pre-support for Shallow Soft Ground Tunneling

CONCLUSIONS

Under WMATA's leadership in design and construction management three NATM soft ground projects were successfully completed on the Metrorail system between the late 1980s and the late 1990s. These projects met challenges of tunneling under shallow overburdens and met the need to limit surface settlements. Often the use of NATM resembled the only feasible tunneling alternative under the given ground, environmental and economic constraints. To allow for controlled and efficient tunneling the projects made use of the latest state-of-art in pre-support and ground improvement practice. Observed success allowed the limits of the NATM to be pushed always a bit further allowing tunneling under a hydrostatic head with only limited grouting and dewatering. The upcoming and most recent soft ground NATM project at Tysons Corner will again build on state-of-the art systematic tunnel pre-support. This pre-support will not only provide a positive control of stand-up time but will also be an integral tool of tunneling risk management and control settlements to permissible levels.

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