MARTA East Line Tunnels under I-285, Atlanta, Georgia

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SYNOPSIS The Metropolitan Atlanta Rapid Transit Authority combined three new technologies-microtunneling, jet grouting, and rock-socketed "minipiles"-to successfully construct twin rail tunnels under an eight-lane highway with as little as 4-1/2 feet of cover. Geotechnical parameters, tunneling method selection, and construction methods are discussed. Ground response and monitoring are summarized.

LOCATION AND DESCRIPTION

The eastward extension of the Metropolitan Atlanta Rapid Transit Authority's (MARTA) East Line rail corridor crossed beneath Atlanta's eight-lane circumferential highway, which carries over a quarter million vehicles daily. The lower limit of the proposed MARTA vertical alignment was constrained on the west by a 100-year flood plain and high ground farther to the west. The upper limit was limited because the alignment had to be compatible with the development of a station approximately 1000 feet to the east of the highway. A relatively sharp curve in the highway to the north of the crossing would make traffic shifts for construction difficult and would require extensive modifications to the highway bridge 400 feet to the north. Therefore, MARTA chose to construct twin tunnels 179 feet long just under the highway. The tunnels had excavated dimensions (including presupport) of approximately 24 feet wide by 25 feet high and a center to center spacing of 40 feet. The tunnels were driven in soil with only 6 to 12 feet of cover.

Significant settlement of the highway was considered intolerable, not only because of the large volume of high-speed traffic, but also because the pavement in the travel lanes was portland cement concrete. The slabs had longitudinal dowels, but no dowels were present from lane to lane, thus "faulting" between lanes could occur. The emergency lanes were asphaltic concrete.

Ground support options proposed by the owner and design team for further consideration were:

1. A multiple pipe arch.
2. A jet grouted arch.
3. A precast box section jacked into place.
4. Ground freezing.

After initial review, preliminary design and the geotechnical exploration were undertaken to evaluate the options and complete the design.

The geotechnical exploration performed from the highway was limited to periods of night-time low traffic volumes. Message boards, extensive lane control and warning lighting were required. In a matter of 6 to 8 hours, traffic controls had to be placed, mobilization, drilling and sampling completed, boreholes grouted and patched, and traffic controls removed.

The exploration confirmed generally the same stratigraphy as had been revealed by preliminary borings on the alignment adjacent to the highway:

1. Compacted fill for the highway embankment.
2. Residual micaceous sand and silt, a product of the inplace weathering of the parent gneisses and schists.
3. A soft rock zone locally referred to as partially weathered rock.
4. Relatively unweathered gneiss and schist.

The geologic profile along the tunnel alignment is illustrated on Figure 1.

During the field exploration, Iowa Borehole shear tests, Handy and Fox (1967) and Ko stepped blade tests Lutenegger and Timian (1986) were performed in the overburden. Conventional thin-walled sampling of soil was also accomplished to obtain samples for laboratory testing.

The ratio of vertical to horizontal stress in-situ was measured by the stepped blade to be approximately 0.75 for the fill and 0.5 for the residual soil.

Laboratory testing and in situ borehole shear testing of the residual soil both indicated an internal friction angle of 28 degrees and essentially no cohesion. In fact, several samples demonstrated a total lack of cohesion by crumbling during removal from the sampling tubes.

Although the overall strength characteristics of the soil were of interest, the deformation characteristics were of primary concern. Field testing, laboratory testing and correlations to pressuremeter modulus values were all used in developing deformation parameters to be used in deflection calculations for the proposed support systems. Table 1 summarizes the geotechnical parameters used in the design.

Experience has demonstrated that the elastic deformations of the soil and supports do not account for the total observed ground movements, Peck (1969). Therefore, consideration was given to settlements observed in similar ground conditions. Furthermore, the "stand up time" was a major concern, because the soil had essentially no cohesion and crumbled readily; the ground was expected to be "slow to fast ravelling" under the Terzaghi classification scheme, Terzaghi (1950).

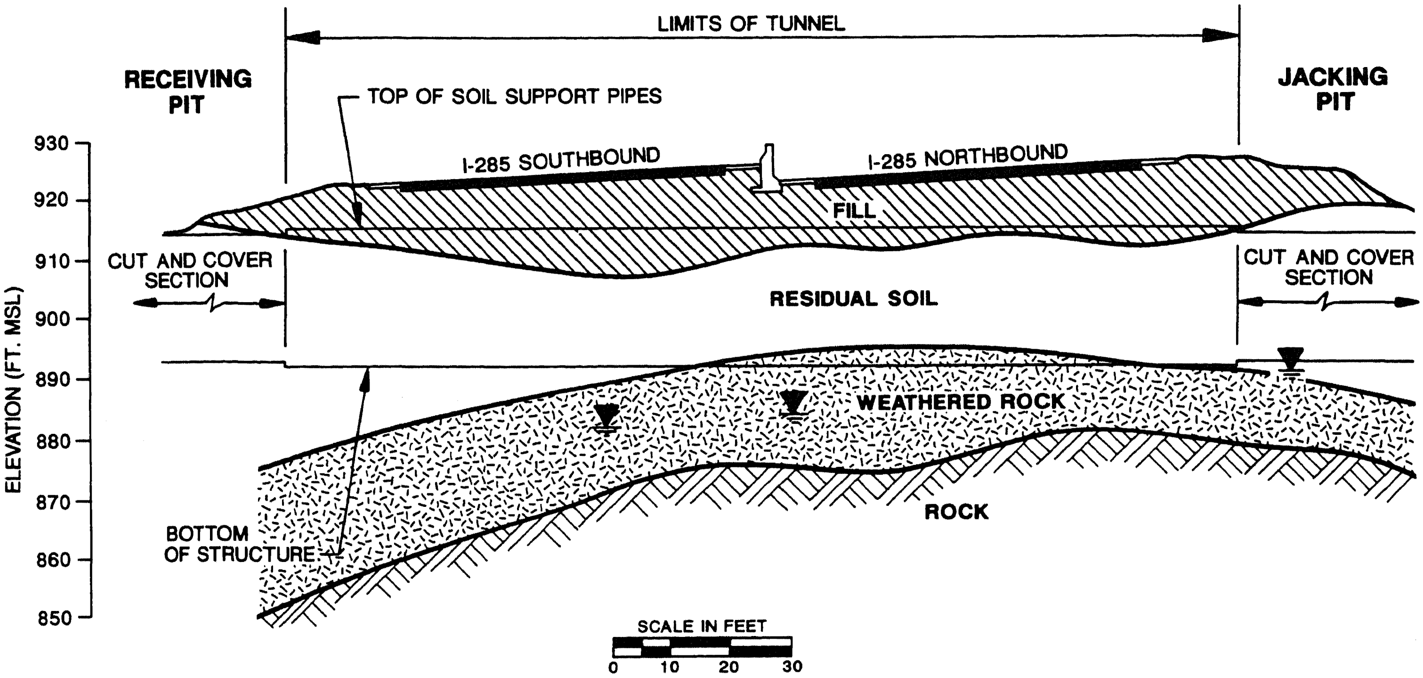


FIGURE 1: GENERALIZED GEOLOGIC PROFILE, VIEW NORTH

TUNNEL DESIGN

Early in the design process, several key considerations emerged as essential for the successful completion of the tunneling with such difficult constraints. Specifically, the tunnel design and construction technique would be required to accomplish the following:

1. Avoid catastrophic ground loss.
2. Avoid excessive deflection of the support system.
3. Minimize settlement due to "lost ground".
4. Be capable of handling obstacles such as boulders in the highway fill, or soft rock near the tunnel invert.

Because of the potential for catastrophic ground loss and immediate stoping to the ground surface, some form of presupport was considered essential. Grouting the soils was ruled out because of limited access to the entire area above and around the tunnels. Grouting was also expected to be relatively ineffective because of the grain size of the soils, Baker (1982). As a part of the evaluation process, the selected tunneling methods were evaluated on the basis of the following criteria:

1. Demonstrated use and availability of equipment.
2. Stability of the crown and face during tunneling.
3. Risks and reliability.
4. Adaptability in dealing with underground obstructions.
5. Ability to control ground movements.

A summary of the evaluation is provided in Table 2. The selected option was the multiple pipe arch, which included the following features (See Figure 2):

1. Presupport by the pipes to avoid catastrophic ground loss.
2. Jet grouting of the soil inside the pipe envelopes.
3. Pipe piles drilled and socketed into rock.
4. Steel sets founded on the pipe piles to support the pipes.

Close cooperation between MARTA, the designer and the geotechnical engineer allowed the development of realistic loadings, expected ground response and the appropriate construction sequencing. The critical construction loading condition was found to be after the face was advanced, and before the next set was placed. In order to limit structural deflections to the desired 1/2-inch maximum, the previously placed set would have to carry the soil and highway load of approximately 22 feet longitudinally along the pipes, even if the face was maintained as steep as 1(H):4(V). This unsupported span is a function primarily of the soil strength and stiffness that can be relied upon to support the pipes beyond the heading.

Assuring reliable ground support was considered essential, and the pipes were expected to provide that. However, minimizing ground surface settlements in such shallow soft ground tunnels was much more difficult.

TABLE 1. GEOTECHNICAL PARAMETERS

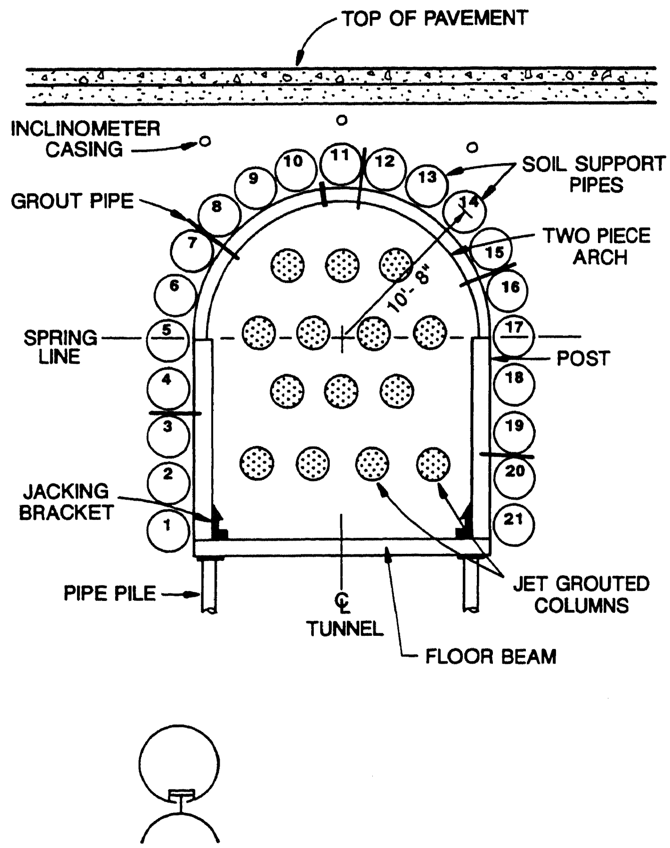
PARAMETER AND VALUE	METHOD OBTAINED	APPLICATION
FRICTION ANGLE AND COHESION		
Fill: 25°, C=0	In Situ Borehole Shear Shear Testing (See Note 1)	Mohr Coulomb Strength Envelope; Stability Analysis; Earth Pressure
Residuum: 29°, C=0		
Fill: 29°, C=0	Laboratory Triaxial Testing	
Residuum: 29°, C=0		
HORIZONTAL STRESS IN SITU		
Fill: 3.6-17 psi	In Situ Ko Stepped Blade	Compute Loads on Lining
Residuum: 5-20 psi		
HORIZONTAL TO VERTICAL STRESS RATIO		
Fill: 0.75	Computed From Horizontal Stress And Soil Unit Weight Data	Compute Loads on Lining
Residuum: 0.5		
GRAIN SIZE DISTRIBUTION		
Predominantly fine sand and silt	Sieve Analysis; Hydrometer	Grouting Feasibility; Ground Freezing Feasibility
SATURATION		
Fill: 85%	Computed From Unit Weight, Moisture Content	Ground Freezing Feasibility
Residuum: 40%		
MODULUS OF ELASTICITY		
Fill: 600 psi	Laboratory Triaxial Testing	Soil Structure Interaction
Residuum: 1000 psi		
MODULUS OF SUBGRADE REACTION (See Note 2)		
Fill: 300 psi	In Situ Ko Stepped Blade	Soil Structure Interaction
Residuum: 400 psi		
Fill: 300 psi	Computed From Laboratory Triaxial Test Data (See Note 3)	
Residuum: 450 psi		

NOTES:
1. These values of strength are for 95% confidence limit from statistical analysis.
2. Not a property of the soil; depends on the load and area.
3. From a method suggested by Vesic (1961).

Ground surface settlements associated with soft ground tunneling result primarily from two causes:

1. Elastic deformations of the supporting elements under applied loads.

2. Ground movements that occur before the supports can be placed and act effectively.



TYPICAL "T" INTERLOCK DETAIL AT ADJACENT PIPES

FIGURE 2: CROSS SECTION OF TUNNEL

Of these two causes, the first is typically much smaller and more readily predictable. Therefore, much effort during the design process dealt with expected ground response as the excavation proceeded and the desired sequence of placement of the support components.

A maximum deflection of the roadway of 1/2-inch had been set. This limit is much smaller than settlements observed in tunnels at greater depths where soil arching could be relied upon. The selected components were intended to minimize ground loss and surface settlement as described in the following paragraphs.

The soil support pipes were to be installed by microtunneling. This would provide the most reliable alignment of the pipes, which would facilitate the installation of other components, and also minimize ground loss often experienced with boring and jacking techniques. However, the microtunneled pipes alone were not expected, or intended, to provide an effective arch. Therefore, steel sets were designed as part of the support system.

Critical loading for the strucutral deflections was dependent on the unsupported length of soil support pipes in the crown. The soil at the heading was required to support the pipes ahead of the last steel set placed so jet grouting of the soil inside the pipe envelopes was selected to enhance the soil mass properties. The jet grouting was intended to sufficiently stabilize the suspected slow to fast ravelling soils so that the face of the tunnel could be maintained

TABLE 2: CONCEPTUAL COMPARISON OF TUNNEL SUPPORT METHODS

	MULTIPLE PIPE ARCH	JET GROUTED ARCH	JACKED PRECAST LINING	GROUND FREEZING	NEW AUSTRIAN TUNNELING METHOD
DEMONSTRATED USE AND AVAILABILITY	Used on small scale locally and in Japan and Europe	Primarily used overseas	Primarily used overseas	Used some in USA	Limited use in USA, mostly in rock
RISKS & RELIABILITY STABILITY:					
CROWN/ARCH	Positive; extends beyond face	Relies on technique	Not assured beyond face	Depends on frozen soil strength, creep	Catastrophic failures have occured with little cover
FACE	Potentially unstable	Potentially unstable	Potentially unstable	Potentially unstable	Potentially unstable
OTHER RISKS	1. Pipe alignment 2. Obstructions 3. Rock at invert	1. Alignment 2. Obstructions 3. Rock at invert	1. Alignment 2. Thrust on soil and pavement	1. Ground heave 2. Freezing roadway	Relies on soil strength for support
ADAPTABILITY	Some	Some	Allows access to obstructions	Good	Quite Adaptable
GROUND MOVEMENT CONTROL					
RING PLACEMENT	Overdrilling	Depends on technique	Minimal, Possible Heave	Heave	
HEADING ADVANCE	Face Slump	Face Slump	Minimal with breasting boards	Irregular support, creep	
LOAD TRANSFER	1. Pipe to pipe 2. Pipe to arch (rib) 3. Rib to foundation 4. Foundation settlement	1. Irregular arch 2. Arch to foundation 3. Foundation settlement	1. Annular space grouting	1. Annular space grouting 2. Settlements on thawing	1. In general, presumes ground movement 2. Depends on timing

at 1(H):4(V). Even with this ground improvement technique, and a four foot maximum advance between sets, the unsupported length of support pipes was estimated to be 22 feet. Given this span, and the need to eliminate virtually any ground settlement due to structural deflections, heavy steel sets and very stiff soil support pipes (250 kip-ft. moment capacity) were required.

Similarly, settlement of foot blocks or other foundations under the steel sets were calculated to be excessive; therefore, loads were designed to be carried to rock by drilled, socketed piles. Space constrictions in the tunnel prevented the positioning of the piles directing below the posts so a floor beam was incorporated into the design to allow the steel sets to be jacked up (pre-stressed) tight against the soil support pipes. A preload of 40,000 pounds (20,000 pounds per post) and welded steel shims between every soil support pipe and the steel sets, and between the steel set and the piles was specifically provided for in the design.

TUNNEL CONSTRUCTION

Equipment and Materials

The tunnel design had considered twenty-two 24-inch diameter soil support pipes for each of the twin tunnels. The contractor proposed the use of the Tunnelherrenknecht AVN 600 microtunneling mole which would be used with 30-inch diameter pipes jacked in 20-foot sections. The mole was 17 feet long and had a 3/8-inch overcut. The other major components of the Tunnelherrenknecht system were the specially fabricated jacking frame, control center, the slurry muck removal system and the grout pumps. Sheet pile bulkheads placed at the ends of the tunnel allowed staged excavation of the jacking pit on the east side and the receiving pit on the west side of the highway. A reaction frame consisting of deep H sections and timber lagging was installed approximately 30 feet east of the east portal which formed the back of the jacking pit. Maximum thrust capacity of the microtunneling system was 375 tons.

The mole was articulated with three hydraulic jacks to allow directional control with a laser guidance system. The bi-rotational cutting head could be fitted with a shutter soil cutting head or roller disk rock cutting head. A coffee mill crusher was located in the mole to reduce excavated particles to 1-1/2 inch size or smaller. Muck removal was then by the slurry system which consisted of two centrifugal pumps, piping, a settling tank and a settling pond. All electronic control cables, hydraulic lines, and grout and slurry pipes were pre-placed in the soil support pipes at the staging area.

The pipes were 30-inch diameter, ASTM A 139, Grade B with 0.375-inch wall thickness and yield stress of 35,000 psi. The interconnecting "T" and channel assemblages were fabricated with WT 5 x 9.5 and MC 6 x 18 structural steel shapes.

Jet grouting was designed to provide fifteen 24-inch diameter grout columns with 4.6-inch outside diameter hollow fiberglass rods. Minimum yield stress of the rods was 30,000 psi. The equipment was a double-tube drill string with a tricone bit, and capable of producing 9000 psi at the nozzle. During jet grouting trials with soil cover comparable to that in the highway, some dramatic breakouts occurred. The grout mix was adjusted to provide at least 500 psi at 28 days.

This strength proved to be sufficient to stabilize the face, but allow excavation with hydraulic backhoes and front end loaders.

The pipe piles were 8-5/8 inch diameter API 5CT threaded pipes with a 1/2-inch wall thickness and minimum yield stress of 55,000 psi. The piles were socketed 8 feet into sound rock. Piles were drilled in place with a hydraulic drill fitted with a down-the-hole hammer and eccentric button bit which could be extracted through the installed piles. Piles ranged in length from approximately 15 feet to 55 feet, and were installed in 6-foot sections. Grouting was performed with accelerated neat cement grout pumped from the grout plant, through the pile cap and pile, and out ports at the bottom of the piles until grout returned outside the pile head. The design load was 185,000 pounds, and load tests on initial piles were carried to 140% of design load.

Contact Grouting

Two different stages of contact grouting were undertaken during the course of construction. Initially contact grouting was performed during microtunneling through 3 ports in the tail can of the boring machine. The fly ash-bentonite-cement grout was pumped with a manually operated screw-type displacement pump located at the jacking pit. The grout was pumped at a rate of approximately 1 cubic foot per 5 feet of advance to compensate for the volume of over excavation of the cutter head. The grout was also used to reduce pipewall friction during jacking.

Potential grout communication to the roadway surface and slab jacking were concerns particularly during the early microtunneling of pipes 8 through 14, as these were closest to the roadway and as close as a few feet to the relatively porous base course. Risk of breakthrough to the roadways was mitigated by the use of a relatively low flow rate pump and stringent control of pumped volumes. Typically, some grout was lost during the first thirty feet of tunneling due to washout from the slurry mucking system returning along the annular space around the pipe and through the bulkhead.

Secondary contact grouting was performed after the soil support pipes were concreted and excavation was complete. This grouting was performed through threaded ports which were welded to the webs of the "T" splines at approximately a 10-foot spacing. A similar 50 psi flyash-bentonite-cement mixture was used, but was pumped with a piston type pump which necessitated much closer monitoring of grout pressures at the injection port.

A summary of approximate grout takes is summarized in Table 3. The calculated overcut volume is based on the 3/8-inch overcut of the mole cutter head multiplied over 11 crown pipes and 10 sidewall pipes for each tunnel.

Tertiary contact grout sleeve pipes were installed prior to concreting to allow the option of future contact grouting, but have not been used to date.

TABLE 3. CONTACT GROUT TAKES

Description	ER Tunnel (ft^3/ft)	EL Tunnel (ft^3/ft)	Over Cut Volume (ft^3/ft)
Tail Can Grouting			
Crown	1.7	2.0	1.3
Sidewalls	1.3	1.7	1.3
Total	3.0	3.7	2.6
Secondary Grouting			
Crown	0.3	0.2	1.3
Sidewalls	0.5	0.5	1.3
Total	0.8	0.7	2.6
Total Contact Grout	3.8	4.4	2.6

INSTRUMENTATION AND GROUND RESPONSE

Monitoring the ground response was made difficult because no access was permitted on the highway. The monitoring program therefore included the following:

- Over 250 survey pins mounted in the highway.
- Thirteen conventional surface settlement markers beside the highway at the portals.
- Inclinometer casings drilled and grouted into place horizontally below the highway and parallel to the tunnels.
- Tape extensometer convergency measurements for the steel sets.
- Survey monitoring of the steel sets at 5 points on every other set.

Two permanent instrument pedestals were installed to assure sufficient repeatability while reading vertical angles to the pin targets. Foresights were on the order of 100 to 400 feet. The practical limit of the method was judged to be on the order of 1/10 to 1/4-inch based on review of the numerous data sets obtained. Additionally, there appeared to be some sort of drift in the setup on the jacking pit side which made the road surface well beyond the tunnel limits appear to tilt 1/4-inch from north to south.

The horizontal inclinometer casings allowed precise and repeatable determination of the relative settlement profile along the tunnels. However, difficulties with survey control of the collar of the casings and no end fixity made determination of absolute settlement with this technique virtually impossible.

The overall ground response at the surface is illustrated on Figure 3 as settlement contours. Numerous contour maps were developed during construction using commercially available software for plotting and contouring. Time-settlement plots were kept to track response at points of interest, with the corresponding construction activities noted. Figures 4 and 5 show pavement movement versus time at several key locations over a years time. These points are also shown in plan on Figure 3.

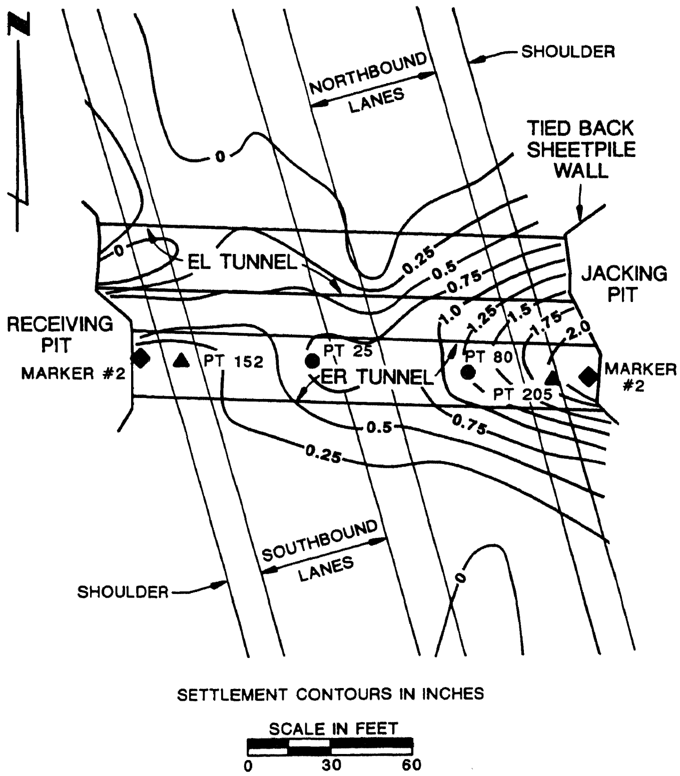


FIGURE 3: CONTOURS OF SETTLEMENT

The following statements regarding ground surface movement as measured by vertical angle and level surveys are made:

1. Almost all of the pavement movements occurred within about 30 feet to the north and south of the tunnel limits.

2. As shown on Figure 3, greater settlements occurred over the eastern portion of the tunnel than the west. The greatest heave (1-1/2 inches) occurred during microtunneling near the receiving pit which then subsided to about 1/2 inch.

3. The largest settlements along the highway (as great as 1.7 inches) have been measured on the northbound lanes. Settlements along the southbound lanes are generally 3/4 inch or less.

4. The greatest pavement settlement, ie the bottom of the settlement trough occurred just south of the twin tunnel centerline.

5. Settlements typically began during microtunneling of the sidewall pipes. Settlement rates appeared to either remain constant or accelerate during jet grouting. Settlement rates tended to decrease or stop during or shortly after excavation and support.

The greatest settlements occurred near the jacking pit, and much of the settlement was attributable to deflection of the bulkhead during excavation of the jacking pit and penetration of the bulkhead to begin microtunneling. Later during microtunneling of the sidewall pipes, larger settlements near the jacking pit were attributable to the inability to form an adequate seal at the bulkhead, which resulted in large grout returns during tunnel driving and likely some ground loss. The embankment fill and the upper portion of the residual soil profile contained more clayey material and was more cohesive than the residual soil below the tunnel springline. It is suspected that less ground loss occurred due to washout in the more cohesive materials in the crown.

Conclusions based on the steel set surveys are as follows:

1. The majority of the measured movements are 0.2 inches or less, with a few isolated readings approaching 1 to 2 inches. Large movements were not corroborated by successive surveys, and large rib movements did not correlate with surface settlements during excavation. Error is suspected for measurements greater than approximately 0.5 inches.

2. Pavement settlement points and portal movement markers indicated that surface settlement rates increased during excavation within the central portion of the tunnels. Settlements near the portals tended to continue at a steady rate or decrease during excavation. The steel rib survey data did not show this trend. However little data was available for the ribs near the center of the tunnels.

Overall, regular surveying of the steel sets during excavation and support indicated that very little movement of the structure occurred. Occasionally some access difficulty was experienced, and initial surveys were delayed or not obtained. Data for steel sets that do not have timely initial readings may not show the total movement experienced.

An analysis of the convergence data is provided below:

1. The majority of the tape extensometer readings show movement of less than 0.2 inches, which agrees well with the steel rib survey data.

2. Most of the sets which showed significant movement were located near the tunnel portals. However, surface settlement data indicates little or no influence of tunnel excavation on surface settlements near the portals. Observed movements began during microtunneling and continued through tunnel excavation.

3. Some measurements of about 1 inch or more (both convergence and expansion) are believed to be due to field reading/recording errors.

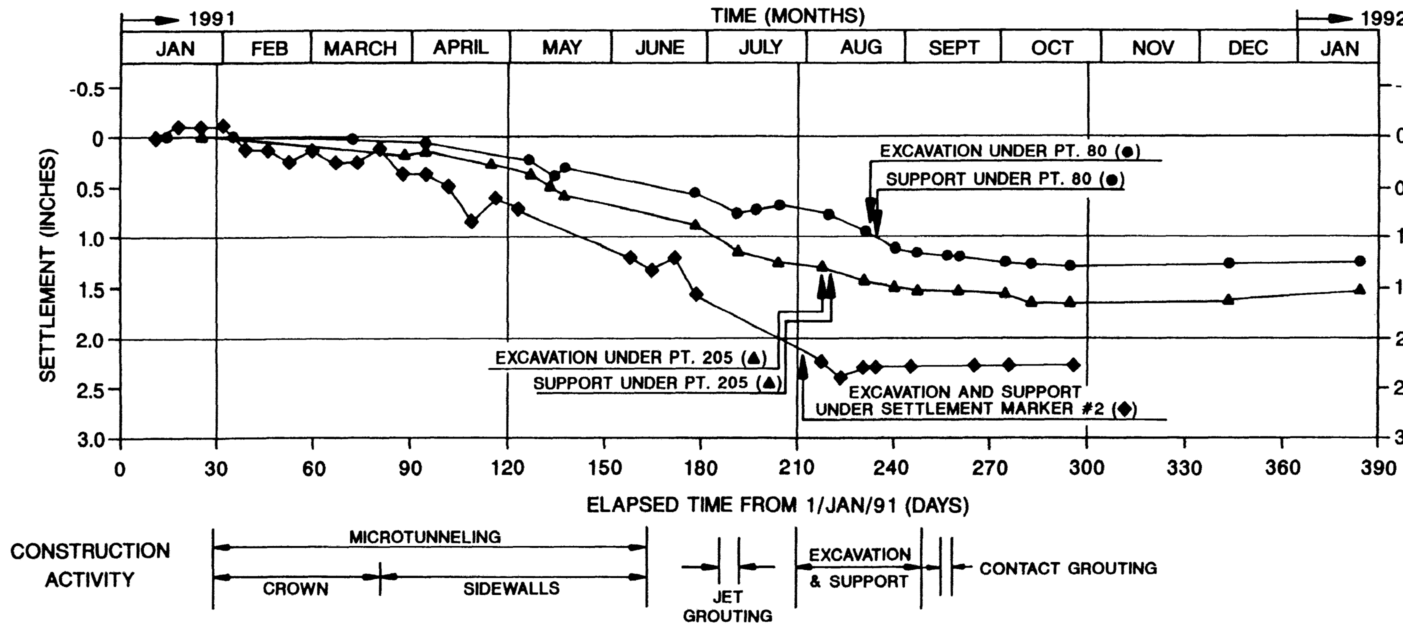


FIGURE 4: SETTLEMENT RESPONSE TO CONSTRUCTION ACTIVITIES - JACKING PIT SIDE

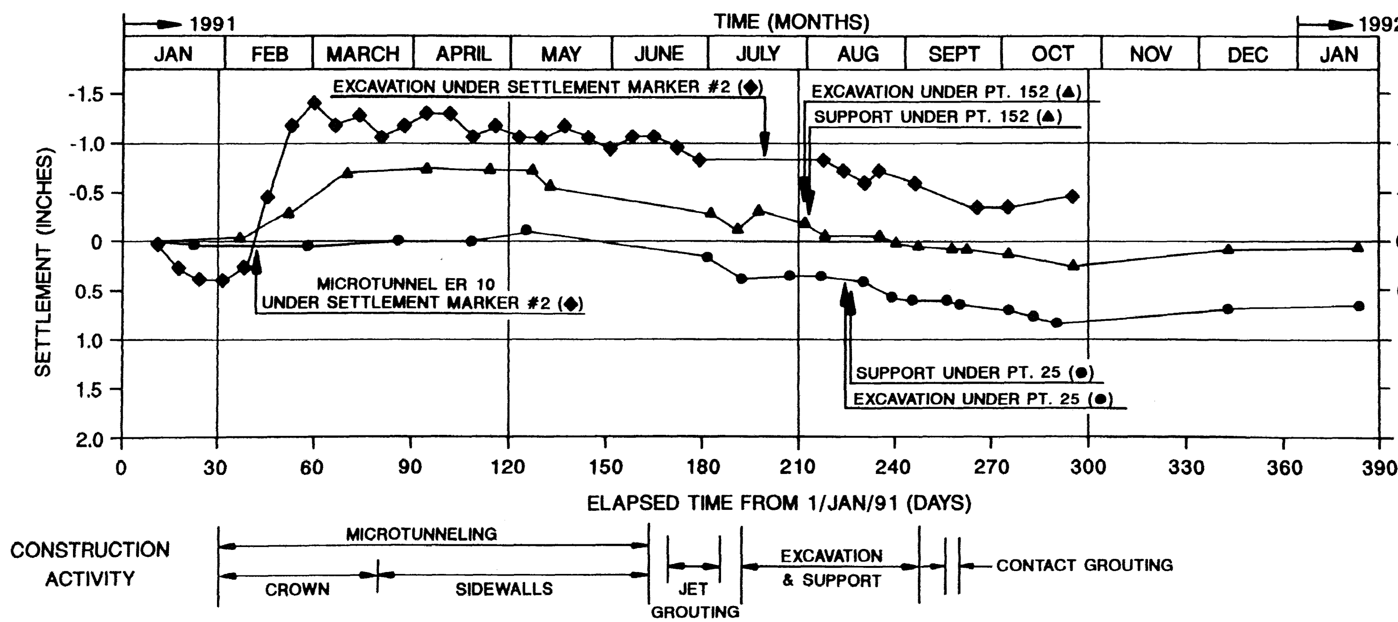


FIGURE 5: SETTLEMENT RESPONSE TO CONSTRUCTION ACTIVITIES - RECEIVING PIT SIDE

4. As with the steel rib survey data, there was no apparent correlation between soil support pipe movements and pavement settlement.

Surface settlements, when evaluated by way of settlement trough cross sections, can be compared to other project case histories. The settlement troughs over the east shoulder (near the jacking pit portal) and near the median (a more typical section) are shown on Figure 6.

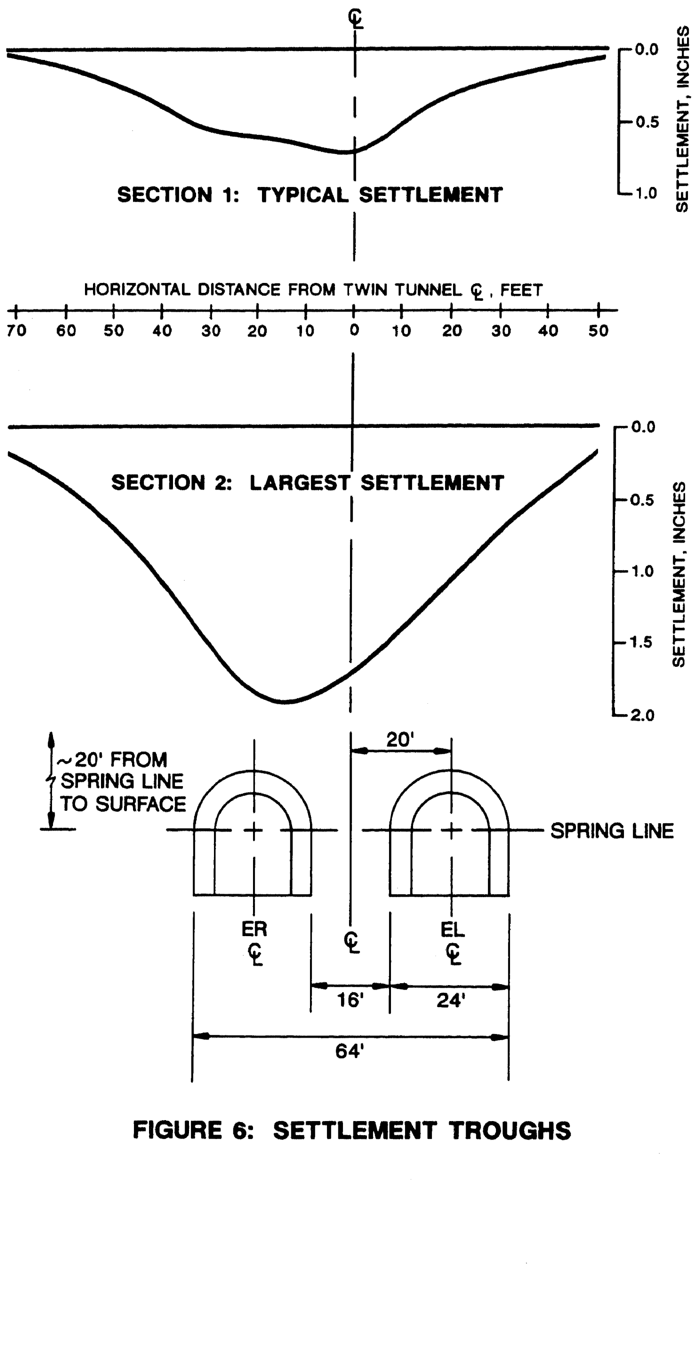


FIGURE 6: SETTLEMENT TROUGHS

The key characteristics of the tunnels and the settlement trough are as follows:

Effective Twin Tunnel
Radius, $R' = R + d/2$ = 32 ft
Tunnel Axis Depth, z = 20 ft
Depth/Diameter Ratio, $z/2R'$ = 0.3
Trough Width, W = 125 ft
Inflection Point, i = 25 ft
Width/Radius Ratio = i/R' = 0.8
Typical Maximum Settlement, = 0.06 ft
Extreme Maximum Settlement, = 0.16 ft
Settlement Volume, Vs :
Typical Trough Volume, Vs = 3.9 ft^3/ft
Extreme Trough Volume, Vs = 10.9 ft^3/ft
Typical Vs as
Percent of Microtunnel Volume = 1.9%
Typical Vs as
Percent of Excavated Volume = 0.4%

The settlements experienced during tunneling compare very favorably to previously documented cases, Peck (1969), and Akins (1983) where settlement trough volumes on the order of 0.5% to 7.0% are reported for tunnels in similar materials with much larger depth ratios (z/2R).

Given the timing of the start of surface settlement and its duration with respect to construction activity, as shown on Figures 4 and 5, it is difficult to quantify the amount of settlement that is attributable to microtunneling, and that which is attributable to excavation, or how much settlement was arrested by the contact grouting.

Monitoring of steel support members indicates that very little settlement should be attributed to support movement, but given the extremely shallow cover a more rapid ground response would be expected if settlements were due solely to lost ground during microtunneling. Undoubtedly, some settlement of the soil support pipes occurred during excavation before the ribs and arch could be installed, but again, given the time of initiation and duration of surface settlements, the attributable amount is difficult to quantify.

SUMMARY AND CONCLUSIONS

The use of the combined technologies by MARTA resulted in tunnel excavation with exceptionally thin cover, and without excessive movement of the roadway.

The microtunneled multiple pipe arch effectively provided pre-support of the relatively weak residual soils and fill. The slurry mucking system, while necessary for the ground conditions, may have caused some of the settlements due to ground loss (wash out) near the jacking pit.

The heavy and closely spaced steel sets founded on grouted pipe piles were effective in supporting the microtunneled pipes. The data indicates that less than 1/4-inch of movement of any steel components occurred at any given station once a rib and arch were welded in place.

Contact grouting to compensate for microtunneling overcut (and lost ground) was undoubtedly at least partly responsible for keeping settlements to acceptable values, but did not completely eliminate them. The extremely shallow cover and associated low overburden pressure precluded increasing grouting pressures to enhance grout takes.

The extensive remote monitoring system was effective in providing timely and accurate ground response information.

What pavement settlements that did occur are considered virtually unavoidable given the ground conditions, tunnel/cover geometry and the current state of the art.

ACKNOWLEDGEMENT

The Metropolitan Atlanta Rapid Transit Authority is the project owner. Parsons Brinckerhoff/Tudor are MARTA's general engineering consultants. Golder Associates Inc., in association with R&D Testing and Drilling, Inc., are the authority's geotechnical consultant. Archer/Western Corp., located in Georgia, was the general contractor. The subcontractor for the microtunneling, contact grouting and pile installation was Salgado Eastern Corp., Columbia, Md. The jet grouting subcontractor was a joint venture of Salgado Eastern and Pacchiosi Drill Sandra Idrolica of Italy.

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