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ABSTRACT

Tunnelling was part of the new Tren Urbano transit system in San Juan, Puerto Rico. Four tunnels in soil were designed and constructed with shotcrete linings using the sequential excavation method (SEM), which uses some aspects of the New Austrian Tunneling Method (NATM). Four 6-m-diameter tunnels of about 100 m in length were required to preserve two historic structures located above the subway alignment. Two of the four tunnels were constructed as part of a turnout to a future line. Cover over the SEM tunnels ranges from 20 to 5 m. Some of the tunnels are located less than 1 m from each other in the turnout section. Detailed analysis of the staged construction was undertaken to design shotcrete lining thickness, shotcrete strength, and reinforcing with welded wire fabric and lattice girders. Several variations in lining section were required, which depended on sequence of tunnel excavation and depth of cover. Further refinement of the lining design was possible by considering the initial lining as permanent since it had been constructed with final structure quality requirements. Compensation grouting effectively mitigated ground movements and building settlement was limited. Tunnel lining convergence measurements revealed the lining displacements due to excavation of adjacent or overlying tunnel construction to be within acceptable limits. Design and construction of the tunnels as sequentially excavated with shotcrete support (SEM) was unprecedented in Puerto Rico and not in widespread practice in the continental United States. Further, this was the first major United States underground transit construction project with design-build project delivery.

PROJECT BACKGROUND

In 1994, the Government of Puerto Rico approved plans for a heavy rail transit system. “Tren Urbano” (translated as Urban Train) was chosen as the name of the project, as well as the organization formed to manage the project. The Phase I alignment of the Tren Urbano Project connects the populous western municipality of Bayamón with Santurce, passing through the municipality of Guaynabo and the districts of southern and central San Juan known as Río Piedras and Hato Rey (Fig. 1). The line is 17.2 km long, has 16 stations, and a centrally located storage and maintenance yard. Most of the Tren Urbano is above ground. The underground section passes through the congested and historic district of Río Piedras.

The Río Piedras design-build contract was advertised in June 1996. Award and Notice-to-Proceed were given simultaneously in April 1997 to the KKZ/CMA joint venture, which comprises three construction contractors: Kiewit Construction Company, Kenny Construction, and H.B. Zachry Company. Managing designer was the Puerto Rico firm, CMA Architects & Engineers. Subcontractor engineering firms included Jacobs Associates (tunnel structural design), Sverdrup Civil (station structural/architectural design and mechanical/electrical design), and Woodward-Clyde (geotechnical exploration and instrumentation). The bid of $225,600,000 (US) was determined to be the best value of the three bidders.

The Río Piedras Contract consists of a 1,500 m long underground rapid transit guideway with two underground subway stations and is situated in a dense urban area, (see Fig. 2.) Geotechnical conditions consist of weathered alluvium (soft-ground) with 3 principal strata as noted in Fig. 3. The entire area is underlain by limestone of the Aquada formation which is known to have solution cavities but did not materially affect the project. Most project structures are below the groundwater table. See Gay et. al. [1999] and Morrison et. al [1999] for other information on this project.

Sections of the guideway and the University Puerto Rico Station were constructed by cut-and-cover methods. The remainder was done by various tunneling methods:

- Twin guideway tunnels: Earth pressure balance tunnel boring machine (EPBM)
- Río Piedras Station: Stacked drift method (Romero and Madsen [2001])
- Guideway and turn-out tunnels: Sequentially excavated, shotcrete supported (SEM/NATM)

Design and construction of the SEM tunnels was unprecedented in Puerto Rico and not in widespread practice in the continental United States. Further, this project was the first major United States underground transit construction project with design-build project delivery.

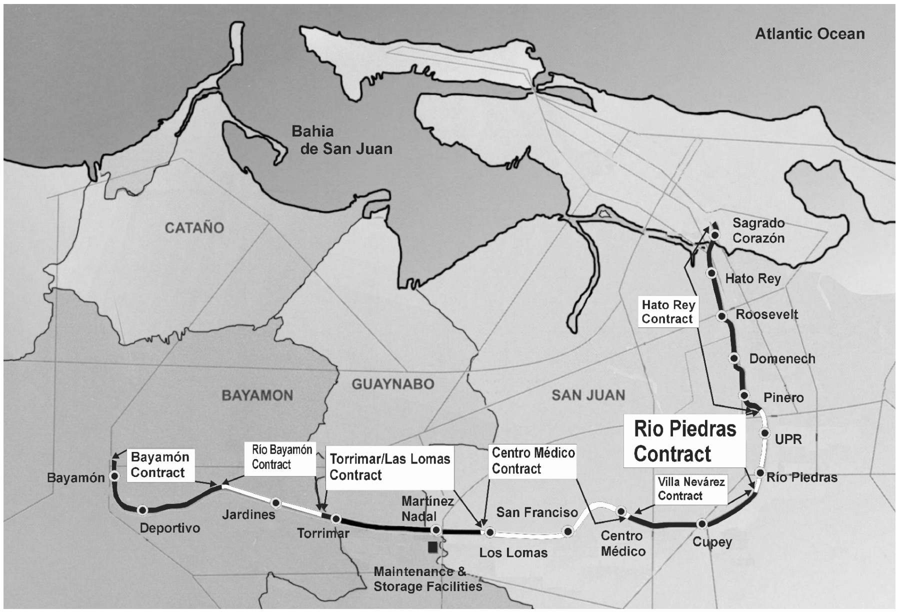

Fig. 1 Tren Urbano Transit System, San Juan, Puerto Rico

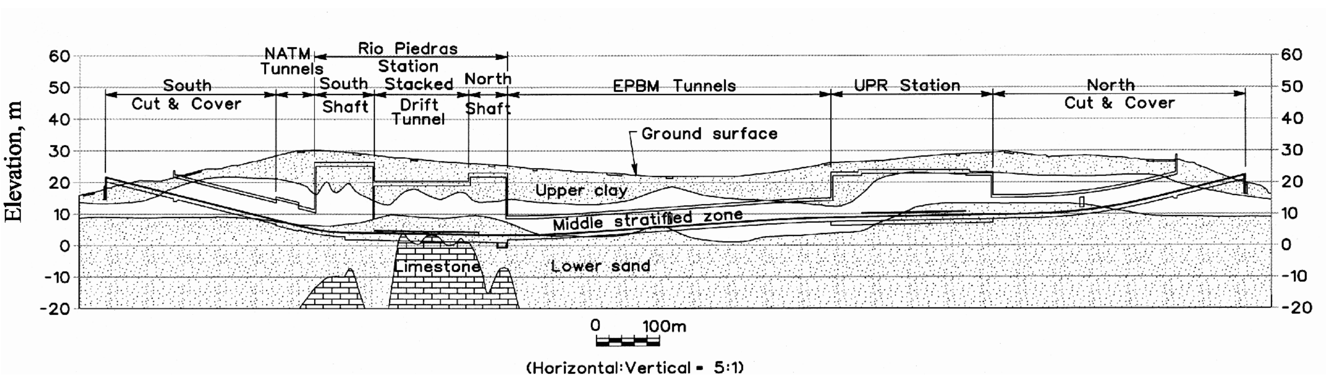

Fig. 2 Underground Section of Tren Urbano, Río Piedras Contract

TUNNEL LAYOUT, CONSTRUCTABILITY AND CONTRACTOR PREFERENCES

As-Bid

An historic structure directly in the alignment mandated tunneling for the turnouts at the south end of the project. Both shield tunneling and cut-and-cover construction methods were suggested in the tender documents. During the tender design, the contractor rejected shield tunneling for the three short tunnel drives, of which two were stub tunnels for a future transit line and thus would not hole-through to a portal or shaft and preclude an easy recovery of a tunnel shield. Mining these three tunnels with shotcrete support was considered to be a much more practical and efficient approach. The bid was submitted with three tunnels being sequentially excavated with shotcrete initial support, which is referred to herein as the SEM. The fourth remained as a deep cut-and-cover constructed tunnel.

Final Design

Final design started with further evaluation of overall project sequence and construction methods. Several project needs in the area of the mined tunnels were considered together to arrive at an optimum scheme. Site conditions were constricted by existing structures and narrow streets that had to remain serviceable. A major underground high voltage power line crossed over the tunnels. Excavation of the large, permanent shaft for transit tunnel services and station entrance required a very large crane to operate at the south end of the shaft.

By the summer of 1997, the mined tunnel configuration was changed significantly from that assumed at bid. KKZ/CMA elected to drive four tunnels instead of three and generally the length of tunnel was increased. The value of this change came in several ways: eliminating the time and significant cost of the power line relocation, simplifying overall construction, and eliminating short retained cuts that could be done more economically by extending tunneling limits.

Turn-Unders, Issues for Start of Tunneling

Turnouts for the future Carolina Line presented a complicated situation for the start of tunneling. In principle, the turnouts are the transition from two tunnels to four tunnels. With no limits on the shaft excavation size, the four tunnels could have been started with a comfortable pillar of soil between each tunnel. But such a scheme would have required the shaft wall to be too far south and would eliminate the area for setting the large crane servicing the shaft. It also would require closing the roadway. Setting the crane on decking or falsework was unacceptable to the contractor. The other extreme was reducing shaft size significantly and constructing the wye transitions from two to four tunnels all by mining. This concept was rejected on the basis of requiring even more complex and costly construction with significant risks.

The solution was to set the shaft wall, and thus the tunnel turn-unders, as far north as possible (decreasing shaft size) to the point where at least the full ring of all four tunnel linings could be constructed. The resulting turn-under was established at Sta. 219+10 as shown in Fig. 3. The implications of the close spacing on lining design were significant. Close spacing of the tunnels meant the sequence of tunneling would have to be considered in detail. Further, the structural capacity of each tunnel lining ring would have to be considered carefully.

Contractor Input and Design Preferences for Tunnel Linings

Initial and Final Lining. The contractor and engineer agreed at the onset that the most efficient design and construction should make best use of all materials installed for both the initial and final linings. This required design details and materials for the initial lining to meet requirements for permanent materials. This concept was a significant departure from most tunnel design practices of the time where an initial tunnel lining of shotcrete is routinely ignored for the final condition and the final cast-in-place concrete lining is designed to take all design loading.

Initial linings were designed to accommodate all ground loads. A waterproof membrane was a specific project requirement and was designed to be installed between initial and final linings. Contract criteria mandated a minimum 300-mm-thick cast-in-place concrete final lining

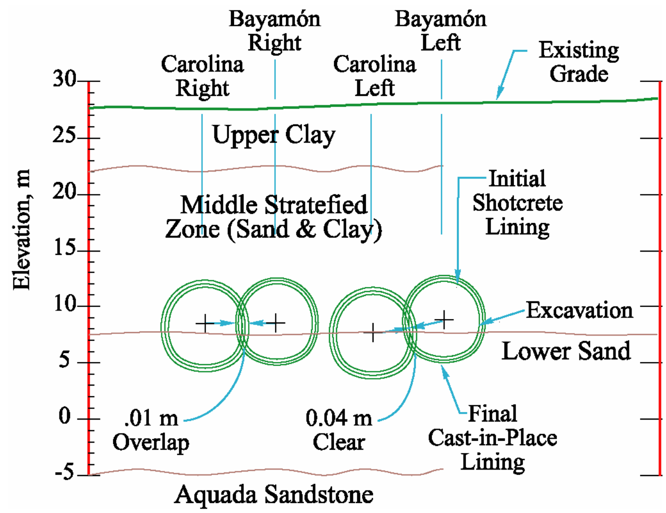

Fig. 3 Tunnel Configuration at Turn-Under, Sta 219+10

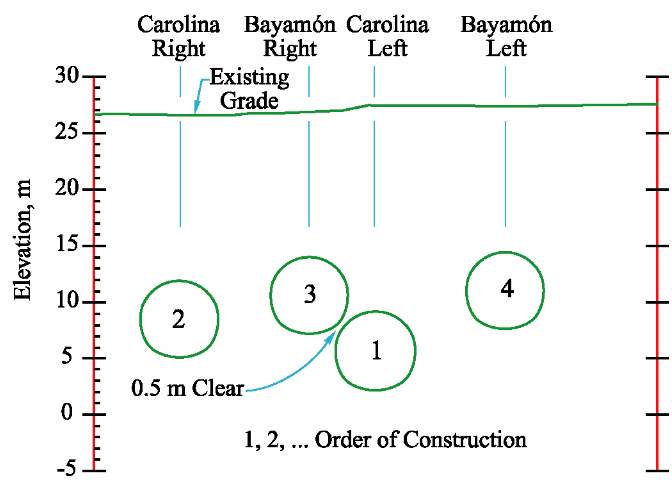

Fig. 4 Tunnel Configuration at Critical Over/ Under Condition, Sta 218+70

With the initial lining taking ground load, the contractor's engineer felt an adequate and sensible basis for design was to require the final lining to carry only the design ground water loading and live train loads, and therefore the concrete could be unreinforced. The contract criteria by the owner, unfortunately, had evolved from past design-bid-build tunnel projects and required the final lining to take all loading and ignored the value of the initial lining. Detailed analysis was able to demonstrate that the unreinforced concrete final lining could accommodate loading by both groundwater pressure and differential ground loads. Design details are presented in the following sections.

Designs Tailored With Varying Sizes and Structural Capacity

With the designer and contractor as a team, the contractor preferences for materials, products, methods, and sequence were incorporated in the design wherever possible. Several items are summarized in the following.

Lining Design Varied for Optimum Efficiency. At the turn-unders, a more substantial initial shotcrete lining was required (325 mm thickness) in order to carry ground loads. As soon as could be justified by analysis, the structural capacity of the initial lining was reduced (less thickness, 240 mm, and less reinforcing).

Curves in the alignment meant that transit vehicle clearances were different. Thus, the tunnels could have different final sizes. Using one tunnel size for all would result in an oversized section for substantial lengths of the tunnel. The contractor felt the minimum size was most economical to construct, even though the sizes varied by small amounts. Design details were developed accordingly, which required different size lining materials (lattice girders) and adjustable concrete forms.

Shotcrete Preferred Over Reinforcing. The initial linings were comprised of varying amounts of shotcrete, lattice girders, and welded wire fabric (wwf). Steel bar reinforcing was considered to be very undesirable because of the labor required to install and was not used in the design. Lattice girders, which have evolved as being integral with shotcreted tunnel linings, were selected on the basis of sizes available. Welded wire fabric was used as a variable design component. The highest loading conditions at the turn-unders required multiple layers of heavier gage wire and in the minimum lining case required only a single layer of lighter gage wwf.

Designed and Constructed as Permanent. Two major requirements had to be met in order for the initial lining to be considered permanent. First, shotcrete had to meet the strength and durability requirements like concrete. The project approach was that shotcrete was no different from concrete and quality requirements could be verified by inspection and quality control testing. Second, the design had to incorporate the details of concrete cover over reinforcing that are essential for the shotcrete lining to perform as reinforced concrete and to provide sufficient cover for long term conditions.

ANALYSIS AND STRUCTURAL DEISGN OF TUNNEL LININGS

Numerical methods were used to analyze complex geometries and excavation sequences required to construct the initial and final linings. Two methods were used.

Finite Difference Continuum Model. The computer program FLAC (Itasca [1996]) modeled behavior of the initial tunnel lining. Two-dimensional models simulated excavation sequence and installation of initial linings at key stages of construction with the soil as an elasto-plastic continua and the initial lining as structural beam elements.

Beam-Spring Model. Structural frame analyses, using the computer program STAAD, were used for evaluating the final tunnel lining. The lining was modeled as beam elements and the surrounding ground was modeled by a series of springs.

FLAC Analyses

Critical cross sections modeled using FLAC are shown in Figs. 3 and 4. The modeled excavation sequence followed the Contractor's proposed construction sequence as indicated in Fig. 4.

For each tunnel, the analysis simulated excavation of each tunnel in a heading and bench sequence. The structural beam elements simulating the initial lining were installed simultaneously with the appropriate stage of excavation. Based on the predicted rate of advance of the tunnels, the beam elements were initially assigned properties corresponding to the 1-day shotcrete strength, modulus, and bending moment capacity and the model allowed to equilibrate. Structural parameters of the subject beam element were then changed to the 3-day values prior to the next excavation stage and similarly for the second stage of excavation. The detailed modeling sequence is summarized below:

1. Excavate top heading and install shotcrete lining (as beam elements) in crown with 1-day modulus and bending moment capacity.
2. Modify modulus and moment capacity of beam elements in crown to 3-day values.
3. Excavate bench and install shotcrete lining (beam elements) in invert with 1-day modulus and moment capacity.
4. Modify modulus and moment capacity of beam elements in crown and invert to 7-day values.
5. Repeat sequence for remaining tunnels.

Properties of Beam Elements

The 28-day strength of the cast-in place concrete final lining was assumed to be 35 MPa. For the initial lining shotcrete, the assumed variation of unconfined compressive strength with time is presented below:

- 0-days, no shotcrete strength
- day, f'c = 18MPa
- 3-day, f'c = 27MPa
- 7-day, f'c = 32MPa
- 28-day, f'c = 35MPa

The relationship used to calculate these strengths was based on the following relationship after Chang and Stille [1993]:

$$Y = 1.105 \cdot f_c' \cdot e^{-0.743/t^{0.7}}$$

where: Y = unconfined compressive strength; f'c = 28-day unconfined compressive strength; and t = shotcrete age in days.

The modulus of elasticity of the shotcrete was calculated using the following equation:

$$E_c = w^{1.5}\, 0.043 \sqrt{f'c}$$

where: E_c = modulus of shotcrete (MPa); w = weight of shotcrete (kg/m^3); and f'c = unconfined compressive strength (MPa).

The constitutive model for the beam elements provides for elastic behavior up to a specified moment capacity beyond which the moment remains constant. Based on the results of the preliminary analyses, the beam elements simulating the initial lining installed in the Carolina tunnels were assigned properties corresponding to a lining thickness of 325 mm. The beam elements simulating the initial lining installed in the Bayamón tunnels were assigned properties corresponding to a lining thickness of 225 mm, which later became 240 mm to achieve adequate concrete cover over lattice girders.

Soil Properties

The soil was modeled as perfectly elastic-plastic continua with a Mohr-Coulomb failure criteria. Drained strength parameters and total unit weights were used on the basis that dewatering will take place ahead of tunnel excavation; pore pressure generation and dissipation are not modeled in this type of analysis. See Table 1 for details.

Table 1. Geotechnical Parameters Used in Analyses

	Soil Type		
	Upper Clay	Middle Stratified Zone	Lower Sand
Total Unit Weight, γ, kN/m^3	18.0	19.5	19.5
Buoyant Unit Weight, γ', kN/m^3	8.2	9.5	9.5
Design Ground Water Elevation	+15 m		
Drained Shear Strength	c'= 0 kPa $\Phi = 36°$	c'= 0 kPa $\Phi = 37°$	c'= 96 kPa $\Phi = 37°$
Subgrade Modulus, Es, MPa	Linear variation: 125 at top of tunnels to 160 at tunnel inverts		
Coefficient of At-Rest Pressure, K_o	0.5	0.5	0.9

Relative Stiffness (Flexibility) of Tunnel Linings

Stiffness of the linings relative to the ground was known be the major factor in how much thrust and moment would result in the tunnel linings (Peck, et. al. [1972]). Using the flexibility ratio as a measure of relative stiffness and comparing the extremes of lining stiffness and ground stiffness (modulus), the flexibility ratio, F, was calculated to range from about 8 to 25. A tunnel lining is generally considered flexible for F greater than 10. In practical terms, this meant that the 325 mm thick linings would require the most reinforcing in order to sustain predicted loads. On the other hand, the thinner 225 mm thick linings would have substantially less bending moment and would have the least reinforcing.

Results of FLAC Analyses

The critical thrusts and moments in the initial linings calculated by numerical models were evaluated using moment-thrust interaction diagrams in accordance with an ultimate capacity analysis using the procedures of ACI 318 (ACI [1992]).

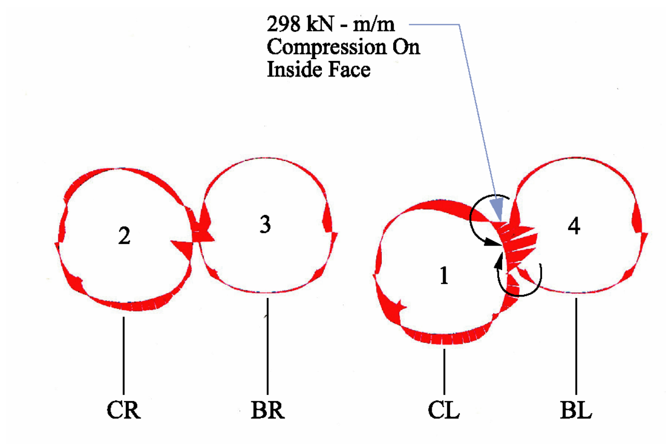

Fig. 5 Calculated Bending Moments in Tunnel Linings for Condition of All Tunnels Excavated, Start of Tunneling, Sta 219+10

Analyses indicated the highest bending moments occur in the initial lining of the Carolina Left tunnel. This tunnel was constructed first and deformed and took load as the next tunnels were excavated. The critical case occurs when the lateral restraint by the ground is substantially reduced as the Bayamón Left tunnel is excavated immediately adjacent to it as shown in Fig. 5. (Note that the apparent sign change and discontinuity in the moments between beam elements installed in the top heading and the beam elements installed in the bench excavation is due to the moment sign convention used by FLAC.) Thrust in the initial lining of Carolina Left also increases sharply following the excavation of the Bayamón Left tunnel. Similarly, significant thrusts and moments are generated in the initial lining of the Carolina Right tunnel following the excavation of the adjacent Bayamón Right tunnel.

The increased thrust that develops in the lining following the excavation of the adjacent tunnel is a function of the relative stiffnesses of the closed ring in the Carolina tunnel and the open ring installed in the heading of the adjacent Bayamón tunnel. Because the closed ring is significantly stiffer than the open ring, the closed ring will tend to carry a larger portion of the overburden load. This modeling of the tunnel construction sequence portrays the load shifting to the completed initial linings (Carolina tunnels) by subsequent adjacent tunneling (Bayamón tunnels). This type of behavior for load interaction between multiple tunnels is supported by tunnel lining research (Ranken & Ghaboussi, 1976), which consisted of finite-element soil-structure interaction modeling supported by field measurements on actual tunnels during construction. The excavation of the Bayamón tunnels also results in some asymmetry of the thrust distribution in the Carolina tunnels. The predicted thrust in the lining of the Carolina tunnels is higher on the side of the tunnel closest to the adjacent Bayamón tunnels.

The increased moments that develop in the Carolina tunnels following the excavation of the adjacent Bayamón tunnels are a result of the increased loads acting on the initial lining of the Carolina tunnels and the decreased lateral confinement that is available for the section of lining closest to the adjacent tunnel. The reduced confinement results in a bulging of the completed tunnel lining towards the adjacent tunnel and an increase in bending in the lining. This effect is most noticeable in the initial lining of the Carolina Left tunnel.

Generally, the pattern of displacements of the final lining is characterized by inward deformation of the ring, except for the previously mentioned bulging of the lining towards the adjacent tunnel. The predicted displacements of the initial linings were in all cases less than 16 mm. It was recognized that the calculated ground displacements would be less than the actual displacements because the models do not account for ground relaxation around the tunnels prior to installation of the lining.

Structural Design

Final design shotcrete linings were 325 mm and 240 mm thick and had several variations to suit specific reaches of tunnel. In the detailed analysis, a 225 mm thick lining had been assumed. When the final choice for lattice girder was made among several alternatives, the thickness had to be increased to 240 mm in order to provide adequate concrete cover over the girder. Spacing of girders was generally 1200 mm. Welded wire fabric was 4x4-W5.5xW5.5. PVC membrane waterproofing was placed between the contract-specified 300 mm thick cast-in-place concrete final lining. As described earlier, the final lining was designed to be unreinforced, but where the tunnel alignment varied and the full 300 mm section did not exist, bar reinforcing was required.

INSTRUMENTATION, MONITORING AND CONSTRUCTION PERFORMANCE

The entire Río Piedras contract had a comprehensive geotechnical instrumentation and monitoring program for construction. Instrumentation associated with the SEM tunnels is shown in Fig. 6 and included:

- 5 multiple point borehole extensometers (MPBX)
- 2 inclinometers
- 4 piezometers (for 5 dewatering wells)
- 10 settlement rods (optical leveling)
- 37 building settlement markers (optical leveling)
- 2 surface settlement markers (optical leveling)
- 38 tunnel convergence points (tape extensometer)

This instrumentation monitored ground and structure movements, in particular movement (i.e., settlement) of existing structures above the tunnels, and deformation of the tunnels themselves (i.e., tunnel lining convergence).

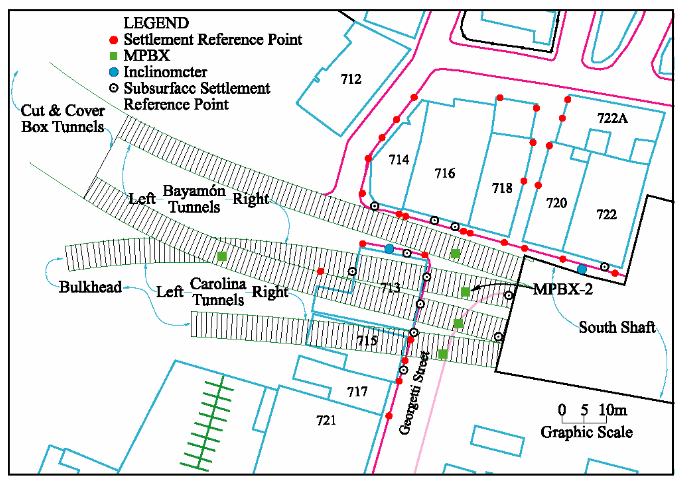

Fig. 6 Plan Showing Tunnels and Geotechnical Instrumentation

Monitoring was initiated prior to construction to establish a "baseline" from which movements associated with construction could be compared. During construction, monitoring schedules were carefully coordinated with excavation and initial lining sequencing of the SEM tunnels. In general, monitoring was very frequent during tunnel excavation, when most ground and lining movements were taking place. During installation of the membrane waterproofing and the final cast-in-place concrete final lining, monitoring was less frequent as most ground and structure movements had already occurred.

During tunnel excavation, data from the instrumentation and monitoring program was reviewed daily by the tunnel designer and compared with predicted movements. Data on building settlement was used by the contractor to implement a compensation grouting program which was very effective at keeping building settlement to acceptable limits. In addition, monitoring data was compared with threshold limits on lining convergence. In the event that convergence threshold limits were exceeded, the contractor could quickly implement a contingency plan to install additional tunnel support.

Two sets of instrumentation data are presented to illustrate the type of data that was analyzed by the designer. Figure 7 shows a plot of vertical ground movement above the Carolina Left tunnel as measured by an MPBX. (The surface settlement measured by this MPBX was not the settlement experienced by the nearby existing buildings above, as these buildings were protected by the compensation grouting.) The data show vertical movement as each of the 4 tunnels was excavated, as well as the cumulative movement associated with all 4 tunnels.

The surface settlement can be compared with traditional settlement empirical prediction theory (such as by Peck [1969]). The data and tunnel experience used by Peck included not only shield-driven tunnels, but ones that were hand-mined without a shield like the SEM tunnels for Rio Piedras. "Ground loss" or "face loss" are general terms that are associated with many different sources of ground movement and settlement such as over-excavation, as well as elastic and non-elastic ground movements – a phenomena predicted by the numerical analysis. Based on Fig. 7, the "ground loss" for the SEM tunnels was back-calculated to be 1 %, which falls within the realm of reasonable construction practices for an open-faced tunnel in soft ground.

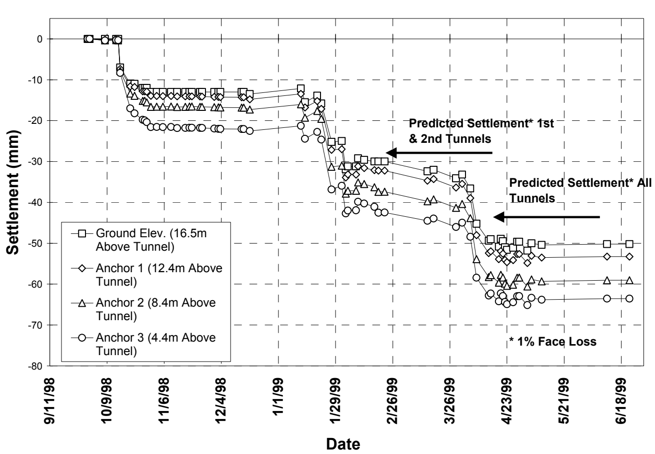

Fig. 7 Deep Settlement, Carolina Left Tunnel, MPBX-2

Figure 8 shows convergence monitoring in the Carolina Left tunnel before and during excavation of the Bayamón Right tunnel above. Positive (outward) movement was experienced at the springline, while negative movement was seen at the crown. This represents the traditional "tunnel squat" phenomena commonly in soft-ground tunnels.

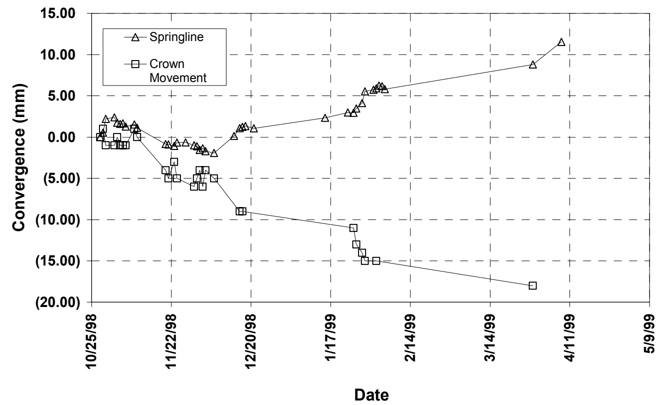

Fig. 8 Measured Tunnel Convergence, Carolina Left Tunnel, Sta 218+70

As shown in Fig. 8, magnitude of the lining deformation was 0.15 to 0.20 % of the tunnel diameter and well within acceptable limits. The magnitude and direction of movements observed in Carolina Left, however, were not in agreement with the numerical modeling. This was expected since assumptions in the model that yield conservative stresses in the lining also result in lower predicted ground movements.

Fig. 9 shows the completed tunnel with cast-in-place concrete final lining, walkway, track, and surface mounted utilities.

Fig. 9 Completed Tunnel

CONCLUSIONS

Two dimensional numerical models allow simulation of complex excavation sequences and can be used to evaluate the interaction effects between adjacent tunnels. Accurately modeling a complex excavation sequence requires not only simulating the physical changes with each stage of excavation, but also accurately characterizing the change in lining properties with time. Excavation sequence is very important as significant loads can be transferred to existing tunnels when an adjacent excavation takes place.

Numerical analysis was sufficient to reliably define loading conditions to structurally design the initial linings. The minimum, yet adequate, tunnel linings were designed for the four tunnels. Linings installed first in the sequence attracted load and required more structural capacity with greater thickness and reinforcing. Linings installed later could be thinner with less reinforcing.

Lessons learned from the instrumentation and monitoring program included:

- Redundant and different types of instruments are needed to get quality data, as some instruments are invariably destroyed or otherwise yield unusable data. This is inherent with the difficulties in underground construction,
- Numerical modeling should not be relied on solely to predict ground movements; there is no substitute for judgment and experience.
- Instrumentation data is very useful for calibrating numerical models on projects in similar conditions. For multiple tunnels constructed by SEM in challenging geotechnical conditions such as in Río Piedras, daily supervision of the data by an on-site design engineer promotes safety and quality in the construction.

Finally, this project was done as a design-build effort. Successful construction of these tunnels was the result of close interaction between engineer and constructor on all details. The basis of design was controlled to a large extent by contract requirements of the owner. However, the designs were not prepared on the basis of textbook methods or as the direct result of computer software. Rather, the tunnel design and construction sequence followed fundamental engineering principles that ensured stable underground structures through the many steps in the construction process. An essential element in that process was the designer and contractor working as a team.

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