

## DESIGN AND CONSTRUCTION PERFORMANCE OF A LARGE DIAMETER TUNNEL CONSTRUCTED IN SOFT-GROUND BY THE STACKED DRIFT METHOD

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### ABSTRACT

The Tren Urbano project in Puerto Rico includes a 19 m diameter tunnel constructed by the stacked drift method, which involves installation of a pre-supported concrete arch using multiple tunnels. Geotechnical conditions consisted of weak rock and highly weathered, unstable alluvial soils. Due to low cover, reinforcement of the arch was required. Ground behavior ranged from firm ground to mixed-face to raveling and flowing ground. Consequently, construction of the individual drifts involved many different soft-ground techniques such as steel set supports, shotcrete support, shield tunneling, forepoling, and consolidation grouting. During construction, the design was modified to improve constructability. Although settlement was experienced during excavation of the drifts, arch movements were within predicted limits and minimal surface settlement was observed upon excavation of the interior.

### BACKGROUND

In 1994 the Government of Puerto Rico approved plans for the “Tren Urbano” (translated as Urban Train) a heavy rail transit system. The Phase I alignment of the Tren Urbano connects Bayamón with Santurce, passing through Guaynabo and central San Juan (Figure 1). While most of the Tren Urbano is above ground, an underground section passes through the heavily congested and historically rich district of Río Piedras.

Within the Tren Urbano project there are seven alignment section contracts and one systems contract that were let as design-build. One alignment section, the Río Piedras contract, was advertised in June of 1996. Three joint ventures of contracting and engineering design firms submitted technical, management, and cost proposals. Award and notice to proceed were given simultaneously on April 1997 to the KKZ/CMA joint venture, which comprises three contractors: Kiewit Construction, Kenny Construction, and H.B. Zachry Company. The managing designer is 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

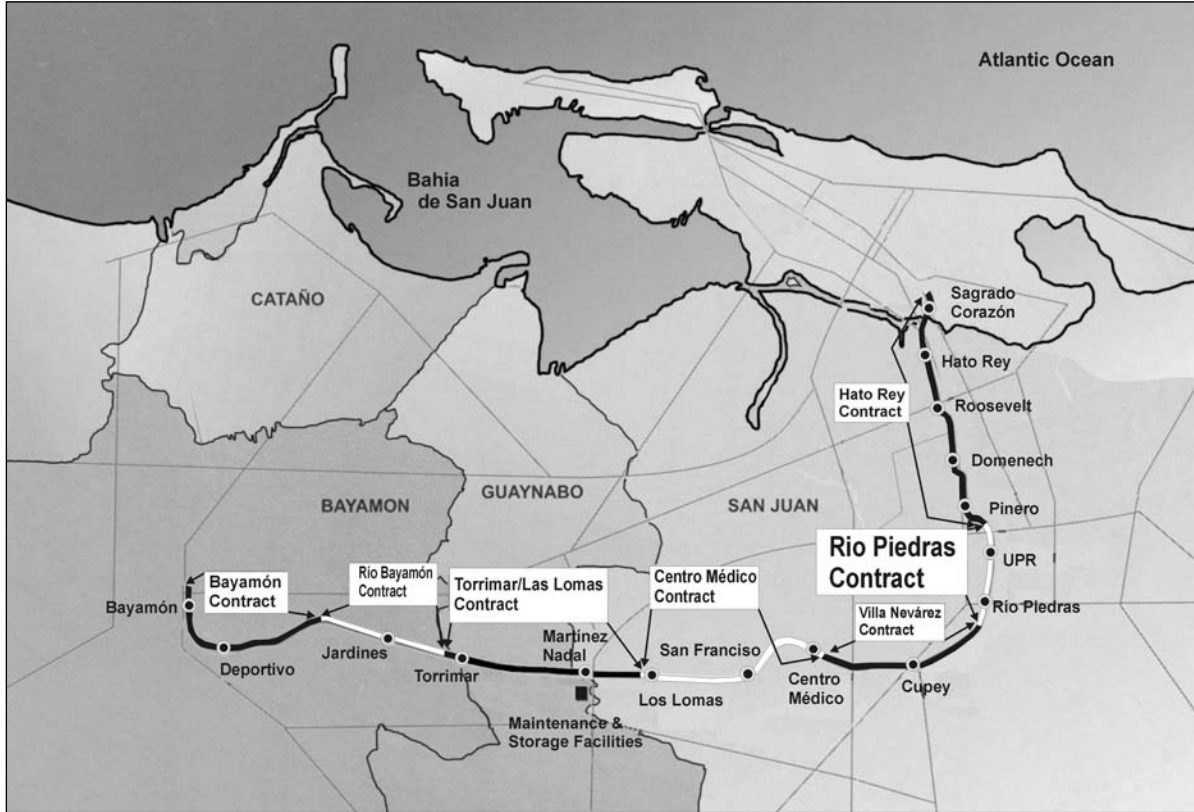


Figure 1. Phase I of the Tren Urbano

exploration and instrumentation). The bid by KKZ/CMA of \$225,600,000 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 (see Figure 2). The project is situated in a dense urban area, and geotechnical conditions consist of weathered alluvium. Most of the project structures are located below the groundwater table. A detailed description of the overall project is given by Gay et al. (1999).

On the Río Piedras Contract, sections of the guideway and the University of Puerto Rico Station were constructed by cut-and-cover methods. The other guideway sections and the Río Piedras Station were constructed by various tunneling methods:

- Guideway and turnout tunnels were constructed by the New Austrian Tunneling Method
- The Río Piedras Station was constructed by the stacked drift method
- Twin guideway tunnels were constructed by an Earth Pressure Balance Machine (EPBM) with a one-pass concrete segmental lining

The tunnel for the Río Piedras Station platform is 150 m long by 19 m wide by 16 m in height, making it one of the largest diameter soil tunnels ever constructed (see Figure 3). This structure was built using the stacked drift method, in which the tunnel lining, consisting of concrete filled drifts, is constructed to form a horseshoe-shaped arch. Construction of the arch is followed by removal of the soil core under the arch. Many factors, which include variable soil conditions, overlying historic structures, relatively shallow tunnel depth (less than 6 m), and small permissible ground settlements, have influenced this design and construction approach.

## GEOTECHNICAL CONDITIONS

Geologic profiles are shown on Figures 2 and 3. Soil conditions at Río Piedras Station consist of a thin layer artificial fill at the surface (generally less than 1 m thick), which is underlain by the Hato Rey Formation, an alluvial deposit of silty clays, silts, and clayey to silty sands with layers and lenses of comparatively clean sands. Below the Hato Rey Formation lies the Aquada Limestone, which forms pinnacles or buried karst features that intrude into the Río Piedras Station invert. At these locations the limestone is slightly to moderately weathered.

Within the Hato Rey formation, sands are variable but usually medium to very dense. The consistency of fine-grained soils ranges from medium to stiff. The most characteristic feature of the Hato Rey Formation is the extensive conversion of non-quartz constituents to clay, although the original sand and gravel texture of the deposits is usually visible despite the advanced state of alteration and weathering.

For the various design analyses, the Hato Rey Formation was divided into three soil layers for simplicity:

- Layer 1 - Upper Clays: Characterized as overconsolidated, stiff silty clays. This layer was assumed to be a cohesive soil ( $c=36$  kPa,  $\phi=0$ ).
- Layer 2 - Middle Stratified Zone: Characterized as alternating, highly lenticular layers of clean sand, silty sand, clayey sand, and clay. Layer 2 was also assumed to be a cohesive soil ( $c=58$  kPa,  $\phi=0$ ).
- Layer 3 - Lower Sand: Characterized as interbedded sands, silty and clayey sands, and clays. Contains clean lenses of coarse-grained deposits up to 2 m thick. Cemented horizons occur in horizontal layers 1 to 2 m thick. This layer was assumed to be a " $c-\phi$ " soil ( $c=96$  kPa,  $\phi=37$  degrees).

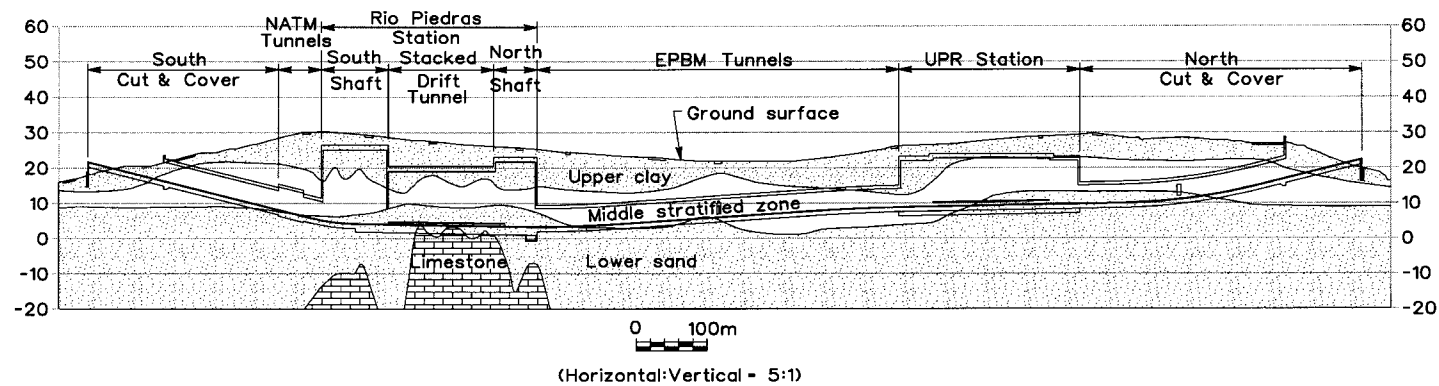


Figure 2. Geologic profile of the Río Piedras contract

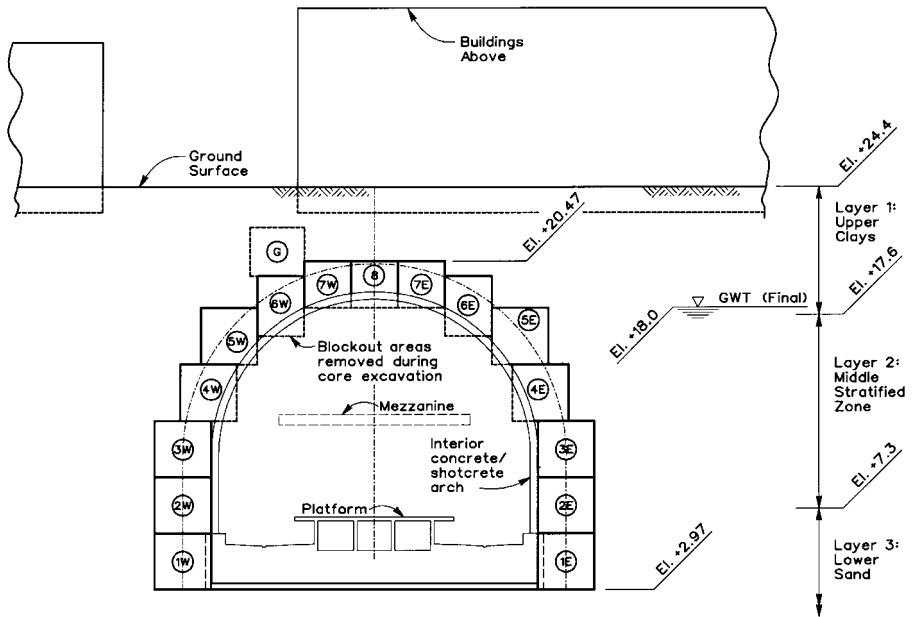


Figure 3. Cross section of stacked drift tunnel (north end)

Observation wells and piezometers indicated a groundwater table at elevation +18 m, which is approximately near the top of drift 5 as shown on Figure 3. Cover over the station crown ranges from 3.9 to 5.8 m.

## PRE-CONSTRUCTION DESIGN

### Owner Supplied Preliminary Design

Tren Urbano engaged a General Management, Architectural and Engineering Consultant (GMAEC) to produce a preliminary design for inclusion with the design-build Request for Proposal (RFP). The GMAEC team members include Daniel, Mann, Johnson, and Mendenhall; Frederic R. Harris, Inc.; Eduardo Molinari y Asociados; and Barret & Hale. The RFP included a suggested design for the Río Piedras Station tunnel that consisted of installing a pre-supported canopy of 67 pipes installed by microtunneling, as shown in Figure 4. It was up to the design-build bidders to either adopt this method or propose an alternate method.

During the first phase of the design-build proposal period, KKZ/CMA investigated the possibility of using the microtunnel canopy concept. Changes to the concept included using larger pipes to reduce the number of microtunnel drives, and interlocking the pipes to maintain alignment. This concept was ultimately rejected by KKZ/CMA for several reasons, including:

- The pipe canopy could not be relied on as a structural arch, and therefore the construction of large steel ribs or a shotcrete structural arch within the canopy would be required. This structural arch had many constructability and cost disadvantages.

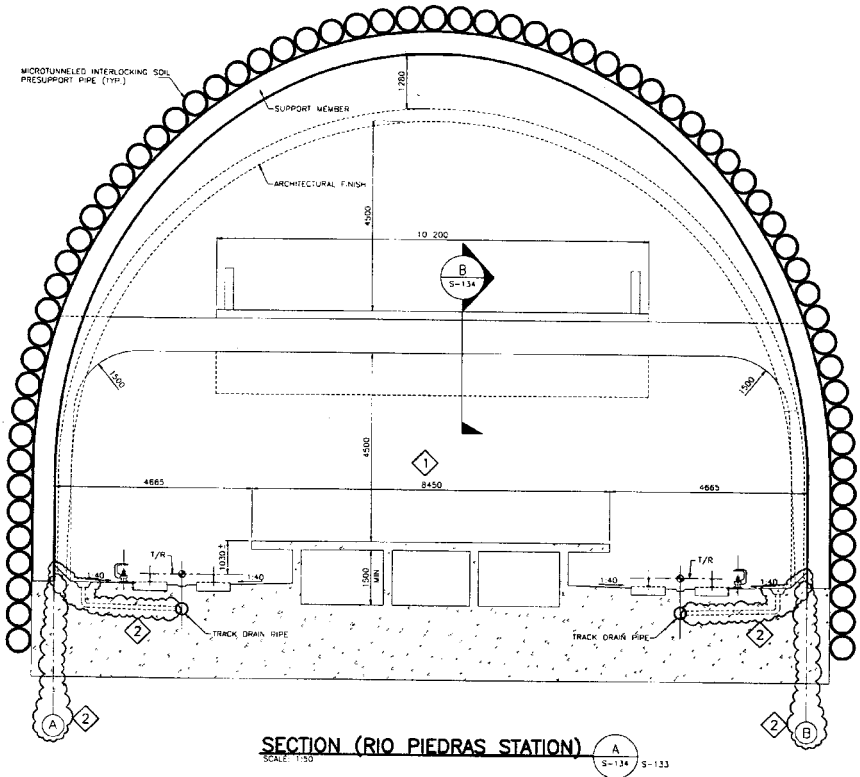


Figure 4. Owner suggested design for mined station

- Microtunnels in the upper portions of the canopy had the risk of encountering buried utilities or other obstacles that could halt tunneling.

Considering these risks in conjunction with a comparative cost analysis resulted in the investigation of alternate construction methods for the Río Piedras Station tunnel, ultimately leading to the selection of the stacked drift method.

### Proposal Design

The stacked drift approach has precedents in the United States in very poor ground conditions for a part of the Eisenhower Tunnel on Interstate 70 in Colorado (Hopper et al., 1972) and for the Mount Baker Ridge Tunnel on Interstate 90 in Seattle, Washington (Robinson et al., 1983; Johnson et al., 1983). Personnel from Kiewit Construction had direct experience with construction of the Eisenhower Tunnel. A preliminary design was prepared for a stacked drift arch consisting of 17 concrete filled drifts, as shown on Figure 5. Square drifts were selected by the contractor for ease of steel set erection, and for fabrication economy. Reinforcement of the arch was shown as continuous threadbar coupled between drifts.

Two design concepts for the Río Piedras Station tunnel were submitted with the initial proposal: construction using the New Austrian Tunneling Method (NATM) or the stacked drift method. Use of the NATM was rejected by Tren Urbano due to the collapse

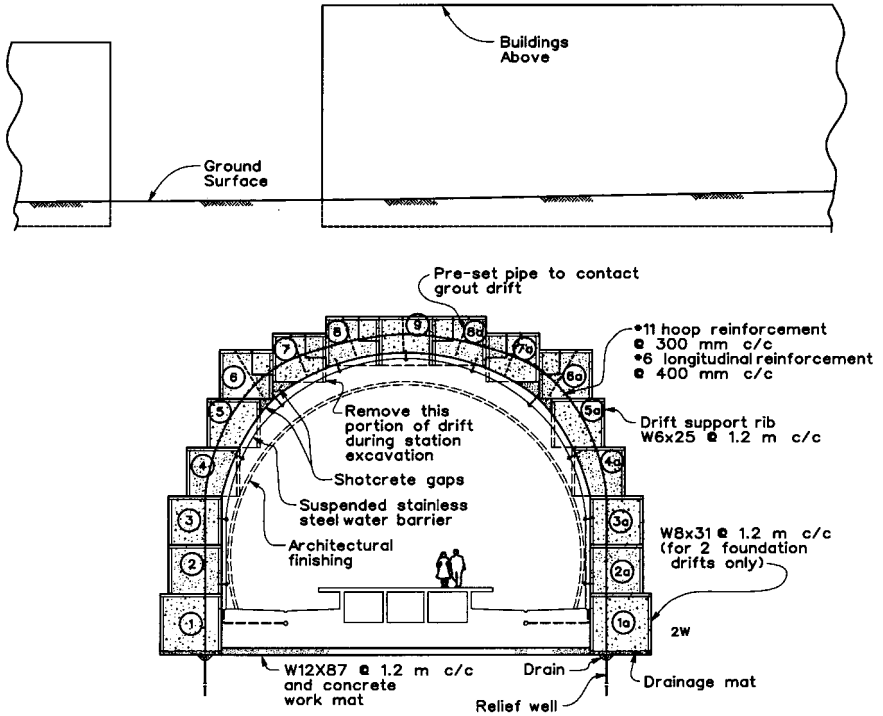


Figure 5. Proposal design for stacked drift tunnel

of the Heathrow NATM works in the United Kingdom, which occurred just before the design-build procurement commenced in Puerto Rico. KKZ/CMA's final proposal therefore included only the stacked drift concept. This proposal design was incorporated into KKZ/CMA's design-build contract in the award of the project.

## DESIGN/CONSTRUCTION PROCESS

### Post-Award Design

The simultaneous award and notice to proceed, along with an aggressive construction schedule, led to the immediate need for detailed design so that materials and equipment procurement could begin. During the first month of the contract, an intensive one-week workshop was held between the contractor and the designer to finalize the construction approach. Drift dimensions were modified so that the number of drifts could be reduced from 17 to 15, thus reducing the construction schedule (see Figure 3). Drifts were sized to achieve a constructable opening yet be small enough to minimize construction impact on overlying structures. Structural requirements for the arch did not control size of the drifts (i.e., for constructability reasons, the drifts were oversized with respect to minimum structural requirements). For the lower drifts forming the sidewall, a nominal size of 3 m by 3 m was selected. In the upper drifts, blockouts were specified where the drift encroaches on the clearance required inside the tunnel.

Various methods for drift support were also determined so that the contractor had a menu of options to choose from during construction. In addition, dewatering from the surface (where access was available) was selected as a means to stabilize soils in drift headings. Finally, threadbar reinforcement of the arch was eliminated, with the understanding that the steel sets used for drift support would serve as arch reinforcement, with connection details between drifts to be finalized before procurement deadlines.

Although it was predicted that the completed stacked drift arch structure would be stiff and therefore result in minimal ground movements during excavation of the core, settlement during excavation of the individual drifts was a significant concern. It was concluded that compensation grouting would be the best way to mitigate settlement that was the inevitable result of drift excavation. To minimize surface disruption, it was decided to excavate a drift solely for the purpose of installing an array of sleeve-ports and conducting grouting. The compensation grouting program is described in detail by Morrison and Madsen (1999).

**Summary of Design Approach and Analyses.** The stacked drift tunnel could be considered to behave like an unreinforced arch where loads are accommodated as compressive forces (thrust) across individual arch segments. As an arch it could be stable without steel reinforcement. This was the case for the Mount Baker Ridge Tunnel. The differences between these two projects, however, are significant: Mount Baker Ridge has a full circular ring, is situated in clayey soils with substantial cover, did not have appreciable groundwater inflow potential, and has less concern for overlying structures. The Río Piedras Station arch is not circular but horseshoe in shape, has significant groundwater concerns, and has occupied commercial and residential structures directly overhead with low cover. In particular, the Río Piedras Station arch, due to its low cover, is subject to unbalanced ground loading in both the short-term (construction stage) and long-term conditions.

Since the arch must be structurally effective before the soil core is removed, it was designed to accommodate loads associated with both short-term and long-term conditions. In conventional tunneling, most of the elastic and some of the inelastic ground deformations occur before tunnel supports are installed and become structurally effective. This results in arching of soil, thus reducing load on the tunnel supports. Because of the construction method planned for the Río Piedras Station, substantial soil arching cannot develop. Thus, regardless of how much the stacked drift tunnel might actually function as an unreinforced arch, reinforcement was deemed to be appropriate for this situation.

Four structural models were used to analyze the behavior of the stacked drift tunnel under the anticipated loading conditions:

1. **Beam-Spring Model:** Structural frame analysis where the stacked drift tunnel and the surrounding ground are modeled as a network of beams and springs. Elastic material properties were used in this model.
2. **Three-Hinged Arch Model:** Simplified representation of the arch as a statically determinant structure, with a hinge assumed at the centerline of the arch at the crown. The three-hinge arch model was used to compute the reactions at foundation and invert slab level. By definition, the results of this analysis are not dependent on material properties or the relative stiffnesses of the elements of the structure.
3. **Soil-Structure Interaction Model:** Finite difference continuum model of soils and structural elements that simulates excavation sequence. Non-linear soil properties were used. This model was utilized to indicate the overall behavior of the structure.

4. **Finite Element Structural Model:** Similar to the Beam-Spring Model, except that structural elements are modeled as a continuum rather than beams. This model was used to perform a racking analysis.

Results from the racking analysis showed that the stacked drift tunnel has adequate structural capacity for differential settlement and for seismic waves propagating perpendicular to the tunnel provided that the drift 3-4 and drift 6-7 interfaces are reinforced. Reinforcement in the sidewall and arch that was provided for static loading conditions will have sufficient structural capacity for earthquake loads. Analyses also indicate that the stacked drift tunnel is structurally adequate to withstand axial and curvature strains resulting from seismic waves propagating parallel or nearly parallel to the tunnel.

**Predicted Lining Displacements.** Figure 6 presents the displacements at various points around the excavation calculated by the soil-structure interaction model for a "base case." Several cases with varying loading conditions and geotechnical parameters were analyzed. The deformation pattern in Figure 6 consists of bottom heave accompanied by inward movement of the sidewall and an initial downward crown displacement that reverses, as the sidewall moves in, to produce a net upward movement of the crown at the completion of the station excavation.

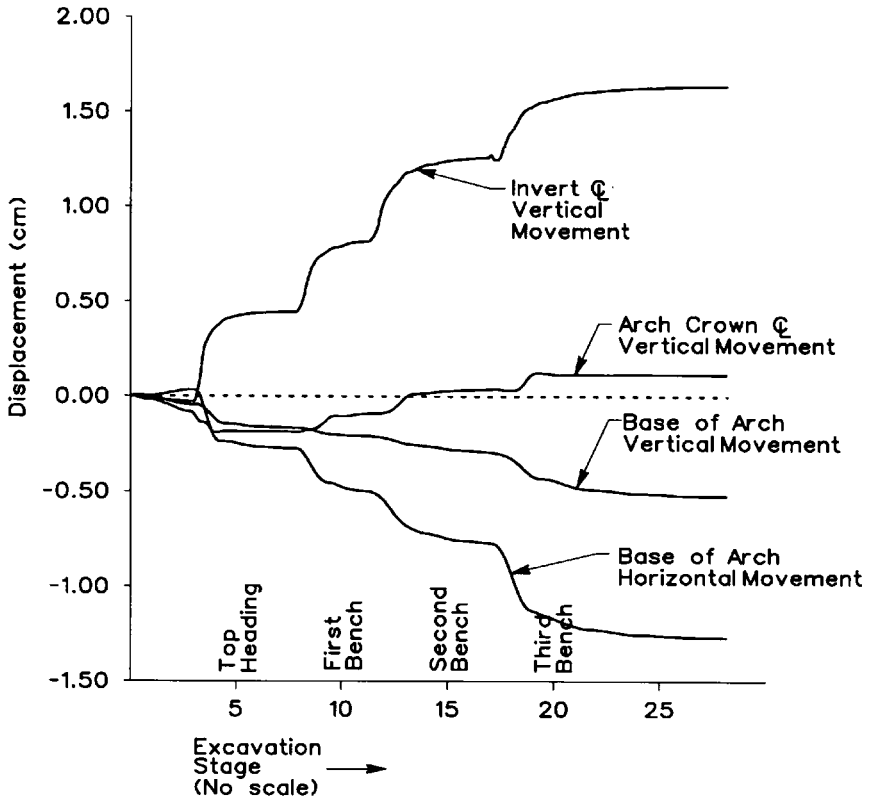


Figure 6. Predicted lining displacements

For the base case, maximum predicted arch displacement at the crown was approximately 2 mm downward during core excavation, followed by a net upward movement of 1 mm after hydrostatic pressures are reestablished. The maximum predicted horizontal movement at the base of the sidewall was approximately 13 mm inward. The construction invert was predicted to heave 16 mm, while the permanent invert was predicted to heave approximately 70 mm after reestablishment of the hydrostatic conditions. The maximum predicted surface settlement was 4 mm.

The displacements calculated by the soil-structure interaction model were not be taken as exact predictions of the displacements of the concrete arch and surrounding ground, but rather as indicative of the overall mode of deformation. The displacements do not represent the true total displacement of the ground or arch because the modeling procedure assumed the entire arch was installed without any ground relaxation.

It was concluded from these analyses that arch is very stiff in relation to the soil, which results in stress redistribution around the arch being minimal. Predicted soil stresses acting on the arch were close to the at-rest, or pre-construction, values.

**Structural Design.** Initially, structural steel sections were relied on to create a reinforced concrete structure. Where possible, parts of the steel sets required for individual drift construction were used for reinforcement of the overall stacked drift structure. Sidewall reinforcement was to be provided by the sidewall drift posts, while portions of the crown reinforcement of the arch were to be provided by drift cap beams. Short-term and long-term loading conditions required reinforcement on different faces of the structure.

Shear transfer was of particular concern between drifts 4-5, 5-6, and 6-7. In the case of the interface of drifts 4-5, a rough joint and full contact was required to achieve transfer of horizontal loads at this critical location. The vertical joints between drifts 5-6 and 6-7 must be capable of transferring shear from vertical loads. This required radial construction joints.

Requirements for concrete cover over the steel reinforcement vary. Long-term conditions require sufficient embedment for bond and corrosion protection. For the short-term condition, full embedment is not required for corrosion protection, however sufficient embedment is required to achieve bond between the steel and the concrete.

Certain connections between the various structural steel members acting as arch reinforcement must develop the full strength of the member in tension. Other connections needed only to be partial-strength connections.

### **Changes to Construction Sequence**

The design submitted in KKZ/CMA's proposal included a construction sequence for the drifts whereby all drifts are constructed from the bottom up. Shortly after award of the project, KKZ/CMA submitted a sequence of 10 drifts constructed from the bottom up and 5 drifts constructed from the top down. Over the first six months of the project, numerous discussions were held between Tren Urbano, GMAEC and KKZ/CMA engineers regarding the drift construction sequence. The contractor's preference was to keep the drift construction sequence flexible, including having the option of constructing the drifts completely top-down. In January 1998, Tren Urbano directed KKZ/CMA to proceed with drift construction in a bottom-up sequence per the proposal design. This required KKZ/CMA to quickly sink a construction shaft at one end of the station (North Shaft) in order to gain access to the bottom of the arch and begin excavation of drifts 1E and 1W.

Construction performance of the lower six drifts convinced KKZ/CMA and Tren Urbano that constructing some of the upper drifts top-down was feasible and would

help to reduce the schedule. The as-built drift construction sequence was ultimately very close to the sequence of 10 drifts bottom-up and 5 drifts top-down that was originally submitted after award.

### Changes to Drift Excavation and Support Methods

The first drift to be excavated was the grouting gallery (drift G), located closest to the ground surface and above the observed groundwater table. Ground conditions were predominantly hard clays that could be excavated by hand with power-spaders (Figure 7). The position of drift G above the groundwater table required the use of a lagging material that would not rot away and leave a potential void space in the long-term after the tunnel was backfilled. This was also required for drifts 5, 6, 7 and 8, which are also above the groundwater table.

Having the option of varying the means and methods was an advantage throughout drift excavations, and enabled the contractor to react to varying ground conditions and equipment availability. In the lower drifts, tunneling conditions ranged from firm ground to raveling and flowing ground which presented great difficulties for construction. Tunnel digging machine (TDM's), specially designed and fabricated by



Figure 7. Hand excavation in drift G

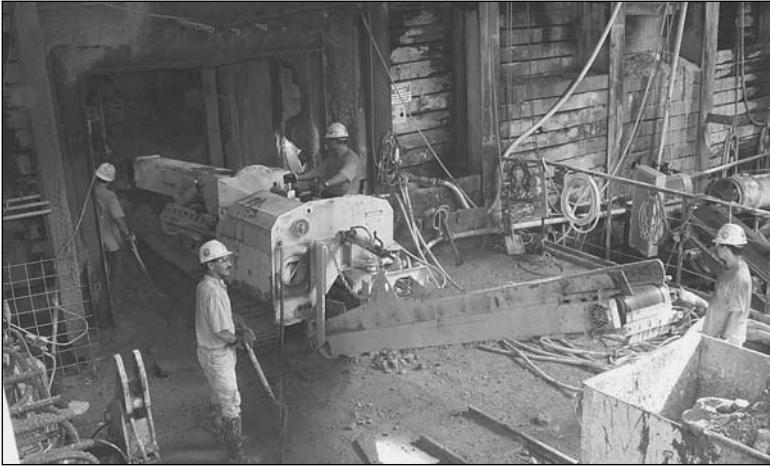


Figure 8. Technicore tunnel digging machine



Figure 9. Square drift shield

Technicore for this project, were used for much of the drift excavations (Figure 8). The various methods for the excavation and support of each drift are listed in Table 1.

The changes to the excavation and support methods were triggered by obvious factors such as production but also by the performance of each method in relation to ground loss and resulting surface settlement. The most favorable production rates were achieved by the TDM and shotcrete combination, relying on full equipment mobilization. The square shields (Figure 9) created a safe working environment, particularly with their ability to breast hydraulic face doors against flowing ground as shown in Figure 10. The shields however, proved difficult to steer on top of the



Figure 10. Flowing ground conditions in drift 4E

Table 1. Drift excavation and support methods

Drift	Excavation Method	Support Methods	Lagging Type
G	Hand excavation, overshot mucker	Steel sets, continuous advancing headboards	Corrugated steel
1E/1W	Hand excavation, conveyors	Steel sets, continuous advancing headboards	Timber
2E/2W	Hand excavation, Tunnel Digging Machines	Steel sets, continuous advancing headboards	Timber
3E/3W and 4E/4W	Hand excavation, Tunnel Digging Machines	Square shields, steel sets	Timber
5E/5W and 6E/6W	Tunnel Digging Machines	Steel sets, shotcrete, pipe spiling	Shotcrete
7E/7W and 8	Hand excavation, overshot mucker	Steel sets, shotcrete, pipe spiling	Shotcrete



Figure 11. Forepole headboards

underlying drifts, and had a long set-up time. The advancing forepole headboards consisted of interlocking 3.7 m long channels advanced by hydraulic jacks, as shown in Figure 11.

### Changes to Structural Design During Construction

During construction, direct observation of foundation materials below the south end of drifts 1W and 1W indicated that this portion of the stacked drift tunnel rests on an approximately 2 m thick layer of silt/sandy clay, below which is a 2 to 4 m thick layer of dense, hard cemented sand/clayey sand. The clay layer was characterized as soft to medium stiff. The dense sand exhibited significant cohesive and frictional strengths.

To provide sufficient bearing capacity and minimize settlements in this area, KKZ/CMA decided to excavate portions of the softer clay material and replace this excavated soil with concrete. A series of 2.0 m long by 0.7 m wide by 2 m deep concrete piers were constructed below drifts 1E and 1W, spaced at 2.4 m apart (one between every other drift set), and founded on the dense layer below (Figure 12). It was assumed that the soil between the concrete piers would provide negligible support. Using these foundations, a bearing capacity factor of safety of between 2.1 and 3.8 can be expected.

The most significant design change during construction was the change of the arch reinforcement materials. As noted above, reinforcement of the arch in the proposal stage consisted of continuous threaded rebar coupled between drifts. In the post-award design it was believed that changing the reinforcement to utilize the drift support steel would have advantages such as eliminating the time required to install rebar, eliminate rebar cost, and eliminate the critical lead time for rebar. However, as the design of drift 1E and 1W progressed, it became clear that this solution had some serious drawbacks. A full tension connection between the posts and the cap beams required the use of heavy butt plates, web stiffeners and high strength bolts. This substantially increased the cost per pound of "reinforcing" steel. To transfer the tension through the cap beam, a W8x67 was required, as opposed to a W8x24, which was needed to carry the ground loads. This created an installation challenge that also required an involved Q/C program for bolt tightening. After reviewing the drift 1E/1W operation and cost, the decision to



Figure 12. Foundation pier pit in drift 1E

change the reinforcement design back to threaded rebar was made. Details of the arch reinforcement were different than envisioned at the proposal stage, a result of the experience gained during construction of the lower drifts (Figures 13 and 14). The operations/cost analysis and subsequent redesign was accomplished in less than two months. The change proved beneficial and was one of the better experiences with the design-build approach.

### CORE EXCAVATION

The excavation of the Río Piedras Station core was restricted in many ways due to the ongoing structural work in the shafts at either end of the stacked drift tunnel. There were also a few unknowns that could have slowed or even halted the excavation of the 43,000 loose m<sup>3</sup> of soil. The soil and groundwater conditions together with the possible deflection of the arch were the major concerns.

The excavation was divided into three lifts of approximately equal height. The top heading was launched from the mezzanine level at the south end and excavated north. The direction of excavation was reversed for the bench and invert excavation which posed a change to the design, but allowed for better flow of subsequent activities.

The encountered soil conditions allowed for fair stand-up time, since no water permeated through the drift canopy. In addition, limestone in the invert did not affect the excavation significantly. Continuous perforated drain pipe preinstalled in the invert of drifts 1E and 1W performed as predicted and acted as a cut-off for groundwater outside the arch. As a result, no piping or heave was observed during excavation.

Removing soils from the inside of the core and preparing the surface for waterproofing not only consisted of excavation but also included removal of the protruding drift steel, various formwork materials, blockouts and sacrificial lagging. This work involved cutting and removing 1,984 pieces of W8x24 weighing 77,620 kg; excavating and removing of 790 m<sup>3</sup> of Styrofoam blockouts; removing of 667 sheets of 75 mm plywood, excavating of 219 m<sup>3</sup> of lean mix concrete blockouts; and removing of

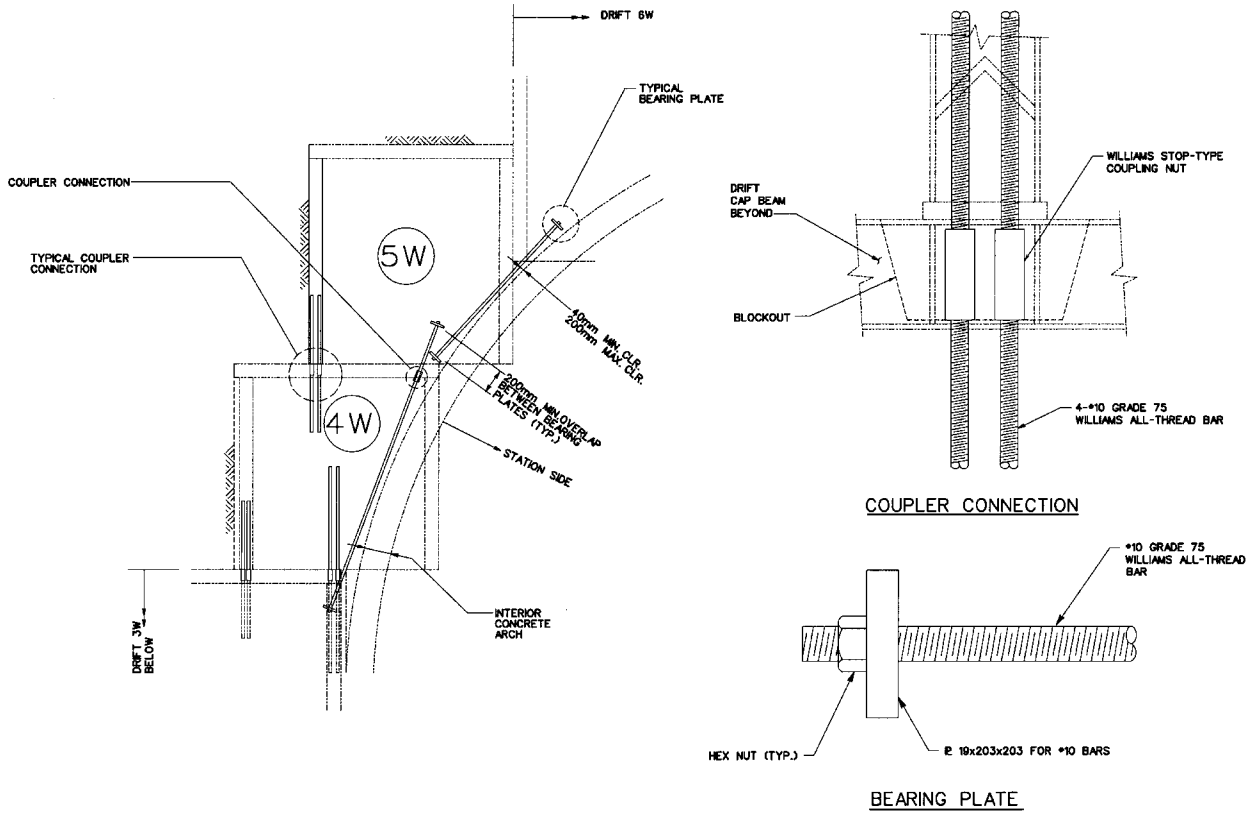


Figure 13. Modifications to reinforcement details



Figure 14. Threadbar reinforcement in drift 7W

2,705 m<sup>2</sup> of 100 mm timber lagging. Core excavation was therefore more of a materials handling challenge than an excavation challenge.

Figure 15 shows the top heading during the turn-under phase, with TDM's excavating and a small electric loci with a side dump car for mucking. A CAT 933 hauled muck to the side dump. Figure 15 also illustrates the drift 8 support steel protruding into the core envelope with lean mix as block-out; close to the face on the left side are the stringers and plywood that made up the form in drift 6W. This last type of block-out was the easiest to install and remove.

The final design called for the lower bench and fine grading to be executed in rounds no longer than 2.4 m, starting from the end of the previously placed construction slab and extending to the toe of the advancing lift. As planning of the core excavation progressed it became clear that the 2.4 m length was too stringent. The design was changed to an observational approach, where the concerns regarding sidewall movement were addressed by implementing a convergence monitoring program. The convergence program and observations of the subgrade by professional geologists during fine grading would provide information to base decisions whether to extend or shorten the lift length. The new plan was executed and resulted in 8 m sections being placed daily without any significant movement of the walls, as shown on Figure 16.

Figure 17 shows a CAT 325 excavating the lower bench while a Liebherr 900 moves a bull hose during placement of a 7.6 m section of the construction slab. The texture of drifts 1 through 3's vertical walls is a product of the timber lagging, compared with the Styrofoam blockout for drift 4. The Styrofoam was more of a challenge to remove than the wood lagging.



Figure 15. Excavation of first lift

Part of the core excavation instrumentation plan included monitoring of vertical crown movement. The only movement measured took place shortly behind the face of the top heading excavation, as the arch took load. This movement was on the order of 2 to 10 mm, which was within the range of movements predicted by the various soil-structure model cases. No further vertical movement of the crown occurred during the remaining excavation phases. Concerns regarding differential settlement and settlement of the entire structure as the core was excavated proved to be unfounded.

Work subsequent to the core excavation is shown in Figure 18, where the stacked drift walls have received a smoothing shotcrete layer and PVC waterproofing installation is ongoing together with bulkhead and rebar installation for the final structural invert. Note the keyways in drift 1 to prevent the final invert from floating due to hydrostatic pressures.

## CONCLUSIONS

Constructing of the stacked drift tunnel by the design-build approach was a challenge. The numerous changes to the design as the construction progressed were mainly the result of the various stakeholders' personal opinions of how the structure should be built. Unfortunately, there were very few preceding projects to refer to, and therefore many of the design and construction approaches were devised along the way. It would not have been possible to successfully complete this project had there had not been a good understanding between the designer and the contractor of the others' limitations and abilities.

From a design standpoint, it was demonstrated that this atypical tunnel structure could be prudently designed using basic principals of geotechnical and structural engineering. Furthermore, it was concluded that numerical models were useful in

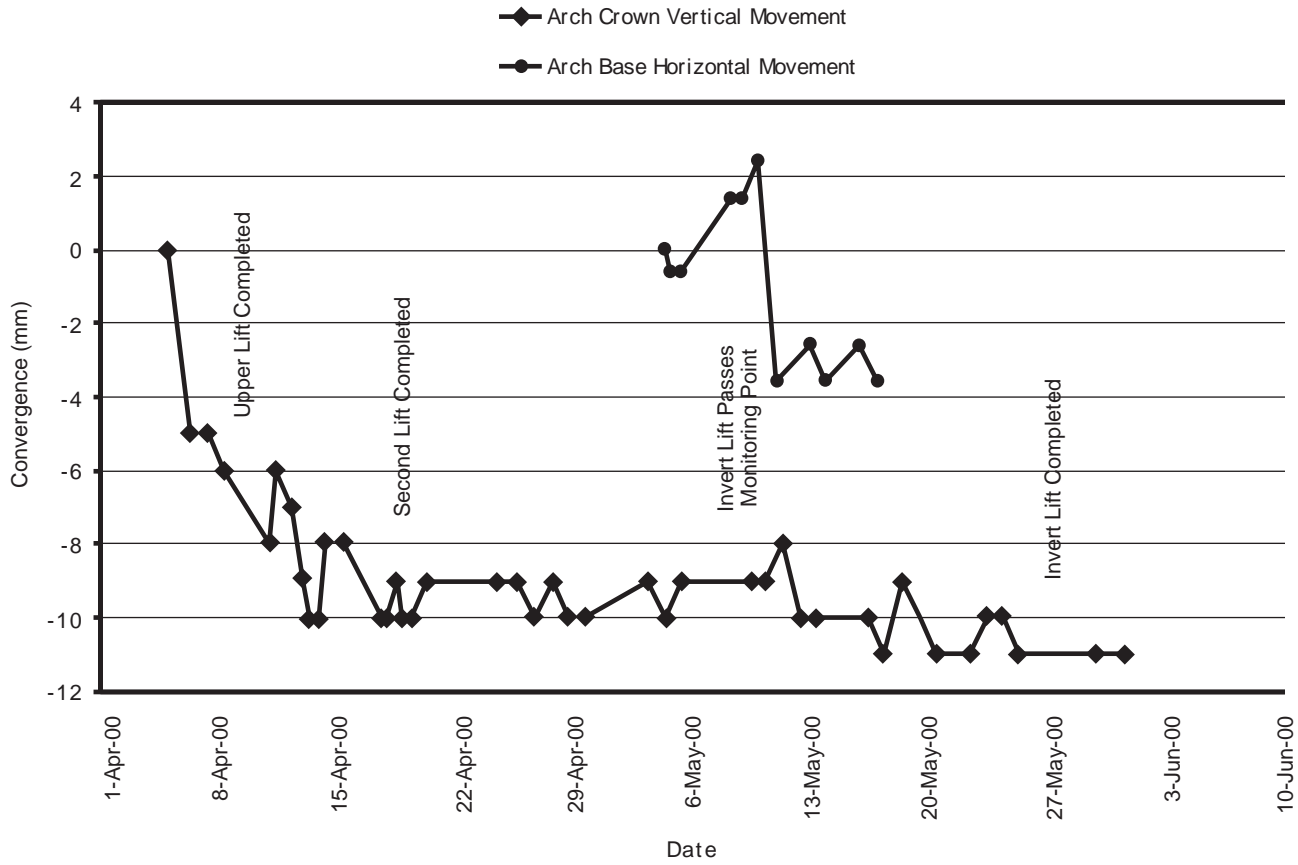


Figure 16. Lining displacements during construction



Figure 17. Excavation of lower lift



Figure 18. Construction of final invert and interior arch

predicting structural behavior, even if no precedent for such a structure existed. Finally, the importance of completing the design in advance of procurement and having the ability to quickly change the design during construction cannot be over emphasized. This schedule awareness was key to maintaining a good relationship with the contractor under the design-build approach.

From a construction standpoint, the following constructability changes would be recommended for similar projects: arching the individual drifts (i.e., as opposed to a flat crown); use of shotcrete for drift support; use of threaded rebar (with couplers and anchor plates) as reinforcement; use of plywood and build-up forms as continuous blockouts; and perform preconsolidation grouting in soft soils that are subject to raveling or flowing. Since almost all surface settlement was a product of excavating the individual drifts and not from the final structure settling and/or deforming, it would be

worth considering excavating any multiple-drift structure from the top down. Last but not least, the construction engineer must recognize the importance of planning for such a large and complex underground structure.

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