DESIGN AND CONSTRUCTION OF THE SR 836 FLYOVER BRIDGES

SCOTT W. DEAN, P.E.

BIOGRAPHY

Scott Dean is a Bridge Project Manager in the Orlando office of HNTB Corporation and served as the structural design manager for this design/build project. He has worked for HNTB for nine (9) years. He graduated from the University of Florida with a BSCE in civil engineering and an ME in structural engineering.

Mr. Dean has more than 12 years of experience designing bridge structures, primarily complex bridges including steel box girders, concrete segmental bridges and aircraft related structures. He has been either the design team leader or the senior bridge designer for a number of signature structures, major multi-level interchanges and design-build proposals.

SUMMARY

This project is one of three major design/build projects recently undertaken by the Miami-Dade Expressway Authority. Now complete, this $39 million project included the construction of three new bridges and one bridge widening.

The two largest bridges in the project are both steel box girder bridges. The 543-ft long Bridge No. 11 and the 528-ft long Bridge No. 12 each utilized several innovations, including post-tensioned integral concrete diaphragms and spread footing foundations for the piers and abutments.

This paper describes the design and construction of these two complex structures, including the architectural constraints stipulated by the Owner and the innovative solutions selected by the design/build team. In addition, the paper will detail the aesthetically-enhanced sign structures required by the Owner.

DESIGN AND CONSTRUCTION OF THE SR 836 FLYOVER BRIDGES

By
Scott W. Dean, P.E.

PROJECT DESCRIPTION

As Miami-Dade County continues its rapid growth west towards the Everglades, local transportation officials find themselves in need of significant capacity improvements to accommodate the increase in vehicular traffic. To address this need, the Miami-Dade Expressway Authority (MDX) has embarked on an expressway expansion program that consists of three design-build contracts. The goal of the expansion program is improved service and heightened aesthetics. This project is the first of the three design-build contracts, is approximately 2.6 miles long and includes the SR 836 Flyover bridges.

Within the project limits there are four bridge sites and several innovative sign structures. Three of the bridges in the project are completely new structures employing aesthetic features such as concrete formliners for piers and retaining walls, smooth and flowing structural lines, unusual color schemes, and inlaid tile. The two largest and most innovative bridges in the project, Bridge Nos. 11 and 12, consist of long-span steel box girder superstructures with post-tensioned integral concrete diaphragms, aesthetically enhanced piers founded on spread footing foundations, and spread footing abutments.

Prime contractor Condotte America, Inc. teamed with HNTB Corporation for this design-build project. By electing to utilize innovative construction methods and structural elements, the project team won the project with a final bid of $36 million, a construction schedule of 775 days, and a score of 83 on the technical proposal. Four months into the contract, the Owner elected to add a lane to the eastern portion of the project, which increased the contract value to $39 million and added 122 days to the schedule.

OWNER'S REQUIREMENTS

Because the expansion plan undertaken by MDX is playing out on the stage of one of the most visible and well-traveled east-west arteries in Miami, the Owner required the use of advanced architectural details that were very specifically defined and described in the Owner's proposal package. For the superstructures of the bridges in the program, MDX stipulated that all long-span bridges utilize either concrete or steel box girders with a closed bottom soffit. For the shorter span bridges in the program, MDX stipulated that the bridges utilize precast Florida U-beam superstructures. MDX allowed the use of only three types of piers in the program: single round columns, rectangular wall piers, and rectangular piers with flared tops. For each of the many bridge sites in the three projects of the expansion program, MDX defined which of the three pier types were allowed.

Other architectural features required by the Owner included:

- textured form liners for piers and MSE wall panels (four from which to choose)
- inlaid colored glass block or ceramic tiles in some piers, as defined by MDX
- a 21-color paint palette from which the colors for all elements must be selected
- extend MSE walls a minimum of 200 feet from the bridges

Early in the proposal process, Condotte made the decision to satisfy the closed bottom soffit requirement for the long bridges through the use of steel box girder superstructures. This decision was made after comparing the respective advantages and disadvantages of steel box girder and concrete segmental box girder bridges with the site conditions and bridge geometries. It was apparent to Condotte that the variation in bridge lengths, widths and curvatures, combined with the relatively small overall quantity of bridges precluded the use of concrete segmental box girders. The decision to use structural steel was further supported by the relative ease by which the girders could be erected over the critical active surface roadways.

BRIDGE NO. 11: RAMP F OVER H.E.F.T.

EXISTING CONDITIONS

Prior to completion of this project, traffic exiting westbound SR 836 to travel south on the Homestead Extension of Florida's Turnpike (HEFT) crossed over the HEFT on an existing four-span AASHTO beam bridge. The existing bridge carried two lanes of traffic over the four existing lanes of the Turnpike. However, future expansion plans for the Turnpike include the eventual widening of the roadway to nine lanes with four additional connector/distributor (C/D) lanes. The existing flyover simply could not accommodate this much expansion beneath it, so replacement of the bridge became a necessity.

NEW STRUCTURE

By far the most complicated bridge in the project, Bridge No. 11 replaces the existing bridge over the HEFT by carrying the expanded three lanes of westbound SR 836 traffic over the mainline HEFT lanes. The bridge is located just south of the existing bridge and is 543 feet long with a span arrangement of 140'-249'-154'. These spans are configured such that the bridge piers are located between the mainline HEFT lanes and the respective future HEFT C/D roadways. The Owner's proposal package recommended that a concrete segmental box girder bridge be utilized for this structure. However, because the proposal package did not restrict the design-build team to the recommended bridge type, nor to the recommended span arrangement, HNTB and Condotte chose very early in the proposal process to change some of the bridge parameters. First, as mentioned previously, Condotte decided that it would be more cost-effective for them to utilize a steel box girder solution. Second, the design-build team fine-tuned the bridge length by slightly shortening the main span and one of the end spans to optimize the span arrangements.

Figure 0. Elevation of Bridge No. 11

The 63'-1" wide bridge utilizes three steel box girders in the cross-section. The girders are 12 feet wide at the top flanges with 8'-10" deep webs that are inclined on a 4:1 slope. For its entire length, the bridge is on a curved alignment with a baseline radius of 1,146 feet and a constant cross-slope of 8.3% (see Figure 1). As constructed, the final unit cost of the bridge was approximately $130 per square foot.

The Owner's design criteria included the stipulation that all designs adhere to the most current design requirements of the Florida Department of Transportation (FDOT). At the time this bridge was designed, FDOT required that all bridges designed in the state use the AASHTO LRFD design method with HL-93 live loading. FDOT's only exception to this rule applied to curved steel bridges, for which the AASHTO LFD design method was to be used in conjunction with the AASHTO LFD curved girder specifications. As design progressed, AASHTO released the second edition of the LFD curved girder specifications, and HNTB elected to incorporate as many of the new requirements as was practicable.

As part of the Owner's aesthetic requirements mentioned previously, our choice of piers for this bridge was restricted to either a single circular column or a narrow rectangular pier with a flared top. Because each of the two pier locations selected by the team will ultimately be located between the HEFT mainline lanes and the HEFT C/D lanes, the space in which to place a pier was limited to approximately nine feet. Because the HEFT roadways are skewed to the bridge axis by nearly 54 degrees, use of the flared pier would extend the top of the pier out into the traffic lanes and violate the vertical clearance of the roadway. In order for this pier type to be utilized, the piers would have to be rotated to match the skew of the roadway, however this would have resulted in skewed box girder framing. Early team discussions indicated that the contractor preferred not to fabricate nor erect skewed box girders because of the added costs due to fabrication complexities and because of the difficulties that often occur during deck casting due to differential deformations. Therefore, the single round column option was selected to be used in concert with non-skewed supports. Use of a traditional pier cap with the round column was not possible due to the vertical clearance requirements over the HEFT lanes adjacent to the pier, so HNTB and Condotte elected to utilize a concrete diaphragm that was integral with the box girders (see Figure 2).

Figure 2. Integral concrete diaphragm at Pier No. 3

INTEGRAL DIAPHRAGM

Doubtless the most complex and innovative feature of this bridge is the use of these post-tensioned concrete integral diaphragms at the piers. Because the pier column is only eight feet in diameter, two of the three box girders fall outside the limits of the pier column. To solve this problem, HNTB and Condotte chose to use an

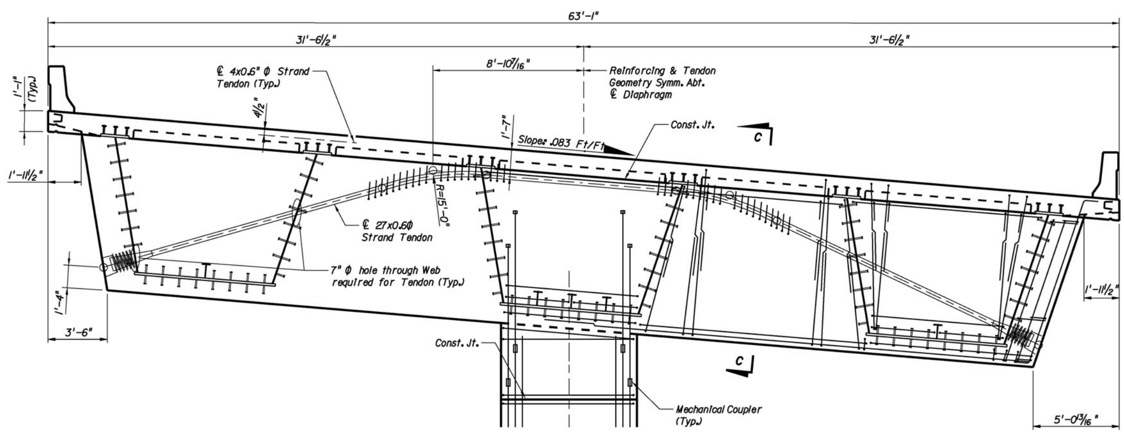

Figure 3. Post-tensioning within integral diaphragm.

integral diaphragm that cantilevered out from the pier to capture the girder forces from the outer girders and transfer the reactions back to the pier. Given the size of the bridge, the loads imparted to the diaphragm required extensive post-tensioning that was installed in two phases (see Figure 3). First, to support the self-weight of the girders, the deck slab concrete, and the diaphragm itself, five 27-strand post-tensioning tendons in a draped configuration were utilized within the diaphragm. These draped tendons pass through 7” diameter holes in the box girder webs. After the deck casting was complete, 12 flat tendons, each containing four strands, were installed above the diaphragm in the deck slab itself to resist the vehicular live loads and the self weight of the concrete traffic railings. In order to properly camber the box girders for the final predicted bridge shape, the diaphragm itself was slightly cambered for the anticipated deflection of the tips of the diaphragm due to the weight of the girders and the deck concrete.

Figure 4. Pier reinforcing extending into integral diaphragm.

Because the pier is small relative to the width of the bridge, HNTB also elected to use an integral connection between the diaphragm and the pier itself. This complicated the detailing of the reinforcing steel in the diaphragm; the dense forest of column reinforcing, shown in Figure 4, shared the same space with the numerous diaphragm shear stirrups and the five, 5 1/2" diameter post-tensioning tendons. During the design process, it became evident that in order to minimize the large overturning moments acting on the pier footings, it was necessary to cast the deck slab concrete in the three positive-moment regions of the bridge prior to casting the integral diaphragms and the integral connections to the pier columns. This served to redistribute the self-weight bending moments prior to “locking” them into the substructure with the integral pier connection. The net effect of this was to preload the columns with dead load moments that occur in the opposite direction as the live load moments, thus limiting the net total moment in the columns and placing less demand on the spread footings.

STRUCTURAL STEEL

The box girders were fabricated by Tampa Steel Erecting Company using Grade 50 steel and were detailed by Tensor Engineering. Early discussions with Tampa Steel indicated that it was their preference to utilize Grade 50 steel throughout the bridge rather than introducing Grade 70 steel into portions of the bridge. In addition, input from the fabricator helped HNTB define field splice locations by identifying the maximum length of curved girder section that could easily be transported from the fabrication yard. For this bridge, the longest piece of the five lengths of girder sections was just over 149 feet.

Like other steel box girder bridges, each box girder in this bridge utilizes all of the traditional steel framing elements, including internal cross-frames, intermediate web stiffeners, longitudinal bottom flange stiffeners and top flange lateral bracing. The post-tensioned integral diaphragms and the integral connection of the center box girder to the tops of the piers introduced several detailing challenges for the box girders.

First, in the center box girder (BG-B), several large holes were provided in the bottom flange to allow the column reinforcing to protrude into the tub of the box girder for the integral connection (see Figure 4). Two short sections of bottom flange stiffeners were added to the girder to help transmit the bottom flange compression through this region where the sectional area of the bottom flange was significantly reduced.

Second, in all three box girders, to maintain girder stability during construction, the top flange lateral bracing and the bottom flange longitudinal stiffeners were designed and fabricated assuming that they would be cast into the diaphragm concrete (see Figure 5).

Figure 5. Lateral bracing and longitudinal stiffeners cast into the diaphragm.

Figure 6. Lateral bracing bolster at the integral diaphragm.

However, because the lateral bracing in the center box girder is in the same plane as the post-tensioning tendons, the diagonal lateral bracing member that passes through the diaphragm was positioned beneath the post-tensioning using a bolster fabricated from a 15” length of W10x45 (see Figure 6).

Between the box girders, there are 6-ft deep welded plate diaphragms spaced at approximately 50-foot intervals. In addition, within each box girder there are full-depth welded steel plate diaphragms at each end of the bridge and 10 feet from the centerline of each pier, measured into each end span. The latter locations correspond to the location of the main temporary box girder supports that were used during the erection of the girders and during casting of the diaphragm concrete.

TEMPORARY SHORING

The selection of the integral diaphragm and the integral connection of BG-B to the pier required that the box girders be erected prior to completion of their permanent support elements. Therefore, it was necessary to temporarily support the box girders. These supports were placed near enough to the permanent piers that the structural behavior was not significantly changed, but far enough from the piers that there was sufficient space to erect the temporary formwork necessary to cast the diaphragm. HNTB designed a permanent interior steel plate diaphragm to provide stiffness and load-transfer capability for the box girders in the temporary condition when the girders were supported at a location other than the permanent pier. The girders were analyzed for this temporary condition and all stresses and forces were within the allowable values. In addition, the steel box girder cambers were adjusted to account for the temporary support conditions; because the girders are first supported 10 feet from the permanent location, a small camber was introduced in the girders at the permanent pier locations to counter the slight deformation anticipated over that 10-ft distance during construction.

Figure 7. Temporary shoring at pier.

Condotte contracted with a specialty engineer, Construction Engineering Consultants (CEC), to design the temporary support system for the box girders (see Figure 7). The system designed by CEC utilized two 42” diameter steel pipe columns at each exterior box girder to transfer the girder reactions down to temporary concrete footings cast specifically for the shoring system. At the center girder, the shoring system used two W14x176 columns. Two 430 ton hydraulic jacks, one beneath each girder web, were used to support the box girders at each temporary column. Transverse forces from wind loads and construction activities were transmitted from the exterior pipe columns to the temporary foundations through W14x73 diagonal struts. The W-section columns beneath the center box girder were braced back to the permanent concrete pier using 8-inch diameter steel pipes and temporary post-tensioning bars. This bracing was necessary in order to transmit to the stout permanent support any longitudinal forces that developed during construction, including those from wind loads, thermal loads, and construction activities.

Figure 8. Temporary horizontal bracing.

During the time that the concrete diaphragm was being cast and was curing, any differential longitudinal movement of the box girders relative to one another would have been detrimental to the diaphragm. At this critical time for the diaphragm, differential movement could cause excessive cracking in the concrete as the diaphragm was "racked" by this movement. To counter this, HNTB designed a temporary horizontal bracing system between the box girders that ensured that all three girders would move together due to longitudinal thermal expansion and contraction. This bracing system consisted of post-tensioning bars diagonally linking the top flanges of adjacent box girders much like the diagonals in a truss (see Figure 8).

SUBSTRUCTURE

The other key innovation utilized by the Condotte/HNTB Team for this structure is the use of shallow, spread footing foundations in lieu of pile-supported foundations. One of the unique geologic features of the Miami area is the close proximity of a hard limestone bearing strata to the natural ground surface. Often only a few feet below natural grade, this dense rock layer provides many challenges to traditional construction. Primarily, bridge contractors find it difficult to drive bridge piling through this dense limestone and ultimately resort to predrilling a hole in the limestone, setting the pile into the hole, and then grouting the pile in-place to obtain bearing. During the development of the project proposal, HNTB and Condotte drew on our local knowledge and realized that significant cost and schedule savings could result if we elected to utilize shallow foundations for the bridge substructure. While not necessarily unusual elsewhere in the United States, the use of shallow foundations at the pier and abutments for this bridge were two significant innovations used by our design-build team on this project.

Figure 9. Surface texture on pier column.

For the foundation of each pier, we chose to construct a large spread footing that transmitted the bridge reactions directly into the limestone bearing layer. Our geotechnical engineers determined that the contractor needed to excavate approximately three feet below the top of the limestone layer in order to obtain the minimum required 10,000 psf bearing capacity for the 28-ft by 28-ft spread footings. This requirement complicated construction of the 5-ft thick footings because the ground water elevation was several feet above the bottom of the footings. Rather than install an extensive dewatering system to temporarily lower the ground water near each footing, Condotte elected to cast a tremie concrete seal at the bottom of each excavation and pump the groundwater out of the hole. Once the hole was dewatered, construction of the foundation and pier quickly followed. As a part of the architectural requirements previously mentioned, the palm-frond form liner was utilized on two faces of the 8-ft diameter pier column to further enhance the aesthetics of this large structure (see Figure 9).

Much like the decision to utilize the spread footings at each pier, the final innovation selected by the team was to use a similar foundation system for the bridge's two abutments. Each of the two abutments utilizes a spread footing that is near the top of the MSE wall soil mass. These spread footings transfer the reactions from the abutment into the reinforced soil. To carry the additional lateral soil pressures induced by the spread footing system, the MSE wall manufacturer simply increased the lengths of the wall reinforcing straps. To limit the potential for settlement at the abutments, the existing overburden overlying the natural limestone formation was excavated prior to placing the embankment fill. This excavation was limited to the material directly beneath the MSE mass within 50 feet of the abutment. As a further means of limiting potential settlement, the embankment fill material in the same 50-foot region was a high-quality limerock fill that provides greater stiffness and strength than traditional embankment material. Use of the material provided a design bearing capacity for the abutment footings of 4,000 psf. Due to the proximity of adjacent roadway embankments, excavation of the in situ material and construction of the MSE walls required extensive use of temporary anchored sheet pilling. However, once the contractor installed the sheeting and excavated the material, construction of the MSE wall embankments began immediately because there was no need for the lengthy operation of mobilizing cranes and driving bridge piling.

BRIDGE NO. 12: RAMP F OVER EB SR 836

EXISTING CONDITIONS

Prior to this project, traffic exiting westbound SR 836 to travel south on the HEFT crossed over the extension of eastbound SR 836 on an existing three-span steel plate girder bridge. The existing bridge carried three lanes of traffic over the two existing lanes of the ramp from southbound HEFT to eastbound SR 836. MDX is currently extending the mainline SR 836 to the west, which will add another three lanes beneath the bridge. Because there was insufficient horizontal clearance beneath the existing bridge to add these future lanes, the bridge was replaced.

NEW STRUCTURE

While not as complex as Bridge No. 11, this replacement structure was not without its own challenges. The bridge was built to the north of the existing flyover and was configured so that it spanned the existing eastbound lanes and the future westbound lanes and eastbound ramp. To reduce the largest span, a pier was placed between the existing eastbound lanes and the future westbound lanes. As with Bridge No. 11, the Owner's proposal package recommended that a concrete segmental box girder bridge be utilized for this structure and suggested that additional piers were required outside the existing and future roadways. It

Figure 10. Elevation of Bridge No. 12.

appeared that the only purpose for these piers was to facilitate balanced cantilever construction, creating flanking spans as a by-product of the balanced cantilevers. Capitalizing on the freedom to change the proposed bridges, Condotte and HNTB again elected to utilize a steel box girder solution and to remove the short end spans from the bridge and build a two-span, steel box girder bridge.

Even though the proposal package only required that the bridge accommodate the two existing and three future lanes of the SR 836 extension, HNTB was aware through our relationship with the Owner that MDX had originally planned to provide three mainline lanes in each direction beneath the bridge. HNTB and Condotte elected to locate the abutments for Bridge No. 12 in such a way that there is sufficient clearance to add one 12-ft lane in each direction beneath the bridge if MDX should choose to widen SR 836 in the future. The final configuration, shown in Figure 10, is a 528-ft long bridge with spans of 255 and 273 feet, which is still significantly shorter than the recommended 640-ft long, four-span structure.

Although 12 feet wider than Bridge No. 11, this bridge uses the same configuration of three steel box girders in the 75'-1" wide cross-section. The girders are 13 feet wide at the top flanges with 8'-10" deep webs that are inclined on a 4:1 slope. The bridge is essentially straight, with 284 feet of the bridge falling on a tangent and the remaining 244 feet on a nearly imperceptible curve with a radius of 11,459 feet. During the design process, the steel fabricator and the contractor indicated their preference to curve the girders the slight amount rather than use tangent girders and variable-width deck overhangs. The team decided that it was less costly to curve the girders than it would be to configure the formwork for the deck overhangs to accommodate a variable width that would have culminated with a maximum overhang of well over five feet. At $130 per square foot, the approximate unit cost of the completed structure matched that of Bridge No. 11.

Because the bridge is partially curved, it technically qualified for the same design code exception that applied to Bridge No. 11. However, early in the proposal phase, the design team elected to design this bridge according to AASHTO LRFD design specifications since the curvature was very slight. As with Bridge No. 11, HNTB incorporated as many of the requirements from the latest AASHTO LFD curved girder specifications as was practicable.

SUPERSTRUCTURE

Aside from a few minor differences, Bridge No. 12 utilizes the same superstructure details as were used for Bridge No. 11, including the innovative use of the post-tensioned concrete integral diaphragm at the pier, the bolster for the top flange lateral bracing that passes through the concrete diaphragm, the full-depth interior diaphragm at the location of the temporary shoring and the use of the post-tensioning bars to create a temporary horizontal tie system for the girders during construction.

As part of the Owner's aesthetic requirements mentioned previously, the pier for this bridge was restricted to a rectangular wall-type pier. Plans for a future expansion of the roadway beneath the bridge limited the amount of width in the future roadway median to approximately 24 feet. The design team opted to go with a 23-ft wide pier, which resulted in two of the three box girders falling outside the limits of the pier column. As with Bridge No. 11, the solution was to utilize an integral diaphragm that cantilevered out from the pier to the outer girders. Geometrically, the diaphragm cantilevers for this bridge are shorter than those for Bridge No. 11, but the girder loads for Bridge No. 12 are much larger, resulting in roughly the same quantity of post-tensioning tendons in the diaphragms. The other significant difference with

Figure 11. Integral diaphragm at Pier No. 2.

the Bridge No. 12 superstructure is the use of high-load pot bearings beneath the integral diaphragm (see Figure 11). Each of these two pot bearings was designed for a maximum vertical reaction of approximately 4900 kips.

The footprint of Bridge No. 12 overlapped the limits of the existing ramp bridge by as much as 22 feet. Although the new bridge was significantly higher than the existing structure and there was sufficient vertical clearance between the top of the existing bridge deck and the bottom of the new box girders, the Contractor elected to partially demolish the overlapped portion of the existing bridge superstructure. This ensured that there was a clean working space in which to erect girders, construct the integral diaphragm, and form and cast the deck slab. It also provided a convenient work platform from which to work.

TEMPORARY SHORING

As with Bridge No. 11, in order to form up and then cast the integral concrete diaphragm, it was necessary to temporarily support the box girders away from the pier and the diaphragm. However, for Bridge No. 12, the details were somewhat less complicated because the pier was not also being made integral with the diaphragm as it was on Bridge No. 11.

In general, the temporary support system designed by the specialty engineer, CEC, was utilized for both bridges. The biggest difference between the two bridges was that CEC's system utilized 42" diameter steel pipe columns beneath all three box girders for Bridge No. 12 instead of using the W14x176 columns beneath the center girder as was done on Bridge No. 11. The center pipe columns were temporarily tied to the rectangular wall pier with two, 1 1/4" diameter post-tensioning bars.

SUBSTRUCTURE

For the foundation of the pier, we again chose to construct a large spread footing that transmitted the bridge reactions directly into the limestone bearing layer. Because the superstructure reactions were larger for this bridge, and because the pier was a wide wall pier rather than a circular column, the footing dimensions for Bridge No. 12 are quite large. To ensure the bearing loads remained within the allowable 10,000 psf, the footing was 26-ft x 40 ft (see Figure 12). As with Bridge No. 11, the contractor utilized a tremie concrete seal to dewater the footing excavation. The 23-ft wide pier is post-tensioned transversely with eight, 1 3/8" diameter post-tensioning bars in order to resist the large splitting forces developed as a result of the 4900 kip reactions from the two pot bearings beneath the integral concrete diaphragm. As a part of the architectural requirements previously mentioned, the two broad faces of the pier contain a colorful tile mosaic to further enhance the aesthetics of this large structure, seen in Figure 11.

Figure 12. Spread footing at Pier No. 2.

The innovative use of spread footing abutments founded at the top of the reinforced soil mass was also utilized for Bridge No. 12 in lieu of traditional pile-supported end bents.

SIGN STRUCTURES

The most visible aesthetic elements in this project are the architecturally sculpted sign structures, which consist of a steel monotube horizontal member and a post-tensioned concrete upright with architectural dimensions, curves and textured reveals (see Figure 13). Development of the proportions and dimensions of the concrete upright was completed by the Owner and included in the proposal request. However, design of the element was HNTB's responsibility, as was development of all of the connection details. To match the concrete surface texture on the MSE walls and the pier columns at Bridge No. 11, the palm frond form liner was utilized on the broad faces of the sign structure upright.

Figure 13. Typical span-type sign structure.

The horizontal monotube is an 18-inch diameter, ASTM A500 Grade B steel pipe, with a minimum yield strength of 42 ksi. The connection at the face of the upright was made using 14, 1 1/2" diameter A325 bolts. The section of pipe embedded in the concrete upright was fabricated with holes in the pipe to allow the concrete reinforcing steel to pass through. After the concrete was placed and cured, the pipe section at the upright was filled with grout to ensure the reinforcement passing through the pipe was adequately protected against corrosion. The upright was post-tensioned with two, 1-inch diameter post-tensioning bars that terminated in the top of the drilled shaft foundation.

SUMMARY

This project demonstrates one of the many benefits of the design-build construction method. The Owner's project requirements included many stipulations about form, color and texture, yet still allowed the contractor to make numerous independent decisions. The contractor and the engineer had the freedom to select structural materials and to optimize geometric parameters to suit the contractor's strengths. The innovative use of integral concrete diaphragms and spread footing foundations allowed Condotte to win the project while still meeting all of the requirements set forth by MDX. With the completion of the bridges, walls, and roadways, this project has ultimately achieved the Owner's intent to improve traffic capacity while adding attractive elements to the western end of their expressway system.

ACKNOWLEDGEMENTS

The author wishes to thank the many members of the design team for their hard work in preparing a quality set of structures plans and the staff and crew at Condotte America for their dedication to the project, not only during construction, but also during the design process. In addition, the project could not have been a success without the assistance and input from the many suppliers and specialty designers, including but certainly not limited to, Tampa Steel, Tensor Engineering, Reinforced Earth Company, VSL, Construction Engineering Consultants and The Corradino Group.