Design and construction of Raritan River Bridge foundations

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ABSTRACT: This paper presents the design and construction aspects of the foundations for the North Jersey Coast Line (NJCL) Raritan River Bridge Replacement project, which will replace the 115-year-old existing bridge connecting South Amboy and Perth Amboy in New Jersey. The new bridge piers are founded on a cluster of 8.0 feet diameter drilled shafts with shaft tip elevations varying from 180 feet to 230 feet below the water line, which will satisfy the American Railway Engineering and Maintenance-of-Way (AREMA) Manual requirements. This includes limiting the computed longitudinal deflection of the superstructure under load to one inch. Additionally, the new bridge abutments are supported on 24 inch closed-ended steel pipe piles. This paper also discusses the construction aspects of the bridge's foundation.

1. INTRODUCTION

The project involves the design and construction of the Raritan River Bridge, colloquially known as River Draw, which is considered a critical link between New Jersey's shore communities and New York City. Figure 1: Project Location Map.

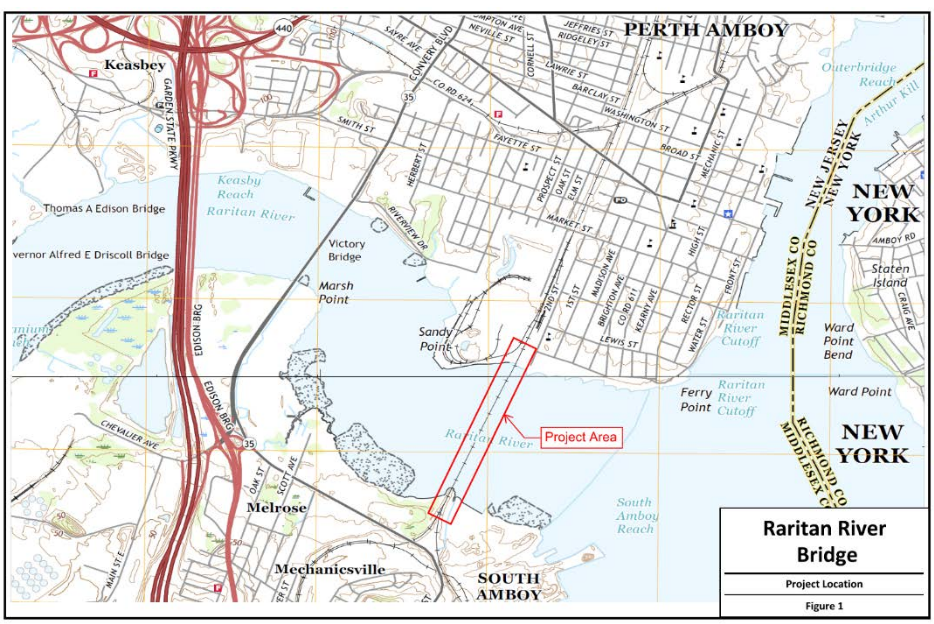

Figure 1. Project location map

The complete reconstruction of the existing 893.5-meter (2,931-foot)-long (abutment to abutment) bridge involves three-phases. Construction Contract No. 1 (GC.01) includes landside and bridge approaches and substructure construction. Construction Contract No. 2 (GC.02) will primarily include construction of the vertical lift bridge (lift/flanking span superstructure). The Construction Contract No. 3 (GC.03), primarily includes demolition of the existing bridge. The existing bridge will remain in service throughout the duration of construction of the new off-line bridge.

The design process began in 2018. In May of 2020, NJ TRANSIT awarded a $248 million contract to George Harms Construction Co. (GHC) of Farmingdale, N.J. for “GC.01” Work, which is expected to be completed in October 2024.

This paper discusses the new bridge foundation design and construction, which includes two abutments and twenty-seven (27) water piers (i.e., Pier 1 thru 27). The new bridge piers and abutments are supported on 2.4-m (8.0 feet) diameter, 70+- meter (200+ feet) long drilled shafts and 70 cm (24-inch) diameter closed-ended steel piles. The drilled shaft foundation design and construction of the new structure is the focus of this paper.

2. NEW BRIDGE

The project includes replacing the 115-year-old existing bridge, which was damaged by Superstorm Sandy in 2012. The new bridge features a vertical lift span providing a 91.5-meter (300 feet) wide navigational channel, in addition to half-mile approach spans and more than 915-meter (3000 linear feet) of track between the Perth Amboy and South Amboy stations in NJ. Figure 2 illustrates the new replacement structure. The new bridge structure will be on a parallel alignment north of the existing structure, (Figure 3).

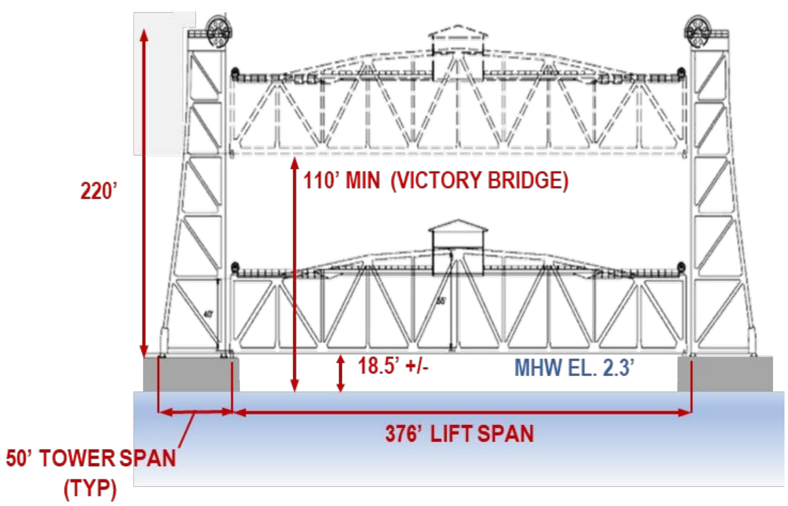

Figure 2. New replacement structure

Figure 3. New bridge alignment

The new bridge design criteria included: 1) increase resilience to flood damage and seismic events; 2) improve navigability (minimum of 33.5-meter (110 feet) of vertical clearance and 91.5-meter (300 feet) navigation channel); 3) optimize operations on the New Jersey Coast Line (NJCL) by facilitating rail speed up to 60 mph on the mainline tracks; and 4) enhance the structural capacity of the bridge to meet current design standards (i.e. Cooper E-80 and 315 Kip Freight live load) to provide long term reliability and service.

3. SUBSURFACE CONDITIONS

The project site is located with the Coastal Plain physiographic province of New Jersey, which is comprised of unconsolidated sediments that range in age from Cretaceous to Miocene (135 to 5.3 million years ago). For engineering analyses, the subsurface conditions at the project site can be divided into five different strata: River Deposits, Raritan Formation, Sand, Intermediate Geomaterials (IGMs), and Highly Weathered to Slightly Weathered Bedrock, (Figure 4).

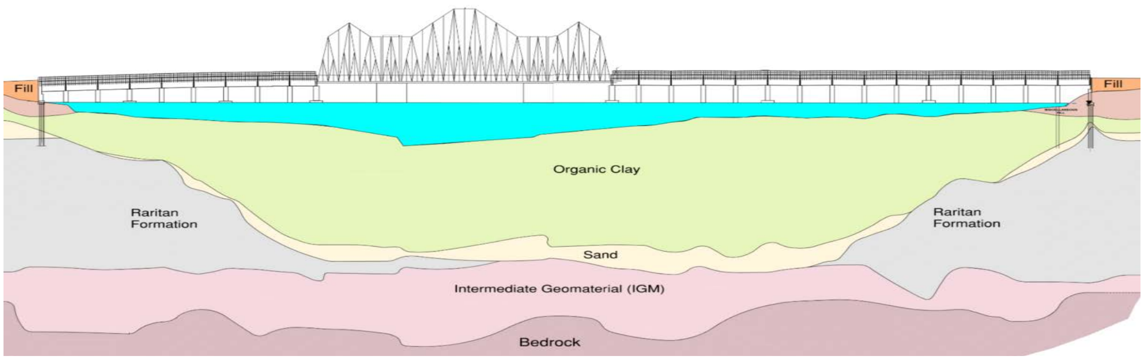

Figure 4. Generalized subsurface conditions

The River Deposits consist of very soft, “weight-of-rod”, fine-grained river muds. The River Deposits overlay the Raritan Formation or the Sand layer. The Raritan Formation generally consists of stiff to hard cohesive material with sand. The stiffness of the layer increases with depth. The layer is thicker near the shoreline and nonexistent near the center of the channel, likely due to scour activity during past glaciations. The Sand layer generally consists of medium dense to dense sand with varying percentages of gravel. The Sand layer is above the Raritan Formation closer to the shoreline and is above the IGMs in the center of the channel. IGMs, which underlie the Raritan Formation and/or the Sand layer, represents Bedrock that has completely weathered and decomposed to soil. It is generally classified as very stiff to hard CLAY & SILT and Silty CLAY with varying percentages of Sand according to Burmister soil classification. IGMs usually contain relic rock structure. The portions of the IGMs having SPT N-values less than 100 indicated soil-like consistency, while the portions of the IGMs having SPT N-values greater than 100 indicated that more rock-like remnants or “corestones” are likely present within the soil fabric. The bedrock underlying this project site generally consists of siltstone/shale of the Stockton Formation. The bedrock was cored up to 12-meter (40 feet) and within this zone was generally observed to be very soft to hard, and slightly to highly weathered. The bedrock is considered prone to rapid degradation and laboratory test results suggest that the bedrock is susceptible to degradation and formation of “smear zones” when exposed to the elements (i.e., water or drilling fluid), (FHWA 2002).

4. BRIDGE SUSBURCTURE DESIGN

The bridge design criteria required a design philosophy in compliance with the requirements of the American Railway Engineering and Maintenance-of-Way (AREMA) Manual, which includes limiting the computed longitudinal deflection of the superstructure under load to 2.54 cm (1inch) maximum.

4.1 *Bridge Piers*

Given the poor surficial soil condition of the very-soft River Deposits and the design load of Cooper E-80 live loads, it was determined that lateral stability and adhering to specified deflection requirements would control the foundation design and the determination of the sizing of foundation members. Therefore, it was determined that deep foundations were required to support the new bridge. The foundation feasibility studies indicated that groups of 2.45-meter (8 feet) diameter drilled shafts with permanent steel casings were needed at the bridge piers to satisfy AREMA requirements regarding limiting longitudinal deflection. The bridge abutments could be supported on 70-cm (24 inch) diameter closed-end steel pipe piles, since bridge abutments were detailed to move freely due to the provision of expansion bearings.

The CSI bridge and FB-MultiPier (FBMP) computerized engineering tools were used to design and size the bridge foundation to meet the one-inch deflection criteria (per AREMA).

Utilizing an iterative process, the sizing of foundation members at each substructure unit was established using FBMP that would yield one inch of deflection while incorporating P-Multipliers, the projected scour at a 100-year storm surge, and the site variability per observed variation in the ground conditions along with drilled shaft flexural characteristics. The stiffness, or lateral capacity, of each substructure unit generated from the FBMP model for a lateral deflection of one inch was incorporated into the CSI bridge model.

The CSI bridge analysis was performed to ensure that the longitudinal deflection of the superstructure complied with the requirements established by the (AREMA Manual 2004). The CSI bridge analysis (developed by H&H's structures group) modeled each approach span as a series of fixed substructure units (except the abutments which were taken as free ends due to expansion bearings). All forces distributed to the piers are based on the relative stiffnesses of the piers and superstructure units. Braking/traction force was applied to this model to determine one-inch maximum lateral deflection in all substructures.

Preliminary analysis determined that using exclusively single row drilled shaft bent type piers for the approach spans did not provide adequate lateral resistance to meet the one-inch maximum deflection criteria. As a result of this analysis, the incorporation of some intermediate, stiffer piers were considered within each set of approach spans to satisfy deflection requirements. A pier using four drilled shafts (quad pier) was included in the approach span layouts to meet AREMA requirements. Refined analyses of single row drilled shaft bent type piers coupled with interspersed quad piers confirmed satisfactory deflections. Figure 5 presents the layout of the modeled bridge integrating quad and bent piers into the analysis.

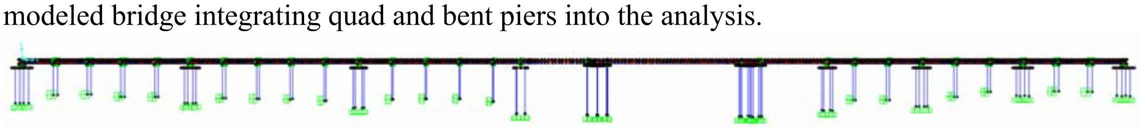

Figure 5. Bridge layout for CSI bridge model

4.2 Bridge Abutments

The very soft River Deposit layer confining the piles at the top was too weak to support the anticipated high lateral earth and hydraulic pressures. Attempts to transfer the lateral load through soft soil to a stronger bearing stratum using reasonably battered piles was found to be not viable. To alleviate concerns over the earth pressure behind the bridge abutments, design details included stabilizing the soil behind the abutment, as shown in Figure 6.

This approach includes detailing straps to resist and relieve the lateral loads on the piles, while designing the abutment structurally as if there were no straps, to address concerns over the creep needed to engage geogrid soil reinforcement.

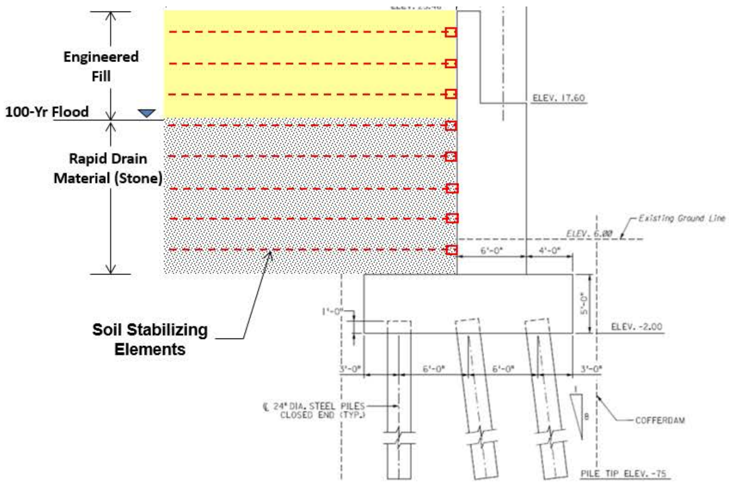

Figure 6. Bridge abutment design details

5. DEEP FOUNDATION DESIGN PROCEDURE

5.1 *Drilled Shaft*

he axial capacity of the shaft was derived from both the skin friction and end bearing, mainly from the portion of shaft embedded in the Sand, Raritan Formation and IGMs while neglecting the friction of the River Deposits to account for scour. The nominal side resistance of the shaft in cohesive soil was estimated per the β-method and the shaft side resistance in cohesionless soil was estimated per the α-method (FHWA 2010). The tip resistance in cohesive soil was estimated by the total stress method. The IGMs were modeled as soft/weak rock. Design strategies included extending the drilled shaft to bear into underlying siltstone/shale bedrock, if axial capacity derived from Sand, Raritan Formation or IGMs was found to be insufficient for a given substructure unit (Figure 7). The bearing capacity of rock was determined using the relationship between uniaxial compressive strength of intact rock and the general condition of the rock at the base of the shaft.

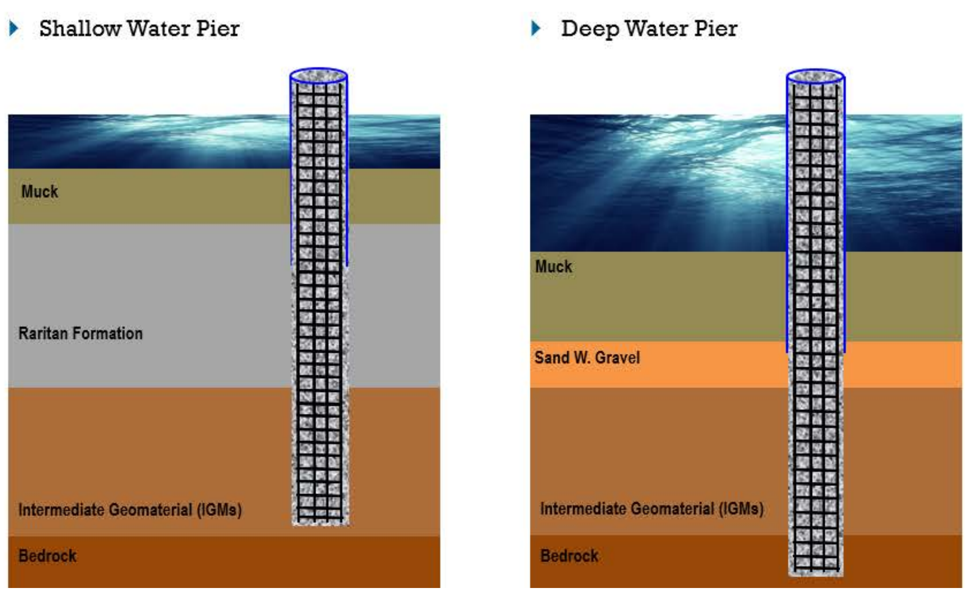

Figure 7. Drilled shaft design approach

5.2 *Driven Pile*

The closed ended pipe pile capacity versus depth for the entire soil profile was computed using the Nordlund and α-methods in cohesionless and cohesive layers, respectively (AASHTO 2017).

6. FOUNDATION CONSTRUCTION

The typical test drilled shaft construction scheme included: 1) installation of the permanent steel casing through the water and advanced into low permeability materials to achieve a positive seal at the bottom of so that there is no intrusion or extrusion of water or other materials into or out of the shaft excavation; 2) drilling using slurry to the required drilled shaft diameter and tip elevation; 3) removal of the excavated materials and excess slurry; 4) cleaning of the excavated hole; and 5) filling the excavated hole with the steel reinforcement cage followed by concrete (Figure 8)

The contract documents required verification of the drilled shaft with Cross-hole Sonic Logging (CSL) and Thermal Integrity Profiler (TIP) tests, which were conducted by GRL Engineers Inc. Results showed no sign of anomalies. The pile was installed vertically with an ICE I-36v2 single acting diesel hammer which was set to be operated using a variable pump.

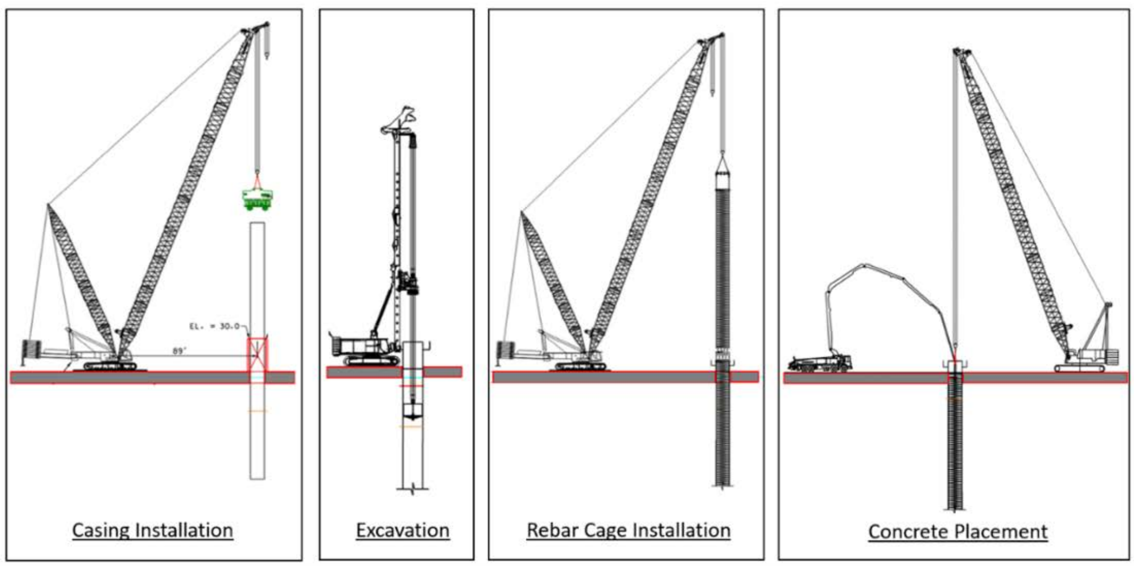

Figure 8. Test shaft construction scheme

7. FOUNDATION LOAD TEST PROGRAM

7. 1 *Drilled Shaft*

To verify the contractor's means and methods of construction and to confirm the shaft's ability to support the applied load without excessive or continuous displacement, the contract documents required conducting a compression load test on two non-productions, instrumented 2.45-meter (96-inch) diameter drilled shafts. The contract document's installation criteria and the maximum testing loads are summarized in Table 1.

The Bi-directional test/Osterberg Cell (O-Cell) testing was conducted by LOADTEST, Inc., and the test shafts were loaded in increments. The loading assembly consisted of three 70-cm (24-inch) diameter O-Cells, each calibrated to approximately 17,800 kN (4,000 kips). The test shafts' instrumentation and assembly were carried out in compliance with the contract documents.

Table 1. Test shafts installation and load test criteria

Test Shaft I.D.	Shaft Tip El. m (ft)	Bi-Directional Cell El. m (ft)	Max. Test Load kN (kips)	Ultimate Capacity kN (kips)	Remarks
DS-1	-60 (-196)	-46 (-150.0)	33,362 (7500)	31,306.5 (7038)	Sacrificial Shaft
DS-2	-72 (-236)	-69 (-226.0)	33,362 (7500)	53,952.5 (12129)	Sacrificial Shaft

LOADTEST technical staff conducted the load test, which began by pressurizing the O-Cells to break the tack welds that hold them closed (for handling and for placement in the shaft) and to form the fracture plane in the concrete at the top of the bottom O-Cell plate. The test shafts were loaded in increments (ASTM D8169 - Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial Compressive Load), bi-directionally, and each successive load increment was held constant for eight minutes by manually adjusting the O-Cell pressure.

DS-1 was loaded in 20 loading increments, resulting in a maximum bi-directional load of 39,314 kN (8,838 kips) applied to the shaft above and below the O-Cells. DS-2 was loaded in 30 loading increments, resulting in a maximum bi-directional load of 62.178 kN (13,978 kips) applied to the shaft above and below the O-Cells.

For DS-1, a displacement of approximately 0.84 cm (0.33 inches) and 3.3 cm (1.3 inches)was noted for the adjusted top loading of 24,465 kN (5,500 kips) and 48,931 kN (11,000 kips), respectively. For DS-2, a displacement of approximately 0.79 cm (0.31 inches) and 1.65 cm (0.65 inches) was noted for the adjusted top loading of 22,241 kN (5,000 kips) and 44,483 kN (10,000 kips), respectively. Therefore, it was concluded that the performance of both demonstration shafts was acceptable (Figure 9).

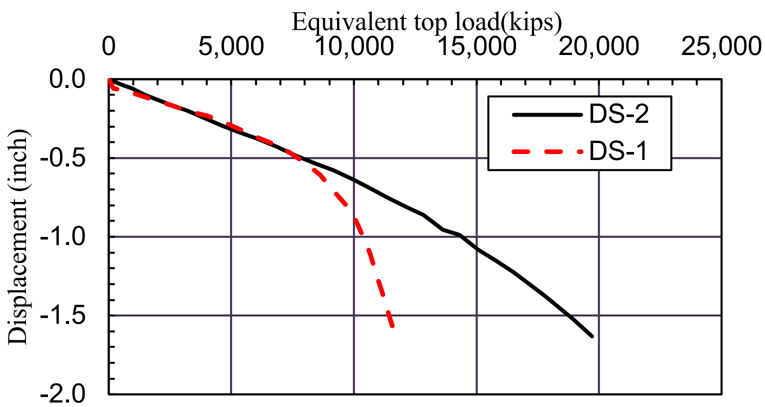

Figure 9. Load versus deflection curve

7.2 Pipe Piles

The pile testing program of the project included performing the Pile Driving Analyzer (PDA) Test with CAPWAP analyses and restrikes on selected production pile(s) at each abutment. GRL Engineers Inc. performed dynamic load testing on the production piles designated as test piles. The tested piles were 70-cm (24-inch) O.D., 1.91-cm (0.75 inch) thick closed-ended steel pipe piles affixed with a conical point. The contract documents required installing the piles to a minimum tip elevation of -15.3-meter (-50 feet) and achieving an ultimate capacity of 2,545 kN (572 kips) with estimated pile elevation of -29-meter (-95 ft). Table 2- Pile Dynamic Test Results presents the summary of the PDA test results.

The results of the PDA test indicated that the estimated pile length was adequate to support the design load and the contractor’s method of pile installation was effective in meeting contractual requirements.

Table 2. Pile dynamic test results

Test Pile ID	Hammer Type	Test Condition	Pile Tip Elevation m (ft)	Capacity from CAPWAP Analysis		
				Total kN (kips)	Skin kN (kips)	Toe kN (kips)
South Amboy Abutment						
TP-1	ICE I-36v2 single acting diesel hammer	EOID	-24.5 (-80.2)	3047 (685)	1334.5 (300)	1712.6 (385)
Perth Amboy Abutment						
18	ICE I-36v2 single acting diesel hammer	EIOD	-27 (-89.0)	3314 (745)	1623.6 (365)	1690.3 (380)
27		EIOD	-21.4 (-70.0)	2331 (524)	996.4 (224)	1334.5 (300)
27		EOR	-21.6 (-70.8)	3300.6 (742)	1877.2 (422)	1423.4 (320)
EOID: End of Initial Drive; EOR: End of Restrike after 48 hr.						

8. CONCLUSIONS

The design methodology to satisfy the AREMA requirements of limiting the computed longitudinal deflection of the superstructure to one inch maximum for E-80 loading included consideration of interaction among bridge piers and between bridge components and supporting soil. The CSI bridge and FB-MultiPier (FBMP) computerized engineering tools were used to assess the anticipated longitudinal deflection of the superstructure under load and to size bridge foundation to fall within AREMA requirements.

The bridge foundation recommendation included single row drilled shaft bent type piers consisting of two (2), 2.45-meter (8 feet) diameters drilled shafts for the approach spans coupled with interspersed quad piers i.e., four (4), 2.45-meter (8 feet) 225diameters drilled shafts up to 69-meter (225 feet) deep to withstand E-80 live loads while limiting anticipated deflection within allowable limits of the AREMA manual.

Contractual requirements to verify both design recommendations and the contractor's means and methods of construction for drilled shaft(s) production included construction of two sacrificial demonstration drilled shafts. Bi-Directional Static Axial Compressive Load tests conducted on two demonstrations shafts verified that the as-built shafts met both design assumptions and proposed construction procedures to satisfy contract requirements.

The contractual requirement of performing PDA tests on the production piles was a cost-effective strategy to verify effectiveness of the contractor's method of pile installation and design assumptions.

9. ACKNOWLEDGEMENTS

This paper is based on the design and construction work of Hardesty & Hanover/Gannett Fleming Joint Venture, New Jersey Transit Corporation, and George Harms Construction Co, Inc.

REFRENCES

AASHTO. 2017. *LRFD Bridge Design Specifications*, 4th Edition.

American Railway Engineering and Maintenance-of-Way (AREMA) Manual for Railway Engineering, 2015.

ASTM D8169/D8169M-18. February 2018.Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial Compressive Load.

FHWA. April 2002. Publication FHWA-IF-02-034, "Evaluation of Soil and Rock Properties", Geotechnical Engineering Circular No. 5.

FHWA. May 2010. Publication FHWA-NHI-10-016, Geotechnical Engineering Circular No. 10, "*Drilled Shaft: Construction Procedure and LRFD Design Method*".