I-495 in Wilmington, Delaware USA – The Leaning Towers; Lateral Squeeze Effects on Pile Supported Bridge Piers Due to Mudwave; Emergency Response and Retrofit

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Abstract. In 2014, the Delaware Department of Transportation was notified of a potential problem with a bridge that was part of the interstate system. After preliminary investigation it was determined that a stockpile located adjacent to the bridge piers caused lateral squeeze effects deforming the pile groups and causing the bridge piers to tilt and displace laterally. Because of the importance of this bridge, an emergency plan was established to enable removal of the stockpile, geotechnical investigations, and design to occur simultaneously. Instrumentation of the bridge was critical to provide real-time monitoring of the existing structure and to modify construction and design decisions. Geotechnical investigation and analysis, design, and construction were completed sufficiently by August 23, 2014 to safely open both directions of the interstate.

1 Introduction

In May 2014 the Delaware Department of Transportation (DelDOT) was notified by a citizen that the barrier walls along the northbound and southbound sides of a bridge on Interstate 495 (I-495) over the Christina River were no longer even with each other. This bridge, part of the north-south interstate system which carries over 90,000 vehicles per day, was constructed in 1973 on deep foundations varying from tapered steel shell piles further inland to steel H-piles.

The impacted section of the bridge, highlighted in Fig. 1, is within the floodplain of the river. It consists of parallel structures supported by pairs of single shaft, hammerhead pier columns founded on groups of battered and plumb steel H-piles, approximately 140 ft long.

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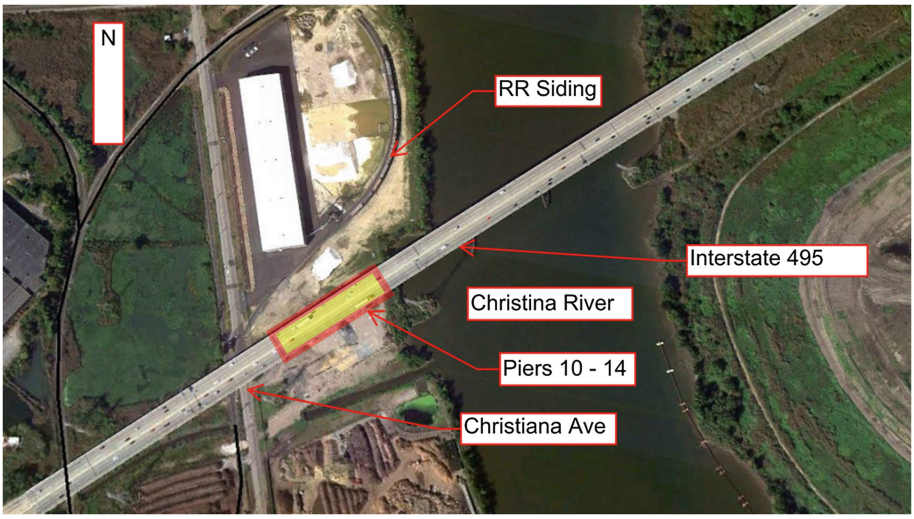

Fig. 1. Impacted area of bridge

2 Response

DelDOT arrived on site on June 2, 2014 and responded by closing the bridge to traffic. DelDOT contracted with AECOM to assist with developing a remediation strategy and designing a retrofit to re-open the bridge as quickly as possible. It was determined soon after that a stockpile adjacent to the piers (Fig. 2) induced lateral deformation and

Fig. 2. Impacted bridge piers with soil stockpile in foreground

produced a mudwave through the saturated clay layer. Multiple activities began concurrently, including removal of the stockpile, geotechnical investigations, contractor selection, and design.

2.1 Stockpile Removal

Work began immediately to remove the soil stockpile to unload the surcharge on the compressible clay layer to stop any additional movement of the bridge. Inclinometers and sensors were installed in the bridge foundation to monitor structure movement during stockpile removal. Work continued 24 h a day until all of the soil had been transported offsite and the area under the affected piers was restored to original grade. Over 50,000 tons of soil was removed in 10 calendar days.

2.2 Geotechnical Investigation

A comprehensive geotechnical subsurface investigation plan was developed in the initial days of the emergency response This was a joint effort between DelDOT's Materials and Research Section, Federal Highway Administration (FHWA), Geo-Technology Associates (GTA), The Walton Group, AECOM, Hillis- Carnes Engineering Associates (HCEA) and the University of Delaware.

The subsurface investigation began on June 5th, 2014 and was performed within the affected area to measure soil disturbance, confirm foundation conditions, and define a competent bearing stratum for the impacted pier foundations. The investigation included test pit excavations at ten pier footing locations (Pier Nos. 10–14); 12 cone penetrometer test (CPT) soundings; six Standard Penetration Test borings with rock coring; installation of four open-standpipe piezometers with drop-in transducers; and installation of three inclinometers with dedicated probes (Fig. 3). In addition to subsurface instrumentation, the bridge piers were instrumented with tiltmeters to measure movements of the superstructure. All of the instrumentation was connected to a data logging hub that uploaded real-time readings to a centralized website.

One of the most significant findings of the investigation was the excess pore water pressure observations during the CPT soundings. The observed pore pressures exceeded a 1.0 psi/ft threshold in numerous locations throughout the entire soft clay layer under Piers 11 through 14 (Fig. 4). This is indicative of shear failure in clay soil. The pore water pressure values (Fig. 5) correlated to an equivalent soil stockpile height of 35 to 40 ft corresponding to the original height of the soil stockpile removed.

Significant cracking (Fig. 6A and B) was found during test pit investigations at each of the pile caps where deformation and displacement of the piers was observed. Exposing the pile caps revealed that each of the impacted pier column foundations were compromised at four pier locations (11 through 14) consisting of eight pile caps. Installation of new deep foundations to underpin and/or reconstruct the piers was required to restore the bridge to service.

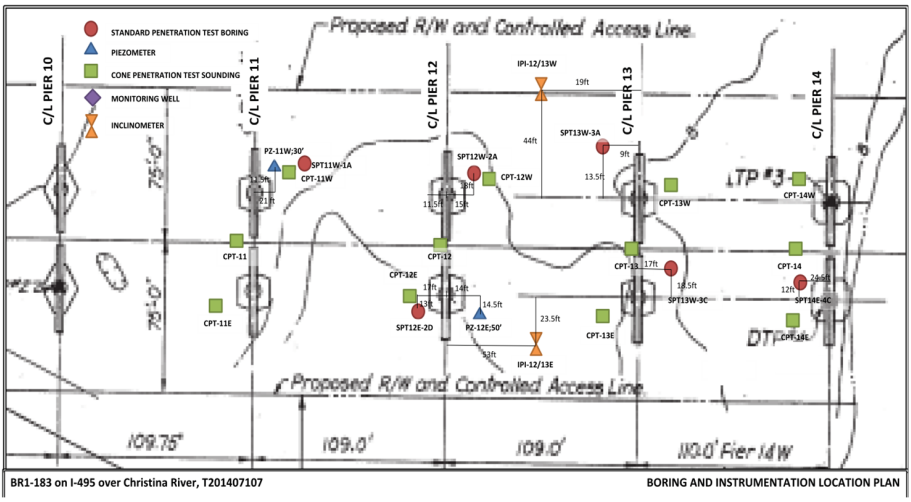

Fig. 3. Boring location and instrumentation plan

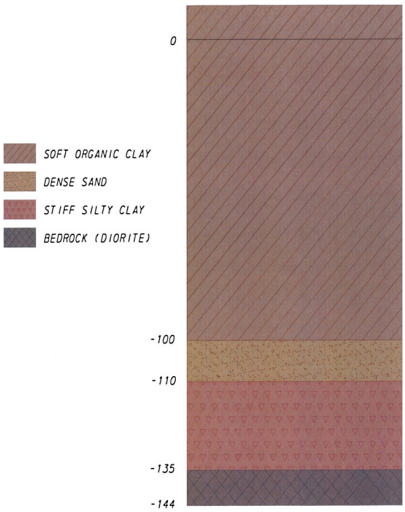

Fig. 4. Soft clay layer

2.3 Design of New Foundation

Drilled shafts bearing on bedrock were the selected preferred foundation due to their superior lateral and axial capacity compared to flexible H-piles. The drilled shafts were designed as end bearing on competent rock.

During the design phase of the drilled shafts, the initial soil and bedrock parameters were based on presumptive values since investigations were still ongoing. Information from published references, correlated values from in-situ testing with a large focus on

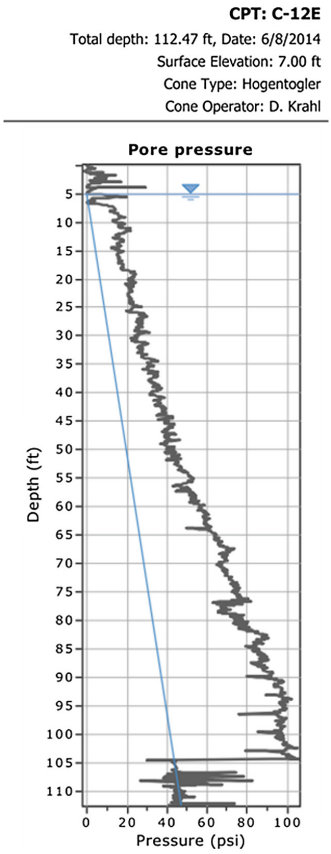

Fig. 5. Pore water pressure reading from CPT C-12E using Hogentogler CPTu Piezocone

the CPT soundings were used. Another bridge, the Wilmington Waterfront Christina River Bridge, located just west of the I-495 site, was in the design phase at this time. That investigation provided recent laboratory test results on similar soil and bedrock in the area. This resulted in almost immediate availability of this data and was crucial in allowing for timely confirmation of assumed parameters used in the preliminary design of the foundations. There was significant correlation and corroboration of the various sources of data.

Using information obtained from existing plans and boring profile sheets, a generalized soil profile was created for the drilled shaft design at the impacted piers. Soil and rock parameters were initially assigned based on published reference values and were extremely conservative. Values were refined as new boring and in-situ testing information was available and finalized when laboratory testing was completed.

A generalized stratigraphic column was created to provide a worst-case model to be used at each of the four pier locations. The drilled shafts were designed as 'end bearing' on bedrock without a rock socket and developed their lateral resistance from the soft marine clay layer that comprised the upper 100 ft of the soil profile. A conservative

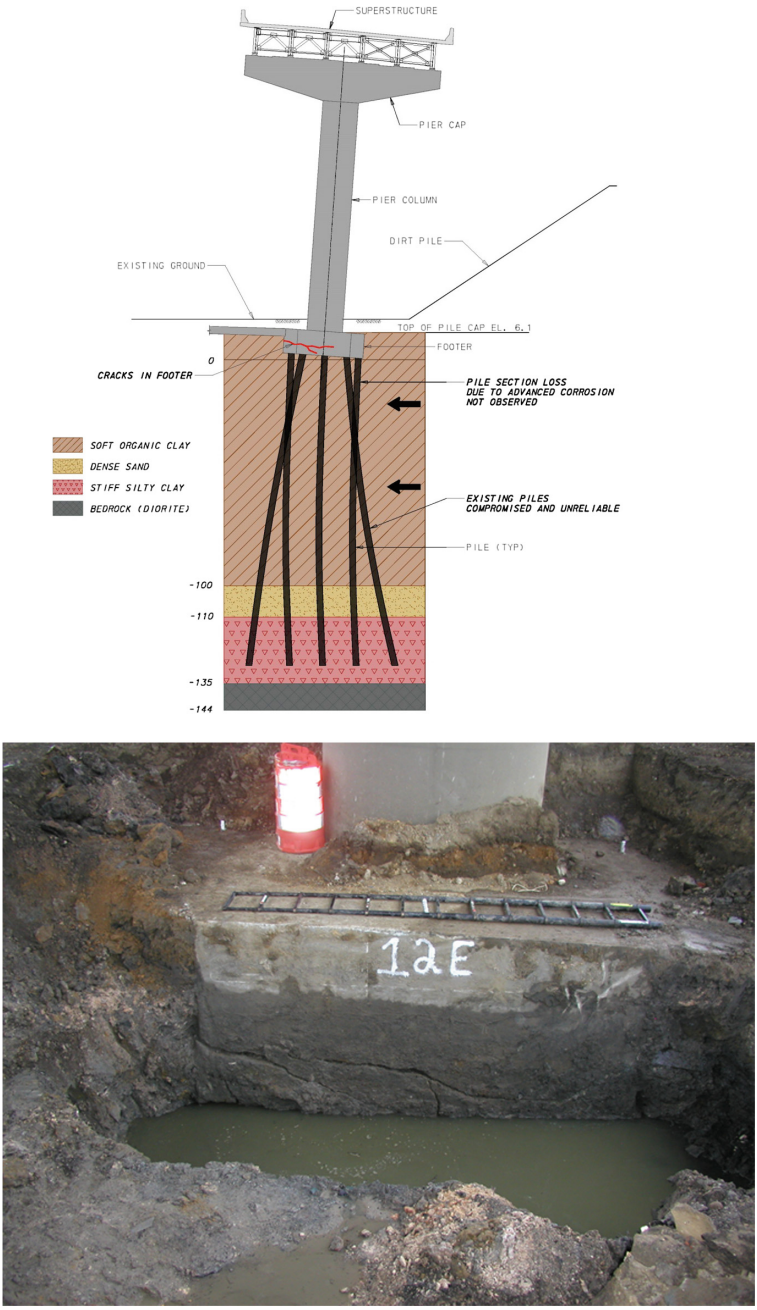

Fig. 6. A: Diagram of impacted pier cap at Pier no. 12E, B: Photo of impacted pier cap at Pier no. 12E

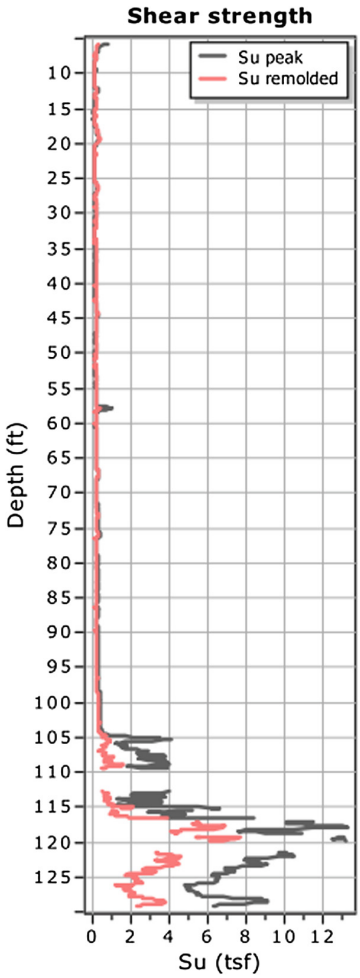

Fig. 7. S_u correlation from CPT C-12E

value of undrained shear strength (S_u) was developed that increased with depth based on correlated estimates of S_u provided by the CPT soundings (Fig. 7).

The S_u values were corroborated by subsequent S_u values produced in the laboratory triaxial testing program performed on undisturbed Shelby tube samples. Unconfined, unconsolidated (UU) tri-axial tests were performed. The significant thickness of soft, compressible marine clay was assigned very minimal S_u values ranging from 0 to 7 psi, increasing with depth. Excess pore water pressure was also accounted for as additional axial load on the drilled shaft using correlated pore pressure readings taken during CPT sounding when the soil surcharge was still in place. This would be considered the worst case scenario, since immediate removal of the soil stockpile resulted in the pore pressures dissipating significantly by the time the drilled shafts were constructed. However, to maintain a conservative approach the initial

excess pore water pressure caused by the soil stockpile was used to approximate the equivalent downdrag load on the shafts. 300 kips of downdrag load was used for each of the shafts.

A static analysis was performed on a single shaft using the AASHTO method [1] which follows the Canadian Geotechnical Society method (1985). The Canadian method accounts for spacing and aperture of discontinuities in the rock bearing stratum and representative unconfined compressive strength (q_u) values from rock core recovered from the project. Since no confirmatory load testing would be performed on the drilled shafts a bearing resistance reduction factor of 0.50 was used.

The worst-case design scenario showed that the shafts could develop in excess of 4,000 kips of axial capacity, much greater than the required design axial loading of approximately 3,000 kips per shaft. All axial capacity was developed from end bearing and any contribution from side resistance was ignored. Final design analysis was performed using the refined values and material properties from in-situ and laboratory tests and indicated that the base resistance (R_P) is 4,261 kips, much greater than the 3,183 kip factored axial loading. Single shaft lateral analysis was performed with LPILE version 6.0 and group analysis was performed with GROUP_v8.0. Input parameters used are shown in Table 1.

Table 1. LPILE input parameters used for shaft analysis

Soil layers								
Type	Depth 1 (in) (ft)		Unit Wt (lb/in^3)	Undrained cohesion/shear strength (Rock), c (lb/in^2)	Strain factor, ε50	P-Y MODULUS, K (LB/IN^3)	Friction angle, φ (deg)	Uniaxial compressive strength (lb/in^2)
Layer 1								
LPILE curve (1) for soft clay	0	0	0.019	0	0.02	30	0	
	1236	103	0.019	6	0.02	30	0	
Layer 2								
LPILE curve [2] stiff clay with water	1236	103	0.033	11	0.004	800	28	
	1356	113	0.033	15	0.004	800	28	
Layer 3								
LPILE curve [4] sand (Reese)	1356	113	0.036	0		125	35	
	1440	120	0.036	0		125	35	
						krm		
LPILE curve (9) weak rock	1440	120	0.051			0.00005		1389
	2100	175	0.051			0.00005		1339

3 Construction

The prime contractor selected for the project, J.D. Eckman, mobilized their caisson contractor, A.H. Beck of San Antonio, TX on June 12, 2014. A.H. Beck performed the caisson installation and another subcontractor, R.E. Pierson, began the installation of thirty-two drilled shafts (Fig. 8). The caissons were fully-cased for their entire depth and excavation was handled with low head, fixed-lead rigs using a 'wet method' with bentonite slurry (Fig. 9).

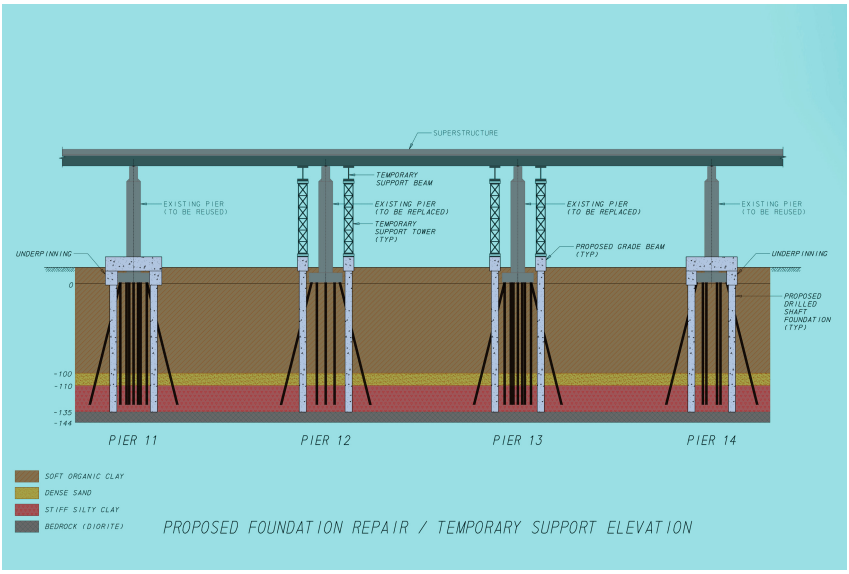

Fig. 8. Elevations of underpinning and new foundations for Pier nos. 11–14

Fig. 9. Photo of cased shaft filled with slurry

A comprehensive Drilled Shaft construction specification was developed by DelDOT and implemented for the entire project. AECOM worked with A. H. Beck to develop termination criteria for each caisson based on the perceived depth to competent bedrock (Brandywine Blue Gneiss shown in Fig. 10) as determined from the existing subsurface profiles and the new borings.

Fig. 10. Bedrock recovered from 1st drilled shaft to reach termination, note light gray Gneiss in center

Initially casing oscillators were used exclusively to limit vibrations and additional damage to existing pile groups which was effective, but very slow. In order to meet the aggressive schedule set, the Contractor asked to switch to vibrating the casing using a Vibro-Hammer. Since the bridge superstructure was already instrumented, it was decided to try vibrating the casing and monitor any movements. When no movement in the bridge superstructure was observed, this method was deemed satisfactory and the remaining casings were vibrated into place, expediting the process. The oscillator was used on subsequent caissons to start the casing and this proved helpful to maintain production.

The caisson contractor's construction methods verified the perceived subsurface conditions by drilling through a weathered rock zone prior to encountering the competent gneiss bedrock. Four feet diameter drilled shafts were installed with an average shaft length of 150 ft, with the deepest shaft going over 165 ft. This was not considered possible with conventional low-head drill rigs and tooling. Permanent steel casing was installed for the entire depth to bedrock to eliminate soil loss during drilling and to minimize localized bridge settlement.

Pre-assembled reinforcement cages from the Tappen Zee bridge project in New York were available and this helped expedite the project. An agreement between DelDOT, J.D. Eckman and the Tappen Zee project authority allowed the pre-assembled reinforcement cages to be shipped directly to the I-495 site where J.D. Eckman spliced the cages and lifted them into place through access holes cut in the bridge deck. This cooperation saved an incredible amount of time.

Real-time monitoring of the superstructure and subsurface was conducted during the entire construction process. Low head clearance under the bridge presented challenges during excavation and installation of the drilled shafts which were overcome by the Contractor's innovative means and methods.

The last drilled shaft was completed on July 16, 2014; just 32 days after the drilling began (Fig. 11). That equated to a production of one shaft/day with an average length of shaft of 150 ft. This duration included the testing and acceptance of each of shafts with crosshole sonic logging (CSL).

Fig. 11. Completed Pier no. 12 from March 24 2015

4 Conclusions

The incident that occurred at the I-495 Bridge confirms the significance of lateral loads imposed by a surcharge load which has been a challenge to estimate by geotechnical engineers. Information obtained from the cause of the bridge tilting shows the effects of lateral squeeze on deep foundations. The innovative remedial methods employed to design and construct new foundations for an existing bridge serve to benefit the entire foundation engineering community.

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Reference

1. LRFD Bridge Design Specification, 6th edn (2012)