DESIGN OF THE DRILLED SHAFT FOUNDATIONS FOR THE COOPER RIVER BRIDGE

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ABSTRACT

The Cooper River Bridge, the longest cable-stayed bridge in North America, replaces two existing bridges along U.S. Highway 17 and connects the City of Charleston and the town of Mount Pleasant, South Carolina. Two diamond shaped towers support he 1,546 ft long main span over Cooper River and allows for a 1000 ft wide navigation channel. The Charleston high-level approach is 4351 ft long and provides a 250 ft wide navigation channel for Town Creek, while the Mount Pleasant high-level approach is 2090 ft long. Due to the high seismicity in the region, potential vessel impact, and hurricanes, large lateral loads are expected. From various types of foundations, 10-foot diameter drilled shafts were selected for their high capacity against vertical and lateral loadings. The shafts are founded on a stiff to very stiff lightly cemented calcareous sandy clay or sandy silt layer, called Cooper Marl, which underlies the coastal sediments of the interbedded clays and sands. A group of 11 shafts supports each main tower, while a rock island protects it from vessel collision. A group of 4 to 8 shafts foundations support the other piers of the main span unit. A pair of shafts supports the High Level Approach piers. The shaft tip elevations vary from 120 ft to 230 ft below water line. An elaborate geotechnical investigation was conducted on the site, including a load test program on full-scale shafts under axial and lateral loads. The results of the tests were used to calibrate the lateral and axial soil resistance. The objective of this paper is to present the different aspects of the bridge foundation design.

INTRODUCTION

The new Cooper River Bridge, stretching approximately 3 miles, connects the city of Charleston and the town of Mount Pleasant along Highway 17 in South Carolina. As shown in Figure 1, the bridge crosses the Town Creek and Copper River channels while passing through Drum Island and is currently the longest cable stay bridge (with a main span length of 1,546 ft) in North America. The wide main span allows for both widening of the navigation channel to 1,000 ft and deepening to accommodate larger vessels. The bridge carries a total of eight lanes of traffic and a shared pedestrian-bicycle lane. Officially opened for traffic on July 16, 2005, the Cooper River Bridge replaces the Grace Bridge, completed in 1929, and the newer 3-lane Pearlman Bridge, completed in 1965.

This $632 million design-build project includes a main span unit bridge structure, high-level approaches, low-level approaches, ramps and interchanges on both sides of the crossing. The contract was awarded to Palmetto Bridge Constructors, a joint venture of Tidewater Skanska and HBG Flatiron by South Carolina Department of Transportation, on July 2nd, 2001. The design team consisted of Parsons Brinkerhoff as the design manager and designer of the main span superstructure, Buckland and Taylor as designer for the superstructure of the high-level approaches, and Ben C. Gerwick, Inc. (BCG) as designer for the foundations of the main span unit and high level approaches, the main pier artificial protection islands, and vessel collision risk analysis.

SITE CHARACTERIZATION AND SUBSURFACE CONDITIONS

Major site investigations were carried out in 1993, 1999 and 2000. These investigations included soil test boring, lab index testing, consolidation testing, and tri-axial shear testing. The last investigation, in 2000, was part of the design-build contract. All together, 148 soil test borings and 63 CPT soundings, ranging in depth from 20m to 107m, have been made in the proximity of the bridge. Figure 2 shows the subsurface conditions established by these investigations along the bridge alignment. An in-depth discussion and summary of site characterization and subsurface conditions for the bridge site was presented by Camp (2004).

Figure 1. Cooper River Bridge Site Plan

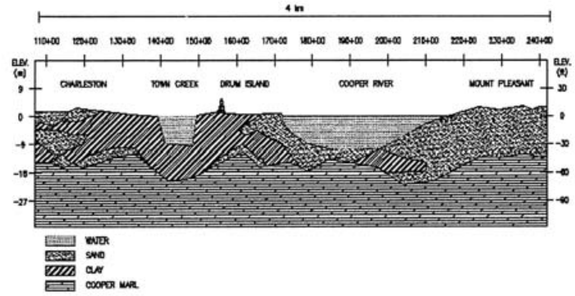

Figure 2. Generalized subsurface conditions along the new bridge alignment.

Most of the bridge alignment crosses over water or low-lying tidal, salt marshes. At the Town Creek and Cooper River channels, the mudline extends down to El. –56 ft and –45 ft, respectively. On the Charleston side, the upper soil strata consists of soft to medium stiff silty clay to sandy clay with an undrained shear strength (Su) in the range of 200 to 700 psf. At Drum Island, an active dredge disposal site, the upper soil consists of 25 ft of clayey sand dredge spoil underlain by a soft to medium stiff clay stratum with Su in the range of 200 to 800 psf. At both the Charleston shoreline and Drum Island, there are very soft clayey marsh deposits up to 35 ft thick. On the Mt. Pleasant side, the upper deposits consist of loose to dense sand with SPT blow counts ranging from 0 to 58. Cooper Marl, a cemented calcareous stiff to hard silty clay and clayey silt underlays these recent deposits.

The surface of the Cooper Marl is at an elevation –42 ft to –55 ft mean sea level, but at Town Creek it is as deep as –77 ft. Throughout the entire site the Cooper Marl is about 350 ft deep. This formation is highly plastic with OCR from about 3 to 6. It has a fines content of 75 to 90%, a calcium carbonate content, which is a measure of the remains of microscopic marine organisms, of 60 to 80 %, and a clay fraction of 20%. The effective friction angle of the formation is about 43 to 46 degrees with a relatively high void ratio of 1 to 2.0 and a virgin compression index (Cc) of 0.6 to 0.8. The undrained shear strength (Su) is high with average values of 2.2 to 5 ksf and reaching 20 ksf below elevation –170 ft. The average liquidity index (LI), plasticity index (PI), and water content are 68%, 34%, and 48%, respectively.

FOUNDATION CONFIGURATIONS

The New Cooper River Bridge is a cable-stay bridge with a main span of 1,546 ft. The bridge has the longest cable-stay span in North America, which offers a 1,000 ft wide navigation channel and a vertical clearance of 186 ft. The approximately three-mile-long bridge, including the main span unit (with a 1,546 ft main span, two 650 ft side spans, and two 225 ft anchor spans for a total suspended span length of 3,296 ft, see Figure 3, the east and west high level approaches, ramps, and interchanges, were designed and constructed in four years.

The Cooper River Bridge is designed to withstand very large axial and lateral loads due to the region's high seismicity and hurricanes. To achieve a large lateral capacity, the bridge is supported by drilled shafts ranging from 8 to 10 ft in diameter. Each drilled shaft has a permanent 10'-6" diameter, 0.75" thick steel casing embedded a minimum of 5 ft into the Cooper Marl. The advantages of these drilled shafts include providing large axial and lateral capacities, minimizing the number of substructure units, and cost-effective installation.

Main Span Tower Foundations

Each main tower foundation consists of eleven 10 ft diameter drilled shafts, and an 80 ft x 110 ft cap with a depth varying from 15 ft to 18 ft. Rock fill islands that extend from the marl surface to elevation of +12 ft protect the main tower foundations from ship impact by rock fill islands (see Figure 4).

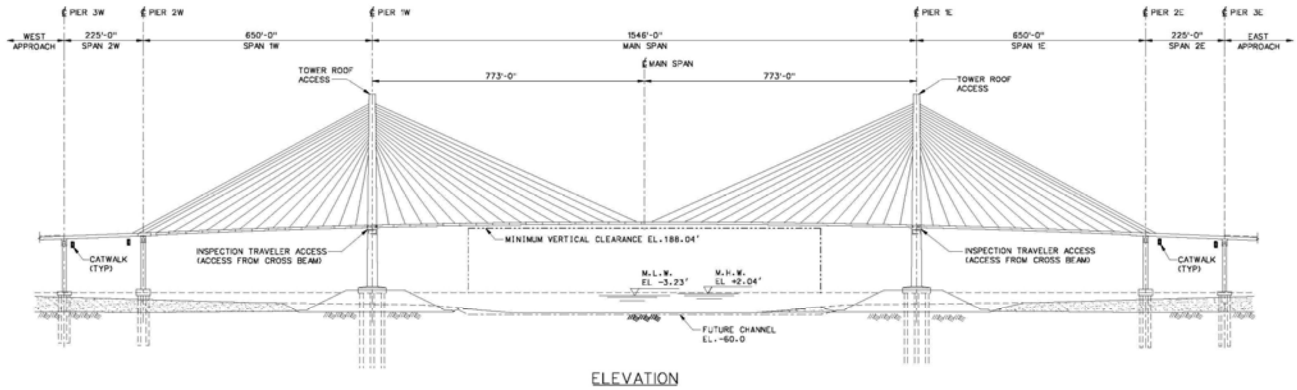

Figure 3. Main Span Unit – New Cooper River Bridge

The rock islands surrounding the foundations serve three purposes. The first is to protect the bridge towers from ship collision, which is a concern due to the large amount of freight traffic along the waterway. The second is to provide full embedment for the drilled shafts to resist lateral loads. The third is to facilitate construction of the large concrete pile caps above the waterline.

Main Span Anchor Piers and End Piers

The dimensions and configuration of the anchor piers (Pier 2W, 2E) and end piers (Pier 3W, 3E) are shown in Figures 5 and 6. Pier 2W consists of two columns sitting on a group of eight 10 ft diameter drilled shafts with a cap of triangular ends capable of withstanding a 16,500 kip ship impact load. The cap is 12 ft thick. A 14 ft x 48 ft hollow space in the middle of the cap serves to reduce the weight of the cap. Each drilled shaft supports about 915 kips of the cap weight. Piers 2E, 3W, and 3E also consist of two columns sitting on a group of four 10 ft diameter drilled shafts with a rectangular shaped cap that is capable of withstanding a 7,800 kip ship impact load. The cap is 12 ft thick at the ends (connecting the column and shafts) and tapers to 8 ft in the middle. The cap is also hollow in the middle to reduce weight. For these caps, each drilled shaft will support about 1,160 kips of the cap weight. All the caps have a top elevation of El +7.77 ft

High Level Approach Piers

Typical high level approach piers are two column bents. Each pier column is supported by an enlarged single drilled shaft (Caltrans refers to this type of foundation as a Type II Pile Shaft). Collision struts are provided at waterline between the columns for all water piers (see Figure 7).

SHAFT AXIAL LOAD TESTING

An extensive load testing program was performed to evaluate the axial and lateral resistance of the drilled shaft foundation (Camp, et al., 2002). The first phase pre-construction load testing took place in the fall of 2000 at three sites: Charleston, Mount Pleasant and Drum Island. This test program included Osterberg load cell tests and Statnamic tests for evaluating both axial capacity and lateral resistance. The test shafts were installed by various methods including wet methods (with bentonite slurry, polymer slurry or plain freshwater as drilling fluid) and a dry excavation method. The test shafts (12 in total) had diameters of 6 and 8 ft, and tip elevations around El. -100 and-150 ft. Two second phase construction stage load tests were performed at each of the two main piers to provide further information for the design of the deeper shafts required at these piers. For practical reasons, 5-ft diameter test shafts were selected and the resulting test data were extrapolated and modified for use in the design of the larger diameter production shafts (Castelli, 2004). The test shafts at the east and west main piers extended to El. -219.2 ft and El. -216.0 ft, respectively. Each test shaft included two levels (at El. –150 ft and approximately 4 ft above the base of the shafts) of Osterberg load cells, with each level consisting of three 21-in diameter

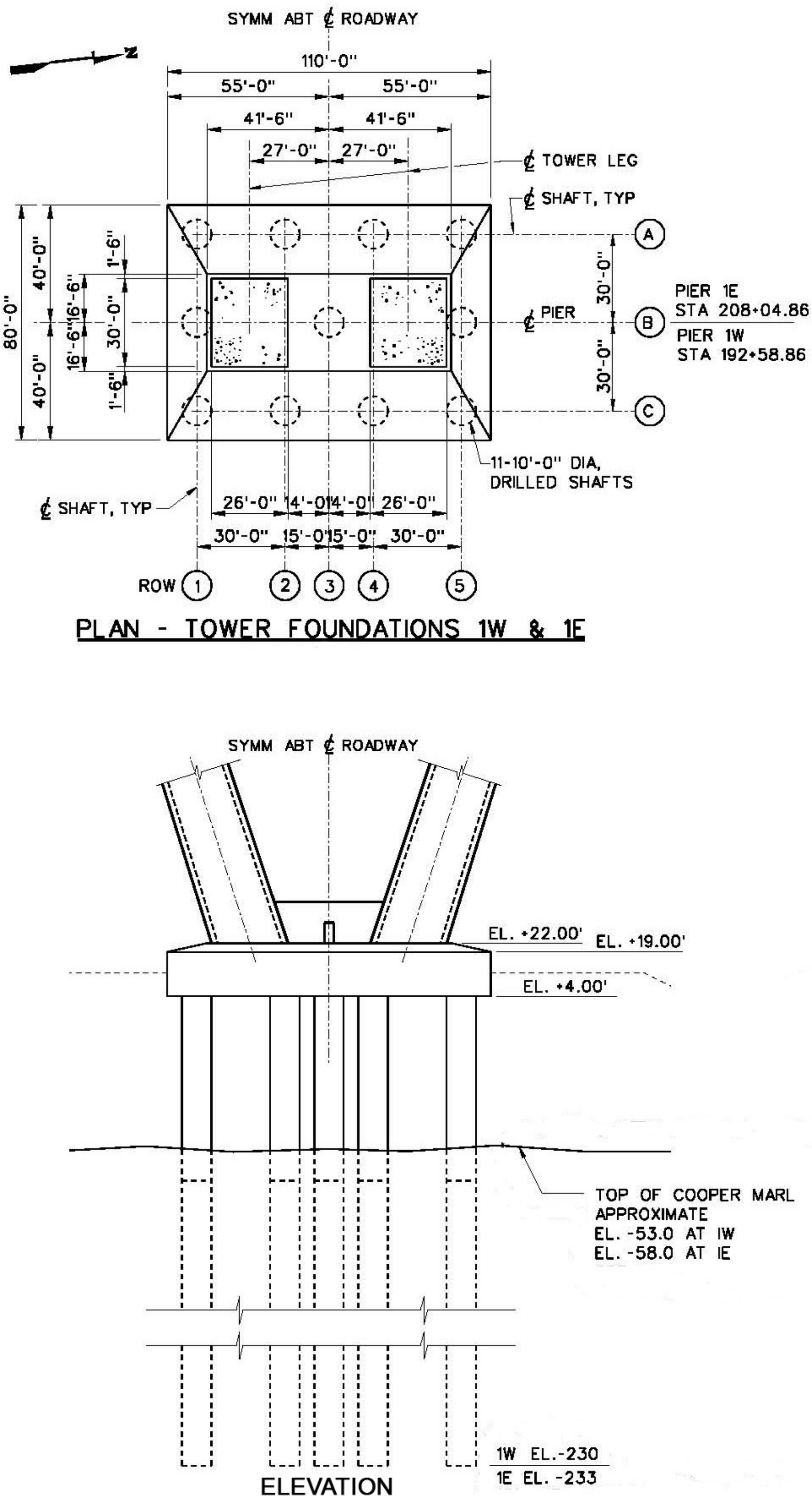

Figure 4. Main tower foundation

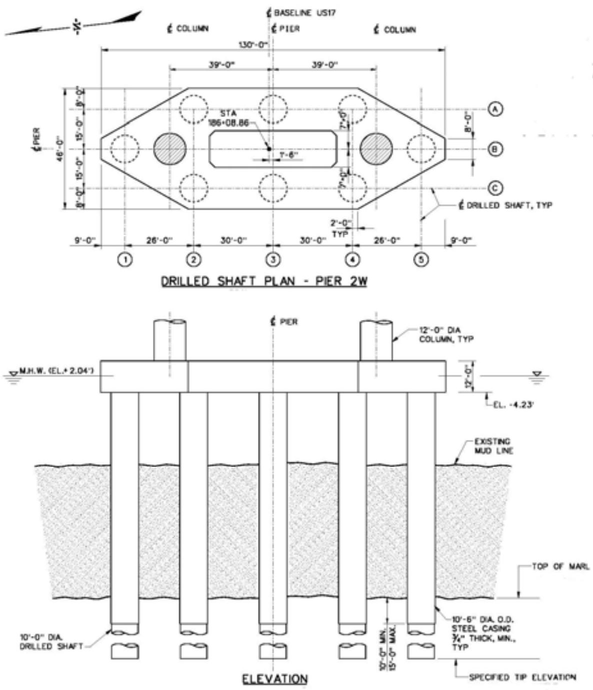

Figure 5. Pier 2W foundation

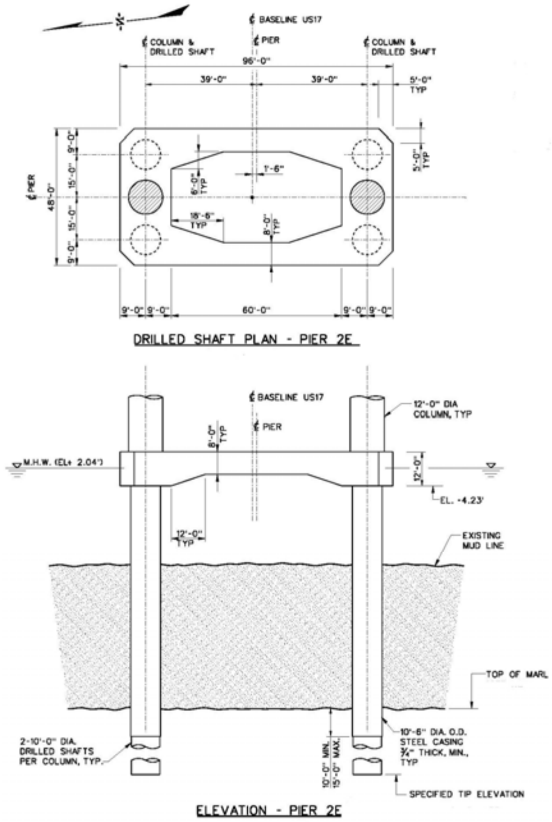

Figure 6. Piers 2E, 3W and 3E foundations

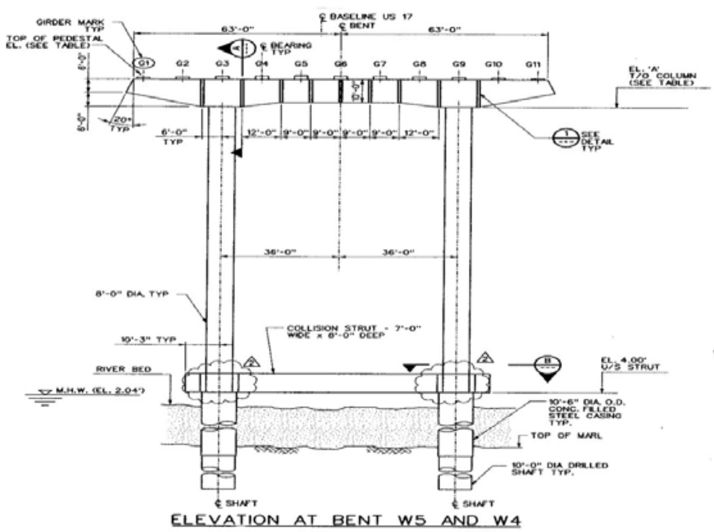

Figure 7. Typical high level approach foundation

load cells with a combined maximum load capacity of 6,000 kips.

Tables 1 and 2 summarize of pre-construction and construction stage axial shaft test results in the Marl formation.

Table 1: Summary of pre-construction axial shaft test results in Cooper Marl

Drilled Shaft Axial Resistance	Range (Ksf)	Average (ksf)
Ult. Unit Side Friction (cased)	-	1.3
Ult. Unit Side Friction (cased)	3.5 to 4.1	3.8
Ult. Unit End Bearing (at El. -100')	43 to 61	54
Ult. Unit End Bearing (at El. -150')	59 to 80	71

Table 2: Summary of contract axial shaft test results in Cooper Marl

Drilled Shaft Axial Resistance	Range (ksf)	Average (ksf)
Ult. unit side friction (cased)	-	1.25
Ult. unit side friction (uncased above El. -150')	3.65 to 3.99	3.8
Ult. unit side friction (uncased below El. -150')	5.24 (before ultimate reached)	5.24
Ult. end bearing at El. -216'	119 (at 3" Settlement)	119

SHAFT LATERAL LOAD TESTING

After completion of the axial load tests, a total of twelve drilled shafts were subjected to lateral loading at the three test sites. The Charleston site is characterized by 50 to 65 ft of soft clays and organic clays overlying the Cooper Marl. The Mt. Pleasant site is characterized by 50 ft of liquefiable loose sand overlying the Cooper Marl. The Drum Island site is characterized by 30 ft of loose to dense sand overlying the Cooper Marl. Figure 8 shows the soil profile at the three lateral load test sites. Shafts were tested to provide measures of soil response under lateral loading with the following variations (Brown, et al; 2002): (1) shafts with and without permanent steel casings, (2) cyclic loading in both sand and soft clays, (3) lateral loading at a high rate of displacement, (4) lateral loading using the split cylinder technique with embedded O-cells, and (5) test shafts in sand subject to blast-induced liquefaction. Cyclic lateral tests, inertial (statnamic) load tests, and split cylinder tests were performed. Detailed descriptions and summary of these lateral load tests were presented by Brown (Brown, et al 2002).

The soil response of the Cooper Marl Formation was most important in design of the bridge foundations. In order to measure soil resistance vs. displacement relationships to significant displacement values, the split cylinder test was performed. This technique was first attempted by O'Neil (O'Neill 1997). With the split cylinder test, the soil resistance vs. displacement relation is observed and used to generate p-y curves for design. The results of the two split cylinder tests are presented in Figure 9. On the same figure, a computed p-y curve using the Reese's criterion for stiff clay below free water, with the following soil properties: c = 30 psi, k = 500 pci, ε_{50} = 0.005, matches the test results very closely up to a displacement of around 0.04 m (1.6 in).

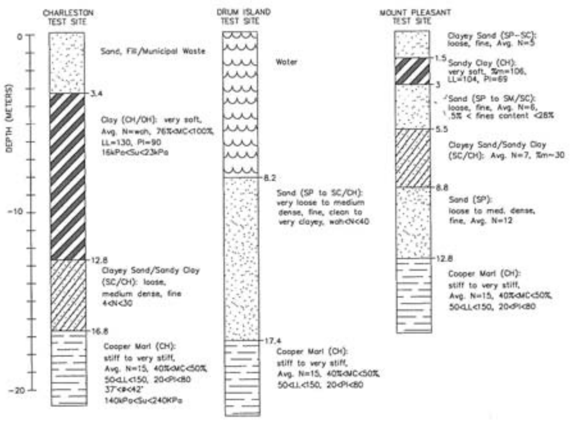

Figure 8. General soil conditions at lateral load test locations

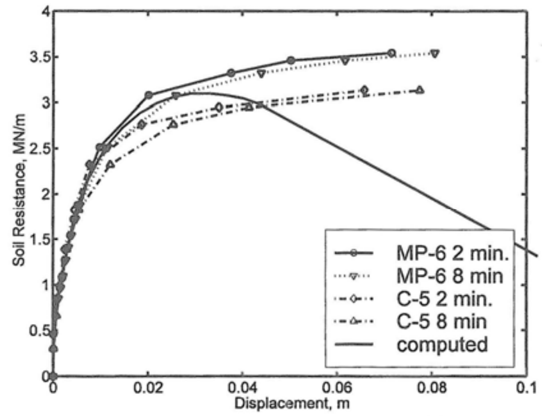

Figure 9. p-y curves from split cylinder test

Major technical conclusions from the lateral testing program include (Brown, et al, 2002):

- permanent casing has a substantial beneficial effect on the structural capacity of a large diameter shaft in bending
- cyclic degradation of the Charleston soft clays was relatively small
- rate of loading effects for 1^{st} cycle dynamic loading of 8 ft diameter shafts in Charleston soft clays were represented by a viscous damping ratio of around 55%
- derived static response captured from inertial lateral loading using the statnamic device was almost identical to static response measured using conventional lateral tests
- split cylinder tests suggest that the Reese p-y criterion for stiff clay below water models the initial static response of the Cooper marl fairly well
- blast-induced liquefaction effects on Mt. Pleasant sands were characterized using p-y curves for sand with R_u values of 0.7 to 0.85
- the higher R_u value is suggested by the higher rate of loading associated with the rapid inertial lateral loading test; the lower Ru value is associated with the relatively slow cyclic loading and may be influenced by dissipation of pore pressures with time after blasting.

DRILLED SHAFTS AXIAL RESISTANCE DESIGN

In the axial design of the drilled shafts, only combined friction and end bearing in the Marl formation were considered. Friction was neglected at the bottom of the shaft for a length of one shaft diameter to account for strains developed in the marl due to loads applied at the base of the shaft. Shaft load tests confirmed a ultimate friction value of 1.3 ksf within the cased section of the shaft, and an average ultimate friction of 3.8 ksf below the casing within the marl above EL. -150 ft. Therefore, these values were used for the design of all of the drilled shafts except for shafts with a tip elevation above EL. -100 ft and the section of shafts below EL. -150 ft at the main piers.

Based on the pre-construction and construction stage load test results and foundation settlement considerations, the following unit friction and end bearing values were developed for use in designing the axial resistance of all the pier shafts:

Ultimate friction resistance within the cased depth:	1.3 ksf *
Ultimate friction resistance in marl, above El. –150:	3.8 ksf
Residual friction resistance in marl, above El. –150:	3.2 ksf
Ultimate friction resistance in marl, below El. –150:	5.8 ksf

Residual friction resistance in marl, below El. –150:	4.9 ksf
End bearing resistance (at 3-inch base settlement):	½ x 119 ksf = 59.5 ksf
End bearing resistance (at 6-inch base settlement):	½ x 130 ksf = 65 ksf

* The ultimate friction value used along the cased length of shaft for determining shaft penetration was reduced to 0.65 ksf (one-half of the value shown) to account for anticipated disturbance to the marl due to installation of the contractor's proposed template support piles in close proximity to the drilled shafts.

Design Factor of Safety

Except at the main piers, factors of safety of 2.0 and 3.0 were applied to the ultimate friction and end bearing resistance values, respectively, for AASHTO Group I loads to account for displacement compatibility at the sides and base of the shaft. At the main piers, compression of the marl bearing stratum and related settlement of the drilled shafts due to the weight of the rockfill island resulted in about 4 inches of settlement at the shaft base. Therefore, for the main piers, a factor of safety of 2.0 was used for both friction and end bearing since displacement compatibility was not an issue. In addition to the above criteria, the design required a minimum factor of safety of 1.3, assuming no end bearing resistance. This later requirement ensures that the shafts have adequate capacity in the unlikely event that undetected problems during construction jeopardize the end bearing resistance.

For AASHTO Group II through VI loads, the factors of safety for friction and end bearing were decreased by 25 to 30 percent, consistent with the AASHTO overstress allowances for these load combinations. For seismic compression loads (2,500 year earthquake), a factor of safety of 1.1 was applied to friction resistance, and a factor of safety of 1.0 was applied to end bearing, consistent with AASHTO guidelines. For seismic tension loads, a factor of safety of 1.5 was applied to friction resistance.

The shaft axial resistance design was carried out by Parsons Brinkerhoff.

Downdrag Loads Induced by Rockfill Island Settlements at Piers 1W and 1E

The weight of the rockfill islands at Piers 1W and 1E will cause settlements of the marl and drilled shaft foundations. Settlements of the rockfill island include immediate settlement due to deformation and elastic compression (estimated to be 5 inches), and long-term consolidation settlement (estimated to be 10 inches).

Due to the settlement of the rockfill islands, the rockfill will settle relative to the drilled shafts, inducing a down drag load to the drilled shafts. At the Piers 1W and 1E islands, these down drag loads are estimated to be 750 kips and 850 kips, respectively, on each drilled shaft. Settlement of the marl relative to the drilled shafts will result in increased settlement of the drilled shafts, and development of down drag forces on the shafts. Along the shaft axis, a "neutral point" will develop that represents the point of zero relative displacement between the drilled shaft and the surrounding marl bearing stratum. For equilibrium, the structure loads and down drag loads above the neutral point will be resisted by a combination of end bearing resistance at the base of the shaft and positive side friction below the neutral point.

The foundation settlement (i.e., the settlement of the top of the drilled shafts after drilled shaft cap construction) was estimated by: (1) determining the elevation of the neutral point along the shaft axis using the unit friction and end bearing resistance values determined from the load tests, (2) determining the settlement of the marl at the level of the neutral point, (3) setting the settlement of the drilled shaft at the neutral point equal to the settlement of the marl at this same level, and (4) adding the elastic shortening of the drilled shaft above the neutral point to the settlement at the neutral point to estimate the top of shaft settlement. Following the above procedure, the foundation settlement was estimated to be approximately 4 inches at both Piers 1W and 1E.

At Pier 1W, the neutral point was estimated to be at El. –116, and the corresponding down drag load from the marl above this level was estimated to be 6,300 kips. At Pier 1E, the neutral point was estimated to be at approximately El. –120, and the corresponding down drag load from the marl above this level was estimated to be 6,200 kips.

The un-factored design axial loads at Piers 1W and 1E are:

- AASHTO Group I: 11,000 kips
- AASHTO Groups II through VI: 14,500 kips
- Seismic Load (from time history analyses): 19,000 kips (compression)
  5,655 kips (tension)

Considering the above design parameters and loads, the penetration of the shafts are governed by the Group I loads and the final design tip elevations for Piers 1W and 1E are -230 ft and -233 ft, respectively.

For axial design of the shafts, only combined friction and end bearing in the Marl formation were considered. In addition, friction was neglected at the bottom of the shaft for a length of 1 shaft diameter.

DRILLED SHAFT LATERAL RESISTANCE DESIGN

The foundation lateral resistance design is dominated by seismic loads; therefore, this paper focuses on the seismic design aspect of the foundations.

The Cooper River Bridge is located in one of the most seismically active regions in the eastern United States. The largest recorded earthquake in the area occurred near Charleston on August 31, 1886 with an estimated moment magnitude of 7.3. Much of the seismicity in this region is related to reactivation of buried faults and sutures under horizontal compressive stress field in the east-northeast direction. Descriptions and discussions of local and regional geologic/seismotectonic setting, earthquake source characterization and paleoliquefaction seismicity data are summarized in Ref. G5, Ref. G19 and Ref. G31 of Supplemental Criteria for Seismic Design for U.S. 17 Cooper River Bridge.

A two-level earthquake design approach was developed by SCDOT, FHWA and the Seismic Review Panel for this project. The higher level performance criteria cover project Critical Access Path (CAP) structure which included the main spans over the Cooper River, high level approaches, low level approaches, one ramp entering and exiting the bridge on the Charleston side and one ramp entering and exiting the bridge on the Mount Pleasant side.

The Safety Evaluation Earthquake (SEE) is defined as an earthquake event that has a return period of 2,500 years. The design earthquake moment magnitude and the source-to-site distance for the SEE are taken as Mw = 7.25 and R = 33 km, respectively. For the SEE, all structures are designed to maintain life safety and prevent major failure. The CAP structures are designed to respond with repairable damage and non-CAP structures may suffer significant non-repairable damage. For the foundations, all structure elements are designed to remain essentially elastic under SEE event and also designed as capacity protected elements to ensure that damage wouldn’t occur in the foundation elements.

The Functional Evaluation Earthquake (FEE) is defined as an earthquake event that has a return period of 500 years. The design earthquake moment magnitude and the source-to-site distance for the FEE are taken as Mw = 6.5 and R = 45 km, respectively. At the FEE, the CAP structures are designed to respond in essentially an elastic manner (minimal damage). All CAP structures will remain fully operational during and after the FEE. The Non-CAP structures are designed to respond with limited damage, with any damage that does occur being accessible and readily repairable. For the foundations, all structure elements are designed to remain elastic under FEE event.

Site specific ground motion hazard analysis was conducted to develop the design response spectra. The two levels of earthquake considered were the FEE and the SEE. The design horizontal acceleration response spectra on the outcrop at the base of Cooper Marl are presented in Figure 10. The vertical acceleration response spectra were taken as about 80% of the horizontal spectra. As indicated in the figure, the ground shaking intensity for the 2,500–year SEE event is significantly higher than that for the 500-year FEE event, with the SEE to FEE peak ground acceleration (PGA) ratio exceeding 6 and the spectral acceleration ratio exceeding 10 for the period equal to 0.75 sec and shorter. As a result, the design of the Cooper River Bridge is governed by the 2,500-year SEE.

Equivalent linear one-dimensional site response SHAKE analysis (Schnabel et al. 1972) was performed primarily to (1) account for the site amplification effect due to the presence of Cooper Marl and recent deposits, (2) provide design spectra for the preliminary response spectral analysis, and (3) provide ground motion time histories for inelastic time histories analysis.

Three sets of ground motion time histories were developed for the safety evaluation earthquake (ICEC, 2000). For the FEE, one set of ground motion time histories was developed. Table 3 lists the seed acceleration time histories and the PGA, PGV, and PGD values of the resulted response-spectrum-compatible bedrock time histories.

Table 3 Response-Spectrum-Compatible and Seed Time Histories

	Source and M_w of Seed Time Histories	Response-Spectrum-Compatible Time Histories		
		PGA (g)	PGV (in/sec)	PGD (in)
SEE	Joshua-Tree, 1992 Landers Earthquake, M_w = 7.3	0.66	26	15
	Cerro-Prieto, 1979 Imperial Valley Earthquake, M_w = 6.5	0.65	22	18
	Tabas, 1978 earthquake in Iran, M_w = 7.4	0.65	25	16
FEE	Carson-Catskill, 1978 Whittier Narrows Earthquake, M_w = 5.9	0.11	2.9	1.5

Main Span Unit Foundations

The substructures of the main span unit include two main piers (1W and 1E), two anchor piers (2W and 2E) and two end piers (3W and 3E). Response spectrum analysis (GTStrudl) was utilized to perform the preliminary seismic design of these main span unit foundations. The final seismic design was based on the results of full inelastic time history analyses, using the computer program ADINA, which considers spatial variation ground motion time histories to account for wave-passage effects, wave scattering / incoherency effects and local site response effects. Two loading effects were considered in the design of drilled shafts and caps: (1) the inertial effect from the structure, and (2) the kinematic effect from the ground displacements. The kinematic coupled soil-structure interaction effect is accounted for by using nonlinear hysteretic p-y springs, t-z and q-z springs along the drilled shafts while the drilled shafts were modeled as moment curvature elements. The parameters used to derive the nonlinear p-y springs are presented in the Table 4. Figure 11 illustrates the modeling of spatially varying ground motions (varying in elevation as well as in horizontal direction) that were applied to the support ends of the nonlinear springs to account for the kinematic interaction effects.

From the above analyses results, it was concluded that the plastic moment at the bottom of the column mainly affects the axial forces in the drilled shafts. On the other hand, the moment and shear forces in the drilled shafts are mainly caused by the inertia forces of drilled shaft cap and are not affected by the plastic moment at the bottom of the column.

Table 4 Soil Parameters Used to Define Nonlinear p-y Springs

Soil Type	Friction Angle	Cohesion C (psi)	50% Strain ε_{50}	Sub-grade K (pci)	Liquefaction p-multiplier *
Loose to Medium Sand (Reese Model)	30 – 34	-	-	20 – 40	0.15 – 0.25
Dense Sand (Reese Model)	36 – 38	-	-	60 – 90	-
Soft to Firm Clay (Matlock Model)	-	2 – 7	0.01 – 0.02	-	-
Cooper Marl (Stiff Clay w/ Water)	-	30	0.005	500	-

*The p-multiplier was used to represent the residual strength/stiffness of liquefied loose to medium sand.

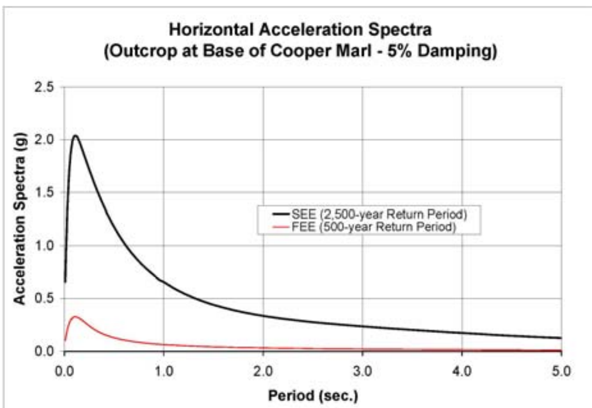

Figure 10. Outcrop horizontal response spectra

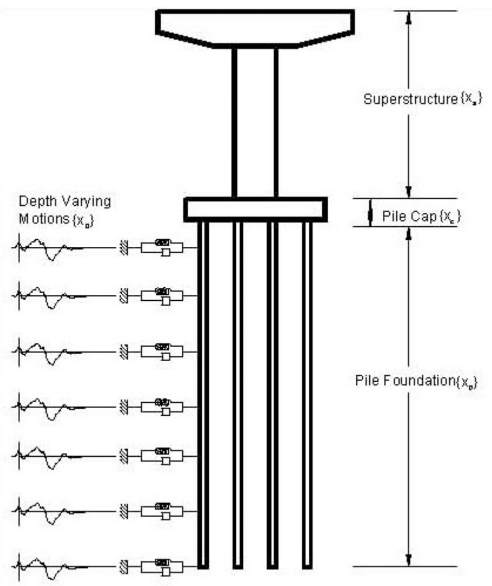

Figure 11. Modeling of kinematic interaction with spatially varying ground motions

BCG developed a computer program for the design of drilled shafts subjected to concurrent loads (axial load combined with flexural bending, and axial load combined with shear forces) that were derived from the elastic and nonlinear time histories results. This program calculates the required longitudinal reinforcement with the assumption that plane section remains plane after the application of loadings and other design assumptions in Section 8.16.2 of AASHTO Standard Specifications. By using this program, the drilled shaft design considers concurrent axial load and flexural bending combination directly retrieved from analysis results of each interval of the time histories. The design cross sections are about 5ft to 10ft along the drilled shafts per GTStrudl and ADNIA models. In addition, the program calculated the required shear strength provided by transverse reinforcement (Vs) at each interval of the time history considering the concurrent axial load effect on the shear strength of concrete.

The drilled shaft caps were designed using the 3-dimensional strut and tie method. In this strut and tie model, the concurrent time history response reactions (axial loads, biaxial bendings and shears) at the top of the drilled shafts were considered to determine the required tie forces and the compressive stress of the struts. For the moment-resisting connections between the members (column/drilled shaft cap and drilled shaft cap/drilled shaft), required amount of horizontal and vertical joint shear reinforcement was determined based on the magnitude of principal tension stress in the joint. BCG also develop a computer program for the cap design which calculated the 3-dimensional strut and tie forces and principal tensile stresses per concurrent time history result at the bottom of the columns and the top of the drilled shafts.

High Level Approaches Foundation

The foundations for the high level approaches (drilled shafts) were design based on the response spectrum analysis and pushover analysis results and the stress and strain demands were confirmed per final ADINA nonlinear time history analysis (similar with the main span unit nonlinear time history models). Along the high level approaches, each pier column is supported by a single drilled shaft. The drilled shaft segment above the mud-line was included in the global model; while the portion of the drilled shaft below the mudline was represented by a 6x6 equivalent stiffness matrix and mass matrix. This modeling technique was found to be very effective in addressing plastic hinging formed at a location above the shaft/column interface, and is a compromise between the stiffness matrix model and the complete model approaches.

Analyses were conducted for SEE seismic loads (Group VII) and non-seismic loads, Groups I, through VI and VIII). In addition, Group VII loads also include results from the transverse and longitudinal pushover analysis. For each drilled shaft, the analyses were conducted for two scour-related conditions: the 50% of the 500-year scour and the non-scour condition.

The computer program LPILE (Ensoft Inc.) was used to compute the drilled shaft 6x6 stiffness matrix at mud-line, also the deflection, shear, bending moment and soil response with respect to depth. The program uses the p-y method in an iterative procedure to model the nonlinear soil response and to analyze the soil-shaft interaction behavior under lateral loading.

BCG developed a computer program for the design of the high level approach drilled shafts. The features of this program are: (1) directly retrieved the LPILE analyses output (shear and bending moment versus depth), (2) calculated the drilled shaft axial load distributions by using the axial loads at the top of the drilled shaft along with its self-weight and skin friction, (3) calculated the required longitudinal reinforcement with was the assumption that plane section remains plane after the application of loading and other design assumptions in Section 8.16.2 of standard specifications of AASHTO, and (4) calculated the required shear strength provided by transverse reinforcement (Vs) while considering the concurrent axial load effect on the shear strength of concrete.

Due to the large design earthquake magnitude (M_w = 7.25), it was determined that all loose to medium dense sand deposits on the Mt. Pleasant side (Piers 5E to 10E) could potentially liquefy under the SEE event (PGA=0.65g). The foundations were designed to accommodate both the direct seismic loading due to vibration and the loadings induced by lateral spreading. It is considered appropriate to design for vibration and lateral spreading in a decoupled manner as the peak acceleration of the structure typically happens before full liquefaction develops, and therefore, before significant soil movement begins.

In the vibration case, the foundation design considered the following: (1) vertical support of soil above the Cooper Marl was ignored, (2) down-drag forces were added to the design load, and (3) lateral resistance of liquefied soil was reduced to its residual strength and is modeled in LPILE with a p-multiplier of 0.15 to 0.25 (see Table 7.2). Also, liquefaction-induced lateral spreading was considered in the design of the drilled shafts on the Mt. Pleasant side with a continuous layer of liquefiable sand.

In order to estimate the magnitude of lateral spreading, Youd's empirical procedure was used for the SEE (M=7.5, R=30 km) and FEE (M=6.5, R=45 km) events (Youd, et al's 1999). Using an average liquefiable thickness of 40 ft, with an $(N1)_{60}$ less than 15, and an average gradient of 2%, lateral displacements on the order of 3 ft for the SEE and 1 to 2 inches for the FEE were calculated.

For the effects of lateral spreading on the drilled shaft design, two scenarios of spreading conditions were considered: (1) soil flow around the drilled shaft - liquefied soil without crust, see Figure 12, and (2) spreading that displaces foundation with soil – liquefied soil with crust (see Figure 13). For scenario 1, an LPILE analysis was performed by prescribing a 3-ft ground displacement profile within the upper 40 ft of the soil, pushing toward a 10-ft diameter drilled shaft via the p-y spring, and modeling the residual strength of the liquefied sand by applying a p-multiplier of 0.15 to 0.3. The calculated maximum deflection at the top of the shaft is just a little more than an inch and it was concluded that the spread movement has no impact during the SEE event in this case. For scenario 2, it was assumed that a 10 ft thick non-liquefied soil crust rides on the top of the liquefied layer. Similar LPILE analysis was performed and concluded that, for a 5-ft diameter shaft, the deflection at the top of shaft would exceed 30 inches, and for 8-ft diameter shaft, the deflection at the top of shaft would be about 5.5 inches and the response of superstructure to the foundation deflection is considered acceptable. These analyses demonstrate the advantages of using large-diameter drilled shafts in potentially liquefiable soils. The maximum pile deflection caused by lateral spreading was reduced, which eliminated the need for additional liquefaction mitigation. Also, the response of the structure to the foundation deflection as a result of lateral spreading was found to be acceptable.

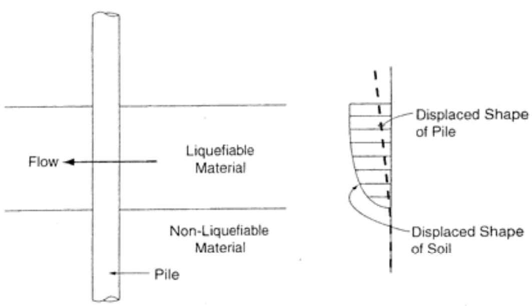

Figure 12. Soil Flow around shaft - liquefied soil without crust

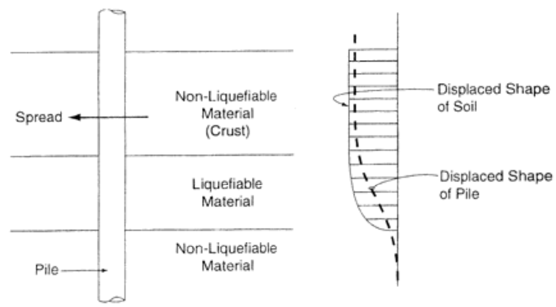

Figure 13. Spreading that displaces foundation with soil – liquefied soil with crust

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REFERENCES

American Association of State Highway and Transportation Officials (1999). AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition, 1999 Interim Revisions.

Applied Technology Council (1996). ATC-32, Improved Seismic Design Criteria for California Bridges: Provisional Recommendations.

Brown, D.A. (1999). "An Experiment with Statnamic Lateral Loading of a Drilled Shaft," Proceedings, OTRC'99 Conf., Austin, TX, ASCE GSP No. 88, pp. 309-318.

Brown, D.A., and Camp, W.M. III (2002). "Lateral Load Testing Program for the Cooper River Bridge, Charleston, SC," Proceedings of the International Deep Foundations Congress 2002, Orlando, FL, GSP 116, Vol. 1, pp. 95-109.

Camp, W.M. III, Brown, D.A., and Mayne, P.W. (2002). "Construction Method Effects on Axial Drilled Shaft Performance," Proceedings of the International Deep Foundations Congress 2002, Orlando, FL, GSP 116, Vol. 2, pp. 193-208.

Camp, W.M., (2004). “Site Characterization and Subsurface Conditions for the Cooper River Bridge,” Geo-Trans Conference, Geo-Institute of ASCE, Los Angeles Vol. 1, pp. 347-360.

Castelli, Raymond J. (2004). “Design of Drilled Shaft Foundations for the Cooper River Bridge,” Geo-Trans Conference, Geo-Institute of ASCE, Los Angeles. Vol. 1, pp. 334-346.

ICEC Inc. (2000). Final Report: Cooper River Bridges Replacement Project – Ground Motion Time Histories, Prepared for Parsons Brinckerhoff Quade & Douglas, Inc. and South Carolina Department of Transportation, 2000.

Lam, I. P, and Martin, G. R. (1986). Seismic Design of Highway Bridge Foundations – Vol. II, Design Procedures and Guidelines, Report No. FHWA-RD-86-102.

O’Neill, M. W., Brown, D. A., Townsend, F. C., and Abar, N. (1997). “Innovative Load Testing of Deep Foundations,” Transportation Research Record 1569, Transportation Research Board, Washington, DC, pp.17-25.

Power, M.S., Wells, D.L., Youngs, R.R., and Chiou, B.S.J. (2004). “Design Ground Motions for Cooper River Bridge, Charleston, South Carolina,” Geo-Trans Conference, Geo-Institute of ASCE, Los Angeles. Vol. 1, pp. 369-380.

S&ME, Inc. (2000). “Phase II Geotechnical Data Summary Report-Cooper River Bridge Replacement Project, Charleston, SC,” Report Submitted to Sverdrup Civil, Inc., New York, NY.

South Carolina Department of Transportation (2000). Supplement Criteria for Seismic Design for U.S. 17 Cooper River Bridge, Charleston, South Carolina.

Wang, J-N., Mesa, L., and Sizemore, J. (2004). “Seismic Design of Cooper River Bridge,” GeoTrans Conference, Geo-Institute of ASCE, Los Angeles. Vol. 1, pp. 381-392.

Youd, T.L., Hansen, C.M., and Barlett, S.F. (1997). “Revised MLR Equations for Predicting Lateral Spread Displacement, Proceedings of the 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facility and Countermeasures Against Soil Liquefaction, August, 15-17, Seattle.