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INELASTIC SEISMIC DESIGN OF FLEXIBLE HIGH LEVEL APPROACHES TO THE COOPER RIVER BRIDGE

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SUMMARY

The New Cooper River Bridge being built in South Carolina connects the City of Charleston and the Town of Mount Pleasant. This design/build project comprises cable-stayed main spans over the Cooper River, high-level and low-level approaches, and interchanges. The bridge site is within one of the most seismically active regions in the eastern US. This paper describes the design challenges for the very tall and slender high level approaches leading to the main spans, which are required to remain fully operational for the Function Evaluation Earthquake and to respond with repairable damage for the Safety Evaluation Earthquake. Such challenges include development of overall design strategy to meet the stringent performance criteria while minimizing construction costs, and optimization and performance verification of the structural systems using sophisticated inelastic analyses. Such analyses include inelastic static pushover and nonlinear time-history analyses incorporating both geometric and material nonlinearities.

INTRODUCTION

Currently under construction in South Carolina, the New Cooper River Bridge replaces the Grace Memorial Bridge and the Silas N. Pearman Bridge over the Cooper River, connecting the City of Charleston with the Town of Mount Pleasant. This $600 million (US) design/build project comprises cable-stayed main spans over the Cooper River, high-level and low-level approaches, and interchanges. The bridge will have the longest cable-stayed span of 471 m (1546 ft) in North America when it opens in 2005.

The bridge site is within one of the most seismically active regions in the eastern US, as represented by the 1886 Charleston earthquake (magnitude 7.3). Stringent seismic performance and design criteria were specified for this project by the South Carolina Department of Transportation [1]. This paper describes the design challenges to develop an economical design, in the competitive design/build environment, for the very tall and slender high level approaches leading to the main spans. Such challenges include

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development of overall seismic design strategy to meet the stringent performance criteria while minimizing construction costs, and optimization and performance verification of the structural systems using sophisticated inelastic analyses. Such analyses include inelastic static pushover and nonlinear time-history analyses incorporating both geometric and material nonlinearities.

SEISMIC PERFORMANCE CRITERIA

The Cooper River Bridge has been identified as a critical facility that should remain in full operation during and after a small to moderate earthquake and provide a route for emergency traffic in the case of a significantly large seismic event. Towards this end, a two-level earthquake hazard design approach has been adopted for this project: the Functional Evaluation Earthquake (FEE) with a return period of 500 years, and the Safety Evaluation Earthquake (SEE) having a return period of 2500 years.

A Critical Access Path (CAP) has been established across the structures to meet the performance requirements in a cost effective manner. The structures comprising the CAP shall be designed to a higher level of performance than those outside the CAP. The high level approaches have been identified as CAP structures that are required to remain in the elastic range under the FEE and to respond with repairable damage for the SEE. The high level approaches should provide access for emergency traffic immediately following the SEE and should be repaired and returned to service shortly after the SEE.

SOIL CONDITIONS

The entire bridge site is underlain by a layer of stiff clay known as Cooper Marl at a depth of 15 to 18 m (50 to 60 ft). The outcropping firm ground (soft rock) at the base of the Cooper Marl is estimated to be at a depth of about 90 m (300 ft) from the ground surface. The Cooper Marl is overlaid by soft alluvial deposits for the riverbed and by soft surficial soils for the land portions. The bearing stratum throughout the site is the Cooper Marl.

SEISMIC HAZARD AND INPUT GROUND MOTIONS

The bridge site is located within one of the most seismically active regions in the eastern US. Seismic hazard in this region is dominated by the Charleston characteristic earthquake as represented by the 1886 Charleston earthquake with an estimated magnitude of 7.3.

Design response spectra (5% damped) for the SEE and the FEE were developed for outcropping firm ground (soft rock) at the base of the Cooper Marl. Three sets of 3-component (longitudinal, transverse and vertical) time histories were developed at the base of the Cooper Marl as input ground motions for the SEE and the FEE respectively. Each set of 3-component input time histories was generated using a set of historical records from a past event as the starting motions and then modified to match the target response spectra at the base of the Cooper Marl. The following three sets of historical records were used:

- Cerro Prieto of 1979 Imperial Valley;
- Joshua Tree of 1992 Landers; and
- 1978 Tabas.

Site response analyses were carried out to propagate the ground motions at the base of the Cooper Marl to the foundation levels. Due to the long length of the approach structures, the effects of spatial variations of ground motions were considered in developing the time histories at the foundation levels. The design response spectra at the foundation levels for the SEE and the FEE were based on the mean plus one standard deviation response spectra of the time histories obtained from the site response analyses. The site response analyses were carried out for horizontal ground motions only. For vertical ground motions at the foundation levels, the rock response spectra and time histories were used. Figures 1 and 2 show the SEE horizontal and vertical design response spectra (5% damped) at the foundation levels for the Charleston and Mt. Pleasant high level approach respectively.

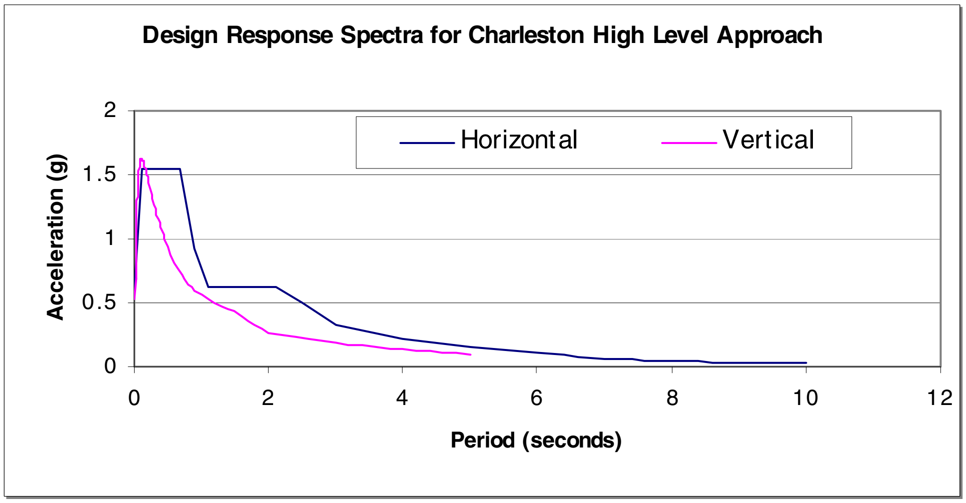

Figure 1 – Design Response Spectra at Foundation for Charleston High Level Approach

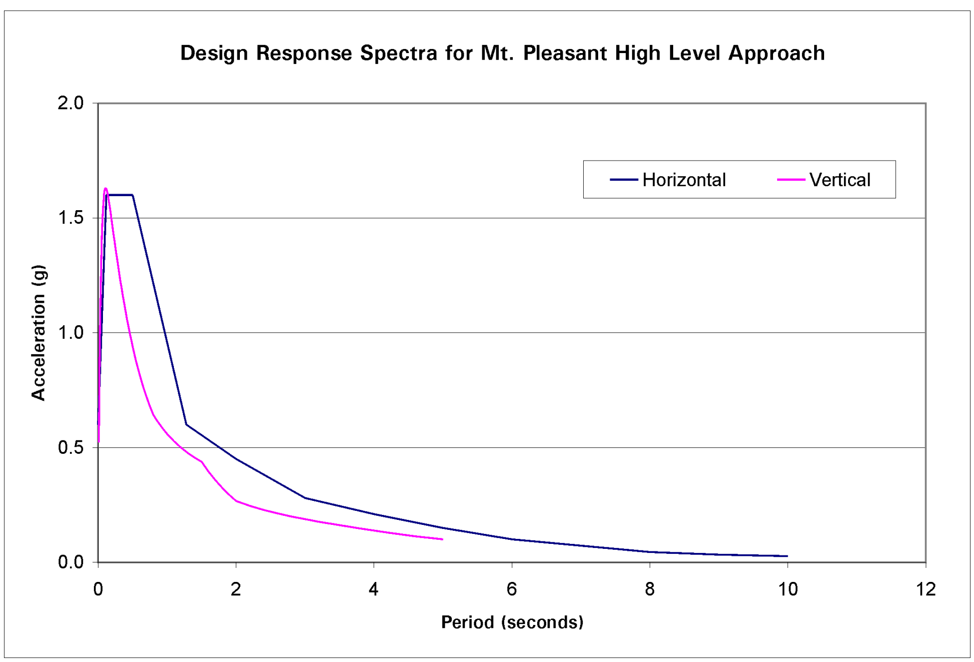

Figure 2 – Design Response Spectra at Foundation for Mt. Pleasant High Level Approach

DESIGN STRATEGY

The Charleston and Mt. Pleasant high level approaches lead to the west and east ends of the cable-stayed main spans respectively and provide eight traffic lanes (four in each direction) and a 4 m (12 ft) wide pedestrian/bicycle lane cantilevering along the south edge.

The main design challenge was to develop an economical design in a competitive design/build environment that meets the specified seismic performance and design criteria. The design strategy adopted was to minimize the weight of the superstructure and to introduce sufficient flexibility in the substructures to reduce seismic force demands. Steel plate girders composite with a concrete deck was used to minimize superstructure weight. Tall slender double column piers founded on drill shafts were used for the substructures. The use of two large-diameter drilled shafts per pier significantly reduced the construction cost for the foundations.

The tallest of the piers reach some 46 m (150 ft) above the water, and they are even taller if one takes into account the 15 to 18 m (50 to 60 ft) of soft alluvial deposits overlaying the Cooper Marl. To prevent dynamic instability for these very tall piers, the strategy was to make each high level approach continuous over its total length. The continuous deck of each approach enables the shorter piers at the lower portion to brace the taller piers and to increase the redundancy of the entire system. To reduce P-delta effects, the taller piers were designed to remain elastic for longitudinal response to the SEE.

STRUCTURAL SYSTEMS

Figure 3 shows the general arrangement of the Charleston high level approach leading to the west end of the main spans. The approach superstructure comprises 11 steel plate girders composite with a 230 mm (9") thick concrete deck. The steel girders are spaced at 3.7 m (12 ft) on center, and the concrete deck has a total width of 39 m (129 ft). The superstructure is jointless over a total length of 1326 m (4350 ft) and supported by 18 concrete piers. The 17 continuous spans range from 61 m (200 ft) to 88 m (290 ft) in length. The concrete piers typically comprise a cap beam supported by two 2.4 m (8 ft) diameter circular columns spaced at 22 m (72 ft), as shown in Figure 4. The columns are founded on 3 m (10 ft) diameter drilled shafts. For the two shortest piers, the cap beam is supported by three smaller size columns on 2.4 m (8 ft) drilled shafts. Pier column height varies from 47 m (154 ft) at the main span end to 19 m (63 ft) at the low level approach end. The drilled shafts are extended well into the Cooper Marl at depths of 37 to 64 m (120 to 210 ft). Six marine piers have a collision strut between the columns at the high water level, as shown in Figure 4. Separate fender structures are provided for the two marine piers of Town Creek. Adequate clearance is provided between the fenders and the piers to prevent pounding during seismic response. The continuous superstructure is pinned to the interior piers in both the longitudinal and transverse directions using pot bearings under the steel girders. At the two end piers, the superstructure is pinned to the substructure in the transverse direction but allowed to slide longitudinally using guided bearings under the steel girders.

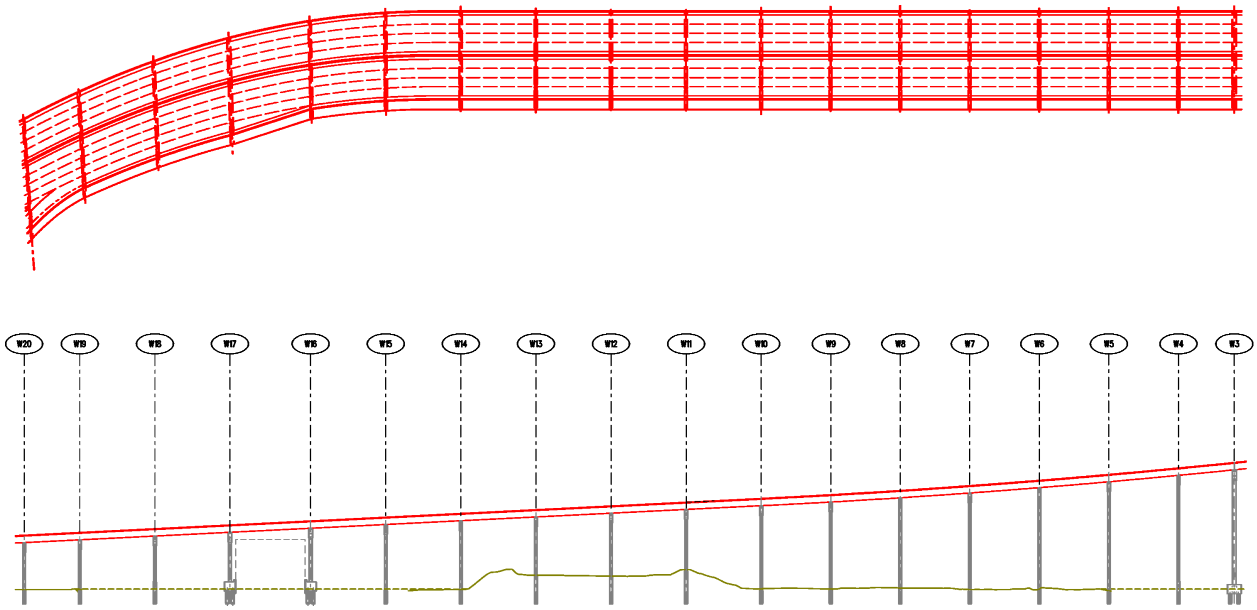

Figure 3 – General Arrangement of Charleston High Level Approach

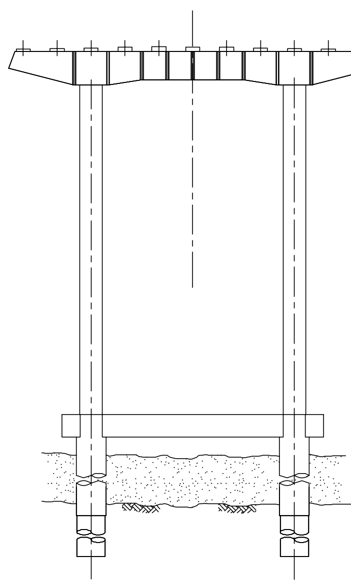

Figure 4 – Typical Concrete Pier

Figure 5 presents the general arrangement of the Mt. Pleasant high level approach leading to the east end of the main spans. The superstructure is jointless over a total length of 637 m (2090 ft) and supported by 9 concrete piers. The 8 continuous spans have a span length ranging from 70 m (229 ft) to 83 m (272 ft). The four spans of the approach closer to the main spans have deck arrangement similar to that of the Charleston approach, whereas the deck of the four spans toward the low level approach is split into two with a width of 18 m (59 ft) and 22 m (71 ft) respectively (see Figure 5). The concrete piers are similar to that shown in Figure 4 except for the split deck portion. The split deck section is supported by pairs of double column bents side by side. Pier column height varies from 47 m (154 ft) at the main span end to 15 m (48 ft) at the low level approach end. The connectivity between the superstructure and substructure is similar to the Charleston approach.

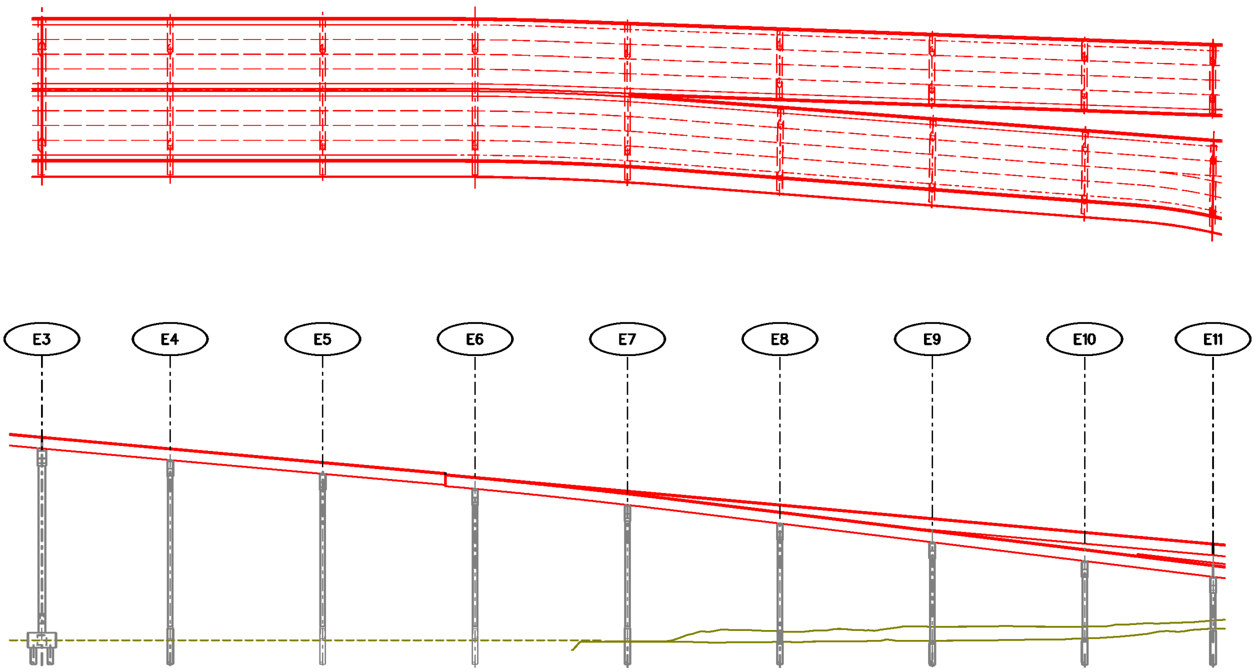

Figure 5 – General Arrangement of Mt. Pleasant High Level Approach

Tall slender piers combined with soft soil conditions yielded very flexible structures with a fundamental period of 8 to 9 seconds. While substructure flexibility reduces the seismic force demands, it leads to significant seismic displacement demands and P-delta effects. The design challenge was to properly establish the relative stiffnesses of the piers and to balance the stiffnesses and strengths of the piers to achieve the desired seismic behavior. To this end, sophisticated inelastic analyses (inelastic static pushover and time-history analyses), taking into account both geometric and material nonlinearities, were performed to optimize the structural systems and to verify seismic performance.

SEISMIC ANALYSES

The basis of performance assessment for the SEE is the limitation of strains in concrete and reinforcing steel to the following allowable values in the plastic hinge regions, as required by the project seismic design criteria [1]:

Concrete: $\varepsilon_{cmax} \le \varepsilon_{cg} = 0.67\ \varepsilon_{cu}$ [1]

Reinforcing Steel: $\varepsilon_{smax} \le \varepsilon_{sg} = 0.67\ \varepsilon_{su}$ [2]

where ε_{cmax} = peak strain demand in concrete; ε_{cg} = allowable strain in confined concrete for column performance goal; ε_{cu} = ultimate confined concrete strain (taken as 0.02 for plastic hinges detailed in accordance with current code provisions); ε_{smax} = peak strain demand in reinforcing steel; ε_{sg} = allowable strain in reinforcing steel for column performance goal; and ε_{su} = ultimate reinforcing steel strain (ranging from 0.09 to 0.12 depending on bar size).

The project seismic design criteria required that an inelastic time history analysis, incorporating geometric and material nonlinearities and multiple support excitation, be used to check these strain requirements.

Seismic analyses for design of the high level approaches comprised the following two phases:

Phase 1 – Assessment of Different Alternatives for Stiffness and Strength Distributions

In this phase, preliminary sizing of the concrete piers was made for other service loads (e.g. wind and thermal). Elastic multi-mode response spectral analysis was then performed to estimate seismic displacement demand at the deck level, Δ_{max}, assuming similar peak displacements between inelastic and elastic responses for long period structures. Inelastic static pushover analysis was carried out to estimate the displacement capacity, Δ_c, at the deck level for the whole system in the longitudinal direction and for individual piers in the transverse direction. Δ_c was taken as the lesser of

- displacement when the ultimate strain capacity for concrete or reinforcing steel has been reached in the governing plastic hinge, whichever comes first; and
- displacement when the lateral load has dropped to 80% of the peak load (after passing the peak load) due to P-delta effects.

The displacement demand was compared with the displacement capacity to see if the following criterion was met:

$$\Delta_{max} \leq 0.67 \ \Delta_c \text{ or} \qquad [3]$$
$$1.5 \ \Delta_{max} \leq \Delta_c \qquad [4]$$

If $1.5\Delta_{max}$ was greater than Δ_c, the stiffness and/or strength distributions were revised. Elastic response spectral and inelastic static pushover analyses were carried out again to estimate the displacement demand and capacity for the revised system. The combination of elastic response spectral and inelastic static pushover analyses provided a quick and effective means of assessing displacement demand vs. capacity for various alternatives of stiffness and strength distributions, converging to an optimum solution.

A 3-D computer model was developed for each of the two high level approaches. Simplified models of the cable-stayed main spans and the low level approach were included to capture the transverse interactions of the high level approach with the main spans on one end and with the low level approach on the other. Soil-structure interaction for each drilled shaft was modeled using an equivalent linear 6x6 foundation stiffness matrix. The stiffness coefficients were iterated for the load level applied, taking into account nonlinear soil behavior. The following upper and lower bound foundation stiffnesses were considered:

- upper bound - no scour and no soil liquefaction; and
- lower bound - 50% of 500 year scour plus soil liquefaction.

The foundation stiffness matrix for each drilled shaft was applied at the existing mudline for the upper bound case and at 50% of the 500 year scour elevation for the lower bound case. The displacement demand (Δ_{max}) and capacity (Δ_c) at the deck level included deformations of both the pier columns and the drilled shaft foundations.

Figure 6 shows the 3-D computer model of the Charleston high level approach.

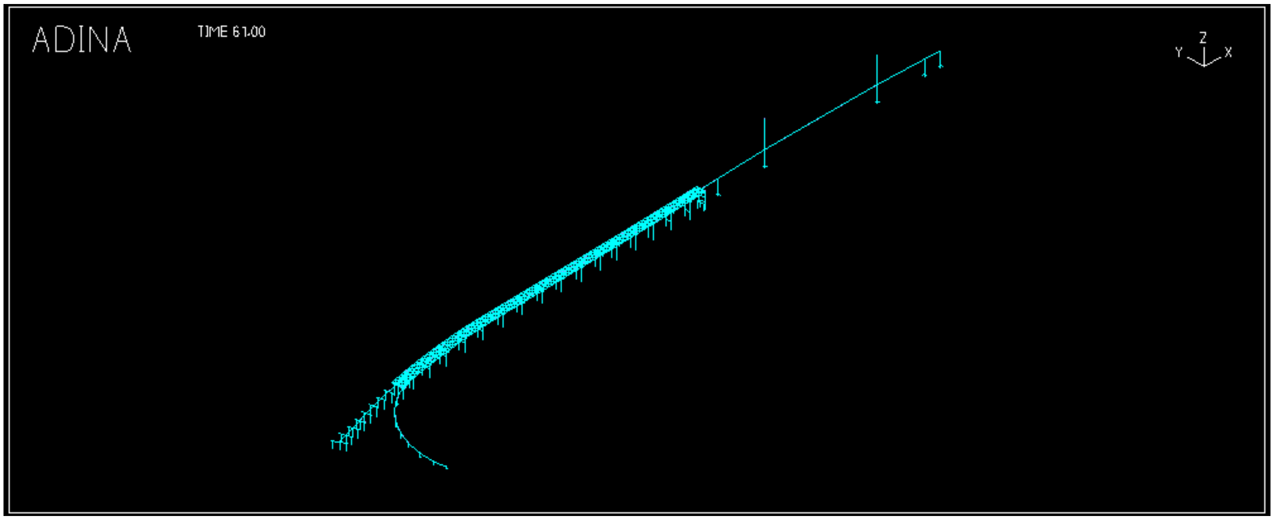

Figure 6 – Computer Model of Charleston High Level Approach

The ADINA program [2] was used to perform the inelastic static pushover analysis. Multi-linear (inelastic) moment curvature curves were developed for the plastic hinge regions in the pier columns. The effects of axial force were considered by generating a set of moment-curvature curves corresponding to different axial force levels, as shown in Figure 7. Plastic hinge length for analysis was established according to the approach proposed by Priestley et. al. [3]. Geometric nonlinearity was considered to account for both large deformation and P-delta effects.

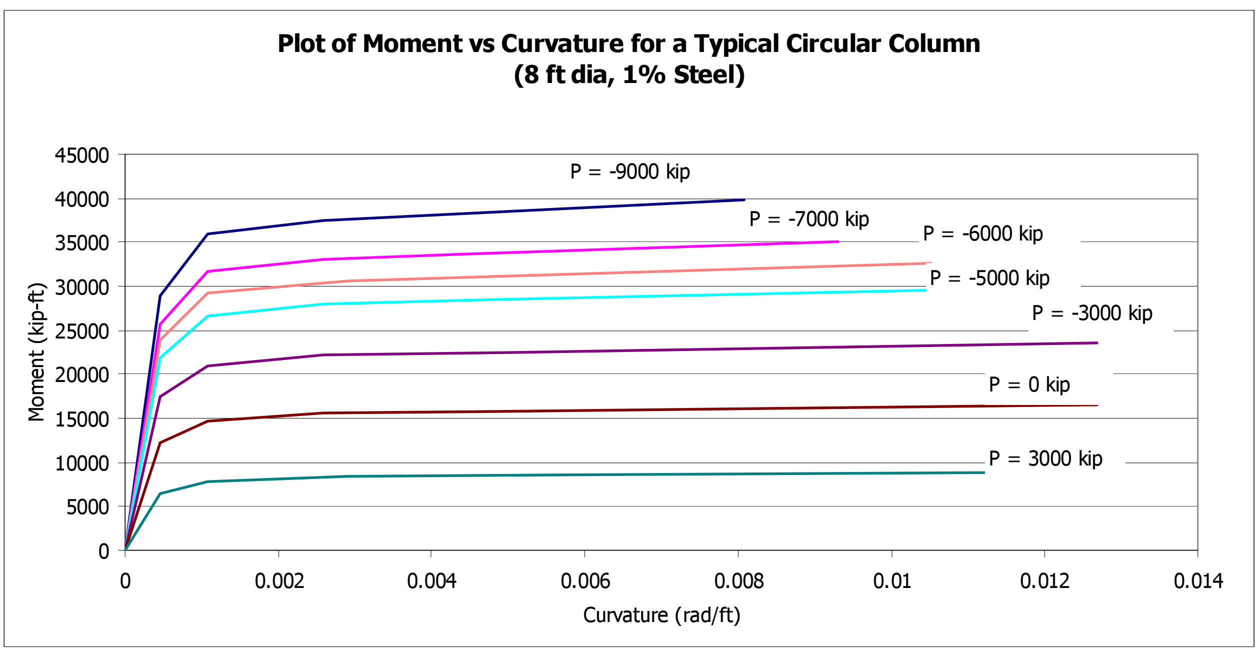

Figure 7 – Moment-Curvature Curves for Column Section under Different Axial Force Levels

Figures 8 and 9 show the longitudinal force-displacement response curve (at deck level) obtained from the static pushover analysis for the Charleston and Mt. Pleasant approach respectively. The curve terminates at the point where the ultimate strain capacity for concrete or reinforcing steel has been reached in the governing plastic hinge. Also shown in the figure is the line corresponding to 80% of the peak load. It can be seen that the displacement capacity is governed by the ultimate material strain capacity for the Charleston approach but by the 80% peak load requirement for the Mt. Pleasant approach. 1.5 times the displacement demand is also shown in the figures. Similar curves were developed for individual piers in the transverse direction where the P-delta effects are less significant.

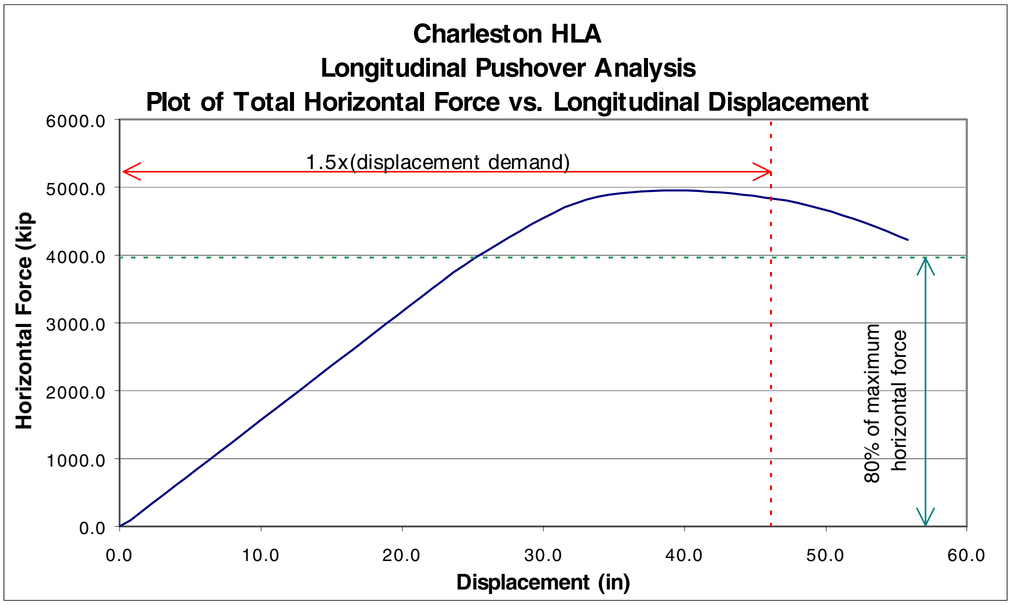

Figure 8 – Force vs. Displacement Curve at Deck Level for Charleston High Level Approach

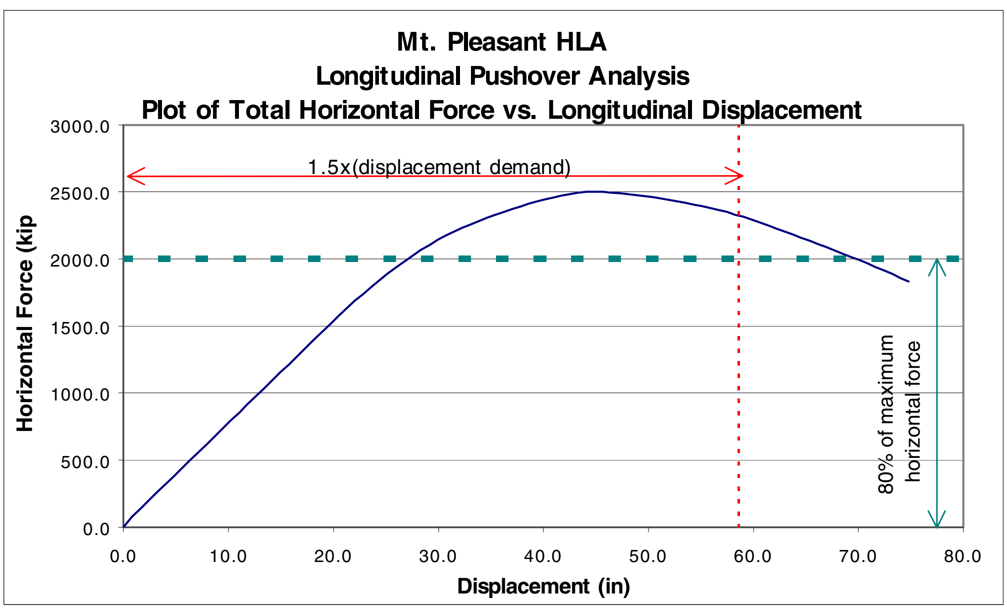

Figure 9 – Force vs. Displacement Curve at Deck Level for Mt. Pleasant High Level Approach

Phase 2 – Final Adjustment and Performance Verification for Optimized Systems

Inelastic time history analyses using the ADINA program were carried out for final adjustment and performance verification of the optimized structural systems.

The model for inelastic time history analysis was similar to that for inelastic static pushover analysis where both geometric and material nonlinearities were incorporated. Rayleigh damping with 5% of critical for the 1st and 200th modes was considered. Multiple support excitation was considered by applying spatially varying displacement time history inputs (obtained from the site response analyses) at pier foundations along the length of each approach model.

Inelastic time history analyses were performed for the three sets of input ground motions (Imperial, Joshua and Tabas). The curvature time history responses were tracked in all column plastic hinge regions for the three sets of input ground motions, and the maximum curvature demands were recorded. The curvature demands were compared with the allowable curvatures that were computed from the allowable material strains (shown in Eqs. 1 and 2) for the applied axial forces. Final adjustment was made to ensure that curvature demand did not exceed the allowable curvature in all plastic hinge regions. Cumulative plastic curvature demands in the plastic hinge regions were also examined to investigate the effects of duration of strong shaking. In addition, it was verified that 1.5 times the peak displacement demands from the inelastic time history analyses were less than the displacement capacities obtained from the inelastic static pushover analyses for the final configurations. Figure 10 shows the longitudinal displacement response time histories of the Charleston approach deck subjected to the three sets of input ground motions. Figure 11 shows the transverse displacement response time histories of the Charleston approach deck at Pier 13W resulting from the three sets of input ground motions. Table 1 compares the curvature demand with capacity in the column top plastic hinges for transverse response of the concrete piers of the Charleston approach.

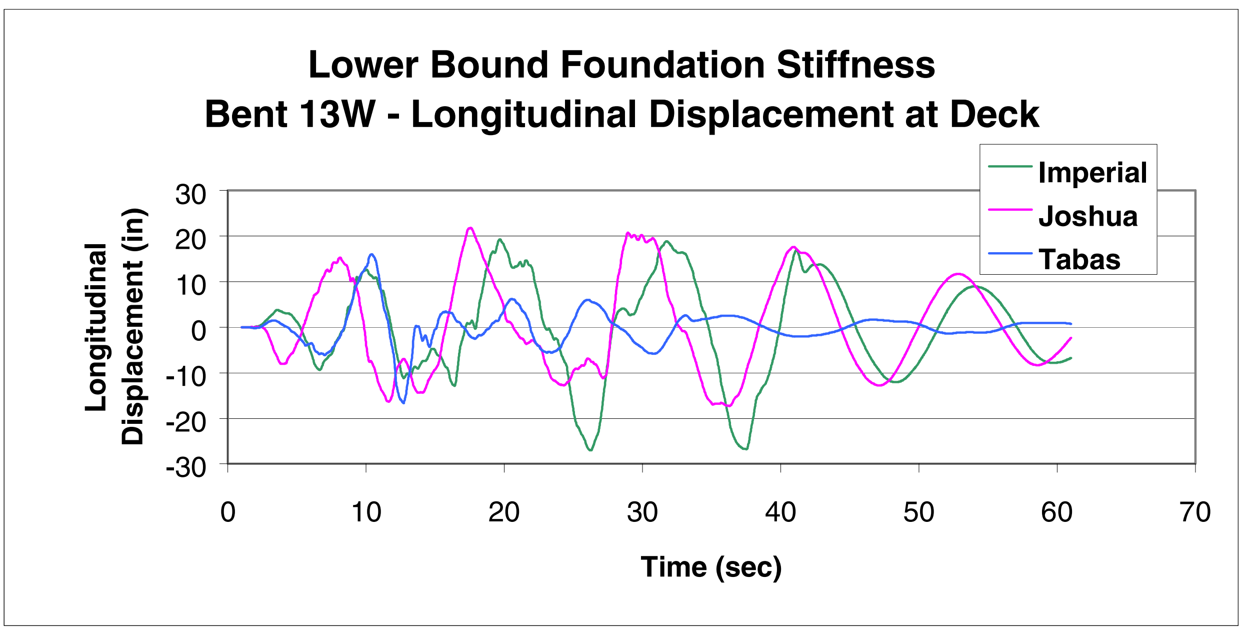

Figure 10 – Longitudinal Displacement Response Time Histories at Deck Level for Charleston High Level Approach

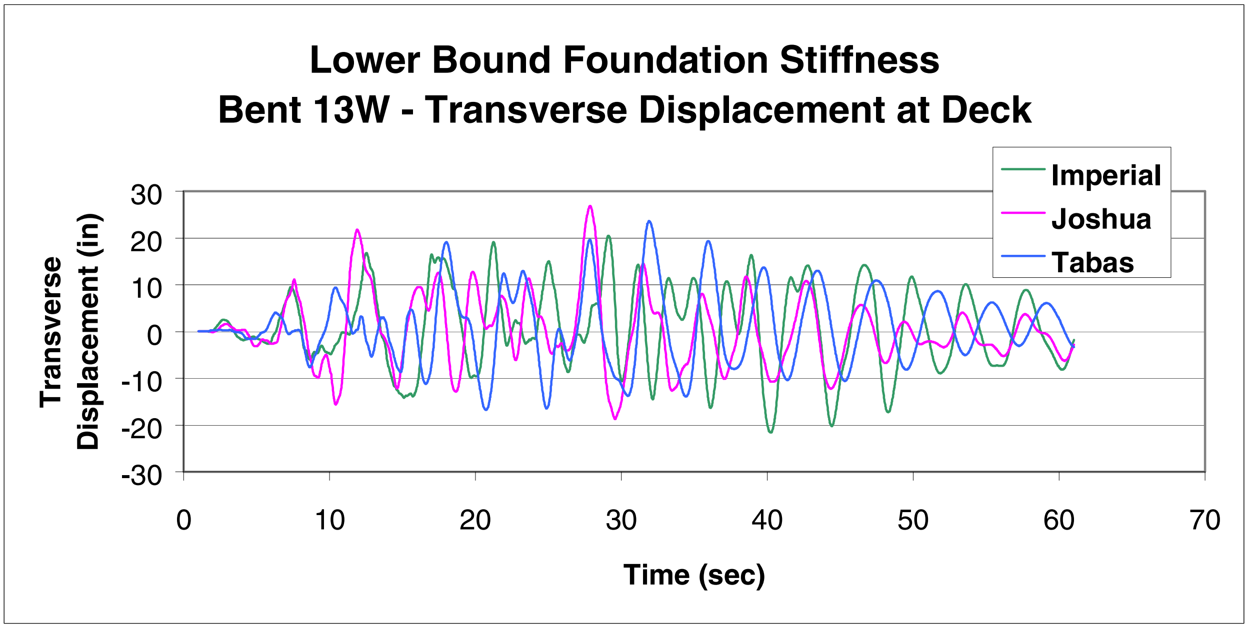

Figure 11 – Transverse Displacement Response Time Histories at Deck Level for Pier 13W of Charleston High Level Approach

Table 1 – Comparison of Curvature Demand with Capacity in Column Top Plastic Hinges for Transverse Response of Charleston High Level Approach

Upper Bound Foundation Stiffness Inelastic Time History Analysis - Transverse Seismic Response Curvature Demands versus Capacities in Top Plastic Hinge Regions of Bent Columns									
	Imperial			Joshua			Tabas		
Bent	Curvature Demand ϕ_{max} 10^{-3}(rad/ft)	Allowable Curvature ϕ_{all} 10^{-3}(rad/ft)	Curvature D/C ϕ_{max}/ϕ_{all}	Curvature Demand ϕ_{max} 10^{-3}(rad/ft)	Allowable Curvature ϕ_{all} 10^{-3}(rad/ft)	Curvature D/C ϕ_{max}/ϕ_{all}	Curvature Demand ϕ_{max} 10^{-3}(rad/ft)	Allowable Curvature ϕ_{all} 10^{-3}(rad/ft)	Curvature D/C ϕ_{max}/ϕ_{all}
W20	3.351	6.110	0.55	3.332	6.110	0.55	2.886	6.500	0.44
W19	2.542	4.598	0.55	2.708	4.598	0.59	2.373	4.890	0.49
W18	4.091	4.195	0.98	2.735	4.195	0.65	2.428	4.366	0.56
W17	2.660	4.207	0.63	1.249	4.207	0.30	1.479	4.116	0.36
W16	3.065	4.116	0.74	1.125	4.573	0.25	1.690	4.207	0.40
W15	2.773	4.299	0.65	1.072	4.024	0.27	1.574	4.665	0.34
W14	1.199	4.390	0.27	0.714	4.390	0.16	0.653	4.756	0.14
W13	1.537	4.116	0.37	2.465	4.116	0.60	1.495	4.299	0.35
W12	1.652	4.207	0.39	3.630	4.207	0.86	1.919	4.299	0.45
W11	3.191	4.390	0.73	3.583	4.390	0.82	3.090	4.207	0.73
W10	3.577	4.665	0.77	3.956	4.573	0.87	2.740	4.848	0.57
W9	3.849	4.390	0.88	3.427	4.939	0.69	3.035	4.848	0.63
W8	3.900	4.482	0.87	4.000	4.299	0.93	2.603	4.848	0.54
W7	3.402	4.390	0.77	2.027	4.482	0.45	1.750	4.665	0.38
W6	2.305	4.848	0.48	1.143	4.848	0.24	1.310	4.573	0.29
W5	1.997	4.116	0.49	1.303	4.665	0.28	2.031	4.756	0.43
W4	1.613	4.207	0.38	1.398	4.085	0.34	1.249	4.207	0.30

EXPECTED PERFORMANCE

Results from the seismic analyses indicate the following expected performance for the high level approaches under the two design level earthquakes:

(a) Function Evaluation Earthquake

The approach structures will remain elastic under the Function Evaluation Earthquake.

(b) Safety Evaluation Earthquake

For longitudinal response, most of the concrete piers will remain in the elastic range, with few shorter piers subjected to very limited plastic deformation at column base.

For transverse response, plastic hinges form at the top of most pier columns and at the two ends of collision struts for the marine piers. Limited plastic deformation occurs in the plastic hinges at column base for some piers.

The extent of damage in these column plastic hinge regions is limited because of the relatively low peak material strain demands (lower than the allowable strains specified by the project design criteria). Such damage can be readily repaired, and the approach structures can be restored for service shortly after the Safety Evaluation Earthquake.

FINAL DESIGN

The top and bottom potential plastic hinge regions of all pier columns were detailed for ductile behavior even though the analyses had indicated that the columns of taller piers would remain elastic at the base under the SEE. The pier columns were detailed in accordance with Division 1A of the AASHTO Standard Specifications [4], supplemented by the provisions of ATC-32 [5]. For example, the AASHTO Specifications restrict the splices in column main rebars to the middle half of the column height. While such restrictions may be appropriate for typical overpass bridges with modest column heights, they are not practical for the high level approaches in this project where columns can reach 46 m (150 ft) in height. As a result, the provisions of ATC-32 were used to detail the potential plastic hinges of the columns, and lap splices were allowed anywhere outside the plastic hinge zones.

Seismic design forces for the capacity protected elements (such as drilled shafts, pier cap beams, bearings, and girder diaphragms at piers) were obtained from inelastic static pushover analyses. In the inelastic static pushover analyses, the following expected material strengths were used to develop moment curvature curves for the column plastic hinges:

$$f_{ye} = 1.1\ f_y \qquad [5]$$
$$f_{ce}' = 1.3\ f_c' \qquad [6]$$

where f_{ye} = expected yield strength of reinforcing steel; f_y = specified minimum yield strength of reinforcing steel; f_{ce}' = expected compressive strength of concrete; and f_c' = specified 28 day compressive strength of concrete. An additional flexural overstrength factor of 1.3 was applied to the plastic hinges. Each approach structure was pushed to 1.5 times the maximum displacement demands obtained from the elastic multi-mode response spectral analysis (Displacement demands from elastic response spectral analysis were typically somewhat higher than those from inelastic time history analyses). Despite conservative design forces from static pushover analyses, adequate transverse reinforcement was provided for the drilled shafts to ensure their ductile behavior in case of inelastic response. Draped post-tensioning tendons were provided in the pier cap beams to ensure that the cap beams remain elastic after formation of plastic hinges in the columns. Large deck expansion joints were provided at the two ends of each approach to accommodate longitudinal movements under service loads and those from the FEE. However, damage to the joints without compromising access requirement for emergency traffic was accepted for the SEE.

CONSTRUCTION

The New Cooper River Bridge is currently under construction. Figure 12 shows some concrete piers of the high level approaches. Figure 13 shows erection of steel plate girders.

Figure 12 - Concrete Piers

Figure 13 – Erection of Steel Girders

CONCLUSIONS

This paper describes the challenges to develop an economical design, in the competitive design/build environment, for the high level approaches of the New Cooper River Bridge that meets the stringent seismic performance criteria. The design strategy adopted was to minimize superstructure weight, introduce flexibility in the foundations, and make each approach continuous over a significant length. While substructure flexibility reduced seismic force demands, it led to significant seismic displacement demands and P-delta effects. As demonstrated through this project, it is important to model both geometric and material nonlinearities to capture response behavior of such flexible systems. Combined elastic response spectral analysis and inelastic static pushover analysis provide a quick and effective means of assessing displacement demand vs. capacity for various alternatives of stiffness and strength distributions, leading to an optimum system. Inelastic time history analysis can be used for final adjustment and performance verification of the optimized system.

ACKNOWLEDGEMENT

South Caroline Department of Transportation is the owner of this project. Palmetto Bridge Contractors is the design/build contractor. Parsons Brinckerhoff Quade & Douglas, Inc. (PBQD) is the design engineering primary consultant. Buckland & Taylor Ltd. (B&T) and Ben C. Gerwick, Inc. (BCG) are the design engineering subconsultants. T.Y. Lin International and HDR Engineering are the design review engineers. PBQD developed the input ground motions and equivalent stiffness matrices for the drilled shaft foundations. B&T carried out analyses and detailed design of the structures except for the drilled shaft foundations that were designed by BCG.

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