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SEISMIC BEHAVIOR OF REINFORCED CONCRETE FILLED STEEL TUBE PILE/COLUMN BRIDGE BENTS

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ABSTRACT

The experimental results from past and on-going experimental research are used to calibrate a fiber-based finite element model capable of simulating the transverse seismic response of multi reinforced concrete filled steel tube (RCFST) pile/column bridge bents. It was found that the limit states are largely controlled by the top hinges, specifically by the tensile strain in the steel longitudinal bars. In any fixed-head pile column the in-ground hinge will rarely govern the design since the column top hinge typically form first and has a shorter plastic hinge length. This situation was found to be more severe in the case of RCFST pile/columns, where the in-ground hinge has a moment capacity ~3-4 times the moment capacity of the top hinge.

Introduction

Reinforced concrete pile or drilled shaft bents are a type of bridge substructure in which the piles or columns are extended from below grade continuously to the superstructure. Depending on the bent configuration (single or multi-column) and the direction of lateral excitation (transverse or longitudinal), plastic hinges will develop in the top of the column (adjacent to the cap beam) and/or below ground. For a transverse loaded multi pile bent (see typical moment distribution in Figure 1), the first yield limit state is characterized by a maximum bending moment at the pile/pile cap connection reaching the flexural strength of the pile and developing plastic hinges at these locations. Further displacement beyond the first yield limit causes a redistribution of internal forces in the pile and a second hinge (below ground) is developed (Song et al. 2005). The location of the maximum moment and formation of the in-ground plastic hinge depends on the relative lateral stiffness of the pile to the soil and are expected at depths ranging from 1.5 to 4.5 times the diameter of the pile/column. The design of this type of structure is usually accomplished using an equivalent depth to fixity and equivalent plastic hinge lengths for the above and in-ground hinges.

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In the case of reinforced concrete filled steel tube (RCFST) pile/columns a steel tube is used as formwork during casting of the concrete. In the majority of the cases a gap is left between the steel tube and the beam–column joint, so that in this potential plastic hinge location the steel tube is only providing shear and confinement strength to the column, and not (in a direct way) flexural or axial strength (which are provided by the concrete and the longitudinal bars). Some of the advantages of RCFST columns are that (1) no formwork is required, (2) the whole concrete section is very well confined which, in theory, will increase the ductility capacity of the section, (3) since the steel tube provides shear and confinement strength a minimum number of conventional ties is required and (4) can be designed to provide adequate blast resistance (Fujikura et al. 2008). Despite the advantages of RCFST piles and their relatively wide use in highly active seismic regions, the seismic response of this type of bridge bents has been scarcely explored.

In this paper the experimental results from past and on-going experimental research are used to calibrate a fiber-based finite element model capable of simulating the seismic response of reinforced concrete filled steel tube (RCST) pile/column bridge bents. The experimental results are also used to evaluate the theoretical member responses based on moment-curvature analyses.

Top Hinge Behavior and Modeling

The modeling of the top hinge behavior is based on the experimental results obtained by Montejo et al. (2009). The RCFST columns tested were designed to emulate typical bent columns of Alaska DOT bridges. As shown in Fig. 1, the tests simulate the part of the column from the cap beam to the inflection point. The columns were designed at half the scale of the actual bridge column/pile. The thickness of the pipe was selected so that the Diameter-Thickness ratio represents that of actual practice in the pile/column design (*D/t*~48). The gap between the steel tube and the cap beam was reduced to 25 mm (1 in) as the typical gap in bridges is 50 mm (2 in). The column diameter of both specimens was 457 mm (18in), cantilever length was 1651mm (65in), and transverse reinforcement was in the form of spirals spaced at 60mm (2.4in) and a steel pipe API-5L X52 of thickness 9.5mm (3/8in). The only variable in columns was the amount of longitudinal reinforcement; one column was reinforced with 8#9 bars (RCFST89) while the other was reinforced with 8#7 bars (RCFST87). Geometric properties and test set up of the units are displayed in Fig. 2. The response of the specimens is summarized in Figs. 3 and 4. Failure of the columns was controlled by buckling of the longitudinal reinforcement and damage in the support block concrete near the column. Fig. 5 shows the condition of the specimens after the tests. Calculations of the equivalent plastic hinge length (L_p) based on the curvatures measured along the columns leaded to an expression (Eq. 1) that predicts values of L_p ~45% smaller than those predicted by the expression proposed by Chai et al. (Eq. 2, 1991) for steel jacket retrofitted columns.

$$L_p = 9.3 d_{bl} \frac{f_{su}}{f_y} + g \tag{1}$$

$$L_p = 0.044 f_y d_{\cdot bl} + g \tag{2}$$

In the above equations, d_{bl} is the diameter of longitudinal bars, g is the gap between the steel jacket and the adjacent member to the column, f_y and f_{su} are the yield and tensile strength in MPa of the longitudinal steel, respectively. Notice that typical ratio of diameter/thickness of the steel jacketed RC columns is $D/t \sim 122$ while RCFST columns have a typical $D/t \sim 48$.

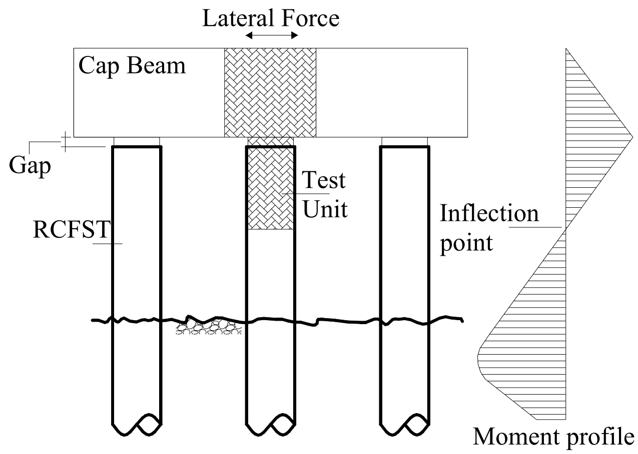

Figure 1. Prototype structure and its representative test model.

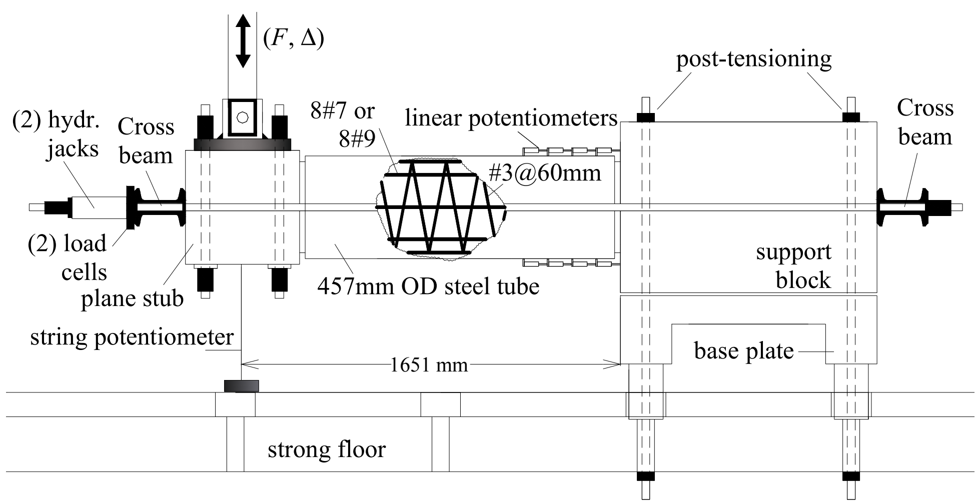

Figure 2. Test set up for evaluation of top hinge.

The experimental results were used to calibrate a finite element model. The element integration method developed by Scott and Fenves (2006) which confines nonlinear constitutive behavior to plastic hinge regions of a specified length while maintaining numerical accuracy and objectivity was used. The section response between hinges is assumed linear elastic; the hinge length used is the obtained from Eq. 1. The formulation utilizes the force-based fiber beam column element formulation and is available in the OpenSees software framework system (McKenna et al. 2000) as the BeamWithHinges element. When a fiber approach is used, separate material rules need to be specified for the reinforcing steel bars, unconfined concrete and confined concrete; material stress is assumed constant between integration points along the fiber segment. No prior moment-curvature analysis is required because the hysteretic response of the section is defined by the material properties, and hence does not need to be specified. Material models used in this research and further discussion on the calibration of the model are available elsewhere (Montejo et al. 2008). The results obtained from the simulations are presented in Figs. 3 and 4. For each unit the force-displacement hysteretic response, the average force-displacement first cycle envelope, the area-based hysteretic damping and the base curvature response were calculated and compared with the experimental results. It is noticed that both the global response (force-displacement) and the local response (base curvatures) obtained from the finite element model, are in close agreement with the experimental results. Regarding the dissipative properties, which were analyzed via area-based hysteretic damping, it is seen that they are generally over predicted by the finite element model.

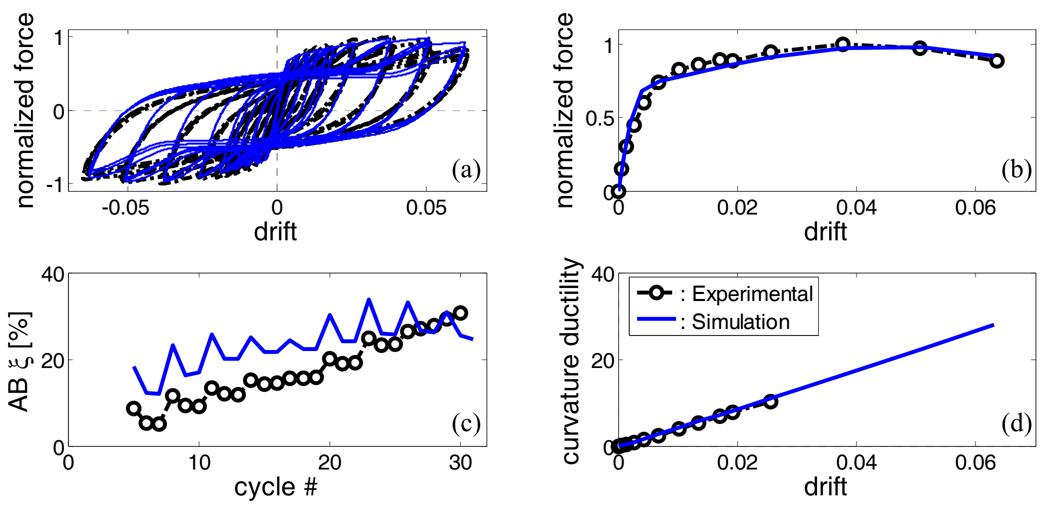

Figure 3. Experimental vs. simulated: (a) hysteretic response, (b) average first cycle envelope, (c) area based hysteretic damping and (d) base curvature ductility for the RCFST87 column.

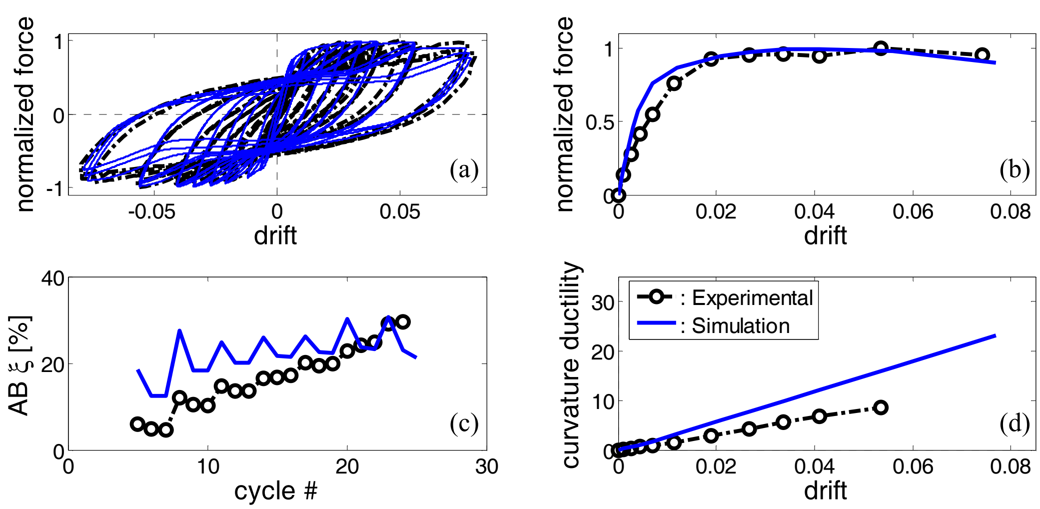

Figure 4. Experimental vs. simulated: (a) hysteretic response, (b) average first cycle envelope, (c) area based hysteretic damping and (d) base curvature ductility for the RCFST9 column.

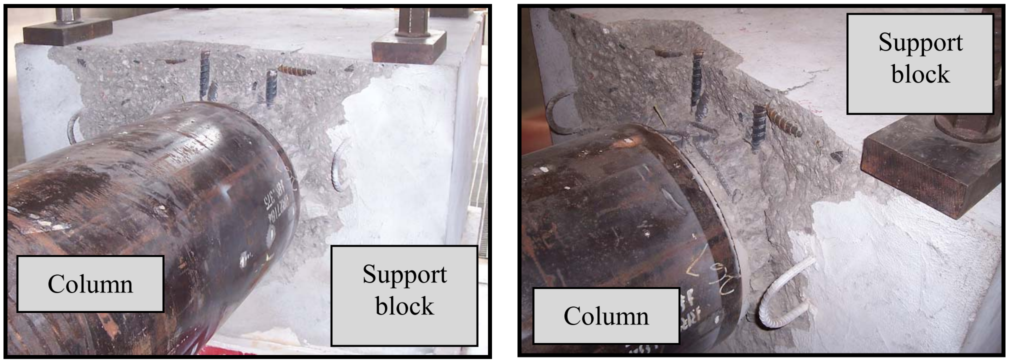

Figure 5. RCFST89 after the test.

The experimental results were also used to evaluate the theoretical monotonic envelopes (Figs. 6a and 6b).The envelopes were calculated using the computer code CUMBIA (Montejo and Kowalsky, 2007). Identified in the theoretical predictions are the stages when the extreme tension reinforcement in the base of the column reached the yield strain, and strains of 0.015 and 0.06, which define the first yield, serviceability and damage control limits, respectively (Kowalsky, 2000). The code calculates the moment-curvature response of the section and then extrapolates the force-displacement response using the equivalent plastic hinge method as described in Priestley et al. (2007). Based on the stain levels on the steel tube and spiral reinforcement during the test, the confined concrete was modeled considering all the confinement provided by the spirals and only half the confinement provided by the steel tube. The value for L_p was calculated using Eq. 1.

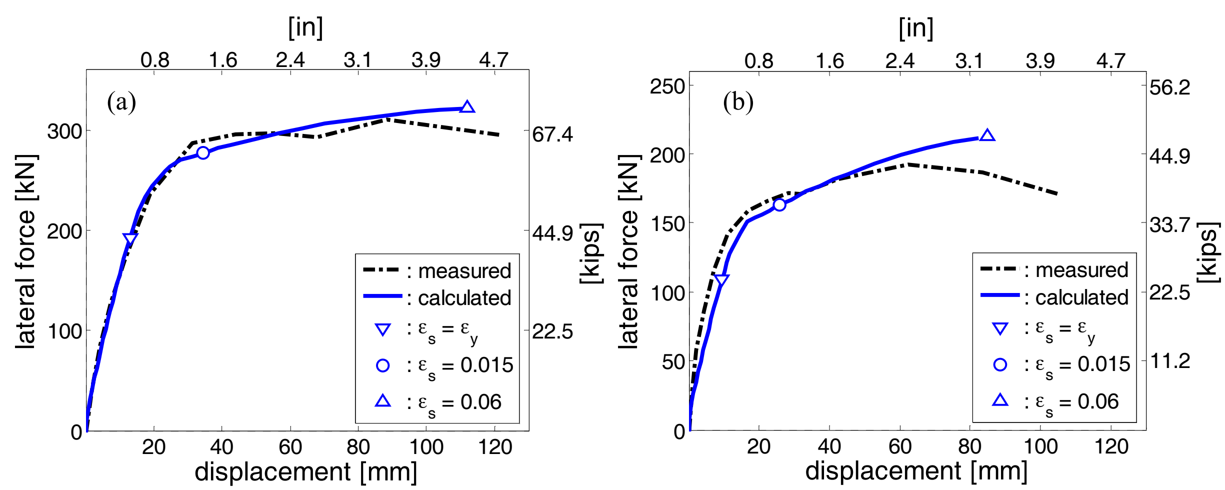

Figure 6. First cycle average envelope and theoretical monotonic envelope: (a) RCFST with 8#9 and (b) RCFST column with 8#7

In-Ground Hinge Behavior and Modeling

The modeling of the in-ground hinge behavior is based on the experimental results reported by González et al. (2008). A series of full scale tests were performed using the four-point bending test setup in Fig. 7. Through this setup, by varying the distance *d*, it was possible to model variable plastic hinge moment to shear ratios and as a result, model the effects of both soft and firm soil on the spread of inelastic action in the below ground hinge. The results obtained for 2 of the 16 specimens tested are presented here. Both units were constructed with a 24in diameter API-5L x52 steel pipe with thickness 0.5in. Actual material test of the pipe revealed a yield stress of 441MPa (64 ksi), each unit was also reinforced with 12#7 bars. The distance *L* between supports was 9144mm (30ft) for both units, the only difference between units was the distance *d* (separation between loads) which was 1828.8mm (6ft) for one specimen and 3657.6mm (12ft) for the other. The response of the specimens is summarized in Figs. 8 and 9. Failure of the units was controlled by buckling and rupture of the steel pipe (Fig. 10).

The tests were modeled in OpenSees using distributed plasticity fiber based elements; due to the test geometry the use of a lumped plasticity fiber based element (BeamWithHinges element) is not viable. The results obtained from the simulation are presented in Figs. 8 and 9 in the same fashion that the previous simulations were presented. The hysteretic response of the specimen is very well captured including the area-based hysteretic damping. The curvature ductility is presented only for the computer simulations as sections curvatures were not measure during the tests, these plots are included to show that there was not strain localization problems in the simulations.

To calculate the theoretical monotonic envelopes in Fig. 11, first the moment-curvature response of the section is calculated and then the force-displacement response is extrapolated using the moment area method. Identified in the theoretical predictions are the stages when the extreme tension fiber of the pipe at the specimen middle point reached the yield strain, and strains of 0.008 and 0.028, which can be used to define the first yield, serviceability and damage control limits, respectively.

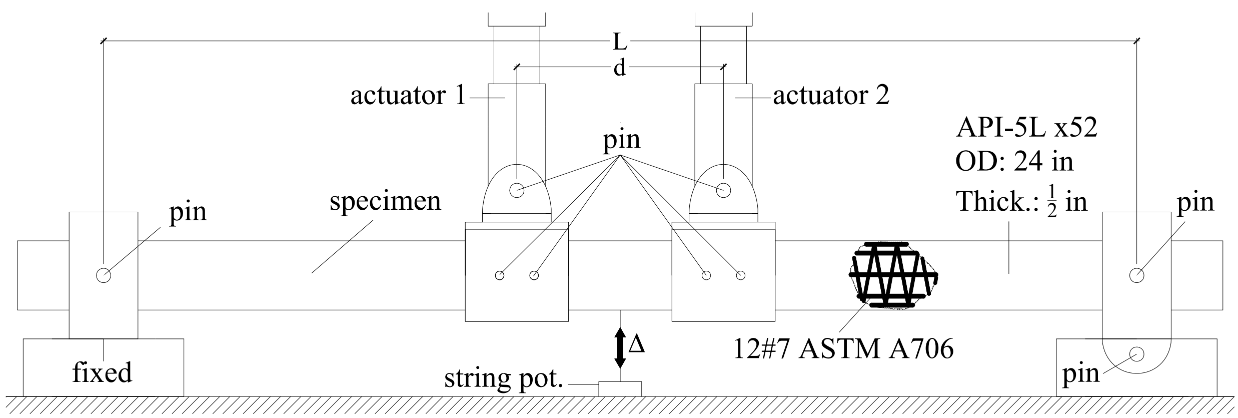

Figure 7. Test set up for evaluation of in-ground hinge.

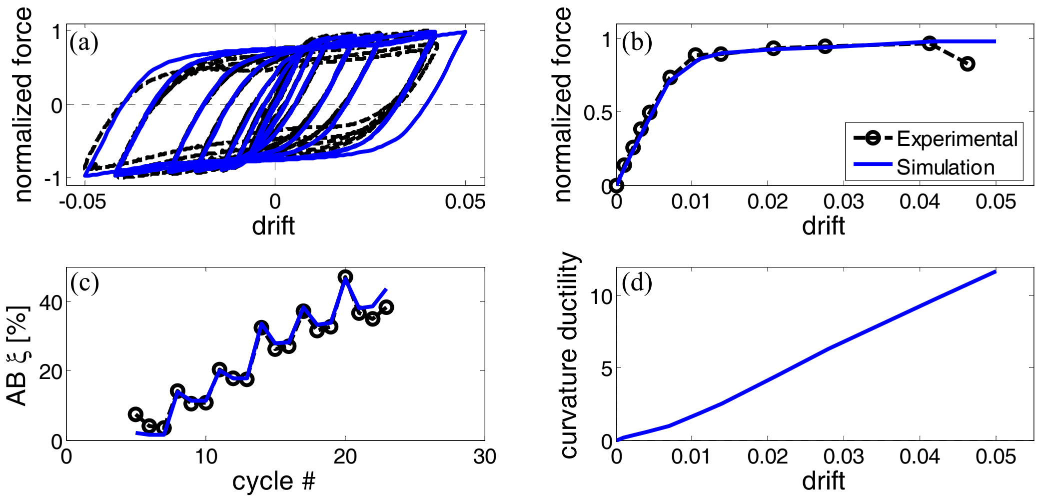

Figure 8. Experimental vs. simulated: (a) hysteretic response, (b) average first cycle envelope, (c) area based hysteretic damping and (d) curvature ductility at mid point for the API-6ft column.

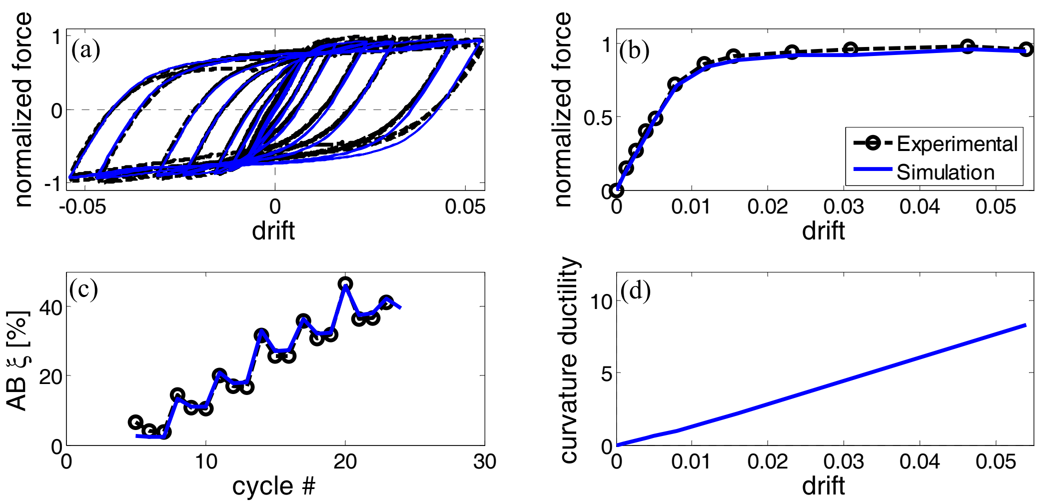

Figure 9. Experimental vs. simulated: (a) hysteretic response, (b) average first cycle envelope, (c) area based hysteretic damping and (d) curvature ductility at mid point for the API-12ft column.

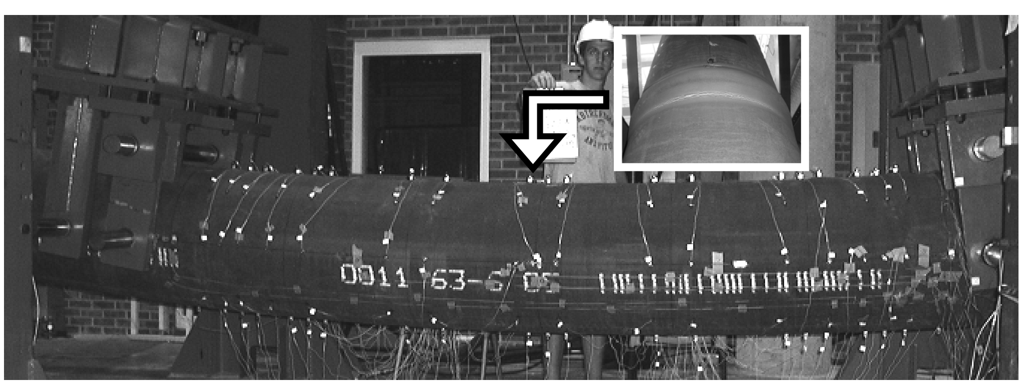

Figure 10. Buckling of the steel tube at ~ductility 6

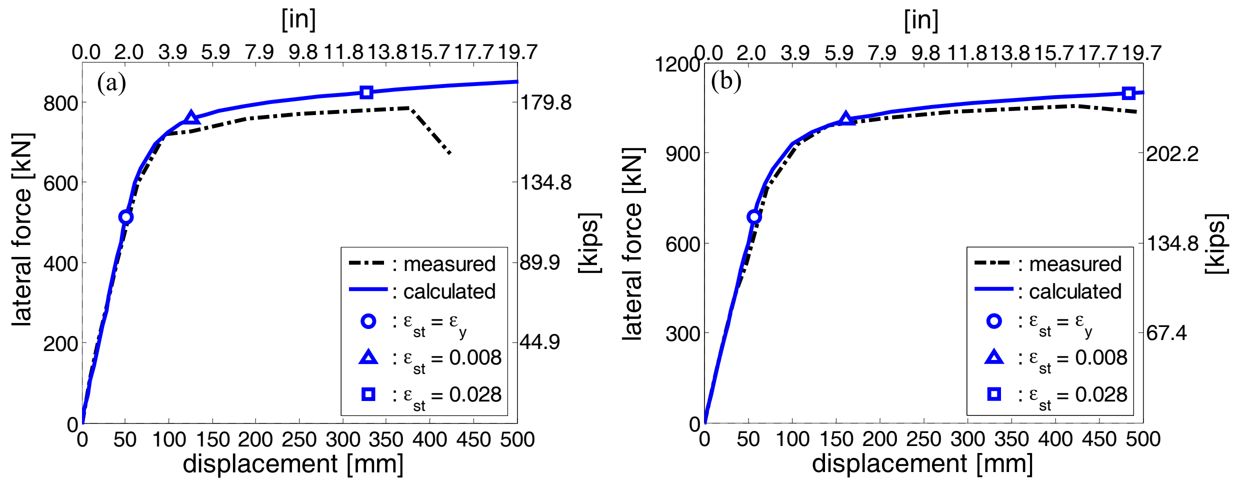

Figure 11. First cycle average envelope and theoretical monotonic envelope: (a) API-6ft and (b) API-12ft

Seismic Behavior of RCFST Pile/Column Bents

To study the seismic behavior of RCFST pile/column bents, non-linear pushover analyses were performed to a typical Alaska DOT bridge bent (Fig. 12). Four pile/columns with diameter 1.067 m (42 in) and reinforced with 30#10 bars (ρ=2.1%) compose the bridge bent, steel tube thickness is 22 mm (7/8 in, D/t=48). A total dead load of 5300kN (1191kips) is distributed on the cap beam, the axial load in each column is then ~1325kN (298kips, ALR=4.1%). The above ground length of the columns is 3m (9.8ft). Fig. 13 shows the moment curvature behavior for the top and bottom hinges, the different limit states are identify based on the material strain limits presented earlier (Figs. 6 and 11). It is seen that the bottom hinge has a moment capacity about 4 times larger than the capacity of the top hinge and a reduced curvature ductility.

The portions of the piles/columns that are located below ground were modeled using fiber-based distributed plasticity elements. The above ground portions are modeled using the BeamWithHinges element. The cap beam was initially modeled as a fiber-force-based element, analysis of the results obtained showed that the beam barely reach the moment required for first yield. It was then decided to model the cap beam as an elastic member with appropriate effective stiffness to reduce the computational effort. The non-linear response of the soil was modeled using lateral springs with appropriate p-y curves. Two different types of soils were used in the analyses: (1) Sand, with a unit weight of. 16.7kN/m3 and a friction angle of 30 degrees, and (2) Clay, with a unit weight of 17kN/m3 and undrained shear strength of 40kPa.

Pushover analysis is used to track levels of strain and formation of plastic hinges to determine displacement limit states, which are initially given in terms of material strain (Figs. 6 and 11). The results from the pushover analyses are presented in Fig. 14. It is noticed that all of the limit states are largely controlled by the top hinges, notice for example that for the bent in soft clay the top hinges reached the damage control limit state before the bottom hinges have even reach first yield. Also, because of the large drifts required to reach some of the limit states, some of the drift limits are likely to be controlled by the superstructure deflection constrains rather than by ductility demand on the columns.

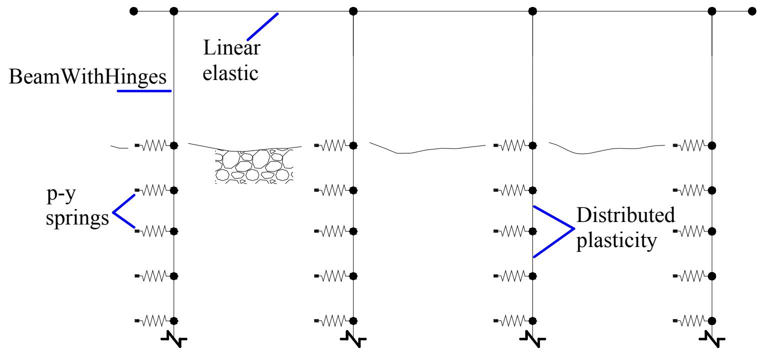

Figure 12. Typical bent model

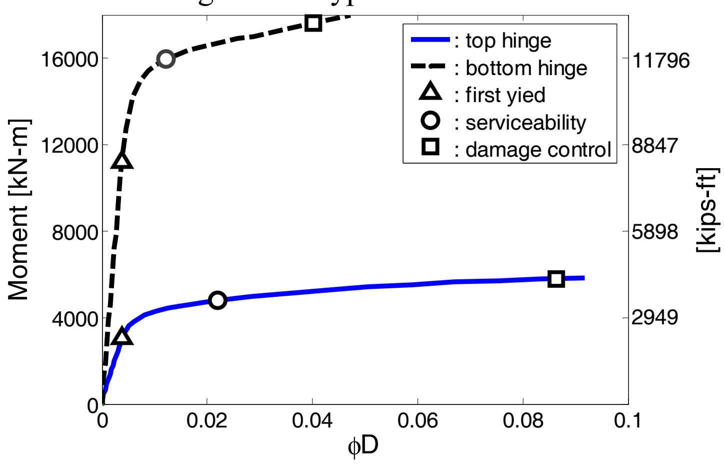

Figure 13. Moment-curvature behavior for top and bottom hinges

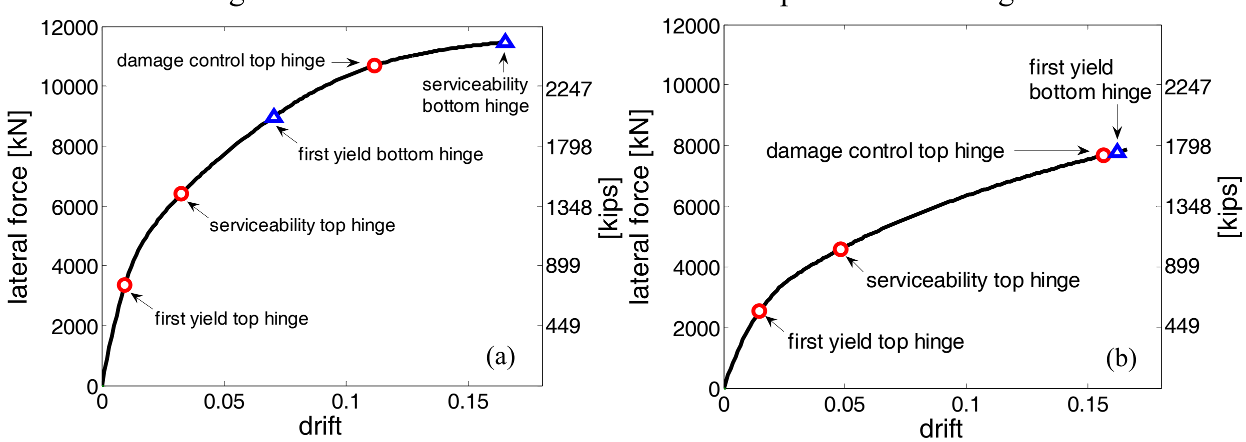

Figure 14. Pushover results for bent in sand (a) and clay (b)

Conclusions

Performance limit states in bridge bents composed of multiple RCFST pile/columns are largely controlled by the top hinges, specifically by the tensile strain in the steel longitudinal bars (as the concrete is very well confined by the steel tube). In any fixed-head pile column the in-ground hinge will rarely govern the design since the column top hinge typically form first, and has a shorter plastic hinge length. This situation is more pronounced in the case of RCFST pile/columns, where the in-ground hinge has a moment capacity ~3-4 times the moment capacity of the top hinge (due to the direct contribution of the steel tube in the flexural strength of the in-ground hinge and not in the top hinge were a gap is left between the tube and the cap beam) and the top hinge has a reduced plastic hinge length. In most cases the in-ground hinge is expected to remain almost elastic while the top hinge has already undergone several inelastic cycles. Elastic response below ground would help preclude the need for inspection of below ground hinges. However, special detailing is required in the transverse reinforcement in the top hinge to avoid early rebar buckling or rupture.

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