Strain Limits for Concrete Filled Steel Tubes in AASHTO Seismic Provisions

Test 4 Summary Report

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I  Executive Summary

Test 4 was conducted at the Constructed Facilities Lab on Friday March 14, 2012. The pile was a concrete filled steel tube; the tube had an outer diameter equal to 24 inches and a thickness equal to 0.375 inches, resulting in a diameter-thickness ratio of 64. Buckling of the specimen began in the first cycle of ductility three. The pile sustained the damage without strength degradation until rebar rupture in the second cycle of ductility five and rupture of the steel pipe followed in the third cycle of ductility five.

II  Introduction

Concrete filled steel tubes are used in bridge design in Alaska and some other states. They serve as both ‘piles’ below ground and as ‘columns’ above ground. This design is preferred because the steel tubes: (a) serve as the formwork for the pile-columns; (b) can be either driven or used in drilled foundations; (c) result in high confinement which results in large deformation capacity; (d) the system has less environmental impact than conventional substructures requiring larger cofferdams.

This research project will address (a) the impact of reinforcing steel on the behavior of the pile, and accuracy of analysis methods (strain compatibility) for prediction of force-displacement response of the pile-column system; (b) the impact of the ratio of tube diameter to tube thickness (D/t ratio) on the performance of the pile-column, including strain limits associated with essentially elastic, repairable and ultimate response; and (c) the plastic hinge length for the below ground hinge developed in the pile-column.

A total of 10 – 12 piles will be experimentally tested in the Constructed Facilities Lab during this project. The first phase will consist of Tests 1 – 5. Test 4 consisted of a concrete filled steel tube with 12#7 ASTM A706 internal reinforcing bars; the had an outer diameter equal to 24 inches and a thickness equal to 0.375 inches resulting in a diameter-thickness ratio of 64. The past tests in Phase 1 had D/t ratios of 48, 128, and 192; the last test in this phase will consist of D/t ratio of 85. The test setup consisted of a six foot constant moment region created by two 440k actuators as seen in Figure 1. A three cycle set load history based on the equivalent yield displacement was applied to the specimen.
The first yield moment of the section was found using the material properties and CUMBIA. The yield stress of the steel tube was 65 ksi and the maximum stress was 77 ksi. The predicted first yield moment was 864 k-ft and the first yield force was 72 kips in each actuator (Equation 1). The self-weight of the pile was taken into account in the cycles up to first yield by setting the moments at the actuator equal to zero. To offset the moment created by the self-weight, 5.8 kips were subtracted from the actuator load in the push direction and added to the actuator load in the pull direction (Equation 2). The experimental first yield displacement was 1.58 inches which resulted in an equivalent yield displacement of 3 inches (Equation 3). The displacement and force histories can be seen in Figures 2 and 3, respectively. At the beginning of the test each actuator pulled up 5.8 kips to account for the dead weight of the pile, all the displacements and forces shown in the remainder of the report are measured from this point after the dead weight was accounted for.

Equation 1:
\[ F_y' = (M_y')/(12ft); \quad M_y' = 864 \text{ k-ft} \]
\[ F_y' = (864 \text{ k-ft})/(12ft) = 72 \text{ k} \]

Equation 2:
\[ F_{add} = (-M_{reaction} + M_{self})/(12ft) \]
\[ M_{reaction} = (R)(d): \text{ Moment due to support reaction} \]
\[ M_{self} = w_{pile}(L_{pile}/2)(d): \text{ Moment due to simply supported self-weight} \]
\[ F_{add} = [(-R)(d_1) + w_{pile}(L_{pile}/2)(d_2)]/(12ft) \]
\[ d_1 = 12ft \]
\[ w_{pile} = 0.54 \text{ k/ft} \]
\[ L_{pile} = 30 \text{ ft} \]
\[ d_2 = 6ft \]
\[ R = w_{pile}(30 \text{ ft})(1/2) + W_{actuator fixture} = (1/2)(0.54 \text{ k/ft})(30 \text{ ft}) + 1.8k = 9.9 \text{ k} \]
\[ F_{add} = [(-9.9k)(12ft) + (0.54k/ft)(30ft/2)(6ft)]/(12ft) = 5.8k \]

Equation 3:
\[ \Delta_y = \Delta_y'(M_n/M_y') = (1.58in)(1637 \text{ k-ft}/864 \text{ k-ft}) = 3 \text{ in} \]
Figure 2. Experimental Displacement History

Figure 3. Experimental Load History
III. Testing Summary

The pile showed signs of buckling at ductility three, a displacement of 9 inches. Buckling initiated in the first pull of ductility three on the underside of the pile (Figure 4), the compressive region, and when the cycle was reversed (the second push) and the top of the pile developed compression stress and small signs of buckling were observed. During the initiation of buckling, buckles only began to form under each loading point. As the pile continued to be pushed and pulled nine inches in each direction during ductility three, smaller ripples began forming in between the two previous formed buckles, both on the underside of the pile during the pull cycles and the top of the pile during the push cycles (Figure 5).

![Figure 4. Buckling initiated under pile during the first pull of ductility three.](image)

![Figure 5. Ripples forming on top of the pile during the third push of ductility three.](image)

When the displacements increased to twelve inches in each direction, in ductility four, the buckles under each loading point increased in size. During the second push and pull set the small ripples in between the loading points were concentrated into two buckles spaced about 20 inches apart- resulting in a total of four buckles on either side of the pile (Figure 6).
The buckles continued to grow in size throughout the loading in ductility four. At the end of ductility four the buckles were all approximately 1 – 1.25 inches in height; the largest buckles occurred at the change in moment gradient under each loading point (Figures 7 & 8).

Figure 6. Buckles spaced at approximately 20 inches, during ductility four (Δ=12 inches)

Figure 7. Constant moment region, ductility four (Δ=12 inches)
After ductility four ended, the buckles which were not located under the loading points showed no significant changes throughout the remainder of the test, the damage became concentrated in the two buckles under each loading point. These buckles continued to increase in size as the pile underwent displacements of 15 inches in either direction in ductility five (Figure 9), reaching a maximum height of approximately two inches. During the second pull of ductility five, one of the longitudinal reinforcing bars ruptured resulting in a 15 kip strength loss of the pile. As the pile reached 15 inches of displacement in the second push and pull cycles, tension cracks appeared on the side of the pile undergoing tensile stress, the underside of the pile during the push cycle and the top of the pile during the pull cycle (Figure 10). These cracks formed along the edges of the buckles. During the third pull cycle, the pile ruptured at one of the loading points along the edge of the buckle where the tension cracks had previously formed (Figure 11). The progression of the pile buckling leading up to rupture can be seen in Figure 12.
Figure 9. Pile profile at ductility five ($\Delta = 15$ inches)

Figure 10. Tension cracks under the pile during the third push of ductility 5 ($\Delta = 15$in).

Figure 11. Pile Rupture, ductility five ($\Delta = 15$ inches)
Figure 12. Progression of pile buckling throughout the test.
IV. Preliminary Results

The force-displacement hysteresis of this test, after the dead weight of the pile had been accounted for is shown in Figure 13. The force displacement envelopes for the first, second and third cycle are shown in Figures 14 - 17. The pile had an average ultimate force of about 150 kips and lost 15 kips of strength after the longitudinal reinforcing bar ruptured on the second pull of ductility five.

Figure 13. Complete Force Displacement Hysteresis.
Figure 14. First Cycle Force Displacement Envelope.

Figure 15. Second Cycle Force Displacement Envelope.
Figure 16. Third Cycle Force Displacement Envelope.

Figure 17. Force Displacement Envelope for First, Second and Third Cycles.
The steel tube was API5Lx42 and had a yield strength of 65 ksi and an ultimate strength of 77 ksi. The stress strain curve of the pile is shown in Figure 18 and the chemical composition of the pipe in comparison to the API5Lx42 regulations is shown in Table 1.

![Steel Pipe Stress Strain Curve](image)

**Figure 18. Steel Pipe Stress Strain Curve**

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<th></th>
<th>Carbon, max, %</th>
<th>Manganese, max, %</th>
<th>Phosphorus, max, %</th>
<th>Sulfur, max, %</th>
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<td>API5LX42</td>
<td>0.28</td>
<td>1.4</td>
<td>0.03</td>
<td>0.03</td>
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<tr>
<td>Test 4 (0.375&quot;)</td>
<td>0.057</td>
<td>0.547</td>
<td>0.013</td>
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Table 1. Steel Pipe Chemical Composition

The longitudinal tensile strains along the length of the constant moment region can be seen in Figures 19-20 during the third push and pull cycles. The peak tensile strains prior to buckling were around 0.018 at ductility three. The compressive strain profiles will require further processing, however, initial analysis imply that the peak compressive strains prior to buckling were around 0.005 at ductility three. In the compressive graphs, which are not shown at this time, the “spikes” along the x-axis are caused by the LED markers moving as a result of buckling. The “spikes” in the tensile graphs show when the buckled regions do not straighten in the reverse cycle (tension strain). For both cases, the strain data is not meaningful after buckling occurs. Any missing data from the LEDs resulted in sudden abruptions along the graph as seen in Figure 20. The strains for cycles after ductility three are not shown in Figure 20 for clarity in the graph.
Figure 19. Tensile Longitudinal Strains on Top of Pile in Third Pull Cycles

Figure 20. Tensile Longitudinal Strains on Bottom of Pile in Third Push Cycles
V. Conclusion

This pile with a D/t of 64 and 12-#7 internal reinforcement (an internal reinforcement ratio of 1.6%) performed in a ductile manner. Local buckling initiated at ductility three, but sustained the damage and did not rupture until ductility five. It is likely that with such a smaller D/t ratio, the pile wall strains would be larger across the thickness than for piles with smaller thicknesses. Hence buckling occurred later than would be the case for piles with smaller thicknesses. These are preliminary findings and a detailed analysis based on the data collected from the Optotrak camera will also be performed. The next pile to be tested has a thickness of 0.28 in and a corresponding D/t ratio of 85, the pipe is longitudinally welded. It has the same internal reinforcement as this pile.