Strain Limits for Concrete Filled Steel Tubes in AASHTO Seismic Provisions

Test 8 Summary Report

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I Executive Summary

Test 8 was conducted at the Constructed Facilities Lab on Thursday, September 20th. The pile was a concrete filled steel tube; the tube had an outer diameter equal to 20 inches and a thickness equal to 0.125 inches, resulting in a diameter-thickness ratio of 160. Unlike the previous piles tested in this research project, the internal reinforcement ratio was 0.78% not 1.7%. Visible buckling of the specimen began in the first pull of ductility one at a displacement of 1.7 inches in both directions. The pile sustained the damage without strength degradation until rupture in the first cycle of ductility six.

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 12 piles will be experimentally tested in the Constructed Facilities Lab during this project. The second phase will consist of Tests 6 – 12. Test 6 consisted of a concrete filled steel tube with 12#6 ASTM A706 internal reinforcing bars; the pipe had an outer diameter equal to 20 inches and a thickness equal to 0.606 inches resulting in a diameter-thickness ratio of 33. The tests in Phase 1 had D/t ratios of 48, 64, 85, 128, and 192 all with internal reinforcement ratios of 1.7%. 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 48 ksi and the maximum stress was 70 ksi. The predicted first yield moment was 171.3 k-ft and the first yield force was 19.0 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, 2.8 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 0.80 inches which resulted in an equivalent yield displacement of 1.67 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 2.8 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:
\[
F_y' = (M_y')/(9\text{ft}); \quad M_y' = 171.3\text{k-ft}
\]
\[
F_y' = (171.3\text{-ft})/(9\text{ft}) = 19.0 \text{k}
\]

Equation 2:
\[
F_{\text{add}} = (-M_{\text{reaction}} + M_{\text{self}})/(9\text{ft})
\]
\[
F_{\text{add}} = [(-R)(d_1) - w_{\text{pile}}(d_1/2)(d_1)]/9\text{ft}
\]

\[
d_1 = 9\text{ft}
\]
\[
w_{\text{pile}} = 0.347 \text{k/ft}
\]
\[
L_{\text{pile}} = 24 \text{ ft}
\]
\[
R = w_{\text{pile}}(24\text{ ft})(1/2) = (1/2)(0.347 \text{k/ft})(24 \text{ ft}) = 4.16 \text{k}
\]
\[
F_{\text{add}} = [(4.16k)(9\text{ft})-(0.414k/\text{ft})[(9\text{ft})(4.5\text{ft})-(3\text{ft})(1.5\text{ft})]]/(9\text{ft}) = 2.77\text{k}
\]

Equation 3:
\[
\Delta y = \Delta y' (M_n/M_y') = (0.80\text{in})(484 \text{k-ft}/171.3 \text{k-ft}) = 1.67 \text{ in}
\]
Figure 2. Experimental Displacement History

Figure 3. Experimental Load History
III. Testing Summary

Visible buckling of the test specimen began in the second push of ductility one, at a mid-span displacement of 1.67 inches. At this point, one small ripple had formed under each loading point, an example of one of these “ripples” is shown in Figure 1. The buckling of the specimen was also detected by the change in temperature of the pipe at the location of buckling, due to energy dissipation. Buckling also appeared on the underside of the pile during the second pull cycle in ductility one. The buckles became more visible as the ductility level continued; a buckle from the third pull is shown in Figure 2.

Figure 4. Initiation of buckling, Ductility 1- 2nd push, Δ = 1.67 in.

Figure 5. Initiation of buckling, Ductility 1-3rd pull, Δ = 1.67 in.
The next ductility cycle increased the mid-span displacement to one and a half the equivalent yield displacement, increasing the deflection to 2.5 inches. Over the course of the three cycle set, five small buckles formed on each side of the pile. A sketch of the buckle locations are shown in Figure 6. As seen in the figure, the buckles were evenly distributed about the centerline of the pile. The pile also exhibited symmetric behavior in the push and pull cycles, forming buckles at the same rate and approximately same locations on both sides of the pile. A photograph of the most significant buckle in the third pull of the cycle can be seen in Figure 7.

![Figure 6. Outline of Specimen Profile, Ductility 1.5, $\Delta = 2.5$ in.](image)

![Figure 7. Outline of Specimen Profile, Ductility 1.5-pull 3, $\Delta = 2.5$ in.](image)
The damage increased as the mid-span displacement increased to 3.4 inches during ductility two. The five existing buckles slightly increased in size during the first push and pull cycles (Figure 8). The second and third cycles created more small ripples on both sides of the pile: eight on the top of the pile and six on the underside of the pile. Although some of the buckles previously existed all appear to be the same size and spread evenly throughout the constant moment region. The location of these buckles within the constant moment region is shown in Figure 9.

Figure 8. Pile buckle, Ductility 2-1\textsuperscript{st} pull, $\Delta$=3.3in.

![Figure 8](image)

![Figure 9](image)

Figure 9. Outline of Specimen Profile, Ductility 2, $\Delta$=3.3in.
During the third ductility cycle, at a mid-span deflection of 5.0 inches, the damage started to concentrate to a few buckles. The first push cycle increased the buckle near the center of the pile and under one of the loading points. The second and third push cycles continued to increase the size of these buckles as well as the buckle under the other loading point to approximately ½” in height. The profile of the constant moment region at the conclusion of the third push cycle is shown in Figure 10 and an example of one of the buckles is shown in Figure 11. The underside of the pile demonstrated the same symmetric behavior during the pull cycles. Three buckles increased in size, one under both loading points and one in the center of the pile, these buckles were larger than those on the top of the pile. All three buckles can be seen in Figure 12 and a close-up of one of the buckles is shown in Figure 13. At the conclusion of the ductility level, three significant buckles increased in size to about 1/2” on the top of the pile during the push cycles, during the pull cycles three buckles increased in size to 5/8” on the underside of the pile.

Figure 10. Profile of the constant moment region, Ductility 3-2nd push, Δ=5.0in

Figure 11. Buckle located in center of pile, Ductility 3-3rd push, Δ=5.0in
The fourth ductility cycle pushed and pulled the pile to a deflection of 6.67 inches in both directions (Figure 14). During these cycles the three buckles on the top and underside of the pile increased in height to about 3/4” and became more narrow (Figures 15 and 16). As the buckles increased in height they also began to spread around the circumference of the pile. As this happened, the concrete was heard crushing as the steel buckled away from the concrete and the concrete lost its confinement. After the third push of the cycle, tension cracks were observed on the sides of the buckles.
Figure 14. Pile profile, Ductility 4-1\textsuperscript{st} push, $\Delta=6.67$in.

Figure 15. Buckle near loading point, Ductility 4-3\textsuperscript{rd} push, $\Delta=6.67$in.
During the fifth ductility cycle, the mid-span displacement was increased to 8.33 inches (Figure 17). The damage in the pile was observed by the increase in size of the buckles on both sides of the pile. The three on each side of the pile increased in height to approximately 7/8” (Figure 18) and tension cracks began to appear on the creases of the buckles, indicating the high tensile strains demanded by the deflection were difficult for the steel pipe to sustain. The concrete continued to crush as the buckles spread around the circumference of the pile and tension cracks began to form on the creases of the buckles.
Ductility five proved to be the maximum deflection the pile could resist, the steel pipe ruptured at a deflection of 8.4 inches, while being pushed to displacement demand of ductility six, 9.67 inches. The rupture occurred on the underside of the pile on a buckle near the loading point; it is shown in Figure 19. At this displacement, a crack had also formed under the other loading point; however it had not ruptured as shown in Figure 20. The pile continued to be pushed to the full ductility six deflection, and the crack opened causing the pile ruptured in a second location (Figure 21). This additional deflection also caused the previous rupture crack to open more as seen in Figure 22. As expected, the pipe ruptured near the loading points where the moment gradient changes. A progression of buckling leading to the first rupture is displayed in Figure 23.
Figure 19. Steel pipe rupture, $\Delta=8.40\text{in.}$

Figure 20. Steel pipe tensile crack, $\Delta=8.40\text{in.}$
Figure 21. Second steel pipe rupture, Ductility 6, $\Delta = 9.67$\,in.

Figure 22. First pipe rupture, Ductility 6, $\Delta = 9.67$\,in.
Figure 23. Progression of buckling
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 24. The force displacement envelopes for the first, second and third cycle are shown in Figures 25 - 28. The pile had an average ultimate force of about 40 kips, and as seen in the hysteresis the pile behaved in a ductile manner and had no strength loss until after rupture.

![Figure 24. Complete Force Displacement Hysteresis.](image-url)
Figure 25. First Cycle Force Displacement Envelope.

Figure 26. Second Cycle Force Displacement Envelope.
Figure 27. Third Cycle Force Displacement Envelope.

Figure 28. Force Displacement Envelope for First, Second and Third Cycles.
The steel tube was ASTM A139 Grade B, had a yield strength of 48.3 ksi and an ultimate strength of 69.5 ksi. The stress strain curve of the pile is shown in Figure 29.

![Stress Strain Curves](image)

**Figure 29. Steel Pipe Stress Strain Curve**

The first signs of visible buckling during testing occurred in the second push of ductility one, however after inspection of the data buckling began to occur right before that in the first pull of ductility one. The buckling is demonstrated by the strain profile shown in Figure 30, the change in the linearity in the compressive region of the profile in ductility four demonstrates the pile beginning to buckle. This behavior is also observed in the longitudinal compressive profiles along the length of the constant moment region (Figures 31 and 32), the later ductility cycles are excluded from these graphs for clarity. The sudden changes in longitudinal strains demonstrate the buckling behavior in the pipe. As seen in Figure 32, these sudden changes or buckles begin to occur in the first pull of Ductility 1, which agrees with the information found from the Strain Profile in Figure 30. The tensile longitudinal strains along the length of the constant moment region are shown in Figures 33 and 34. The “spikes” in the tensile graphs show when the buckled regions do not straighten in the reverse cycle (tension strain). For both the tensile strains and compressive strains, the strain data is not meaningful after buckling occurs. As seen in these figures, the peak tensile strains prior to buckling at the beginning of ductility one were 0.0024. The peak tensile strain prior to rupture was 0.0209. The peak compressive strains prior to buckling were -0.00086 during the first push of ductility one.
Figure 30. Strain Profile until buckling

Figure 31. Compressive Longitudinal Strains on Top of Pile in First Push Cycles
Figure 32. Compressive Longitudinal Strains on Bottom of Pile in First Pull Cycles.

Figure 33. Tensile Longitudinal Strains on Top of Pile in First Pull Cycles.
V. Comparison to Previous Tests

This pile had the same thickness as the last specimen tested, resulting in a D/t ratio of 160. This pile had an internal reinforcement ratio of 0.78%, about one-half of the internal reinforcement of the previous pile which had a ratio of 1.67%. Both of the pile buckled and ruptured at the same ductility levels (Table 1). The mid-span displacements at these limit states were higher for the pile with a 1.67% internal reinforcement ratio because the yield displacement for the pile as 0.9 inches in Test 7 and 0.8 inches in Test 8 (Table 2). The pile with less reinforcement had slightly lower limit state strains and curvatures than that of the previous pile with 1.67% (Tables 3 and 4). The limit state strains and curvatures of the pile with 0.78% were 20% - 30% lower than that of the pile with 1.67% internal reinforcement.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement Ratio (%)</th>
<th>Ductility Prior to Buckling</th>
<th>Ductility Prior to Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.67%</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>8</td>
<td>0.78%</td>
<td>1</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 1. Limit State Ductility Comparison
Table 2. Limit State Displacement Comparison

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement Ratio (%)</th>
<th>Displacement prior to Buckling (in)</th>
<th>Displacement Prior to Rupture (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.67%</td>
<td>1.85</td>
<td>9.44</td>
</tr>
<tr>
<td>8</td>
<td>0.78%</td>
<td>1.67</td>
<td>8.33</td>
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</table>

Table 3. Limit State Tensile Strain Comparison

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement Ratio (%)</th>
<th>Strain prior to Buckling</th>
<th>Strain Prior to Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.67%</td>
<td>0.00308</td>
<td>0.02498</td>
</tr>
<tr>
<td>8</td>
<td>0.78%</td>
<td>0.00240</td>
<td>0.02090</td>
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Table 3. Limit State Curvature Comparison

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement Ratio (%)</th>
<th>Curvature prior to Buckling (1/in)</th>
<th>Curvature Prior to Rupture (1/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.67%</td>
<td>0.00025</td>
<td>0.00179</td>
</tr>
<tr>
<td>8</td>
<td>0.78%</td>
<td>0.000162</td>
<td>0.00110</td>
</tr>
</tbody>
</table>

V. Conclusion

This pile with a D/t of 160 and 8-#5 internal reinforcement (an internal reinforcement ratio of 0.78%) performed in a ductile manner. Local buckling initiated at ductility one, but sustained the damage and did not rupture until ductility six. The displacements, strains and curvatures at the buckling and rupture limit states were 20-30% lower than those of the pile with the same D/t ratio and twice the internal reinforcement (1.67%). Although the pile had lower strains and curvatures, the pile did behave in a ductile manner as shown by sustaining the damage while not losing strength in the pile. 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 0.125in, a diameter of 24in and a corresponding D/t ratio of 192, the pipe is spirally welded. It has the same internal reinforcement as this pile.