Strain Limits for Concrete Filled Steel Tubes in AASHTO Seismic Provisions

Test 9 Summary Report

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I Executive Summary

Test 9 was conducted at the Constructed Facilities Lab on Monday, October 8th. The pile was a concrete filled steel tube; the tube had an outer diameter equal to 24 inches and a thickness equal to 0.125 inches, resulting in a diameter-thickness ratio of 192. Unlike the previous pile with this D/t ratio 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 2.8 inches in both directions. The pile sustained the damage without strength degradation until rupture in the third cycle of ductility four.

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 46.6 ksi and the maximum stress was 68.8 ksi. The predicted first yield moment was 294.7 k-ft and the first yield force was 32.7 kips in each actuator. 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, 6.1 kips were subtracted from the actuator load in the push direction and added to the actuator load in the pull direction (Equation 1). The experimental first yield displacement was 1.49 inches which resulted in an equivalent yield displacement of 2.82 inches (Equation 2). The displacement and force histories can be seen in Figures 2 and 3, respectively. At the beginning of the test each actuator pulled up 6.1 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 2:**

\[ F_{\text{add}} = \frac{(-M_{\text{reaction}} + M_{\text{self}})}{(12\text{ft})} \]

\[ F_{\text{add}} = \frac{[-(R)(d_1) - w_{\text{pile}}(d_1/2)(d_1)]}{(12\text{ft})} \]

\[ d_1 = 12\text{ft} \]

\[ w_{\text{pile}} = 0.494 \text{ k/ft} \]

\[ L_{\text{pile}} = 30 \text{ ft} \]

\[ R = w_{\text{pile}}(30 \text{ ft})(1/2) = (1/2)(0.494\text{k/ft})(30 \text{ ft}) = 7.41 \text{k} \]

\[ F_{\text{add}} = \frac{[(7.41\text{k})(12\text{ft})-(0.494\text{k/ft})((12\text{ft})(6\text{ft})-(3\text{ft})(1.5\text{ft}))]}{(9\text{ft})} = 6.18\text{k} \]

**Equation 3:**

\[ \Delta_y = \Delta_y'(M_n/M_y') = (1.49\text{in})(560 \text{k-ft}/294.7 \text{k-ft}) = 2.82 \text{in} \]
Figure 2. Experimental Displacement History

Figure 3. Experimental Load History
III. Testing Summary

Visible buckling of the test specimen began in the first pull of ductility one, at a mid-span displacement of 2.8 inches. At this point, one small ripple had formed under each loading point, an example of one of these “ripples” is shown in Figure 4. Buckling also appeared on the top of the pile during the second push in ductility one, as seen in Figure 5. These buckles remained visibly the same throughout the conclusion of the first ductility cycle.

Figure 4. Initiation of buckling, Ductility 1- 1st pull, $\Delta = 2.80$ in.

Figure 5. Initiation of buckling, Ductility 1-2nd push, $\Delta = 2.80$ in.
The mid-span displacement was increased to 4.2 inches, one and a half times the equivalent yield during the next ductility cycle. The increased displacement resulted in more damage shown by the growth of the existing ripples and formation of new buckles. The first push did not result in significant change in the buckling behavior since the top of the pile had not been subjected to higher tensile strains at that point. During the first push the underside of the pile was subjected to higher tensile strains which resulted in more buckling during the second pull of ductility one and a half. The buckles under both loading points increased in size and one small buckle formed about 9 inches south of the centerline (Figure 6). The increase in damage was also evident in the second push of ductility 1.5. The buckles under the loading points increased in size and two new buckles formed in the constant moment region. The new buckle formed 12 inches north of the centerline is shown in Figure 7. The pile sustained the damage and the buckled behavior was consistent throughout the remainder of the ductility cycle.

Figure 6. Formation of buckle 9 in. south of centerline; ductility 1.5 pull 1, Δ=4.2 in.

Figure 7. Formation of buckle 12 in. north of centerline; ductility 1.5 push 2, Δ=4.2 in.
After the conclusion of ductility one and a half, the mid-span displacement was increased to 5.7 inches in each direction to meet the demands of ductility two. During this cycle, the existing buckles continued to increase in size and no new buckles were formed. At the conclusion of the cycle the buckles located under the loading points were 1/2” to 5/8” in height (Figure 8) and the buckles located in the constant moment region were 1/4” to 3/8” in height. The majority of the buckles formed perpendicular to the pile however one of the buckles on the top of the pile formed parallel to the spiral weld near it. This buckle is shown in Figure 9. Each side of the pile exhibited symmetric behavior with four total buckles on each side centered about the centerline of the pile. The pile profiles showing the location and heights of these buckles is shown in Figure 10.

Figure 8. Buckle located under one of the loading points, ductility 2, $\Delta=5.7$ in.

Figure 9. Buckle located between loading points, ductility 2, $\Delta=5.7$ in.
The damage continued to increase as the displacement increased in ductility cycles three. Ductility three demanded a mid-span displacement of 8.4 inches, the profile is shown in Figure 11. At the conclusion of ductility three, the buckles had grown in size and become narrow. The buckles under the actuators were about 3/4” in height on both the top and underside of the pile (Figure 12). The buckles in the center of the constant moment region were smaller on the underside of the pile (3/8”-1/2”) than those on the top of the pile which were about 5/8” in height as seen in Figures 13 and 14 respectively. Figure 14 also displays the buckle that formed parallel to the weld instead of perpendicular to the pile. The location and heights of the buckles are seen in Figure 15.

Figure 10. Pile profile outline, ductility 2, Δ=5.7 in.
Figure 11. Displaced pile profile, ductility 3, $\Delta = 8.4$ inches.
Figure 12. Buckle on top of pile under loading point, ductility 3, $\Delta = 8.4$ in.

Figure 13. Buckle on underside of the pile in between loading points, ductility 3, $\Delta = 8.4$ in.
Figure 14. Buckle on top of pile in between loading points, ductility 3, $\Delta = 8.4$ in.

Figure 15. Pile profile outline, ductility 3, $\Delta=8.4$ in.
Ductility four demanded a displacement of 11.2 inches in each direction as seen in Figure 16. The buckles did not change in height during the first cycle of this ductility three cycle set however they did become more narrow. After the pile was pushed to 11.2 inches for the second time in ductility four, small tension cracks begin to appear on the underside of the pile under the loading points where the buckles formed when the underside of the pile is in compression (Figure 17). During this second push the buckles under the loading points on the top of the pile grew by approximately 50% to about 1 inch in height. The same damage continued in the second pull of ductility four. Tension cracks formed on the top of the pile under the loading points where buckles form when the top of the pile is in compression and the buckles under the loading points increased in height to about 1 inch (Figure 18).

Figure 16. Displaced pile profile, ductility 4, Δ=11.2 inches.

Figure 17. Tension cracks observed on the underside of the pile, ductility 4, Δ=11.2 in.
Figure 18. Buckle on the underside of the pile, ductility 4, $\Delta=11.2$ in.

The increase in damage proved to be the maximum damage the pile could sustain and the underside of the pile ruptured during the third push of ductility four at a displacement of 8.6 inches. As seen in Figure 19, the rupture occurred on the crease of the buckle under the north loading point. The crack in the pile grew in width and length as the pile was pushed to the full 11.2 inch displacement demand of ductility four (Figure 20). The increase in crack width shows the concrete is still intact under the portion of the pile that had not buckled. The pile lost approximately 15 kips of load after the rupture occurred in the push cycle. The pile lost no strength during the pull cycle since the crack closed as seen in Figure 21, which allowed it to sustain the steel’s compressive strength. The full spectra of damage from initiation of buckling to rupture is shown in Figure 22.
Figure 19. Rupture on the underside of the pile, during the first push of ductility 4.

Figure 20. Rupture on the underside of the pile at full displacement of ductility 4, $\Delta=11.2$ in.

Figure 21. Steel pipe crack closing when in compression, ductility 4, $\Delta=11.2$ in.
Figure 22. Progression of 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 23. The force displacement envelopes for the first, second and third cycle are shown in Figures 24 - 27. The pile had an average ultimate force of about 50 kips, and as seen in the hysteresis the pile behaved in a ductile manner and had no strength loss until rupture which resulted in a loss of approximately 15 kips.
Figure 23. Complete Force Displacement Hysteresis.

Figure 24. First Cycle Force Displacement Envelope.
Figure 25. Second Cycle Force Displacement Envelope.

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

The first signs of visible buckling during testing occurred in the first pull of ductility one, the data agreed with this observation. The buckling is demonstrated by the strain profile shown in Figure 29, the change in the linearity in the compressive region of the profile in ductility one 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 30 and 31). The sudden changes in longitudinal strains demonstrate the buckling behavior in the pipe. As seen in Figure 31, 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 29.

The tensile longitudinal strains along the length of the constant moment region are shown in Figures 32 and 33. 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 0.0036. The peak tensile strain prior to rupture was 0.0251.
Figure 29. Strain Profile until buckling

Figure 30. Compressive Longitudinal Strains on Top of Pile in First Push Cycles
Figure 31. Compressive Longitudinal Strains on Bottom of Pile in First Pull Cycles.

Figure 32. Tensile Longitudinal Strains on Top of Pile in First Push Cycles
V. Conclusion

This pile with a D/t of 192 and 8-#6 bars as 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 four. 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 0.138in, a diameter of 24in and a corresponding D/t ratio of 192, the pipe is spirally welded. It has more internal reinforcement: 14-#8 bars creating an internal reinforcement ratio of 2.5%.