EVALUATION OF A SPLICE CONNECTION FOR PRECAST CONCRETE PILES

Mikayla Bladow, EIT, PES Structural Engineers, Atlanta, GA
Brandon E. Ross, PhD, PE, Clemson University, Clemson, SC
Thomas E. Cousins, PhD, PE, Clemson University, Clemson, SC
Mahmoodreza Soltani, EIT, PhD Candidate, Clemson, SC

ABSTRACT

Two flexural tests were performed for a connection for splicing precast concrete piles. The splice consisted of steel male studs and female sockets attached with steel locking pins. Internal forces in the splice were transmitted to the piles through #10 bars welded to the back side of the splice assemblies. The splice connection exhibited experimental flexural capacities of 273 kip-foot and 287 kip-feet. Peak capacities corresponded to crushing of the concrete compression zone adjacent to the splice location. This report presents details of the test program and theoretical calculations of flexural capacity. Comparison between experimental and theoretical results suggest that the #10 bars were near ultimate capacity at peak load.

Keywords: Connections, Testing, Flexural Capacity, Piles

INTRODUCTION

Due to fabrication, transportation, and handling constraints, it is common to build precast concrete piles from multiple pile segments. Splice connections between the segments must be capable of transmitting axial and flexural loads. Rapidity of construction is another primary concern when designing and specifying splice connections. This paper presents details of testing and analysis of a splice connection. The work was conducted to evaluate the splice connection for compliance with flexural strength requirements of the Florida Department of Transportation (FDOT) *Standard Specifications for Road and Bridge Construction*[1]. FDOT requires that mechanical splice connections for 18 inch x 18 inch piles be cable of resisting 245 kip-ft of moment, which is approximately equal to the design flexural strength of the pile at sections away from the splice.

PILE AND SPLICE DETAILS

Two specimens were tested. Each test specimen consisted of two 18 inch x 18 inch x 20 foot long prestressed concrete pile segments. The segments were fabricated in July 2015 by a precaster in the Southeastern US according to standard pile details from the FDOT[2]. After fabrication the segments were trucked to Clemson University where they were spliced and tested. Material properties for the piles are presented in Table 1.

Table 1 - Material properties of piles

Material	**Specified minimum strength (ksi)**	***Tested Strength (ksi)**
Concrete	6	7.7
Prestressing strand	270	NP
Spiral reinforcement wire	80	NP
**Auxiliary reinforcement	70	NP

*Based on test results and documentation provided by precaster; NP: not provided
**Does not include #10 bars attached to splice assembly

The FDOT Standard Specifications for Road and Bridge Construction[2] require that mechanical splice connections be cable of resisting 245 kip-ft for 18 inch x 18 inch piles. The splice is made by connecting steel assemblies placed at the end of each pile segment during fabrication (Fig. 1). Each assembly is comprised of #10 bars, male studs, female sockets, pines, and a cap plate. The #10 bars are welded to the back of the studs and sockets. Material properties of the splice assembly components are summarized in Table 2.

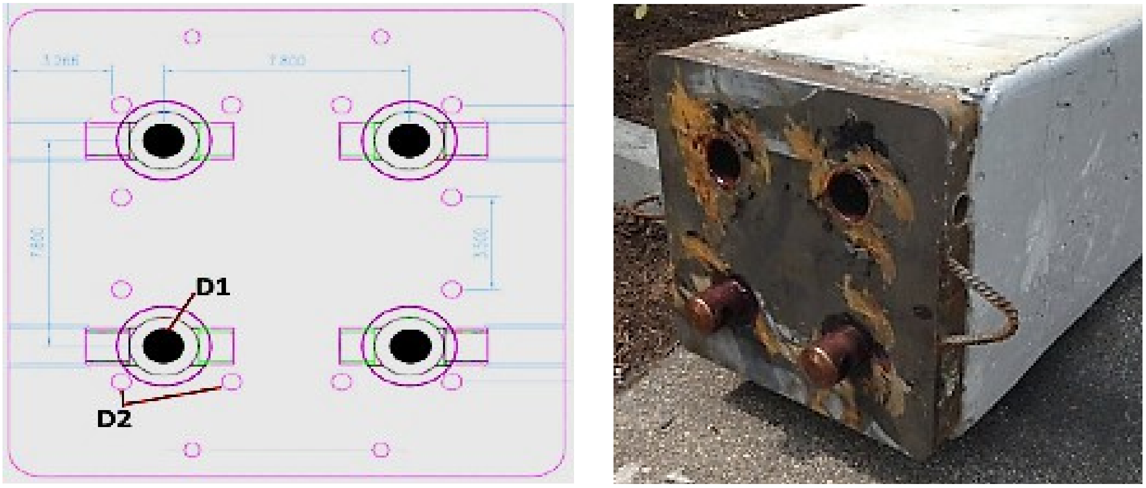

Fig. 1 –Pile Splice Connection

The steel assembly on each side of the splice contains both the male and female components. This can be seen in Fig. 1 wherein the assembly in the picture has male studs at the bottom and female sockets at the top. This arrangement is reversed on the opposite assembly. By detailing the assemblies in this manner, the assemblies are interchangeable.

Working left-to-right in Fig. 2, the load path through the splice connection is as follows: Flexural-tension force from the pile is transferred to #10 bars, which are welded to the back of the socket. Forces are transferred between the socket and stud through a locking pin which is inserted from the side after the two splice assemblies are joined. A #10 bar is welded to the back side of the stud to receive forces from the stud and deliver them to the pile. The #10 bars extend four feet into each pile.

Flexural-tension forces are carried by prestressing strands beyond the termination point of the #10 bars. During fabrication, the prestressing strands extended through holes in the cap plates. Strands were cut flush with the surface of the cap plate by the pile fabricator, but were not connected to the splice assembly. Yielding of the #10 bars was designed to be the controlling limit state for flexural-tension forces.

The piles were oriented horizontally end-to-end on the ground during the splicing process. One of the segments was supported on wood dunnage and the other was supported by a crane (Fig. 3). In order to get the locking pins to fit into the holes, the cap plates needed to be flush against each other. This required some adjustments to the crane-supported pile segment. Once aligned, the pins required 10-12 hits with a 2 lb hammer to be secured in place (Fig. 4). The time from when that the segments were oriented end-to-end (Fig. 3) until the locking pins were secured in place was approximately 10 minutes. After the segments were spliced, they were lifted by crane to the testing bed (Fig. 5).

In practice the pile segments would be oriented vertically during the splicing process. This would likely decrease the effort and time required to complete the splice because gravity would pull the cap plates flush, which would also align the holes for the locking pins.

Table 2 - Splice material properties

Component	**Specification**	***Fy (ksi)**	***Fu (ksi)**
Locking Pin	ASTM A311	100	115
Male Stud	EN S355 J2G3	51.5	91
Female Socket	EN S355 J2G3	51.1	74
#10 bars	ASTM A706	72 (82 tested)**	90 (110 tested)**
Weld Metal	AWS ER80S-X	65	80
Cap Plate	ASTM A572-Gr50	50	65
*All properties are specified properties unless otherwise noted. **Based on test data provided by bar supplier			

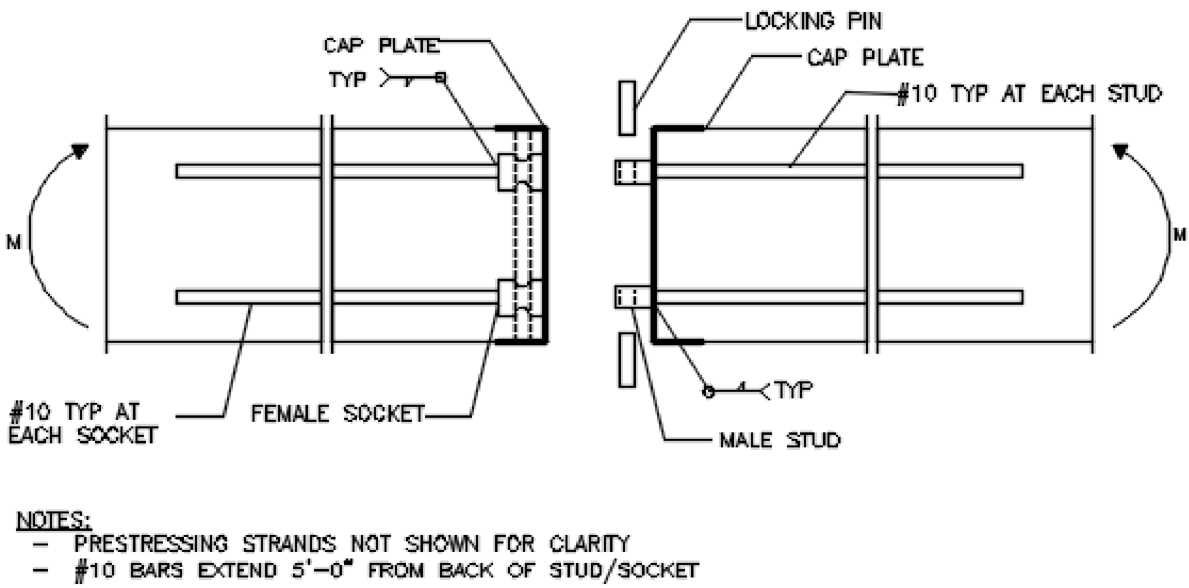

Fig. 2 – Components in Pile Splice Connection

Fig. 3 - Pile supported by crane and wood dunnage

Fig. 4 – Splice connection during alignment (left) and installation of locking pins (right)

Fig. 5 – Segments spliced together and positioned on test bed

Two test specimens were built from four pile segments. The specimens were effectively identical. The only difference between the specimens was orientation of the pins during testing. Pins were located on the sides for Test 1 (Fig. 6 left) and on the top/bottom for Test 2 (Fig. 6 right). For piles supporting a bridge or other structure, flexural moments in the piles can be applied to the splice along either or both axes. For this reason, both orientations were tested in the experiments.

Fig. 6- Pin orientations for Test 1 (left) and Test 2 (right)

TEST SETUP

As shown in Fig. 5 and Fig. 7, the specimens spanned horizontally and were supported at each end. The splice was located at mid-span. Load was applied through a steel spreader beam that was centered over the splice. A hydraulic actuator applied load to the center of the spreader beam (Fig. 8). Pressure in the hydraulic system was monitored and recorded during testing using an electronic pressure gauge. The applied force was calculated by multiplying the gauge pressure by the internal cross-sectional area of the jack. The pressure gauge was calibrated prior to testing and found to be accurate to within +/- 0.5%.

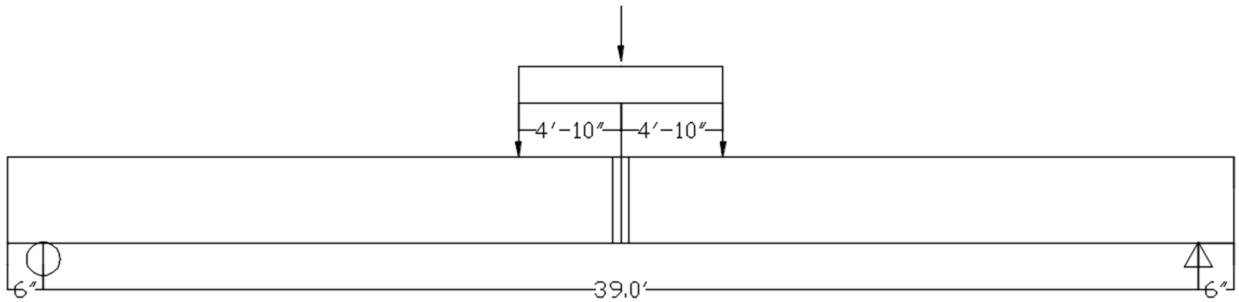

Fig. 7 – Support and load geometry

Fig. 8 – Hydraulic jack and spreader beam

String potentiometers were placed on both sides of the specimen at the splice location to measure mid-span vertical displacement during testing. Displacements were effectively identical for each side, indicating that the piles did not rotate during testing. Displacement was also measured periodically during testing using a tape measure; the tape measure readings were consistent with those reported by the string potentiometers.

TEST RESULTS

The specimens supported peak moments of 273 kip-feet and 287 kip-feet for Test1 and Test 2, respectively. These values include moment from both self-weight and applied load. Moment-displacement responses of the piles are presented in Fig. 9. The target moment based on FDOT requirements (M_{target}), calculated nominal moment (M_n), and calculated upper-bound moment ($M_{up\text{-}bnd}$) are also labeled in the figure; these values will be discussed in the "Theoretical Comparison" section of this paper.

Both specimens exhibited similar behavior during testing. For both tests, the load response was initially linear-elastic until the pile lost stiffness due to cracking. Cracks were primarily vertical and were located within the constant moment region. The prestress force limited formation and growth of cracks at locations away from the splice, however, closer to the splice the full prestress force is not present, which allows crack initiation.

For both tests, peak load corresponded to crushing of the compression zone adjacent to the splice. The portion of the concrete that spalled away at peak load was outside of the transverse reinforcement that supported the core of the piles. Displacement at peak moments for Test 1 and Test 2 were 4.5 inches and 7.3 inches, respectively. Crushing occurred on the same side of the splice as the largest of the flexural cracks (Fig. 10). The additional displacement observed in Test 2 is attributed to the separation between the concrete and the splice assembly (Fig. 11).

For Test 1, popping noises were heard as the test approached the peak moment. The noises seemed to come from the splice location, and are attributed to bending of the locking pins and separation of the splice assembly and #10 bars from the concrete. For both tests, the locking pins were initially centered in the holes, but testing caused permanent deformation of the pins towards the side of the holes (Fig. 12). Deformation was more obvious in the pins on the flexural-tension side.

The post-test width of the cracks adjacent to the splice in Test 1 and Test 2 (Fig. 10) were approximately 0.5". The cracks were wider while the piles were under load; however, no measurements of crack width were taken during loading. The width and location of the crack indicate that the #10 bars and splice assembly did not maintain strain compatibility with the concrete at peak loads. Separation of the concrete and splice observed in Test 2 also supports this notion. Loss of strain compatibility is considered in the theoretical calculations presented in the next section.

Fig. 9 – Moment-displacement response of piles

Fig. 10 – Failure of compression zone of Test 1 (left) and Test 2 (right)

Fig. 11- Gap between splice assembly and the concrete in Test 2

Fig. 12 – Test 1 location of locking pin prior to (left) and after (right) load tests

THEORETICAL COMPARISON

The experimental moment capacities of the splices were 273 kip–ft for Test 1 and 287 kip-ft for Test 2. These values account for bending moments caused by self-weight and applied load, and were calculated as:

Table 3 – Values used for calculating loads

Variable	**Definition**	**Value**
w_{sw}	Uniform load from self-weight	0.337 klf
L	Span length	39.0 ft
P	Load applied by jack	28.5 kip at peak load for Test 1 30.3 kip at peak load for Test 2
a	Distance between load point and support	14.7 ft

The experimental moment capacity was compared to two different theoretical capacities: 1) Nominal capacity using specified material properties, and 2) Upper-bound capacity using ultimate material properties. These capacities are compared to the experimental results in and summarized in Table 4. Comparisons are also made to the FDOT-required flexural capacity (M_{target}) of 245 kip-ft. The experimental capacity of both specimens exceeded the FDOT-required capacity.

Table 4 - Comparison of theoretical and experimental bending moments

$\mathbf{M_{exp}}$	$\mathbf{M_{exp}}$	$\mathbf{M_n / M_{exp}}$	$\mathbf{M_{up\text{-}bnd} / M_{exp}}$	$\mathbf{M_{target} / M_{exp}}$
Test 1	273 kip-ft	0.74	1.02	0.90
Test 2	287 kip-ft	0.70	0.97	0.85
Note: M_n = 201 kip-ft, $M_{up\text{-}bnd}$ = 278 kip-ft				

The #10 bars were designed to have the smallest tensile capacity of the components within the splice system. Accordingly, the splice capacity was modeled using the reinforced concrete section shown in Fig. 13. Prestressing strands were discontinuous at the splice location and were not considered in the flexural capacity calculations.

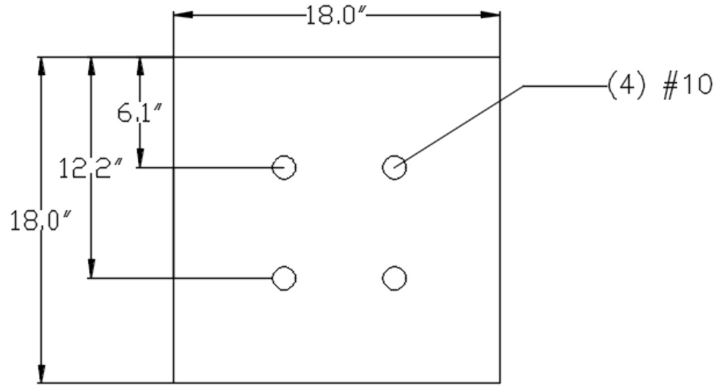

Fig. 13 – Cross-section used for calculating flexural capacity

The nominal moment capacity (M_n) was calculated based on the material properties specified in the design documents provided by the splice and pile suppliers. Yield strength of #10 bars was taken as 72 ksi, and the concrete compressive strength was taken as 6000 psi. The nominal capacity of 201 kip-ft was 26% and 30% lower than Test 1 and Test 2 experimental values, respectively. This result is attributed to the conservative values used for the material properties, and to the loss of bond between the lower #10 bars and the concrete. This loss of bond allowed the #10 bars to reach stress values much higher than the specified yield stress. Thus the nominal capacity provides a very conservative value for the flexural capacity of the pile splice.

The upper-bound capacity ($M_{up\text{-}bnd}$) of the splice connection was based on tested material properties and on physical observations from testing. As such, the upper-bound capacity is intended to be a more accurate description of the experimental results than the nominal capacity. Details of the upper-bound calculations are shown in Fig. 14, and key concepts from the calculations are described below:

- <u>The bottom #10 bars reached ultimate stress</u>. The large crack observed behind the cap plate indicated that bond between the lower #10 bars and the concrete was lost at higher load levels. Accordingly, strain compatibility was not assumed for the bottom bars. Rather, stress in the bottom bars was assumed to equal the tested ultimate stress of 110 ksi. This is the maximum possible stress that the bars could support, hence the resulting moment capacity is termed the “upper-bound” capacity.

- Strain compatibility of top #10 bars. The crack behind the cap plate was much smaller at the level of the top bars and it is assumed that bond was maintained between the concrete and top bars. Stress in the top bars was determined using the constitutive relationship shown in Fig. 15. The figure is based on tested data for the #10 provided by the bar supplier. Stress in the top bars was calculated to be 30 ksi at the upper-bound capacity.

- Concrete compressive strength of 7700 psi. Test data provided by precaster shows that typical compressive strengths for the concrete mix used in the piles was approximately 7700 psi at 28 days.

- Equilibrium. Force equilibrium is satisfied at a neutral axis depth of 4.54 in. This value is based on the approach and material properties described above, and the geometry shown in Fig. 13.

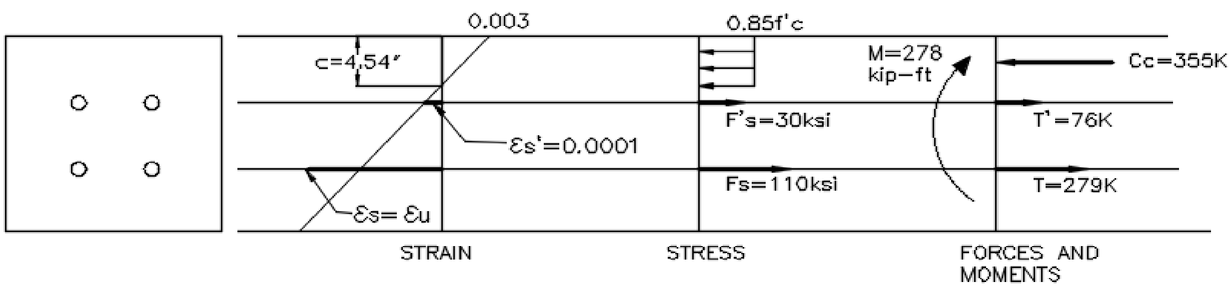

Fig. 14 – Mechanics at ultimate capacity

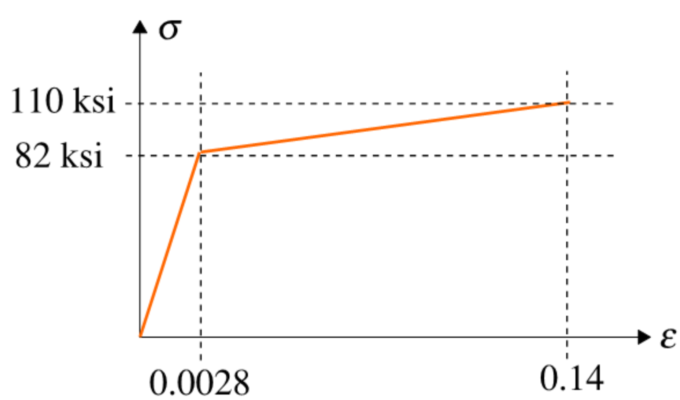

Fig. 15 – Stress-strain relationship for top bars in upper-bound calculations

The calculated upper-bound moment capacity is 278 kip-ft. This value is 2% larger than the experimental capacity of Test 1 and 3% lower than Test 2. This level of agreement suggests that the assumptions made in the upper-bound calculations are reasonable descriptions of the physical behavior of the splice at ultimate capacity. Thus it is concluded that the bottom bars were approaching ultimate stress when the concrete compression zone crushed.

SUMMARY AND CONSLUSIONS

Tests were conducted to identify the flexural capacity of a splice connection for precast concrete piles. Two specimens were tested, each consisting of two 18 inch x 18 inch x 20 foot pile segments. After positioning the segments end-to-end with a crane, the splice connections were completed within minutes. The resulting 40-foot long specimens were load tested in four-point bending to determine the flexural capacity of the splice connection. The connections exhibited experimental moment capacities of 273 kip-ft and 287 kip-ft for Test 1 and Test 2, respectively. For both tests, peak moment corresponded to crushing of the concrete in the flexural compression.

Nominal and upper-bound flexural capacities were calculated for the splice connection. The nominal capacity was calculated based on specified material properties and was 201 kip-ft, or 26% and 30% lower than the experimental capacities of Test 1 and Test 2, respectively. The upper-bound capacity was calculated using tested material properties and by assuming that the lower #10 bars were at ultimate stress. This assumption is supported by the observation made during testing that the concrete and splice assembly separated at higher loads. The separation indicated loss of bond (strain compatibility) between the #10 bars and the concrete, and allowed the bars to reach stresses much higher than the specified yield stress. The upper-bound capacity was 278 kip-ft, or 2% larger than the experimental capacity of Test 1 and 3% lower than the experimental capacity of Test 2. These results suggest that the lower #10 bars were near ultimate stress when the concrete compression zone crushed at peak load.

The Florida Department of Transportation requires that mechanical splice connections in 18 inch x 18 inch piles have a flexural capacity of at least 245 kip-ft. Both of the splice specimens exceeded this value.

ACKNOWLEDGEMENTS

Testing and analysis were sponsored by Emeca/SPE-USA and Aver Technologies, Inc. The pile specimens were fabricated by Gulf Coast Pre-Stress.

REFERENCE

1. Florida Department of Transportation, *Standard Specification for Road and Bridge Construction*. 2015 Tallahassee, FL.

2. Florida Department of Transportation, *Design Standards Index No. 20618*. Tallahassee, FL.