Experimental investigation of a splice for ultra-high-performance concrete H piles under flexure, shear, and tension

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- An experimental study was conducted to validate the anticipated behavior and confirm the capacity of the ultra-high-capacity concrete splice connection for use in bridge foundation applications.
- The splice details were developed to perform in tension, compression, bending, and shear both during installation and when in service.
- Full-scale laboratory testing included a total of six tests consisting of one direct tension test, bending tests about both the weak and strong axes, and three shear tests.

A significant portion of the U.S. bridge infrastructure is rapidly approaching the end of its intended design life. Furthermore, a recent evaluation of the condition of bridges in the United States by the American Society of Civil Engineers resulted in a C grade, which indicates infrastructure in fair condition that requires sustained investment and systematic intervention.[1] According to the National Bridge Inventory database at the end of 2024, approximately 623,000 bridges were in service, of which 43.7% were classified as being in good condition, 49.6% in fair condition, and 6.7% in poor condition.[2] The aging infrastructure, in combination with the continuous increase in traffic volume, demands rapid improvements to the nation's bridge infrastructure, with an emphasis on increasing bridge longevity. In addition, the 2005 American Association of State Highway and Transportation Officials' strategic plan for bridge engineering[3] identified extending the service life of bridges as one of the greatest challenges facing state departments of transportation (DOTs). Producing safer, economical bridges at a faster rate, with a minimum service life of 75 years and reduced maintenance costs, is a driving objective to satisfy the infrastructure needs of the country. Several state DOTs and the Federal Highway Administration (FHWA) are engaged in the development of broadly applicable bridge components that are economical, highly durable, and used in rapid construction using high-performance materials. Ultra-high-performance concrete (UHPC) is an advanced cementitious material with enhanced compressive and postcracking tensile strengths, and durability properties. UHPC has compressive strengths exceeding 22 ksi

(150 MPa) due to the dense packing of its constituent materials. Due to the presence of steel fibers, this material also has a ductile tensile behavior unlike traditional concrete and exhibits a sustained postcracking strength of 1.5 to 1.8 ksi (10.3 to 12.4 MPa). The very low porosity of the UHPC improves its durability compared with normal concrete and high-performance concrete. UHPC has a chloride ion penetration resistance that is 28 times lower than that of normal concrete and a reinforcement corrosion rate 120 times slower than that of normal concrete.[4] Previous use of UHPC for bridge applications[5–8] in the United States has proved effective with minimal maintenance requirements.

Foundations in a routine bridge can contribute up to 30% of the overall bridge cost. There are many different types of piles used to support structural loads, including concrete, steel, timber, and composite piles. The most common deep foundations chosen for building and bridge foundations are steel H piles and precast, prestressed concrete piles. Both of these pile types have limitations in terms of durability and drivability. In the case of precast, prestressed concrete piles, the ends of the piles are not effectively prestressed due to the transfer length of the prestressing strands, resulting in reduced tensile capacity in these regions. Tensile stresses can develop in concrete piles during diving in certain conditions, such as going from a hard soil layer to a soft soil layer or driving in soft clays, which can lead to the cracking of piles, affecting the integrity and longevity of pile foundations. Also, prestressed concrete piles can fail in compression due to the use of large driving hammers or hard driving conditions. Improper handling of the concrete pile during shipping and installation can also cause cracking of the pile. Exposing these piles to chlorides will result in corrosion of steel reinforcement and lead to loss of bond between steel and concrete, as well as a reduction in pile capacity.[9] Similarly, steel H piles, which rely only on skin friction, can experience local flange buckling under harsh driving conditions. Corrosion is also a major problem for steel piles embedded in fill materials or above the water table. The water table fluctuation zone[10] is the zone where the most corrosion occurs on steel H piles. Thus, not only can the driving conditions influence the performance of steel H piles but their corrosion can also dramatically influence bridge service life. Therefore, increasing the longevity of bridges requires an increase in the durability of foundations as well.

To achieve the target service life, minimize drivability challenges, and ensure durability, the use of UHPC pile foundation elements was first investigated at Iowa State University (ISU). By taking advantage of the high compressive, tensile, and bond strengths of UHPC and giving consideration to prefabrication options, a tapered H-shaped section was introduced as the most viable cross section.[4] Given the most frequent use of HP 10×57 steel piles (with a depth of 10 in. [254 mm] and weight of 57 lb/ft [86 kg/m]) in the Midwest for bridge foundations, an equivalent UHPC H pile with comparable outer dimensions, weight per linear foot, and flexural rigidity was designed and tested.[4] The tapered H-shaped, heavily prestressed section was designed to minimize the formation of air pockets during production while increasing longevity and reducing the maintenance cost of bridge foundations. Because it can resist loads through skin friction and end bearing, the UHPC pile can be shorter in length than its steel counterpart and cost-competitive with steel pile foundations.[4]

The 10 in. (254 mm) deep UHPC pile section from the ISU investigation was prestressed with 10 standard 0.5 in. (13 mm) diameter prestressing strands with no mild steel or shear reinforcement. A reduced cover of 0.75 in. (19 mm) was used for prestressing strands by taking advantage of UHPC's high strength and durability. Standard 2 in. (51 mm) center-to-center spacing between the prestressing strands was maintained to allow the UHPC to flow freely through the section during casting. The final UHPC pile cross-section details and a comparable steel pile (HP 10×57) are shown in **Fig. 1**. More details on the design of the UHPC pile cross section are presented in Vande Voort et al.[4] The full-scale vertical- and lateral-load tests on UHPC piles revealed several benefits of the UHPC pile, including a reduced risk of damage during driving, drivability with a greater range of hammers and strokes, and the ability to use existing equipment for pile handling and driving.[4,11] Following successful verification of UHPC pile performance, to accelerate field implementation of the UHPC piles for bridge and building foundations, an experimental research program was initiated at ISU to develop a dry connection for the field splicing of UHPC piles. The summary of this experimental study is presented in this paper.

UHPC pile splice design

Splicing of piles is commonly done in practice due to limitations in handling and transportation of long piles and the associated costs. Also, splicing of piles is done when pile lengths required are too long to drive in one piece with the available equipment. Splicing also alleviates the need to calculate exact pile lengths prior to installation, reducing waste and potentially saving costs. The splice must be capable of resisting stresses induced during driving and service and ultimate loads. Splices should be effective without significantly extending construction time, be as durable as the piles, and be cost-effective. There are several splicing details available for precast concrete piles; however, with the optimized H shape of the UHPC pile, there were no suitable splice connection details available for field splicing, especially for a dry connection. A review of the literature on available splice details indicated that to minimize construction delays, dry connections (such as welding, bolting, or the use of quick-setting epoxy grout) are generally preferred methods for extending piles in the field. It is common practice to use welding when steel piles are spliced because this is considered an efficient field technique. Consequently, a welded splice concept was preferred for the UHPC piles.

Prototype bridge

To develop splice details, a 223 ft (71 m) long and 40 ft (12.2 m) wide integral abutment bridge in Sac County, Iowa,

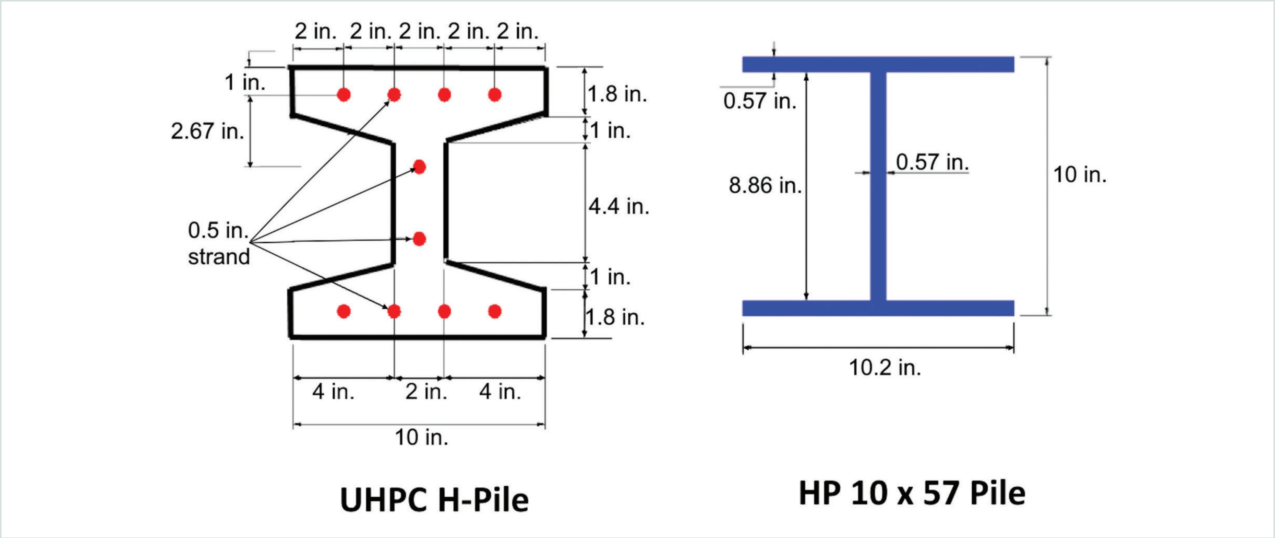
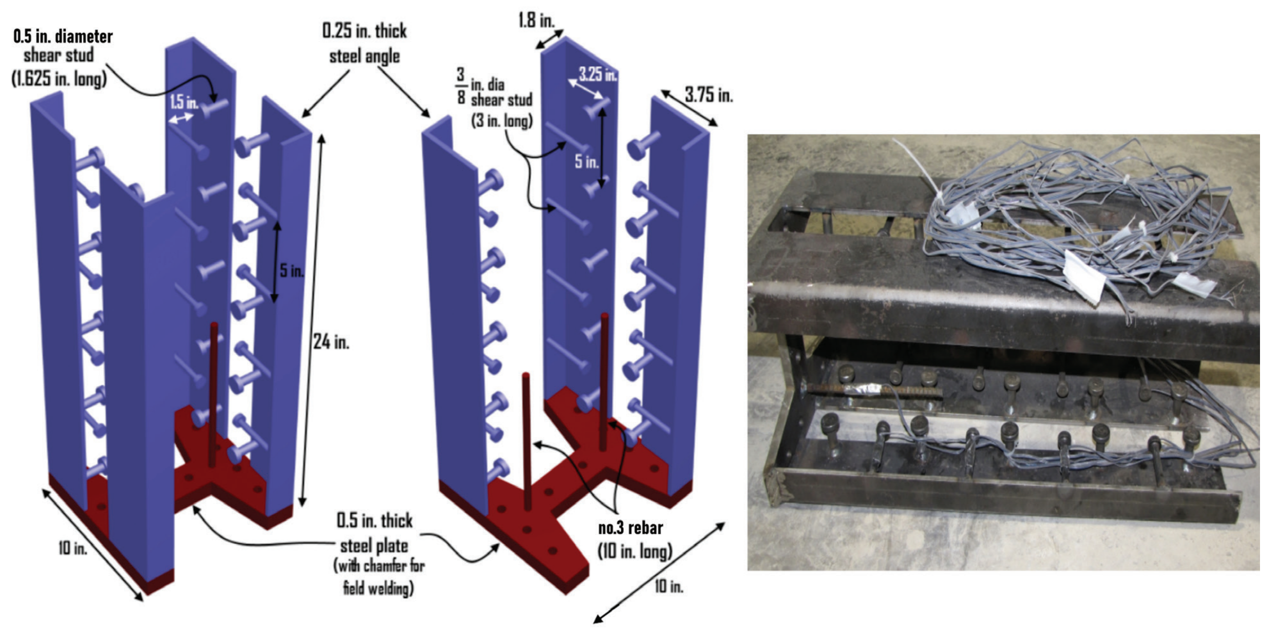


Figure 1. Cross-section details of the ultra-high-performance-concrete (UHPC) H pile compared with the steel HP 10×57 pile. Note: 1 in. = 25.4 mm.

was selected as the prototype bridge for designing the UHPC piles. Given the length, the bridge was expected to subject the piles to lateral deformation of up to 1 in. (25 mm) due to thermal expansion and contraction. The bridge consisted of three spans of 55.75, 106.5, and 60.75 ft (17, 32.5, and 18.5 m) in length from west to east. The abutments were designed according to the standard Iowa DOT details[12] with HP 10×57 steel piles in the weak-axis direction and a few UHPC H piles. The soil at the Sac County bridge site consists of cohesive clay and silty clay. More details about the soil profile and prototype bridge can be found in Garder.[13]

Pile splice detail

The splice details were developed to provide adequate performance in tension, compression, bending, and shear during installation and when in service. To design the splice for the tensile capacity, the forces expected during the driving process were determined using structural and geotechnical software with the measured soil parameters and hammer type used for pile driving. Based on the results and authors' experience with the level of tensile stress observed in piles while driving in Iowa soils in previous research projects, the pile splice details were designed to have a minimum of 50% of the UHPC pile capacity in tension. For the prototype bridge designed to Iowa DOT standards, the piles were not expected to see any tension force in service, though they can experience tension during driving. Therefore, the chosen value for the splice capacity in tension was conservative. The moment and shear capacity of the spliced pile section was designed to reach the full moment and shear capacity of the unspliced UHPC H pile section under both 0 and 200 kip (0 and 890 kN) axial loads. The moment capacities of the UHPC H pile with 0 kip axial load in weak- and strong-axis directions were determined to be 1200 and 1700 kip-in. (135.6 and 192 kN-m), respectively. With a 200 kip axial load, the corresponding values were 1300 and 2300 kip-in. (146.9 and 260 kN-m), respectively. The details of the UHPC pile splice embedment are shown in **Fig. 2**. The UHPC pile splice, which was cast at the pile end, consisted of a 0.5 in. (13 mm) thick H-shaped A50 steel plate with a yield strength of 50 ksi (345 MPa) and four 0.25 in. (6.3 mm) thick A50 steel corner angles with shear studs. The corner angles were welded to the H-shaped steel end plates with a $^3\!/_{16}$ in. (4.8 mm) thick fillet weld, along their entire length. Standard 3 in. (76 mm) long, 0.375 in. (9.5 mm) diameter shear studs and 1.625 in. (41 mm) long, 0.5 in. (13 mm) diameter shear studs at 5 in. (122 mm) spacing were used along the long and short sides of the corner angles, respectively. The steel splice embedment was 24 in. (610 mm) (Fig. 2). Edges of the H-shaped end plate were beveled to accommodate a $^5\!/_{16}$ in. (8 mm) full-penetration weld along the entire perimeter to splice the UHPC piles together. Two 10 in. (254 mm) long, Grade 60 (420 MPa), no. 3 (10M) reinforcing bars with a diameter d_b of 0.375 in. (9.5 mm) were welded to the end plate 3.25 in. (82.5 mm) from the pile centerline along the web centerline to help distribute the tension forces across the depth of the pile. The fabricated UHPC splice embedment is also shown in Fig. 2. The splice embedment was placed in the pile formwork at the ends of UHPC piles, which facilitated welding between two H-shaped steel plates to establish the connection. The tensile forces developed in the pile were first transferred to the shear studs and corner steel angles through bond and shear transfer mechanisms and then to the welds between the splices. At the splice location, all of the tension force was taken by the welds between the splice plates. The cross sections of corner steel angles, weld lengths, and thicknesses were calculated to meet the strength requirement for the splice. The compressive forces between the spliced pile sections were designed to be transferred via full bearing of the end steel plates of the splice.



Schematic of the splice detail

Schematic of the splice detail with corner angle not shown

Fabricated UHPC pile splice

Figure 2. Schematic and fabricated ultra-high-performance-concrete (UHPC) splice embedment details. Note: No. 3 = 10M. 1 in. = 25.4 mm.

Design of test units and precast concrete fabrication

Four identical UHPC piles, each measuring 4 ft (1.22 m) in length and incorporating the aforementioned splice embedment details, were designed and fabricated at a precasting facility in Omaha, Neb. These specimens were produced to experimentally assess the performance of the splice connection under tensile, shear, and flexural loading conditions. The required number of splice embedments was fabricated by the precast concrete producer. Welding of the corner plates and shear studs was carried out by a certified welding technician at the precasting facility. Following fabrication, steel strain gauges were installed on selected long and short shear studs as well as on the reinforcing bar embedment. One end of each splice specimen was configured to transition into a full 10 × 10 in. (254 × 254 mm) square cross section to accommodate four 0.75 in. (19 mm) diameter high-strength threaded rods arranged in a square pattern. These threaded rods were positioned within the formwork before casting and subsequently embedded in the UHPC over a length of 9 in. (230 mm). Wooden side forms with expanded polystyrene foam inserts were used for casting the UHPC pile splice specimens and were assembled by the precast concrete producer at the plant. The test specimens were instrumented with standard foil strain gauges. The splice embedment was positioned within the wooden formwork, and prestressing strands were threaded through openings in the splice embedment end plate. The bottom four prestressing strands were first aligned in their designated configuration and tensioned to an initial prestress of 202.5 ksi (1400 MPa), corresponding to approximately 75% of their ultimate tensile strength. Following the tensioning of the bottom strands, expanded polystyrene foam inserts were affixed to the wooden forms using double-sided tape and sealant to create the desired H-shaped cross section. Once the inserts were secured, the remaining six prestressing strands were arranged and tensioned to the same initial prestress level of 202.5 ksi. UHPC mixing was performed at the precast concrete facility using a 4.0 yd^3 (3.0 m^3) mixer. Upon completion of batching, the fresh UHPC was discharged into a transport bin and conveyed to the casting bed via an overhead crane. The material was then placed into the formwork for all laboratory test specimens, with careful rodding of the web regions to minimize air entrapment and prevent the formation of voids. Immediately after placement, the top surfaces of the UHPC specimens were covered with plastic sheeting to minimize moisture loss. A tarp was then placed over the specimens, and propane heaters were used to maintain an initial curing temperature of approximately 86°F (30°C). Concurrently, 3 in. (75 mm) diameter UHPC cylinders were cast to monitor strength development. These cylinders were tested periodically during the initial curing phase to determine compressive strength. After the UHPC achieved a compressive strength of 14 ksi (97 MPa), the prestressing strands were detensioned by cutting them at the member ends. The specimens were subsequently subjected to steam curing for 48 hours at 194°F (90°C). On the day of testing, the measured compressive strength of the UHPC in the specimens ranged from 21.7 to 22.8 ksi (147 to 157 MPa).

Experimental testing

Full-scale laboratory testing was conducted to validate the anticipated behavior and confirm the capacity of the UHPC splice connection. The experimental program included evaluation of the splice region under direct tensile loading, as well as under critical shear and flexural demands. A total of six tests were performed, consisting of one direct tension test, bending tests about both the weak and strong axes, and three shear tests. Two test specimens were used to complete this program. For each test, two 4 ft (1.22 m) long UHPC pile segments incorporating steel splice embedments (Fig. 2) were fabricated and subsequently joined at their ends using a $\frac{5}{16}$ in. (8 mm) continuous weld around the splice interface.

Splice direct tension test

To simulate the stresses encountered by the splice region while driving in the field, a self-reacting test frame with stiff steel end sections was constructed in the ISU structural laboratory. The self-reacting test frame was supported on rollers, and neoprene pads with Teflon sheeting were provided underneath the rollers to minimize friction forces. The schematic of the self-reacting frame is shown in **Fig. 3**. An 8 ft (2.44 m) long UHPC splice pile specimen was produced by welding two 4 ft (1.44 m) long UHPC specimens with splice embedment. The welding was performed by a certified welder. The test specimen was then subjected to direct tension force using the test setup shown in Fig. 3. The UHPC splice pile was attached to the steel end beams of the reacting frame using the 0.75 in. (19 mm) diameter high-strength threaded rod embedded in the UHPC pile at the unspliced end. Two 110 kip (490 kN) hydraulic jacks were used to apply the load onto the steel H sections on the rollers, which in turn applied the direct tension force on the UHPC pile. The total load applied in each jack was monitored using the load cells mounted on each of the two steel beams of the reaction frame. Several instruments, including displacement gauges and strain gauges, were used to measure the response of the spliced UHPC pile. A total of 10 linear variable displacement transducers (LVDTs) and two string potentiometers were placed along the length of the spliced pile, across the weld splice joint, to measure any gap opening between the steel splice details and UHPC. The external instrumentation details are shown in Fig. 3. The corner steel angles of the splice were also instrumented with external steel strain gauges (SG 1 to SG 7 in Fig. 3) to measure the strain demand in the steel angles during loading. Also, the shear studs and the reinforcing bars in the splice embedment were instrumented with strain gauges to measure the critical stresses under direct tension loading. The details of the strain gauges on the shear studs are provided in Fig. 3.

Testing sequence, observations, and results

The UHPC spliced pile was subjected to a direct tension load in 10 kip (44.5 kN) increments using the hydraulic jacks. Both hydraulic jacks were controlled using a single pump to have equal displacements and thus apply pure tension force on the specimen. During the test, the data from all gauges and displacement devices were recorded using a computer-based data acquisition system at 1 Hz frequency. At a total load of 45 kip (200 kN), oil leakage from one of the hoses connected to loading jacks was observed. The hydraulic jacks were then unloaded to 0 kip (0 kN) force, and the leaking hose was replaced. All instrumentation was reinitialized before the loading was reapplied. During loading, at a total load of 103 kip (458 kN), noises were heard and fine hairline cracks appeared at nearly 6 in. (150 mm) from the right end of the specimen (**Fig. 4**), where the cross section transitioned to a 10 in. (254 mm) square section. Upon continued application of this loading magnitude, the crack widened significantly, reaching up to 0.5 in. (12.7 mm). The loading was stopped, and the load jacks were unloaded to 0 kip force. Following the unloading, a closer inspection revealed the absence of steel fibers at the crack location (Fig. 4). The applied maximum load of 103 kip created 1.03 ksi (7.1 MPa) of tensile stress at the crack section location, which was close to the expected cracking strength of UHPC matrix without any fibers reported in literature. The maximum applied tension load of 103 kip corresponded to 27.4% of the unspliced pile tension capacity.

To subject the pile splice to tension loads up to 50% of the pile capacity, a retrofit to load application using stiff steel angles was designed and implemented. The retrofit included welding four steel angles to 1 in. (25 mm) thick angles, which were welded to the end beams of the test setup (**Fig. 5**). The four steel angles were overlapped and welded to the corner angles of the splice embedment over a length of 6 in. (150 mm). This retrofit transferred the load directly from the load jacks to the splice region. Following the retrofit, all of the displacement measuring devices (LVDT and string potentiometers) were reinstalled and reinitialized. The external strain gauges on the corner angles of the splice embedment were damaged during the welding of retrofit angles due to excessive heat. New gauges were not installed because the surfaces were not easily accessible for gauge installation. The splice specimen was then subjected to tension loading in increments of 10 kip (45 kN). All of the data from the instruments were recorded at 1 Hz frequency. The pile specimen was subjected to a load of 226 kip (1005 kN), equivalent to 60% of the pile tension capacity. The test was stopped as the maximum capacity of the hydraulic jacks was reached. There was no observed visible cracking in either the pile or the splice region at the maximum load.

The measured gap or crack opening between the splice end plate and UHPC as a function of load is shown in **Fig. 6**. The measured gap opening in the web region at 187.5 kip (834 kN) of load, corresponding to 50% of pile tensile capacity, was around 0.012 in. (0.3 mm), whereas the measured gap opening on the flange side was less than 0.005 in. (0.13 mm). These crack widths are much smaller than the crack widths allowed in bridge elements. This observation supports that the splice detail performed sufficiently and will not cause any durability issues. The total elongation of the pile in the

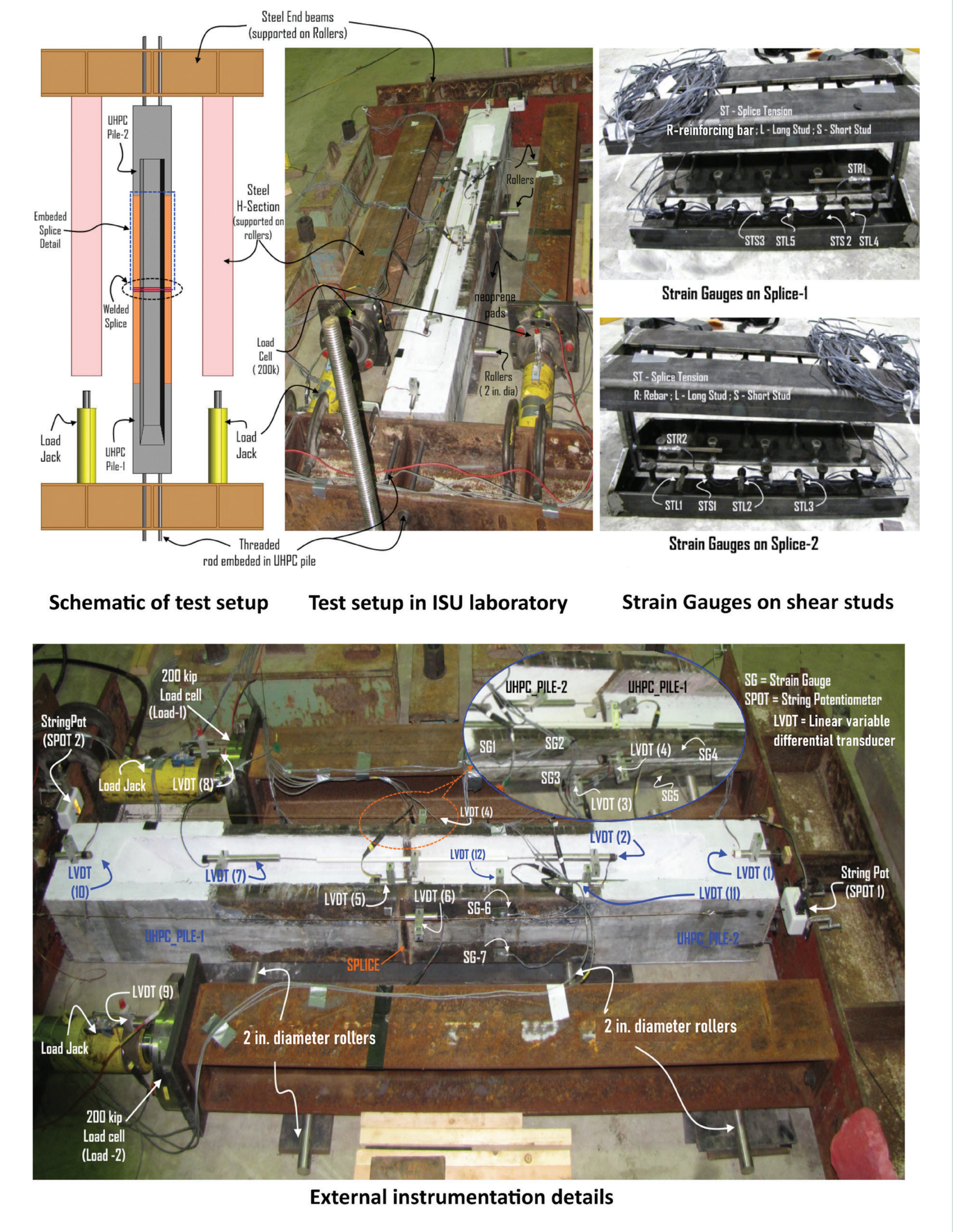


Figure 3. Direct tension test of the pile-splice segment in a self-reacting test frame, along with internal and external instrumentation details. Note: LVDT = linear variable displacement transducer; SG = strain gauge; SPOT = string potentiometer; STL = splice tension long stud; STR = splice tension reinforcing bar; STS = splice tension short stud; UHPC = ultra-high-performance concrete. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

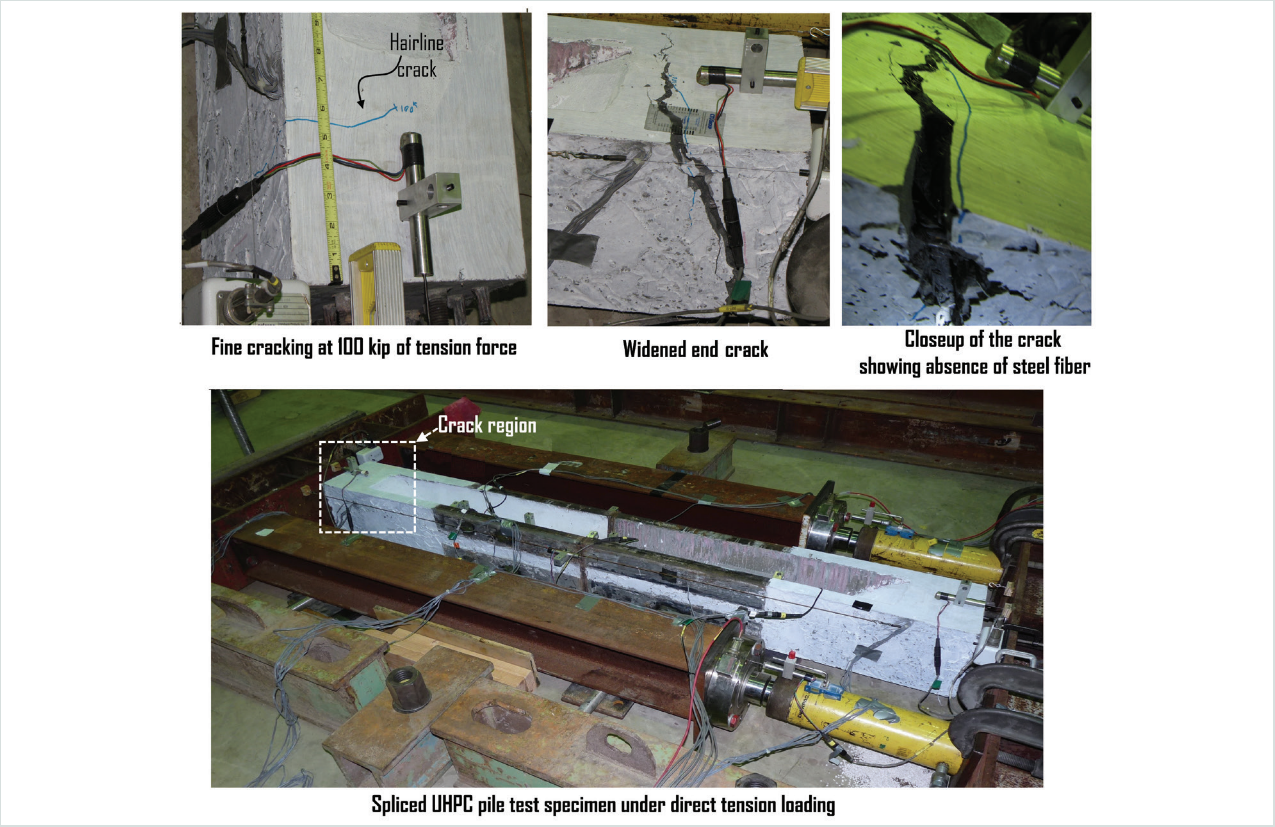


Figure 4. Crack formation in the end region of the spliced tension ultra-high-performance-concrete (UHPC) pile specimen. Note: 1 kip = 4.448 kN.

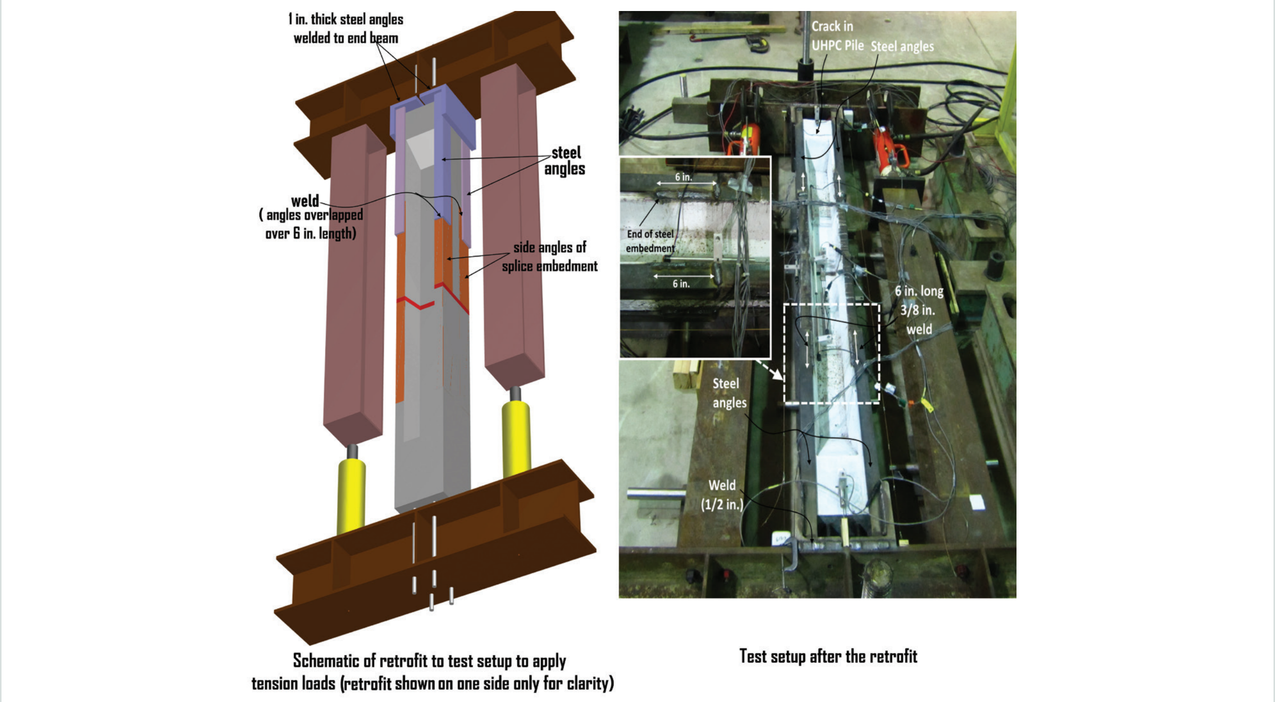


Figure 5. Schematic of the retrofit to direct tension setup for load application and retrofitted test setup in the laboratory. Note: UHPC = ultra-high-performance concrete. 1 in. = 25.4 mm.

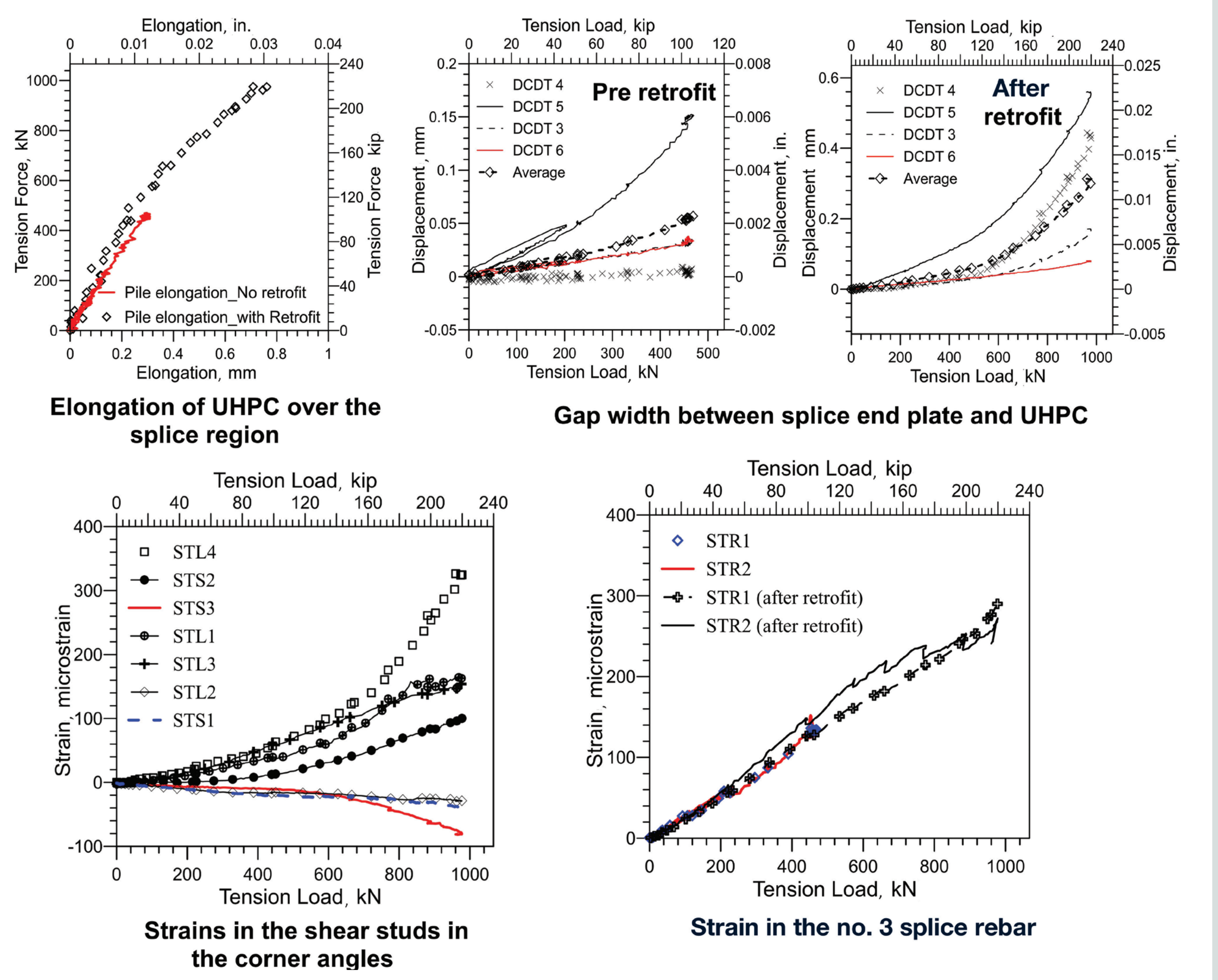


Figure 6. Measured critical strains and crack widths during the direct tension testing of the spliced ultra-high-performance-concrete (UHPC) pile. Note: DCDT = direct current displacement transducer; STL = splice tension long stud; STR = splice tension reinforcing bar; STS = splice tension short stud. No. 3 = 10M.

splice region over a 4 ft (1.22 m) length during both phases of testing is shown in Fig. 6. The splice region's stiffness remained similar in both phases, confirming that the retrofit applied tension forces as intended.

The measured strain in the long and short shear studs along the corner angles with respect to applied loading is shown in Fig. 6. The measured maximum strain in shear studs at the peak load of 226 kip (1005 kN) was 330 microstrain, corresponding to 15% of the yield strain of the shear studs. This suggests that the quantity or diameter of shear studs specified exceeds the necessary requirements and may be optimized in future splice detail designs. Reducing the number of shear studs can decrease the length of the splice embedment from 24 in. (610 mm) and could minimize the total cost of the splice detail.

The measured strain in the no. 3 (10M) reinforcing bar welded to the splice end plate is shown in Fig. 6. The maximum strain measured in the reinforcing bar was 290 microstrains, or 14% of the reinforcing bar yield strain. This indicates that the bar size could be reduced. However, it is not recommended to reduce the bar size or eliminate the bar because it helps to contain the gap opening between end plate and UHPC in the web region (Fig. 6). Further, the authors believe that the gap opening could be reduced by using a larger-diameter reinforcing bar instead of no. 3 reinforcement.

Splice shear test

The shear capacity of the splice connection was evaluated with a spliced UHPC pile subjected to a point load about the strong axis in a simply supported configuration (**Fig. 7**). The 8 ft (2.44 m) long spliced UHPC pile was simply supported over a 3 ft (0.91 m) span and was subjected to three-point bending using a 120 kip (530 kN) hydraulic jack. The load point was located at 8 in. (200 mm) from the splice location, subjecting the splice to a combined shear force and moment.

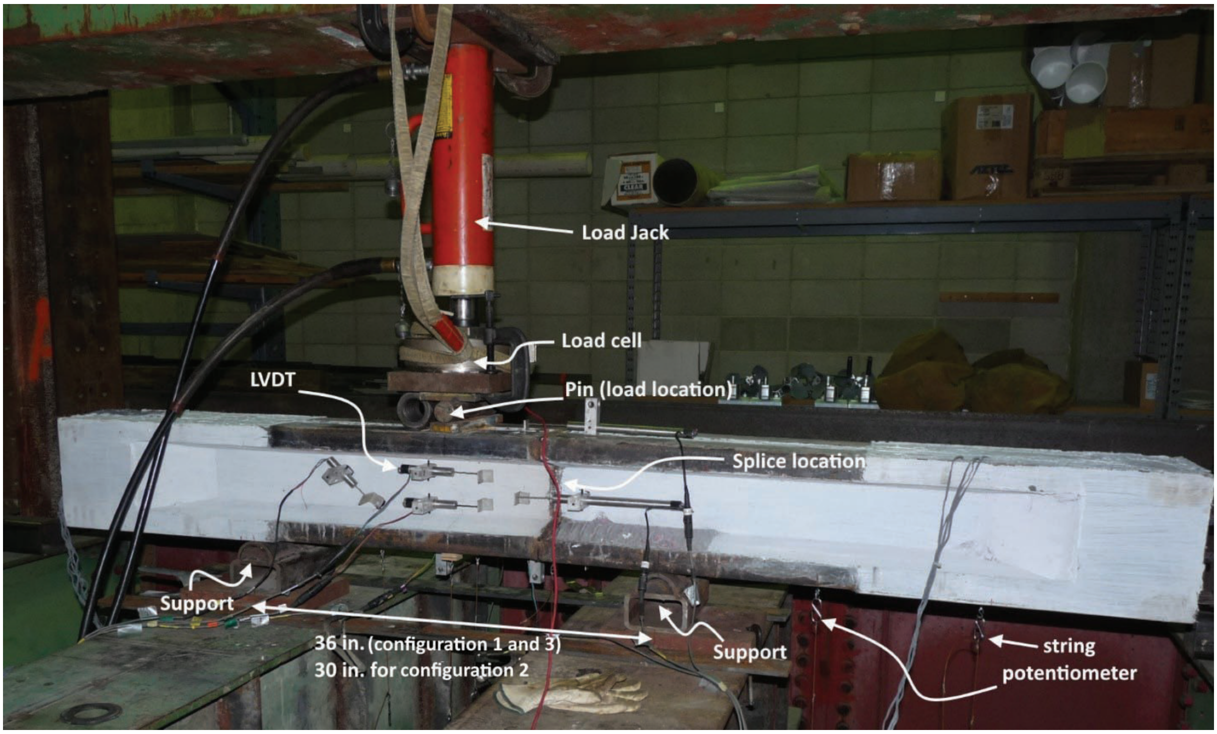


Figure 7. Shear testing setup for spliced ultra-high-performance-concrete (UHPC) H pile in the laboratory. Note: LVDT = linear variable displacement transducer. 1 in. = 25.4 mm.

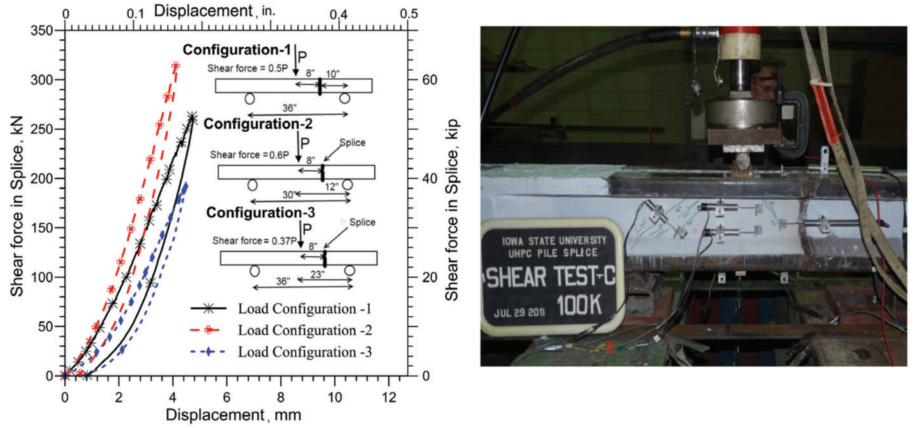
To examine how varying the moment-to-shear ratio *s* influences splice performance, shear tests were conducted using three distinct setups by changing the spacing between the supports and the load point. These different configurations simulated the expected moment, and shear demands at various locations along the pile length in the prototype bridge. The specimen span length and load location points for each of the three load configurations are given in **Table 1**.

Several different types of instruments, including LVDTs and string potentiometers, were used to capture deflections and curvature at the splice region and load location. A total of six string potentiometers were used to measure vertical displacements along the span of the spliced specimens. Three LVDTs at both the load application point and splice location measured average strains and their distribution along the section height. During testing, data from all of the instrumentation was captured at 1 Hz frequency. The loading was applied in 5 kip (22 kN) increments until hairline shear cracking was observed in the pile web region. The maximum load applied in all the load configurations was 110 kip (489 kN). The observed shear force versus displacement under point load for different load configurations is shown in **Fig. 8**. Hairline shear cracks appeared in the web region at a shear force of 48 kip (213 kN) during all load configuration tests; however, the cracks closed completely upon load removal. The splice region was subjected to a maximum shear of 66 kip (294 kN) in load configuration 2, which is nearly 200% more than the typical shear force in the UHPC pile in the prototype bridge. This value is also 37% more than the shear corresponding to web shear cracking capacity of the prestressed UHPC pile.

Table 1. Details of load configurations used for splice shear testing

Load configuration	Span, in.	Location of the splice from the right support, in.	Load location from the right support, in.
1	36	10	18
2	30	4	12
3	36	15	23

Note: 1 in. = 25.4 mm.



Shear Force vs. displacement response

Shear cracking in the web region of UHPC Pile

Figure 8. Displacement and cracking behavior during shear testing. Note: P = load; UHPC = ultra-high-performance concrete. 1" = 1 in. = 25.4 mm; 1 k = 1 kip = 4.448 kN.

Splice flexural tests

The flexural capacity of the splice connection in both weak- and strong-axis directions was evaluated using a simply supported configuration (**Fig. 9**). The 8 ft (2.44 m) long spliced pile was simply supported over a 6 ft (1.83 m) span and was subjected to four-point bending using a 120 kip (534 kN) hydraulic jack. The load points were located 2 in. (50 mm) on either side of the splice, subjecting the splice region to pure moment. The loading was applied in 5 kip (22 kN) increments until the specimens experienced failure. Several different types of instruments, including LVDTs and string potentiometers, were used in both flexural tests to capture deflections and curvature at the splice locations. A total of five string potentiometers were used to measure vertical displacements along the span of the spliced specimens. The string potentiometers were located at the quarter points (17 in. [430 mm] from the supports), at the load points (2 in. [50 mm] from center), and at the center (splice location). The locations and identifications used for these string potentiometers are shown in Fig. 9. Four LVDTs were used at the center of the specimen along the height of the section to measure the average strains and their distribution in the constant bending region. During testing, data from all the instrumentation was captured at 1 Hz frequency.

The spliced UHPC pile specimen tested under direct tension loading was used to evaluate the performance of the splice in the weak-axis bending direction. Reusing the test specimen for this bending test was deemed acceptable because the splice region did not experience any noticeable damage during the previous tension tests. The end crack observed in the tension pile had no effect on the overall response of the specimen because the crack fell outside the support locations.

Figure 10 shows the measured force-displacement and moment-curvature responses of the splice region in the weak direction. The curvature at the splice location was calculated using the LVDTs along the height of the splice region. The splice region experienced failure at a maximum load of 52 kip (231 kN) with the fracture of the weld between the corner angles and the 0.5 in. (13 mm) end plate (**Fig. 11**). This load corresponded to a bending moment value of 918 kip-in. (104 kN-m) on the splice region. This value, when compared with the bending capacity of the unspliced pile section, represents 76.5% and 70.6% of the capacities at 0 and 200 kip (0 and 890 kN) axial load, respectively. On further inspection of the failure surface, it was observed that the shop welding at the precasting plant did not adhere to the requirements and resulted in a shorter weld length between the corner angles and the plate, leading to the failure of the splice as shown in the close-up detail in Fig. 11. It is worth noting, however, that the weld between the two pile pieces did not experience any damage, indicating that the connection had enough strength. Based on the observed damage mode, a full-penetration weld between edge angles and the end plate is recommended for future splice details.

The spliced UHPC pile specimen tested under shear loading was used to evaluate the performance of the splice in the strong-axis bending direction. Similar instrumentation and test setup (Fig. 9) to the weak-axis bending test were used for the strong-axis bending test. Reusing the test specimen for this bending test was deemed acceptable, as the splice region did not experience any noticeable damage during the shear loading tests. The measured force-displacement and moment-curvature responses of the splice region in the strong-axis direction are shown in **Fig. 12**. The curvature at the splice location was calculated using the LVDTs along the height of the splice region. Similar to the weak-axis bending specimen, the splice region experienced failure at a maximum load of 78 kip (347 kN) with the fracture of the weld between the corner angles and the end plate. This load corresponds to a bending moment value of 1326 kip-in. (150 kN-m) on the

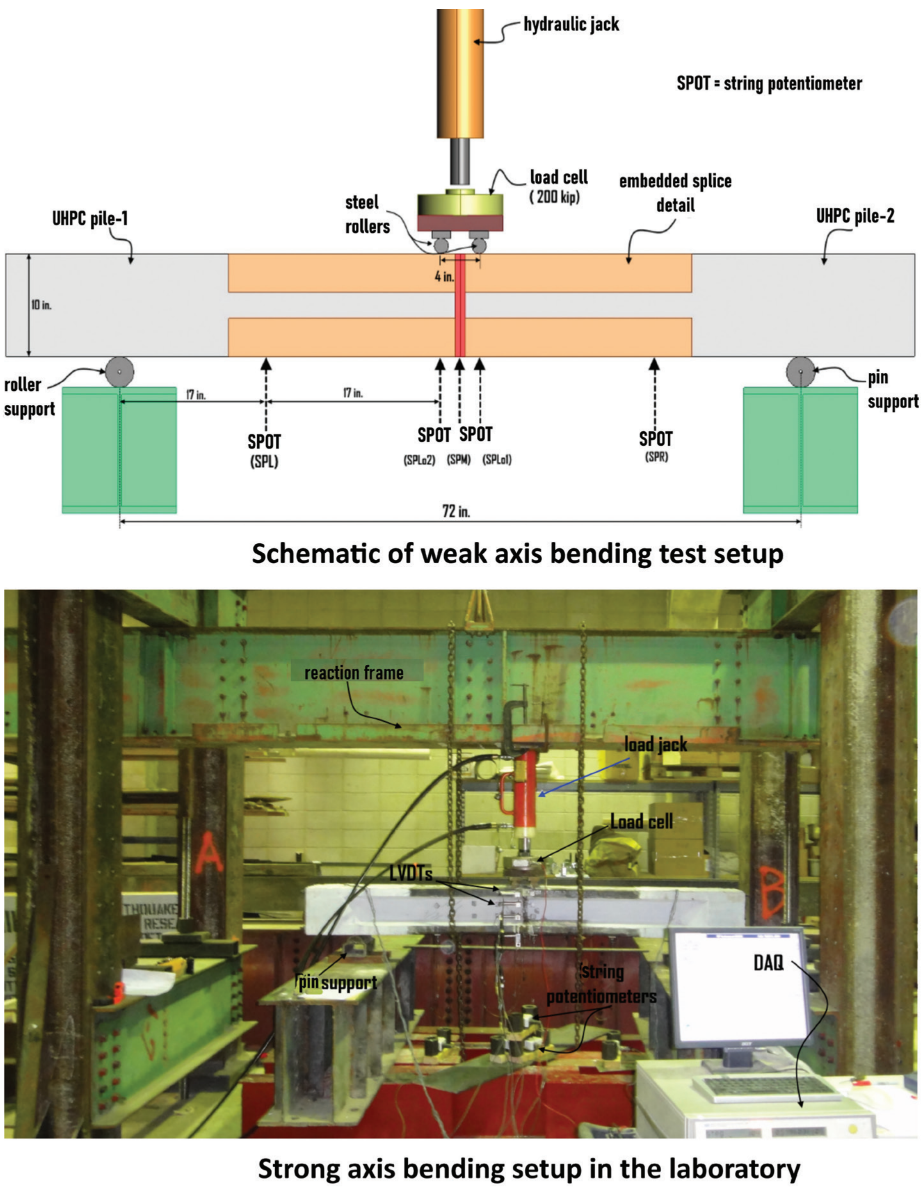


Figure 9. Test setup for weak-axis and strong-axis bending. Note: DAQ = data acquisition; LVDT = linear variable displacement transducer; SPOT = string potentiometer. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

splice region. This value, when compared with the bending capacity of the unspliced pile section, represents 78% and 57.6% of the capacities at 0 and 200 kip (0 and 890 kN) axial load, respectively.

Simplified analysis

A moment-curvature response program for UHPC piles was developed in data analysis software based on the following assumptions:

- Plane sections remain plane.
- Prestress losses occur due to only elastic shortening and shrinkage of UHPC.
- Strands have perfectly bonded to UHPC outside of the transfer regions.
- Effective prestress is applied at the centroid of the section.
- Initial prestressing force does not induce any inelastic strains on the strands.
- Axial loads are applied through the centroidal axis of the pile.

The cross section was discretized into a series of small segments (**Fig. 13**), and the corresponding stress and strain in each segment were computed for a given curvature. These

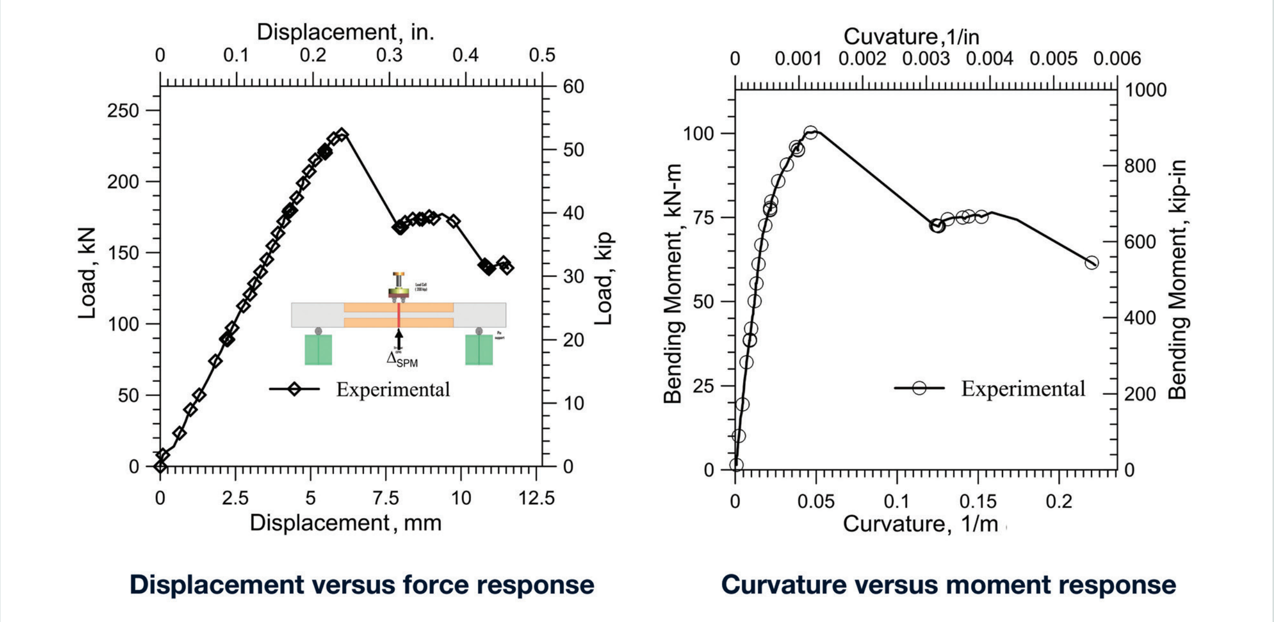


Figure 10. Testing results for the spliced ultra-high-performance-concrete (UHPC) pile under weak-axis bending. Note: Δ_{SPM} = deflection under the load location.

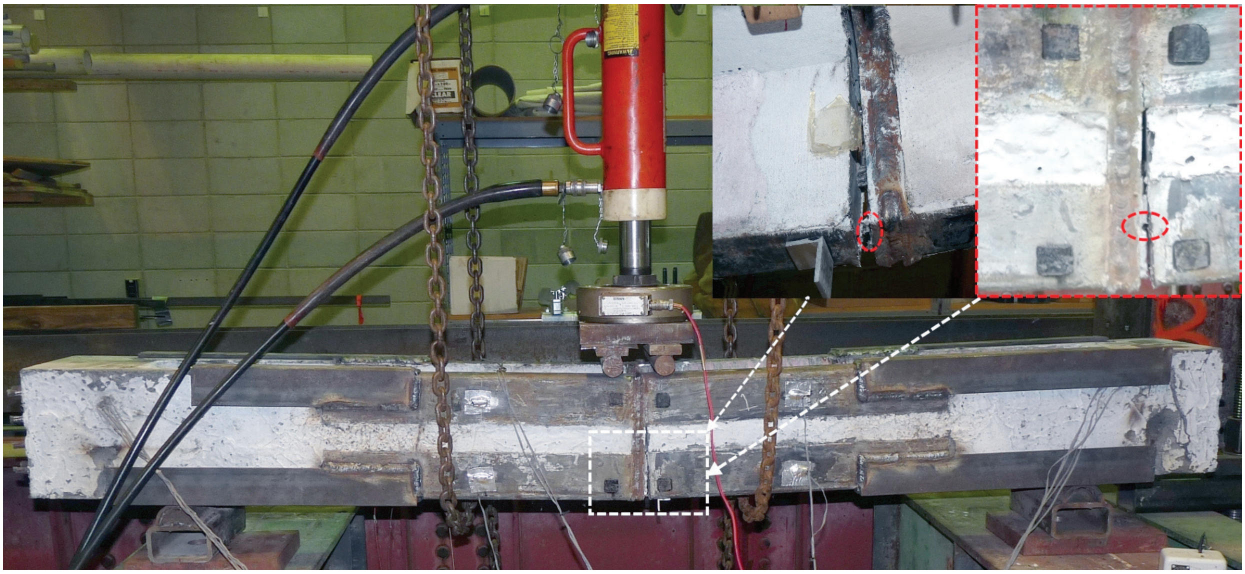


Figure 11. Failure of the test specimen in weak-axis bending and a close-up of the failed welds.

stresses and strains were subsequently converted into forces and moments by multiplying by the appropriate segment area and lever arm. Prestress, associated prestress losses, and applied axial load contribute to a uniform strain in the section, referred to as the zero-curvature strain for both UHPC and prestressing steel. Following determination of the zero-curvature strains, additional tensile and compressive strains induced by curvature were calculated. At each analysis step, stresses and strains in each segment were evaluated using the constitutive relationships for UHPC and prestressing strands (Fig. 13). Standard stress–strain behavior for prestressing strands and experimentally measured UHPC material properties were adopted in the analysis. Segmental forces and moments were then computed based on these strain distributions. The program iteratively determines the curvature and corresponding neutral axis location for each step. Equilibrium is satisfied when the internal force balance is achieved, at which point the summation of moments across the section equals the total moment resistance associated

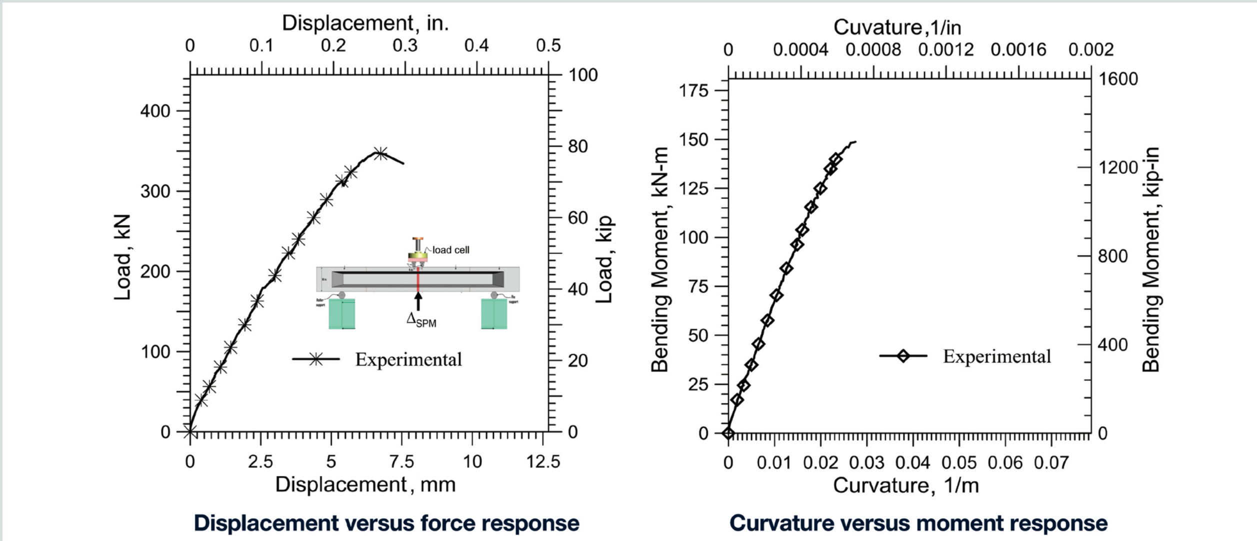


Figure 12. Testing results for spliced ultra-high-performance-concrete (UHPC) pile under strong-axis bending. Note: Δ_{SPM} = deflection under the load location.

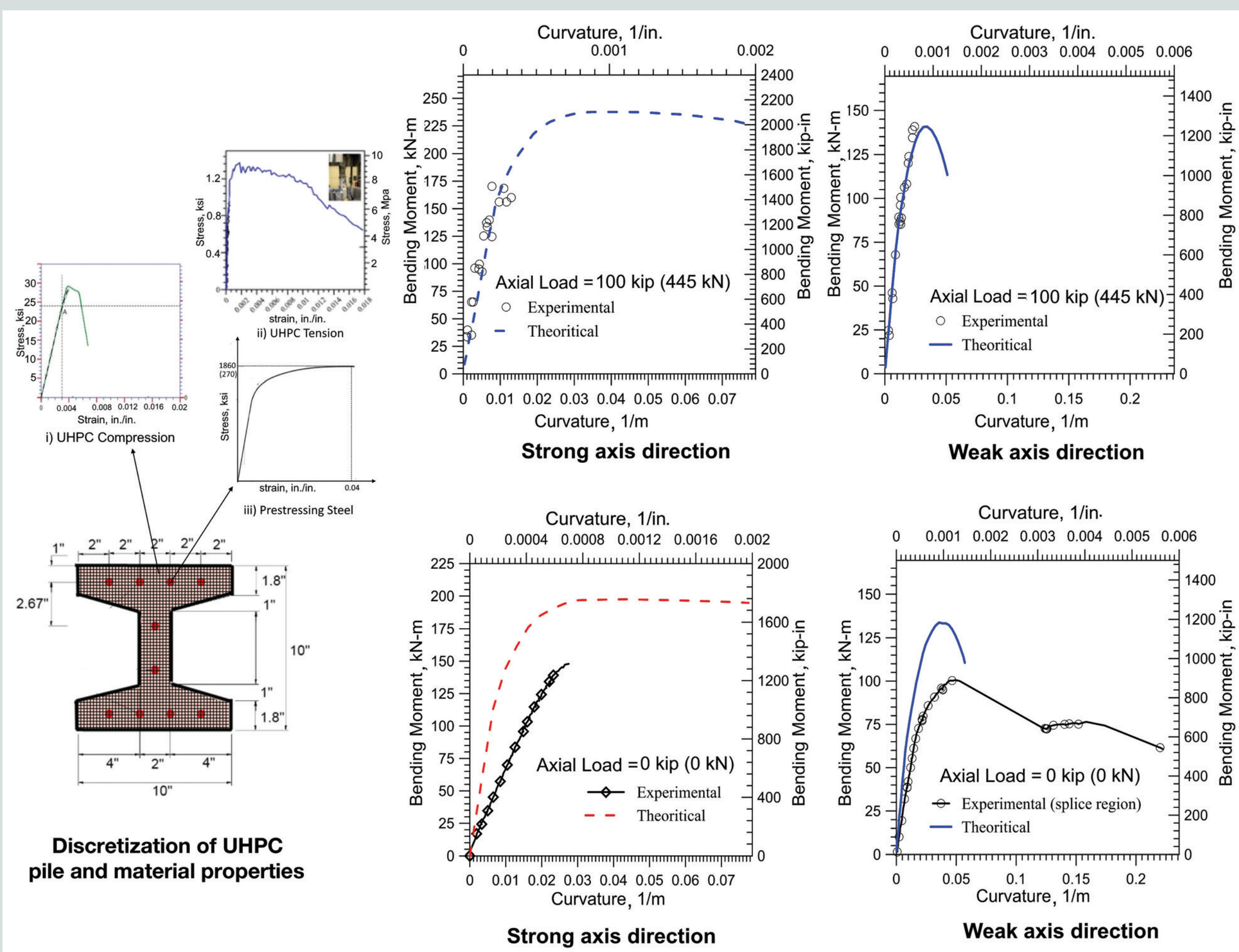


Figure 13. Discretization of ultra-high-performance-concrete (UHPC) pile section and comparison of measured to calculated moment-curvature response of UHPC pile with and without a splice. Note: 1" = 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

with the imposed curvature. Comparisons between measured and calculated moment-curvature responses for the UHPC pile section without a splice, under an axial load of 100 kip (445 kN), and in both weak- and strong-axis bending are presented in Fig. 13. The measured moment-curvature responses about both the weak and strong axes of the UHPC H pile under an axial load of 100 kip were obtained from lateral-load tests conducted on UHPC pile-to-abutment connection specimens. Detailed descriptions of these tests are provided in Aaleti et al.[14] and are not repeated herein. As illustrated in Fig. 13, the moment-curvature analysis based on fundamental principles accurately captures the behavior of the UHPC pile, with calculated values generally within 90% of the experimental results. Comparisons between measured and predicted responses for the splice region, under both weak-axis and strong-axis bending, are presented in Fig. 13. For the splice region, the analytical predictions indicate a stiffer response compared with the experimentally measured behavior. In contrast, the measured moment-curvature response in the splice region is more compliant than that of an unspliced UHPC pile, suggesting reduced stiffness at the splice. This behavior is expected, as the moment-curvature response at the splice is influenced by the quality of the welds and the stiffness of the steel angles. As observed during experimental testing, defects were present in the welds between the splice plate and the angles, which likely contributed to the reduced stiffness of the splice region.

Conclusion

This paper presents the development of a splice detail for the field extension of prestressed UHPC piles and the results from an experimental study to quantify the performance of the UHPC splice under direct tension, flexural loading, and shear loading. Based on the experimental testing of the UHPC splice detail, the following conclusions were drawn:

- Comprehensive tests on splice connections demonstrated that they performed well when subjected to both direct tension and shear. The splice had reserve capacity in excess of 225 and 66 kip (1000 or 294 kN) under direct tension and shear, respectively. The tensile capacity of the splice is more than 60% of the tensile capacity of the UHPC pile.

- The critical strains measured in shear studs and the embedded reinforcement during the direct tension testing were less than 30% of their yield capacity. Based on the measured strain demands, the length of the corner angles can be reduced to 15 in. (380 mm) instead of 24 in. (610 mm), reducing the cost of the splice.

- The measured crack widths and gap opening between UHPC pile and splice embedment under direct tension loading at 225 kip (1000 kN) were less than 0.012 in. (0.3 mm). This observation supports that the splice has enough axial stiffness and a UHPC spliced pile is not expected to experience any damage during driving in the field.

- The UHPC pile splice failed prematurely under flexural loading, around 70% of the UHPC pile moment capacity, due to a poor-quality weld between the corner angles and the end plates in the pile splice. Weld quality should be monitored to ensure that the splice performs as quantified in the tests. Also, a full-penetration weld is recommended between the corner angles and the steel plate.

- A simple moment-curvature analysis based on first principles and measured material properties confirmed that the response of the pile can be accurately captured; however, the simplified analysis did not capture the splice-region moment-curvature response with sufficient accuracy and a more detailed approach would be required. Because the spliced region is relatively short and stiff, it can be assumed to remain elastic with appropriate quality assurance. Consequently, the entire pile may be modeled accurately using its moment-curvature response.

- Based on the experimental evaluations reported herein, the splice investigated can be used for the UHPC pile with modifications to the length and weld requirements. When the pile dimensions are increased, the splice and connections should be appropriately modified.

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Notation

d_b = diameter of reinforcing bars

s = moment-to-shear ratio

Δ_{SPM} = deflection under the load location

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Abstract

Ultra-high-performance concrete (UHPC) has enhanced compressive and postcracking tensile strengths and durability properties. A tapered 10 in. (254 mm) deep, H-shaped prestressed UHPC pile was developed as an alternative to conventional steel HP piles with several benefits. To facilitate field implementation of the UHPC piles, a dry splice detail with steel angles and shear studs was developed with a focus on constructibility. Its performance under flexural, shear, and direct tension loading was evaluated through large-scale testing of full-scale spliced UHPC pile units. Flexural tests were conducted on spliced UHPC piles in weak- and strong-axis directions using a four-point loading configuration. A total of three shear tests were conducted using three-point loading, subjecting the spliced region to different shear demands. The flexural and shear tests confirmed the adequacy of the pile-splice capacity under shear and flexural loading. A direct tension test was also conducted on a spliced UHPC pile by subjecting it to 225 kip (1000 kN) of tension. This resulted in a few microcracks in the test pile with no damage observed in the splice region. A simplified analytical method based on first principles accurately captured the observed performance of the test piles.

Keywords

Full-scale testing, H pile, pile foundation, pile splice, UHPC, ultra-high-performance concrete.

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