Field Investigation of Ultra-High Performance Concrete Piles

Ng, K. W.[1], Garder, J.[2], and Sritharan, S.[3]
[1]Assistant Professor, Dept. of Civil and Architectural Engineering, University of Wyoming, Laramie, WY, 82071. Email: kng1@uwyo.edu
[2]Assistant Structural Engineer, Burns & McDonnell, Kansas City, MO, 64114. Email: jagarder@burnsmcd.com
[3]Wilson Engineering Professor, Dept. of Civil, Construction and Environmental Engineering, Iowa State University, Ames, IA 50011. Email: sri@iastate.edu

Abstract: In response to the recent calls for increasing service life of bridges and optimizing structural systems by the American Association of State Highway Transportation Officials (AASHTO), a field investigation was conducted in Iowa to evaluate the application of Ultra-High Performance Concrete (UHPC) on two test piles. Comparing other pile materials, UHPC has excellent durability characteristics and very high compressive and tensile strengths. The UHPC piles were designed with dimensions and weight similar to that of a reference steel HP 250×85 pile. The first test pile (P3) was 13.7 m (45 ft) long while the second test pile (P4) consisted of two 4.6 m (15 ft) sections welded end to end at a structural steel splice. Both piles were driven into a Wisconsin glacial till. The surface was characterized using a Standard Penetration Test and a Piezocone Penetrometer Test. Driveability analysis was conducted using Wave Equation Analysis Program (WEAP). The responses of both piles during installation and subsequent five restrikes were monitored using a Pile Driving Analyzer (PDA) with subsequent signal matching analyses using CAse Pile Wave Analysis Program (CAPWAP). Immediately after all restrikes, a vertical static load test was performed on P3. Both driveability analysis and PDA confirmed that the maximum pile stresses were well below the allowable driving limits. No visible structural damages to both piles were observed during driving and restrikes. The analytical results reveal that the axial resistances of P3 and P4 increased by about 98% and 70%, respectively, in 6 days after the end of driving. Comparing the axial resistance determined from the static load test based on the Davisson failure criterion, the axial resistance of P3 was about 3% over-estimated using PDA and 6% under-estimated using CAPWAP. The field test verifies the performance of UHPC piles and facilitates the application of UHPC on production piles.

Keywords: UHPC, Pile, Load test, WEAP, PDA, CAPWAP.

1. Introduction

Grand challenges issued by the AASHTO Highway subcommittee on bridge and structures (2005) revealed that a quarter of our nation's 590,000 bridges, including their substructures and foundations, were classified as structural deficient or functionally obsolete, primarily due to material deterioration. To overcome these challenges, innovative methods, such as the use of an advanced Ultra High Performance Concrete (UHPC) material that has been applied to bridge superstructures (e.g., Wipf et al. 2009; Perry and Seibert 2011; Ozyildirim 2011), are being investigated to extend the service life of a structure. Since the UHPC has better durability properties than those of a conventional concrete, structures using UHPC are expected to have a longer service life and require less maintenance (Russell and Graybeal 2013).

Many existing and older bridges were supported by pile foundation systems made of timber, steel and concrete. Each pile type has its advantages and limitations. Timber piles are susceptible to damage and decay when they are installed above the water table and are subjecting to alternate wetting and drying cycle while its durability is a function of site-specific conditions. Timber pile splices are difficult to install and generally avoided. However, timber piles are recommended for the construction of bridge fender systems due to the good energy absorption properties of wood (Hannigan et al. 2006).

Although steel piles are commonly used in the US (AbdelSalam et al. 2010), they are vulnerable to corrosion (White et al. 2007), local bucking under harsh driving conditions (Huck and Hull 1971), and the tendency to deviate from the designed location when obstructions are encountered (Hannigan et al. 2006). However, steel H-piles can easily be extended or reduced in length, has strong splices to resist compression and bending, and are effective when driven into soft rock or dense materials (Hannigan et al. 2006).

Precast/prestressed concrete piles have relatively high breakage rate, especially when they are to be spliced (Hannigan et al. 2006). Furthermore, they are susceptible to cracking as a result of large compressive and tensile stresses developed during driving (Salgado 2006). However, concrete piles are usually resistant to corrosion and exhibit high load capacity (Hannigan et al. 2006). When the limitations of these conventional pile foundations are facilitated by the site-specific condition and the average age of these foundations approach their service life, maintaining and replacing bridge substructures becomes a challenging task.

To minimize drivability challenges, extend a target service life, and possibly reduce maintenance costs, piles made of UHPC material can be considered as an alternative to the conventional piles. The foundation system can be optimized by utilizing the advantages of UHPC, such as 1) excellent durability characteristics as a result of small capillary porosity; and 2) very high compressive (180 to 207 MPa or 26 to 30 ksi) and tensile (12 MPa or 1.7 ksi) strengths (Vande Voort et al. 2008). Recognizing the benefits of UHPC, the first UHPC pile research project (Phase I) was conducted in Iowa to understand the behavior of two 10.7 m-long UHPC piles (i.e., UHPC-1 and UHPC-2), driven in loess on top of a hard glacial till clay soil and subjected to both vertical and lateral load tests. The UHPC piles were designed with dimensions and weight similar to that of a referenced steel HP 250×85 pile (see Figure 1). The UHPC pile section was reinforced with ten 13 mm (0.5 in) diameter prestressing strands with no shear reinforcement. The concrete cover was reduced from 32 mm (1.25 in) to 19 mm (0.75 in) due to the high strength and durability of UHPC.

Recognizing the benefits of UHPC piles and the positive outcomes of Phase I research, additional research on the UHPC pile (Phase II that is discussed in this paper) was untaken to further verify the performance of UHPC piles, and facilitate the implementation of UHPC pile foundations in future bridges. Among the many objectives of the Phase II research project, this paper focuses on 1) the production of two UHPC piles and a newly designed pile splice; 2) driveability analysis of UHPC piles with a full H-shape section; 3) the performance of the pile splice connection under a lateral load test; and 4) testing the UHPC piles to failure in field.

2. Test Site and Subsurface Description

The Sac County Bridge Project was selected for testing two UHPC piles (P3 and P4) because of the bridge's geometry, soil conditions and construction timeline. The bridge project was just north of Early, Iowa, at the intersection of U.S. 20 over U.S. 71. The prototype bridge is 68 m (197 ft) long and 12 m (39 ft) wide with a 24 degree skew. The bridge consists of three spans and the span lengths are 17 m (56 ft), 32.5 m (107 ft), and 18.5 m (61 ft) from west to east. HP 250×58 steel piles were designed to support the two abutments and the two bridge piers. Figure 2 illustrates the approximate locations of the UHPC test piles with respect to each other.

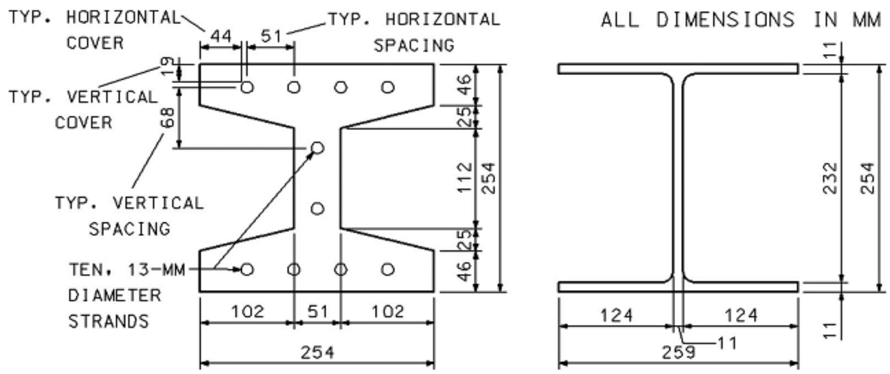


Figure 1. Cross-sectional details of a UHPC pile compared with a steel HP 250×85 pile (adopted from Suleiman et al. 2010)

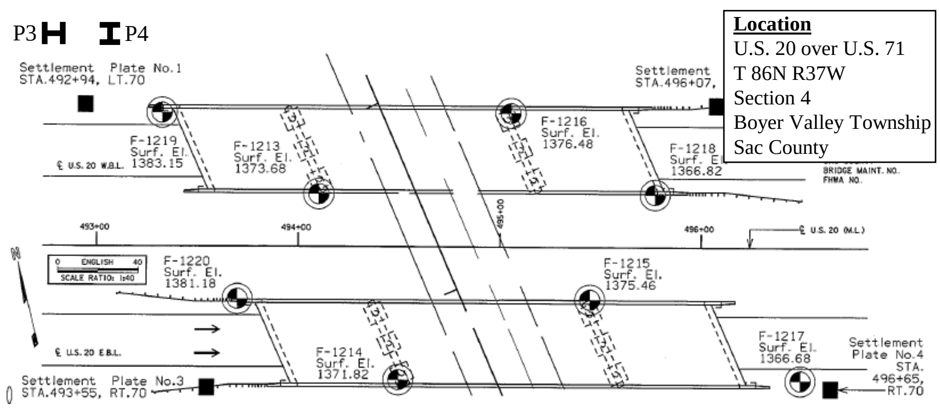


Figure 2. Location of two UHPC piles for field Testing at Sac County, Iowa

The subsurface was characterized using a Standard Penetration Test (SPT) and a Piezocone Penetration Test (CPT_u), which was terminated at 16.75 m (55 ft). The soil is primarily a Wisconsin glacial till and its profile is shown in Figure 3(a). The ground water table was encountered at approximately 2.44 m (8 ft) deep. Figure 3(a) shows the gradual increase in uncorrected SPT N-value from 8 to 15 blows per 300 mm (12 in). The tip resistances (q_t) and skin frictions (f_s) were described in Figure 3(b) and Figure 3(c), respectively.

3. Field Testing

Based on the vertical load test previously completed by Vande Voort et al. (2008) in Phase I, the ultimate capacity of the UHPC pile was found 86% higher than that of the HP 250×85 pile with the same length due to the increased toe area and perimeter. As a result, a shorter UHPC test pile P3 to achieve a target vertical load capacity was designed and manufactured. The second UHPC test pile (P4) was designed and installed with a structural steel splice. Both UHPC piles were instrumented, and their installations were monitored using a Pile Driving Analyzer (PDA). Restrike tests were performed at the end of driving on both UHPC piles as well as on two anchor steel H piles installed for use in a subsequent static vertical load test. Finally, a lateral load test was performed on the UHPC piles to verify the laboratory performance of the proposed splicing detail (Sritharan et al. 2012) and to characterize the weak-axis bending of the UHPC pile.

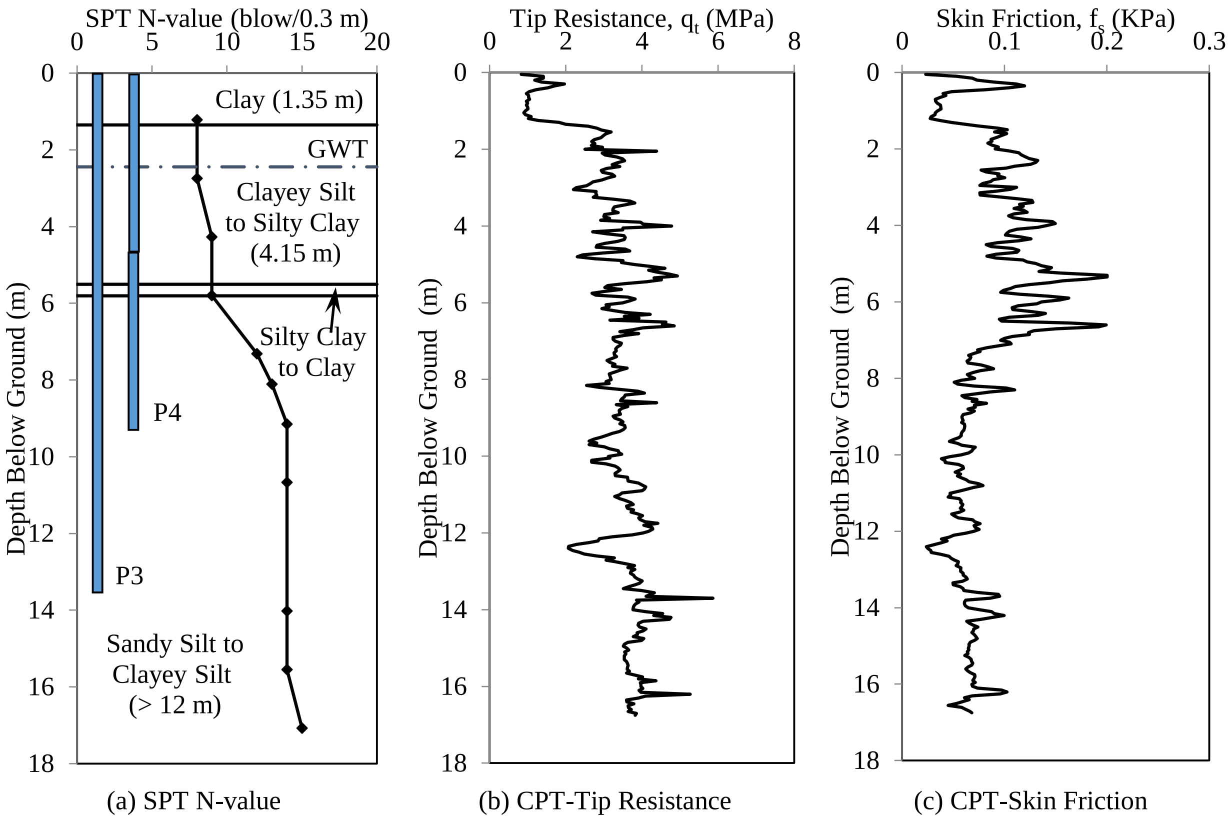


Figure 3. SPT and CPT data at the test pile location

3.1 Production, instrumentation and handling of UHPC piles

The two UHPC piles were cast at Coreslab Structures in Omaha, NE. Test pile P3 was 13.7 m (45 ft) long with a full H-shape section. Test pile P4 consists of two 4.6 m (15 ft) UHPC sections welded end to end at a structural steel splice as shown in Figure 4. Embedded concrete strain gages were suspended between two prestressing strands that were stressed to 1,396 MPa (75% of their ultimate strength; 202 ksi). Gages were placed on a diagonal at each level of instrumentation, as shown in Figure 5(a), to measure the curvature of the piles during the lateral load test. Ten pairs of gages were installed along P3. Only three pairs of gages were installed along the second 4.6 m (15 ft) UHPC section (i.e., above the pile splice) of P4.

UHPC was mixed and poured into the completed forms. The top surface of the test piles were covered with plastic wraps to prevent moisture loss. Propane heaters were used for the initial curing at 30˚C. Six 76 mm (3 in) diameter UHPC cylinders were cast to determine the target compressive strength of 97 MPa (14 ksi) before the prestressing strands were cut and released. The test piles were steam cured at 90ºC (194°F) for 48 hours. To help with handling the UHPC piles, lifting hooks made of #10 rebar were installed 460 mm (18 in) away from both pile head and pile toe before casting. The average compressive strength of the UHPC at 28 days was 183 MPa (26.5 ksi), which satisfied a typical design compressive strength of 179 MPa (26 ksi).

3.2 Installation of UHPC piles

Two steel HP 310×79 anchor piles (RPS and RPN) were driven first on December 6 to 7, 2011 using a Delmag D16-32 diesel hammer, followed by driving P3 and then P4 on December 8, 2011 at the location indicated in Figure 6. The installation process of UHPC piles was similar to that of the anchor piles. The field test was arranged to compare the UHPC piles with the steel H-piles rather than a normal strength concrete pile because steel H-pile is the most commonly used pile type in Iowa as well as the United States (AbdelSalam et al. 2010). All piles were monitored using the PDA, and the measured pile responses were used in a subsequent analysis using the CAPWAP to estimate the pile resistance and its distribution.

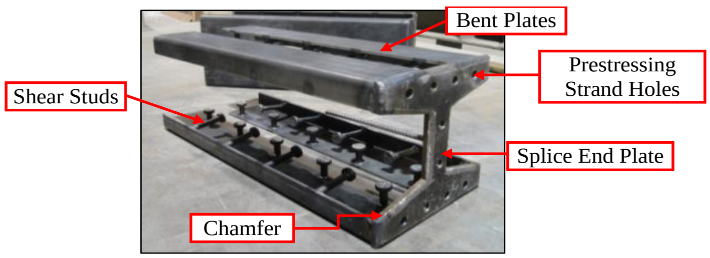
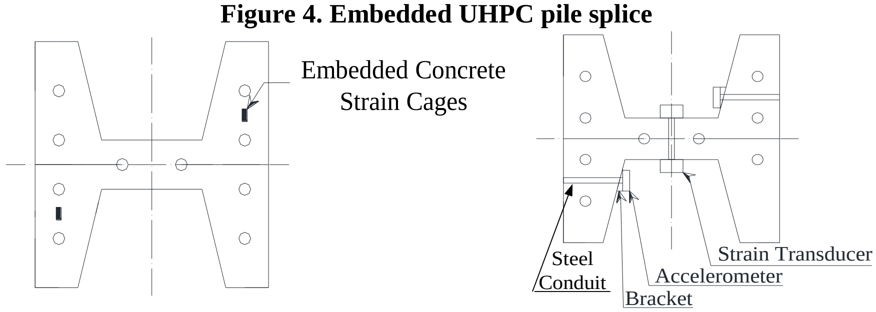


Figure 4. Embedded UHPC pile splice



(a) Location of gages (b) Location of steel conduits

Figure 5. Location of embedded concrete strain gages and steel conduits for PDA testing

3.3 Driveability Analysis

Prior to pile installation, a driveability analysis was conducted using Wave Equation Analysis Program (WEAP) to estimate the maximum stresses during driving for the UHPC and steel anchor piles as summarized in Table 1. PDA was used to monitor pile responses during driving, and the measured maximum stresses were summarized. Table 1 shows that the predicted and the measured maximum stresses of all piles are well below the allowable driving limits. No visible structural damage to all piles was observed after driving. Notably, the driveability analysis concludes that the UHPC test piles performed extremely well during driving.

3.4 Dynamic restrike testing

Five restrikes were performed on P3 and P4 at approximately 8 minutes, 20 minutes, 1 hour, 4 days and 6 days after the end of driving (EOD). Six restrikes were performed on the anchor piles at approximately 8 minutes, 20 minutes, 1 hour, 1 day, 5 days, and 7 days after the EOD. The objective of performing a series of dynamic restrike tests is to evaluate the increase in axial pile capacity known as pile setup. The results of the dynamic restrike tests on both UHPC piles and anchor piles are presented as a percent increase in the pile resistance with respect to the resistance estimated by CAPWAP at the EOD in Figure 7. All four piles experienced pile setup with pile resistances increased logarithmically as a function of time immediately after the EOD. The slope of the best fit line describes the rate of pile setup (i.e., the rate of increase in pile resistance), and P3 experienced the highest rate of pile setup. Also, P3 experienced the highest pile setup with 98% increase in the pile resistance estimated by CAPWAP and 110% measured by the static load test described in next section. P4 experienced about 70% pile setup. Although the embedded lengths of P3 (12.8 m or 42 ft) and P4 (8.2 m or 27 ft) were shorter than the anchor piles (22.3 m or 73 ft), the pile setup rate of the UHPC piles were higher. Also, the percent increase in pile resistance of P3 was higher than both anchor piles. This observation was attributed to a larger cross-sectional area of 364.5 cm^2 (56.5 in^2) of the UHPC pile as compared with 100 cm^2 (15.5 in^2) of the anchor pile. A larger cross-sectional area exerted a greater disturbance to the surrounding soil during the pile installation and caused a larger pile setup.

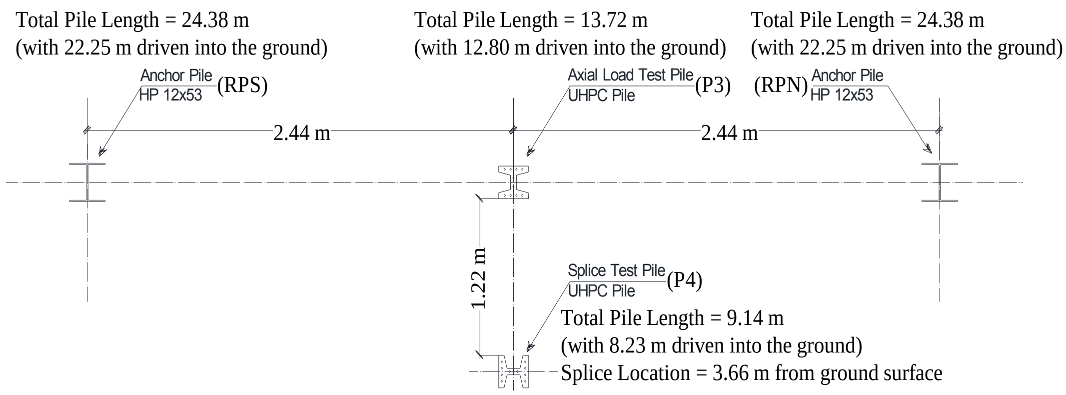


Figure 6. Location of test piles P3 and P4 as well as steel anchor piles RPS and RPN

Table 1. Maximum stresses during driving of the UHPC and steel anchor piles

Pile	Stress	Predicted stresses using WEAP (MPa)	Measured stresses using PDA (MPa/ksi)	Allowable driving limits (MPa/ski)
RPS	Compressive	203	197	310[a]
	Tensile	12	8	310[a]
RPN	Compressive	203	212	310[a]
	Tensile	12	12	310[a]
P3	Compressive	50	37	122[b]
	Tensile	0.7	1.4	37[c]
P4	Compressive	41	39	122[b]
	Tensile	0.0	0.7	37[c]

[a] = $0.9f_y$; [b] = $0.85f'_c - f_{pe}$; [c] = 6.9 MPa + f_{pe}; where f_y = yield strength of steel (345 MPa), f'_c = compressive strength of UHPC; and f_{pe} = effective prestressing after losses.

3.5 Vertical load testing

The vertical load test was completed on P3 following "Procedure A: Quick Test" as outlined in the ASTM D 1143 (2007). The ultimate pile capacity was determined to be 1,321 kN (297 kips) based on the Davisson failure criterion (1972). Accordingly, the total side resistance and end bearing were determined to be 1,234 kN (277 kips) and 87 kN (20 kips), respectively. Table 2 shows that the total pile capacity estimated using the Iowa Blue Book was underestimated by 33%, while PDA and CAPWAP, based on the last dynamic restrike test, provide a relatively good pile capacity estimation. Similar pile capacity estimations were performed on the steel anchor pile RPS. Although higher total capacities were anticipated for RPS, which has a longer embedded pile length of 22.25 m (73 ft), UHPC test pile P3 has higher total pile capacity per unit length, ranging by 31% to 38%. This comparison further suggests that the application of UHPC piles will reduce the total pile length required for a foundation system.

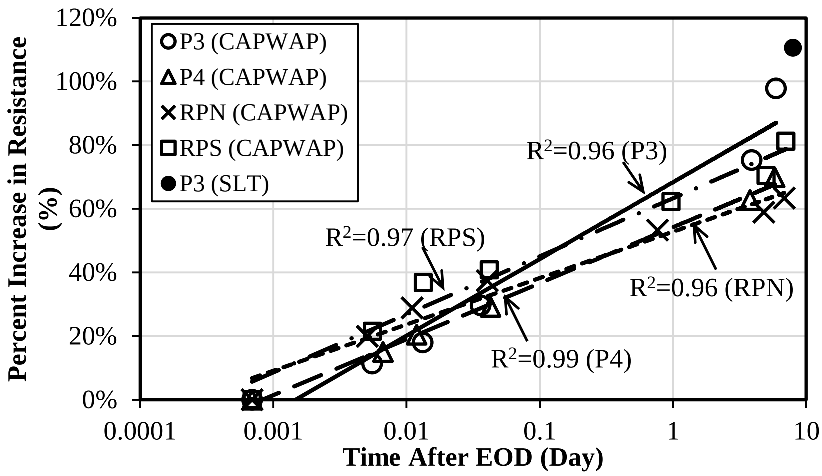


Figure 7. Percent increase in pile resistance as a function of time after the EOD

Table 2. Comparison of pile capacities of UHPC pile P3 and steel anchor pile RPS

Method	UHPC P3 (12.8 m into ground)				RPS (22.3 m into ground)			
	Side resistance (kN)	End bearing (kN)	Total capacity (kN)	Total capacity per meter (kN/m)	Side resistance (kN)	End bearing (kN)	Total capacity (kN)	Total capacity per meter (kN/m)
Iowa Blue Book	704	186	890	70	997	167	1164	52
PDA (LS)	1361	0	1361	106	1032	672	1704	77
CAPWAP (LS)	1111	128	1239	97	1411	232	1643	74
Static load test	1234	87	1321	103	-	-	-	-

LS = last restrike.

3.6 Lateral load testing

Three days after completing the vertical load test, a lateral load test was performed on P3 with a strong-axis bending as well as on P4 with a weak-axis bending and a splice at 4.57 m (15 ft) from pile head. The lateral load test was performed after the vertical load test to minimize the effect of test sequence on the lateral performance of the UHPC piles. The completed lateral load test setup is shown in Figure 8. The lateral load was applied using a 445 kN (100 kips) actuator placed approximately 800 mm (31.5 in) above ground. Along the line of the lateral load, two 254 mm (10 in) stroke displacement gages, mounted to independent, wooden reference beams behind each test pile, were used to measure the lateral displacement of each pile. The lateral load test was performed following "Procedure A: Standard Loading" of ASTM 3966 (2007). For the first load cycle, both piles were loaded up to 200% of the proposed lateral design load of 45 kN (10 kips) unless failure occurs first. For the remaining three cycles, the piles were displacement controlled based off the measurements taken from P4 at 100 mm (4 in), 178 mm (7 in), and 254 mm (10 in). Between each cycle the UHPC test piles were unloaded to 0 kN.

Figure 9 shows that P4 with a greatly reduced lateral stiffness displaced about five times (211 mm or 8 in) more than that of P3 (43 mm or 1.7 in) at the maximum lateral force of 92 kN (21 kips) during the 1st cycle. The lateral force-displacement curves for the remaining cycles of

P3 shown in Figure 9(a) are within the force-displacement loop of the 1st cycle, and the final residual displacement was significantly small. In contrast, P4 had a maximum displacement of 254 mm (10 in) and exhibited a relatively large final residual displacement of 60 mm (2.4 in), which was confirmed by a noticeable heaving of the soil on one side of P4 during the test. The increase in stiffness at about 200 mm (8 in), especially observed in P4 shown in Figure 9(b), was attributed to the continuous densification of the surrounding top soil layer during the three cycles of lateral loading and the contribution of soil stiffness to the pile system when P4 was pushed through the void distance of about 200 mm (8 in) created from previous load tests before exerting against the soil as illustrated in Figure 8. Due to space limitation, the numerical analysis performed using LPILE and result discussions were not included in this paper. However, they were explicitly described by Ng et al. (2015).

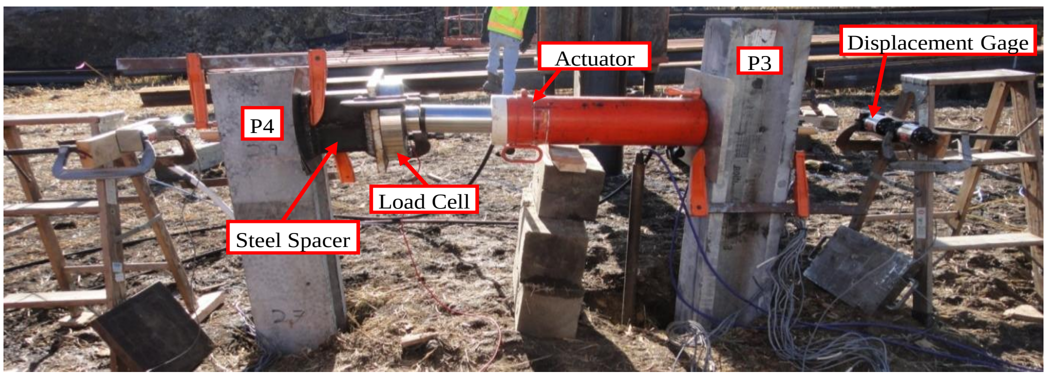
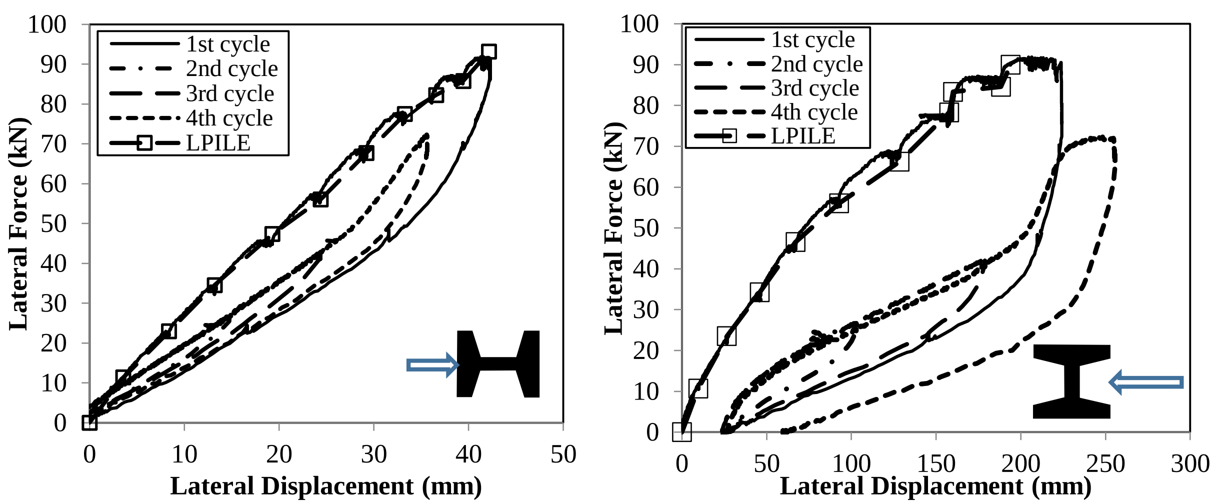


Figure 8. Completed lateral load test setup



(a) P3: strong-axis bending (b) P4: weak-axis bending

Figure 9. Lateral force-displacement responses measured during lateral load test and estimated using LPILE

3.7 Splice performance

The structural splice on P4, which was located 4.57 m (15 ft) from the pile head and driven to 3.66 m (12 ft) below the ground surface, performed well during installation. The maximum compressive stress of 39 MPa (5.7 ksi) and the maximum tensile stress of 0.7 MPa (0.1 ksi) are significantly smaller than the allowable driving limits of 122 MPa (18 ksi) and 37 MPa (5.4 ksi), respectively, as indicated in Table 1. Also, no damage was detected by the PDA during driving and restrike tests as the integrity factors (BTA) that describe the degree of convergence of measured pile force and velocity records were 100%. During the lateral load test, the splice was subjected to 5.92 kN-m (0.4 kip-ft) bending moment and a shear force of 11.6 kN (2.6 kips) (Ng et al. 2015). The splice proved to be very robust with a reserve shear capacity of 200 kN (45 kips), which exceeds the maximum shear demand from the lateral load field test of 91.6 kN (21 kips) by 218 percent (Sritharan et al. 2012). Careful visual inspection was conducted by all authors on site to identify cracks and fractures on both the splice and UHPC in the vicinity of the splice. No damage was observed on or near the splice. Unfortunately, non-destructive methods were not available to detect any micro-scale crack on and near the splice.

4. Conclusions

The research further verifies the performance of UHPC piles for integral abutment bridges. The application of UHPC will increase the service life of bridges and optimize foundation systems. The major conclusions of this research are summarized as follows:

1) Driveability analysis performed using WEAP concluded that the measured maximum stresses of UHPC piles were well below the allowable driving limits. The UHPC test piles performed extremely well during driving.
2) The static load test results concluded that the UHPC test pile has higher total pile capacity per unit length than that of a steel H-pile. This conclusion suggests that the application of UHPC piles could enhance the efficiency of the foundation construction by reducing the total pile length of the foundation system or reducing the number of piles needed in a pile group.
3) The lateral load test results confirmed that P4 with a greatly reduced lateral stiffness displaced about five times more than that of P3. The lateral force-displacement curves for the remaining cycles of P3 are within the force-displacement loop of the 1st cycle, and the final residual displacement was significantly small. Additionally, P4 exhibited a relatively large final residual displacement than P3.
4) The pile splice performed well during installation and no visible damage was found after driving. The structural performance of the splice exceeded the required shear, moment, and tensile demands.

5. References

Grand Challenges: A Strategic Plan for Bridge Engineering, AASHTO Highway Subcommittee on Bridges and Structures, Washington D.C., 2005.

AbdelSalam S.S., Sritharan S. and Suleiman M.T., “Current Design and Construction Practices of Bridge Pile Foundations with Emphasis on Implementation of LRFD,” *Journal of Bridge Engineering*, Vol. 15, No. 6, 2010, pp. 749-758.

Standard Test Methods for Deep Foundations under Static Axial Compressive Load, American Society for Testing and Materials (ASTM) D1143-07, West Conshohocken, PA, 2007.

Standard Test Methods for Deep Foundations under Lateral Load, American Society for Testing and Materials (ASTM) D3966-07, West Conshohocken, PA, 2007.

Davisson, M.T., “High Capacity Piles,” *Proceedings, Soil Mechanics Lecture Series on Innovations in Foundation Construction,* ASCE, New York, 1972, pp. 81-112.

Hannigan, P.J., Goble, G.G., Thendean, G., Likins, G.E. and Rausche, F., "Design and Construction of Driven Pile Foundations–Volume I," FHWA-NHI-05-042. National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 2006.

Huck, R.W. and Hull, J.R., "Resonant Driving in Permafrost," *Foundation Facts*, Vol. 7, No. 1, 1971, pp. 11-15.

Ng, K.W., Garder, J. and Sritharan, S., "Field Investigation of Ultra High Performance Concrete Piles for Bridges," *Engineering Structures*, Elsevier, Vol. 105, December, 2015, pp. 220-230.

Ozyildirim, C., "Evaluation of Ultra High Performance Fiber Reinforced Concrete," Final Report, Virginia Center for Transportation Innovation & Research. Charlottesville, VA, 2011.

Perry, V.H. and Seibert, P.J., "Working with Ductal Ultra-High Performance Concrete," Concrete Plant International, North American Edition, February, 2011.

Russell, H.G. and Graybeal, B.A., "Ultra-High Performance Concrete: A State-of-the-Art Report for the Bridge Community," Report No. FHWA-HRT-13-060, Federal Highway Administration, Washington, D.C., 2013.

Salgado, R. The Engineering of Foundations. McGraw-Hill, 2006.

Suleiman, M.T., Vande Voort, T. and Sritharan, S., "Behavior of Driven Ultrahigh-Performance Concrete H-Piles Subjected to Vertical and Lateral Loadings," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 136, No. 10, 2010, pp. 1403-1413.

Sritharan, S., Aaleti, S., Bierwagen, D., Garder, J. and Abu-Hawash, A., "Current Research on Ultra High performance Concrete (UHPC) for Bridge Applications in Iowa," *Proceedings of the 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Kessel, Germany, March 7-9, 2012, pp. 857-864.

Vande Voort, T.L., Suleiman, M.T. and Sritharan, S., "Design and Performance Verification of UHPC Piles for Deep Foundations," Final Report. Center for Transportation Research and Education, Iowa State University, Ames, IA, 2008.

White, D.J., Mekkawy, M.M., Sritharan, S. and Suleiman, M.T., "Underlying Causes for Settlement of Bridge Approach Pavement Systems," *Journal of Performance of Constructed Facilities*, ASCE, Vol. 21, No. 4, 2007, pp. 273-282.

Wipf, T.J., Phares, B.M., Sritharan, S., Degen, B.E. and Giesmann, M.T., "Design and Evaluation of Single-Span Bridge Using Ultra High Performance Concrete," Final Report. Center for Transportation Research and Education, Iowa State University, Ames, IA, 2009.

6. Acknowledgements

This research project was sponsored by the Iowa Highway Research Board. The authors would like to thank the technical advisory committee of the research project. Special thanks are due to Lafarge North America, Coreslab Structures, Inc. of Omaha, Howe's Welding of Ames, Iowa State Ready Mix Concrete, and Graves Construction Co, Inc. of Spencer, IA.