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N.P. López-Acosta et al. (Eds.)

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Technical Session #6 "Foundations and Retaining Structures"

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Laterally Loaded Pile Behavior at Soil-Rock-Impedance Contrast

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Abstract. At zones of strong impedance contrast in which there is a significant change in stiffness between adjacent geomaterial layers, Winkler-based analysis methods predict abrupt changes in the internal pile reaction force effects for laterally-loaded foundation elements. In particular, the sudden de-amplification of moment when transitioning from a soft to stiff layer is accompanied by amplification of pile shear. From a design perspective, this is problematic when considering large lateral loads and moments acting on drilled shafts, because it can result in bulky transverse reinforcement designs that pose constructability challenges. This paper will review the challenges associated with the lateral performance of piles in zones with strong stiffness contrasts and present a large-scale experimental research program that investigates the lateral load transfer of rock-socketed deep foundation elements. The study seeks to better understand the ability of numerical and analytical methodologies in capturing the behavior at impedance interfaces, compare such with experimental observations, and derive lessons for the construction industry in how to optimize the design requirement using performance-based predictions for deep foundations embedded in stiff materials.

Keywords. Pile foundations, lateral loading, impedance contrast, rock sockets

1. Introduction

Pile foundations are extensively used in the construction of various types of superstructures, including tall buildings, bridges, freeways and offshore structures. Hereby, a fully constrained tip embedment such as a rock-socket offers an attractive solution for achieving maximal tip resistance and improving the load transfer capabilities of the foundation element. In soil profiles with very soft surface soils, rock-socketing often provides the only reliable source of axial and lateral resistance.

Previous research predominantly focused on geotechnical response aspects of rock-socketed piles without considering complex interaction effects inherent to the pile curvature integration and differentiation procedures and their effect on the pile's structural response behavior. Impedance contrasts between strong rock layers and softer surface soils yield abrupt changes in the pile's moment profiles which translate into amplified shear forces at the rock-socket interface, originating from differentiation of the fourth order differential beam equation. The validation of potential shear amplification (or the lack thereof) with field load tests and verified advanced numerical analysis is very

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limited in existing literature. However, the correct evaluation of shear demands at soil-rock socket interfaces is vital since the shear demand may govern the drilled shaft's structural design and the overall constructability of the foundation system. This amplification behavior is not unique to rock-soil interfaces but accompanies most deep foundation elements when interfaces separating soil layers have substantially different stiffnesses, or when head or tip restraints control the lateral pile bending behavior. It is particularly pronounced when predicting a pile's lateral load behavior with the $p-y$ method of analysis and inherent to the mathematical process by which internal pile reactions, such as shear forces are derived.

2. Experimental Studies

To better understand the lateral load behavior of deep foundations embedded in stratigraphies with strong impedance boundaries, an experimental research program consisting of three reinforced concrete pile foundations was developed. Testing was executed at the Structural Engineering Testing Hall at the University of California, Irvine. The test specimens were embedded in a two-layer soil system and subjected to reverse cyclic lateral loading until complete structural failure was reached. This paper will present the iteration of the specimen geometry, the test setup, specimen configuration, material properties, test results, failure observations, and an abbreviated comparison between analytically predicted and experimentally observed specimen behavior for two of the pile foundations.

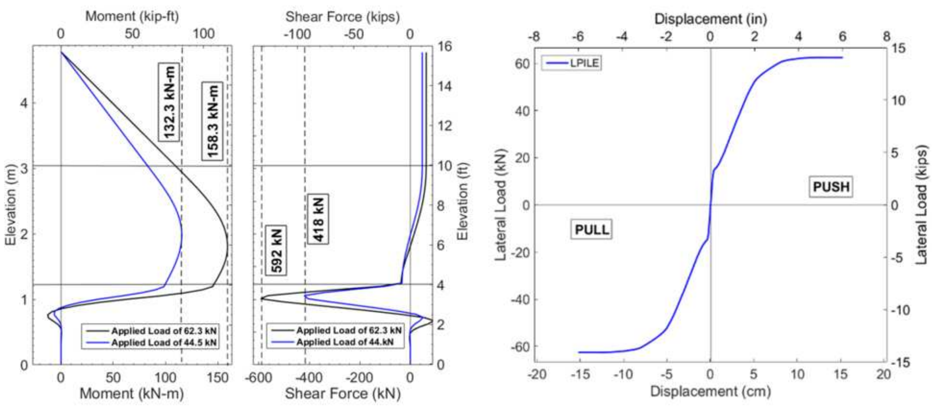
2.1. Specimen Selection and Analytical Pre-test Predictions

The *p-y* method of analysis, as implemented in the software platform LPile [1], was used to predict the general foundation response of two pile specimens due to cyclic lateral loading at the pile head. The analytical results enabled the iteration of suitable specimen geometries and reinforcement configurations that would serve two objectives: (1) to gain general insight into the piles' nonlinear performance behavior and failure development under small and large lateral deformations, and (2) to learn whether the potential amplification of shear forces at the impedance boundary is experimentally verifiable. The soil layers were analytically replicated using *p-y* springs for weak rock [1] as well as sand [4] which are readily embedded in the LPile software. The soil properties were initially estimated based on supplier data sheets for typical sand-type fill materials. Concrete strengths were taken as design strengths with an overstrength factor. One pile specimen, hereafter referred to as Specimen 1, was designed to satisfy all predicted internal pile reactions (moment, shear) and reinforced extensively to provide sufficient capacity to accommodate all amplified internal pile reactions, while another specimen (Specimen 2) was designed to only satisfy the code requirements associated with the applied lateral load. If the amplification of shear demands at the interface between soft and stiff soil layers is real, this specimen would be expected to fail due to insufficient shear capacity at the respective boundary. Table 1 presents the structural design summary of Specimens 1 and 2. In accordance with typical reinforcement ratios used in U.S. foundation deep foundation design, the longitudinal reinforcement ratio was chosen as $\rho = 1.41$ %.

Figure 1 shows the predicted load-deflection relationship of both pile specimens (Figure 1a) and suggests a lateral load of 13.34 kN [3kips] at the point of concrete cracking, a load of 40.92 kN [9.2kips] at specimen yield and a load of 62.27 kN [14kips] when reaching the ultimate pile capacity. Moment and shear profiles are predicted in Figures 1(b) and (c), respectively. Figure 1(c) illustrates the shear amplification prediction explained above. For an applied lateral pile head load of 62.27 kN [14 kips], the Winkler-type analysis predicts an internal pile shear magnification of 591.61 kN [133 kips], which corresponds to 9.5 times of the applied head load. This strong analytical amplification ratio was chosen to exaggerate the pile reaction effects and obtain maximum pile reactions experimentally. The soil was selected in a manner to support this amplification effect (i.e., a relatively loose sand material across a limited specimen height overlaying a stiffer rock layer, as described hereafter).

Table 1. Predictions of pile capacities using LPile [1] and selected reinforcement.

	Specimen 1	Specimen 2
Longitudinal reinforcement	8#6	8#6
Transverse reinforcement (Inside rock socket)	#4@11.43 cm [4.5 in]	#4@15.24 cm [6.0 in]
Transverse reinforcement (Outside rock socket)	#4@15.24 cm [6.0 in]	#4@15.24 cm [6.0 in]
Nominal shear strength, V_n, kN [kips]	618.30 [139]	395.89 [89]
Nominal flexural strength, M_n, kN-m [kip-ft]	157.27 [116]	157.27 [116]
Predicted failure mechanism / limit state	Flexure	Amplified shear
Predicted lateral head load @ failure	62.3 kN [14 kips]	44.5 kN [10 kips]



(a) (b) (c)

Figure 1. Prediction of pile head load-displacement relationships (a), pile moment (b) and pile shear forces (c) due to lateral loads of 44.5 kN (10kips) and 62.3 kN (14 kips) at the pile head.

2.2. Specimen Geometry and Reinforcement

The two reinforced concrete piles were 4.57 m [15.0 ft] in length and 0.46 m [18.0 in] in diameter. As shown schematically in Figure 2a, the piles were embedded in 1.20 m [4.0 ft] of "rock", simulated experimentally through high strength concrete (f'_c = 48.3 MPa [7 ksi]). The concrete blocks (i.e., the "rock sockets") had dimensions of 1.83 m [6.0 ft] in length, 1.22 m [4.0 ft] in width, and 1.22 m [4.0 ft] in height. The blocks were secured to the reinforced concrete floor of the testing facility using pre-drilled, epoxy grouted, high strength steel anchors. The piles extended a total of 3.35 m [11.0 ft] outside the rock. A pile cap with cross-sectional dimensions of 0.61 m by 0.61 m [24 x 24 in] and a height of 0.41m [16 in] was constructed at the pile head and used for lateral load application. The two pile specimens had different reinforcement configurations as explained in section 2.1 above and shown in Figure 2(b) and (c) below. The longitudinal reinforcement of both specimens consisted of eight number 6 bars ($A_{s,\,total}$ = 22.71 cm^2 [3.52 in^2]), Grade 60 A 706 steel, aligned in equal spacings around the circumference of the pile. The longitudinal reinforcement ratio was 1.41 %. The clear concrete cover was 5.08 cm [2.0 in]. The transverse reinforcement consisted of Number 4 spirals. Specimen 1's transverse reinforcement was spaced at a pitch of 11.43 cm [4.5 in] in the zone of the rock socket, i.e., approximately up to 1.2 m [4.0 ft], and at a pitch of 15.24 cm [6.0 in] along the remaining pile height. The closer spiral pitch at the bottom of the pile was satisfies the amplified shear demand within the rock socket as predicted by the *p-y* analysis in Figure 1. Specimen 2 was reinforced with transverse spirals at a pitch of 15.24 cm [6.0 in] along the entire specimen height. The reinforcement scenario of Specimen 2 ignores the amplified shear demand in the impedance zone and satisfies the code requirements for shear and confinement based on the applied lateral load per ACI 318-14 [4] only.

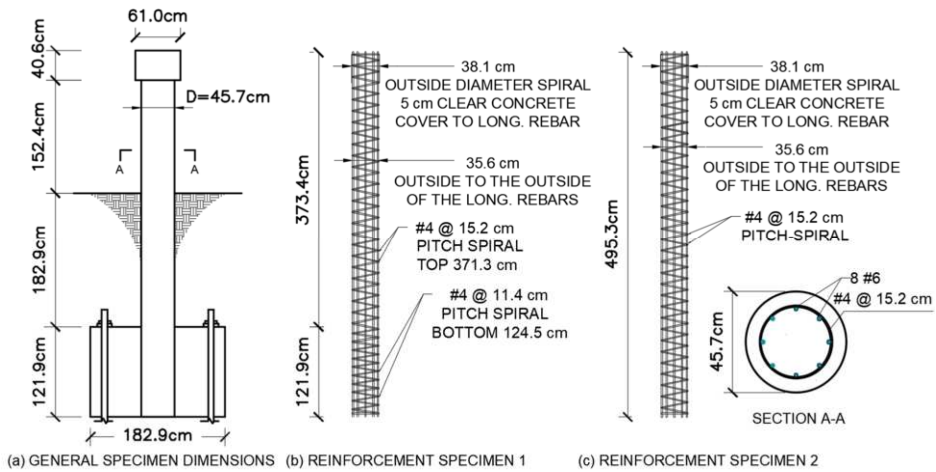


Figure 2. Schematic specimen configuration.

2.3. Specimen Instrumentation

Specimen instrumentation consisted of internal and external sensors, such as inclinometers, linear voltage differential transducers (LVDTs), strain gauges in longitudinal, rosette, and tetrahedral configurations, and string potentiometers. An example schematic instrumentation plan for Specimen 1 is shown in Figure 3.

2.4. Specimen Construction

Figure 4 shows photographs during specimen construction. Upon instrumenting all rebar cages outside the soil pit, the cages were placed inside Sonotubes and aligned along the walls of the soil pit. Concrete was poured and manually vibrated to avoid sensor damage. The pile caps were constructed after the piles had cured for approximately one week. Piles were then placed and secured inside the formwork of the rock sockets prior to casting the rock socket concrete as shown in Figure 4.

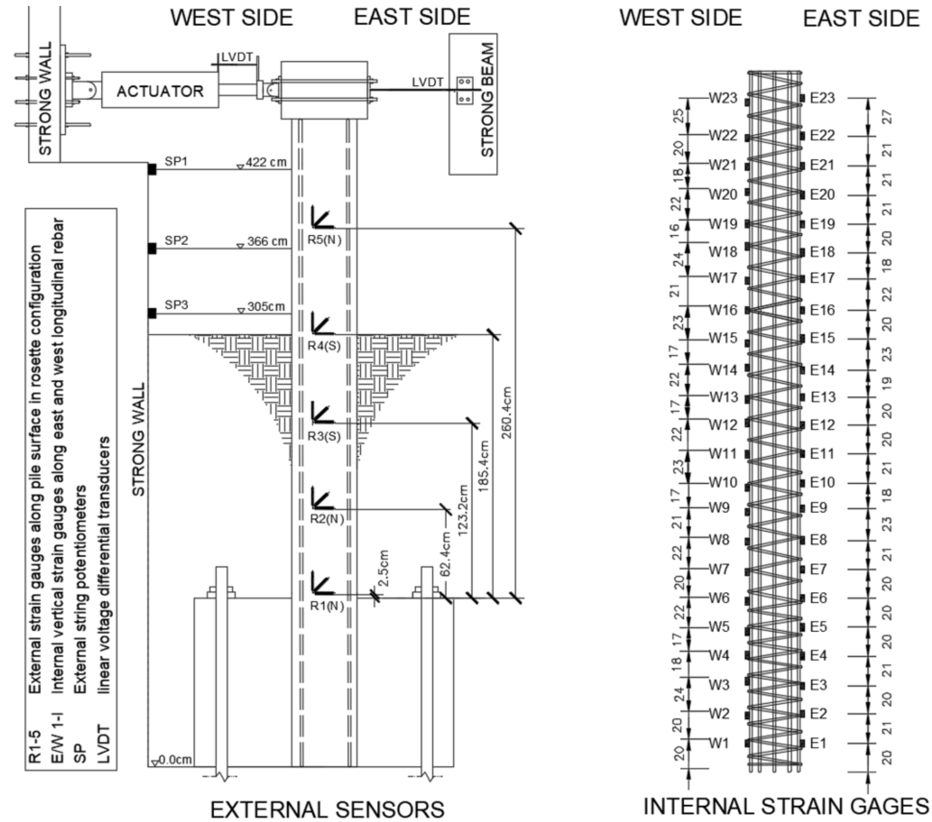


Figure 3. Instrumentation layout for Specimen 1 (tetrahedral sensors not shown, all dimensions in cm).

Along the height of the rock socket (i.e., along the bottom 1.22 m [4ft] of the pile) the concrete surface was roughened with a pneumatic needle scaler to ensure a good concrete to concrete adherence. The rock-socket concrete had a design strength of 34.5 MPa [5ksi], and an average in-situ strength of 48.3 MPa [7ksi] at the day of testing. Pile Specimens 1 and 2 had a design strength of 27.58 N/mm^2 [4ksi], and an in-situ compressive strength of 40.7 MPa [5.9ksi] and 39.3 MPa [5.7ksi] at the day of testing respectively.

The space between the rock-sockets, which is unaffected by the load application at the pile cap was filled with geofoam. The remaining soil pit was then filled with sand pluviated from a minimum height of 3.66 m [12ft] to yield a relative average in-situ density of 20% (Figure 4(e)). Pluviation was accomplished by designing a sieve system to be attached at the bottom of a concrete hopper and calibrating the fall heights as well as the sieve openings until the desired relative density was reached. The relatively low in-situ density of 20% provided a strong impedance contrast between the soil and rock materials. Additional laboratory testing of the soil material revealed a friction angle of 34.6 degrees (direct shear testing), a cohesion of 4.96 kN/m^2 [0.72 psi], and a max dry density of 21 kN/m^3 [134 pcf] (Modified Proctor test). Cone penetration (CPT) and Dilatometer testing (DMT) were used to estimate the soils E-modulus and shear wave velocity; test results are omitted for brevity. The in-situ moisture content of the soil was 6%.

Figure 4. (a) Pile rebar cages, (b) Instrumented specimens prior to concrete pouring, (c) Concrete piles placed in rock-socket formwork, (d) Specimens with rock-sockets anchored into the floor after formwork removal (e) Sand pluviation, (f) Completed test setup for SP 2.

2.5. Load Application

The loading protocol was developed based on the predictive analyses shown in Figure 1 and followed the general guidelines of ASCE 41-06, in which applied displacement levels are selected as fractions or multipliers of the anticipated yield displacement. Quasi-static, reverse-cyclic loading was applied at the pile head by three cycles per displacement level up to ultimate capacity. Hereafter two cycles per displacement level were performed until substantial degradation of the load-displacement relationship was noticeable.

Loading was applied under displacement control at the center of the pile cap using a 76.2 cm [30 in] stroke, 667 kN [150 kips] capacity hydraulic actuator (see Figure 2a). The strong wall of the UCI laboratory served as reaction wall. The actuator was controlled by an MTS 407 dual channel controller and data were recorded using a National Instrument data acquisition system. A total of 115 channels were utilized for each test. An externally installed LVDT, mounted between an independent reference frame and the backside of the pile cap was used to control the experiment and record the pile head displacement.

3. Test Results

3.1. Post-test Failure Observations

To better track the crack patterns around the pile specimen a spray-paint grid was printed on the sand surface as shown in Figure 5. The grid spacing was 15.24 x 15.24 cm [6 x 6 in]. An example of the circumferential crack expansion and the formation of a "crater-type" hole around the pile for displacements beyond 4 in displacement is the depicted in the photographs. Each pile was manually excavated (in push direction) to identify substantial cracking and the location of the plastic hinge (Figure 5 d, e). The most substantial cracking concentrated within 61 cm [24 inch] above the rock socket but extended to higher elevations with larger spacings. Almost all cracks formed perpendicular to the pile axis, i.e. very few diagonal cracks were recorded. There was no sign of cracking or damage along the socket surface or within the rock socket itself.

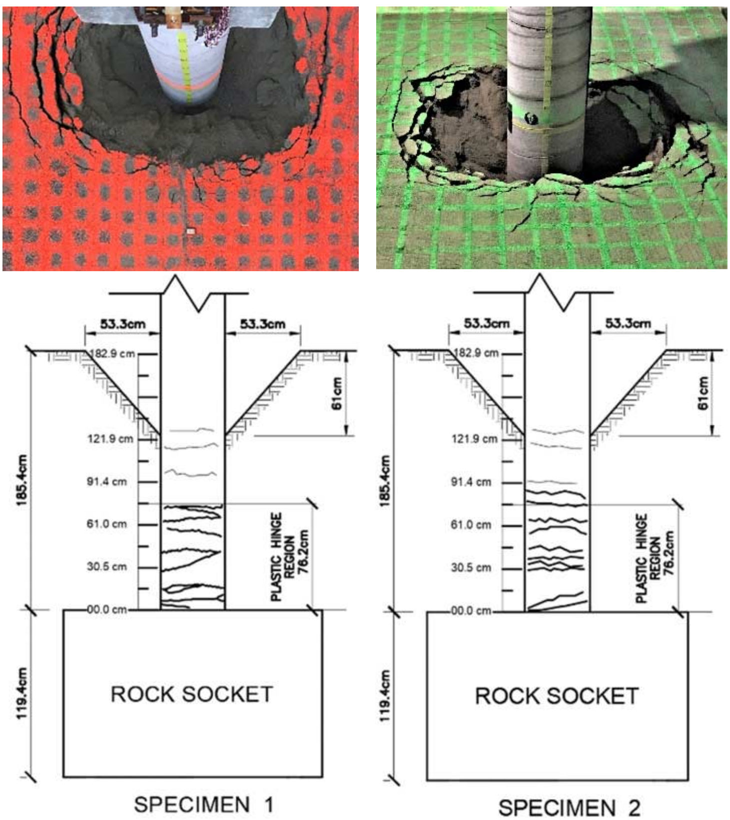
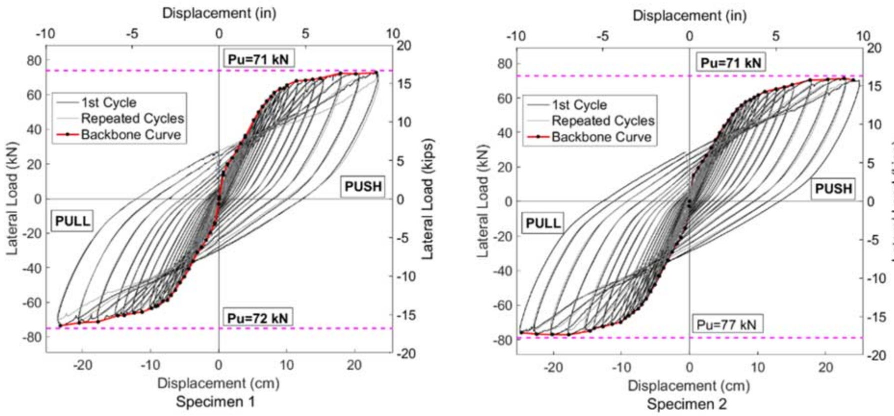


Figure 5. Major cracks along the pile depth and around the surface soil at test completion.

3.2. Selected Experimental Data

Figure 6 shows the experimental load displacement behavior of Specimens 1 and 2 with their respective backbone curves. Specimen 1 reached an ultimate load of approximately 68.5 kN [15.4 kips] at a corresponding pile head displacement of 11.4 cm [4.5 in] in push direction, and approximately 65 kN [14.7 kips] at a corresponding pile head displacement of 11.4 cm [4.5 in] in the pull direction. Similarly, Specimen 2 reached ultimate resistance at 64.5 kN [14.5 kips] and 11.4 cm [4.5 in] in push direction, and approximately 71.2 kN [16 kips] at a corresponding pile head displacement of 11.4 cm [4.5 in] in pull direction.



(a) (b)

Figure 6. Experimental load-displacement relationships with backbone curve: (a) specimen1, (b) specimen 2.

Both specimens behaved identically up to “concrete cracking”, i.e., up to a displacement level of 0.64 cm [0.25 in] and a corresponding load of 13.34 kN [3 kips] (about 20% of the ultimate load). The yield displacement was approximately 6.35 cm [2.5 in] at a corresponding load of 8 kips (about 50% of the ultimate load) after which the piles accumulated substantial permanent deformations for repeated loading cycles.

Figure 7 shows the lateral deformation profile recorded through the inclinometer instrumentation. Measurements indicate that no deformation occurred within the rock socket. Small lateral pile deformations are noticeable beyond 15 cm [0.5 ft] above the rock socket. Deformed shapes were similar in both “pull and push” directions, as well as for SP1 and SP 2. Curvature profiles, as shown in Figure 8 suggest the formation of a plastic hinge within 60 cm [2 ft] above the rock-socket which corresponds to 1.2 m [4 ft] below the ground surface (i.e., about 3 pile diameters (3D)) which agrees well with crack patterns observed upon excavation.

3.3. Comparison with Analytical Predictions

Figure 9 compares the experimental backbone curves for Specimens 1 and 2 with the predicted load-displacement curve. Relatively good agreement can be observed up to specimen yield. The predictions slightly underestimate the specimens’ ultimate capacity by about 10%. This deviation could be attributed to the yield strength assumption of the longitudinal reinforcement. Figure 9 also illustrates the predicted failure limits in flexure and shear. The identical specimen behavior suggests flexural failure modes for both, SP1 and SP2. Specifically, the predicted shear failure due to potential shear amplification near the rock-socket interface would have caused an early failure of Specimen 2 at approximately 44.5 kN [10 kips], which was not observed experimentally. Instead the pile specimen SP2, which was insufficiently reinforced for the potential shear amplification, performed identically to the pile (SP1) which was sufficiently reinforced for the shear amplification and resisted a lateral load of 1.6 times the predicted failure limit state.

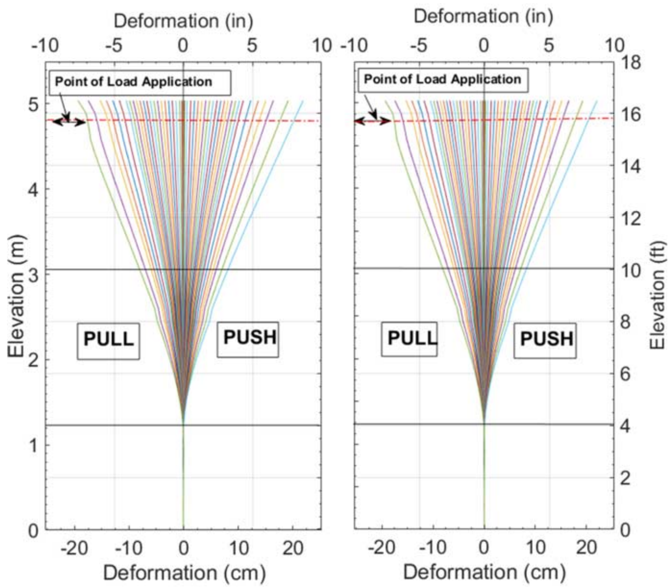


Figure 7. Deformed shape of Specimen 1(left) and Specimen 2 (right) at each applied displacement level (inclinometer readings).

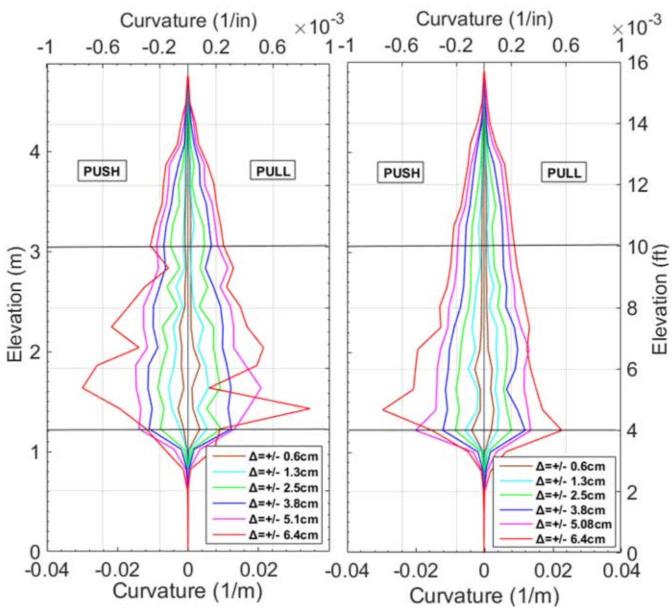


Figure 8. Curvature profiles derived from longitudinal strain gauge instrumentation for Specimen 1 (left) and Specimen 2 (right).

4. Summary

Two pile specimens with different transverse shear reinforcement were examined under identical test conditions and subjected to reverse cyclic lateral loading. The specimens were installed in a two-layer soil system with a strong stiffness contrast, namely a loose, silty sand underlain by rock, experimentally simulated through high-strength concrete. Specimens were tested to complete structural failure and excavated after test completion. No damage (i.e., cracking) and no lateral pile deformations were observed in the rock socket. Predominantly flexural cracking occurred along the pile within 60 cm [2 ft] above the rock socket. This elevation corresponds to a depth of three pile diameters below ground surface, which is a typical location of plastic hinges of flexible piles. The differently reinforced pile specimens were expected to fail in different failure modes according to their transverse reinforcement ratio and the predicted shear amplification by the Winkler-based analysis. The experimental specimen behavior suggests that the analytically predicted shear dominated failure did not occur. These preliminary observations could be of future benefit to the construction industry as heavy transversely instrumented section could be minimized and potential issues such as restrictions of concrete flow or air-pocket formation due to too closely spaced hoops could be reduced.

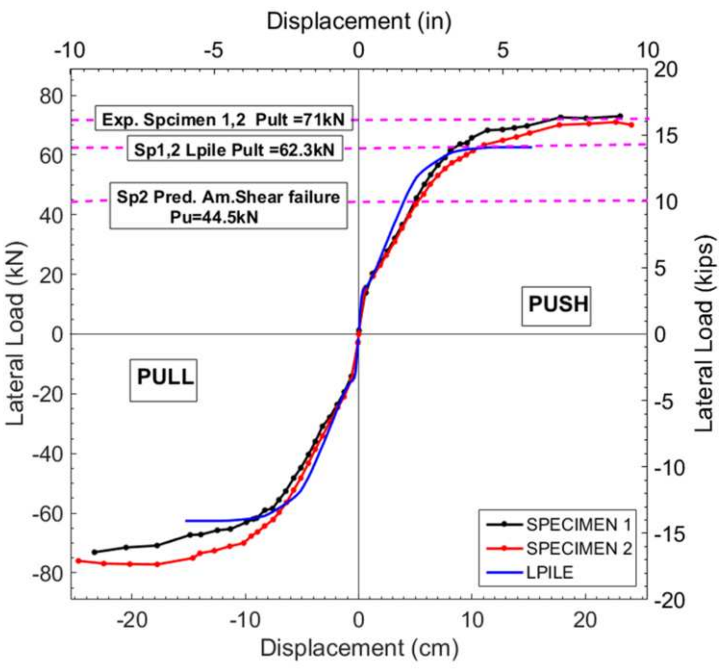


Figure 9. Comparison of experimental and predicted load-displacement responses.

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