Jet Grouting and Soil Mixing for Increased Lateral Pile Group Resistance

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ABSTRACT

Lateral load tests were performed on a full-scale pile cap in clay before and after construction of shallow soilcrete walls produced by soil mixing and jet grouting on either side of the pile cap. This relatively simple approach increased the lateral resistance of the pile cap by 60% for soil mixing and 160% for jet grouting. For the soil mixed wall, essentially all of the increased resistance was due to passive pressure and side/base shear against the soil mixed wall as the pile cap pushed the wall laterally. However, for the jet grout wall, about 25% of the increased resistance came from soil-pile interaction because the jet grout wall extended under the cap and against the piles. Soil mixing and jet grouting provide a means to significantly increase the lateral resistance of existing pile group foundations with relatively little investment of time, effort, and expense relative to adding more piles.

KEYWORDS: Pile Group, Lateral Loading, Load Testing, Jet Grouting, soil mixing, Soil Improvement, Deep Foundations

INTRODUCTION

The lateral resistance of pile groups in soft clay is important in the design of structures subjected to loads produced by earthquakes, waves, wind, landslides and ice flows. Typically, when analyses indicate that the lateral resistance of a foundation is inadequate, additional piles, drilled shafts or micro-piles are added to increase the lateral resistance. For retrofit of existing structures, an expanded pile cap is often required to structurally connect the new piles to the existing pile group. While this approach produces the required lateral resistance, it is also relatively expensive and time consuming.

An alternative approach is to use soil improvement techniques to increase the strength and stiffness of the surrounding soil, thereby increasing the lateral resistance of the pile group. The improved zone could potentially be relatively shallow and narrow because the lateral resistance of piles is typically transferred within a depth of 5 to 10 pile diameters and within 6 diameters laterally. Although this relatively simple approach has the potential of being more cost-effective, few full-scale tests are available to guide engineers in evaluating the actual effectiveness of this approach. In addition, numerical models to evaluate this approach have not been fully validated. To provide basic test data, full-scale lateral pile group load tests were performed on a 9 pile group in soft clay and then after installation of soilcrete walls produced by jet grouting and soil mixing techniques on either side of the pile cap. The jet grouting arrangement has the advantage of being able to treat the soil under the pile cap immediately adjacent to the piles themselves.

GEOTECHNIAL SITE CONDITIONS

A generalized soil boring log at the test site is provided in Figure 1. The depth is referenced to the excavated ground surface which was 0.76 m above the base of the pile cap as shown in the figure. The soil profile consists predominantly of cohesive soils; however, some thin sand layers are located throughout the profile. The cohesive soils near the ground surface typically classify as CL or CH materials with plasticity indices of about 20 as shown in Figure 1. In contrast, the soil layer from a depth of 4.5 to 7.5 m consists of interbedded silt (ML) and sand (SM) layers. The water table is at a depth of 0.60 m.

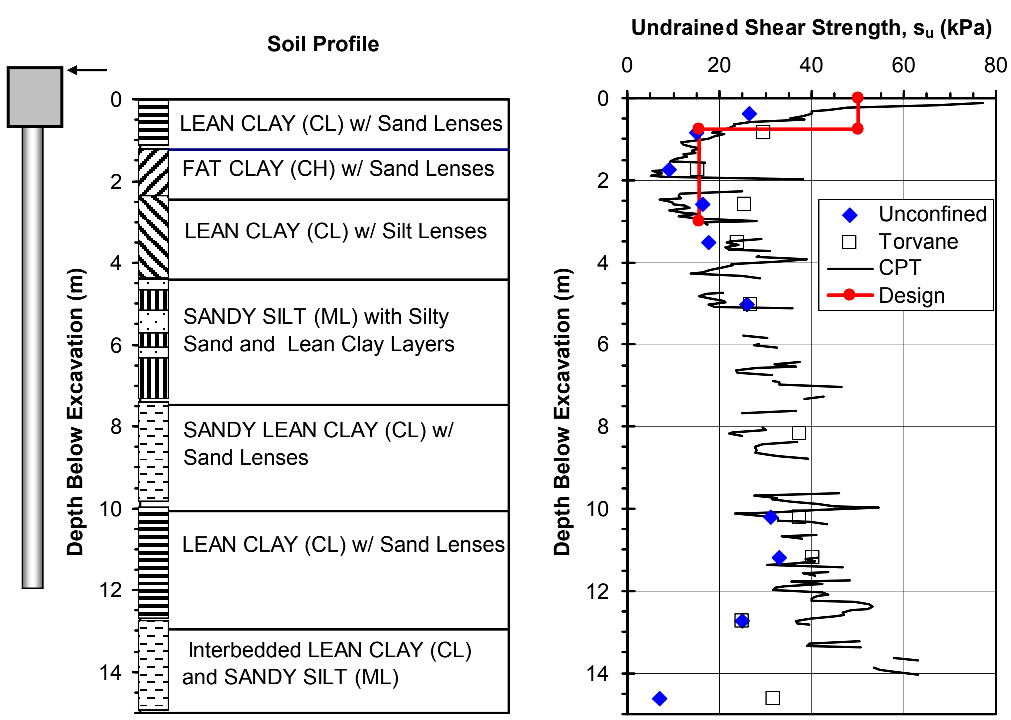

Figure 1. Soil profile and undrained shear strength profile for the test site.

The undrained shear strength is also plotted as a function of depth in Figure 1. Undrained shear strength was measured using a miniature vane shear (Torvane) test on undisturbed samples immediately after they were obtained in the field. In addition, unconfined compression tests were performed on most of the undisturbed samples. Both the Torvane and unconfined compression tests indicate that the undrained shear strength decreases rapidly from the ground surface to a depth of about 2 m but then increases with depth. This profile is typical of a soil profile with a surface crust that has been overconsolidated by desiccation. The undrained shear strength was also computed from the cone tip resistance using a correlation equation. The undrained shear strength obtained from the CPT correlation is also plotted versus depth in Fig. 1 and the agreement with the strengths obtained from the Torvane and unconfined compression tests is reasonably good. Nevertheless, there is much greater soil variability. The drained strength in the interbedded sand layers is not plotted.

TEST PILE GROUP CHARACTERSITICS AND TEST LAYOUT

The pile group consisted of nine test piles which were driven in a 3 x 3 orientation with a nominal center to center spacing of 0.9 m as shown in Figure 2. The tests piles were 324 mm OD pipe piles with a 9.5 mm wall thickness and they were driven closed-ended with a hydraulic hammer to a depth of approximately 13.4 m below the excavated ground surface. The steel conformed to ASTM A252 Grade 2 specifications and had a yield strength of 400 MPa based on the 0.2% offset criteria. The moment of inertia of the pile itself was 11,613 cm^4; however, angle irons were welded on opposite sides of two to three test piles within each group which increased the moment of inertia to 14,235 cm^4.

A steel reinforcing cage was installed at the top of each test pile to connect the test piles to the pile cap. The test piles typically extended about 0.6 m above the base of the pile cap and the reinforcing cage extended 0.7 m above the base of the cap and 2.7 m below the base. The steel pipe pile was filled with concrete which had an average unconfined compressive strength of 34.5 MPa.

A pile cap was constructed by excavating 0.76 m into the virgin clay. The concrete was poured directly against vertical soil faces on the front and back sides of the pile cap. This construction procedure made it possible to evaluate passive force against the front and back faces of the pile caps. In contrast, plywood forms were used along the sides of each cap which were braced laterally against the adjacent soil faces. This construction procedure created a gap between the cap sidewall and the soil so that side friction would be eliminated. A corbel, 0.55 m tall and 1.22 m wide, was constructed on top of the cap to allow the actuator to apply load above the ground surface without affecting the soil around the pile cap.

Testing Procedure and Instrumentation

The lateral load tests on the pile group were conducted using a 2700 kN hydraulic actuator which applied load to the pile group. A similar adjacent pile group provided a reaction for the applied load. The reaction pile group was located 10 m away from the test pile group to reduce interaction effects. The lateral load tests were carried out with a displacement control approach with target pile cap displacement increments of 3, 6, 13, 19, 25, and 38 mm. During this process the actuator extended or contracted at a rate of about 40 mm/min. In addition, at each increment 10 cycles with a peak pile cap amplitude of ±1.25 mm were applied with a frequency of approximately 1 Hz to evaluate dynamic response of the pile cap. After cycling at this small amplitude level at each increment, the pile group was pulled back to the initial starting point prior to loading to the next higher target displacement.

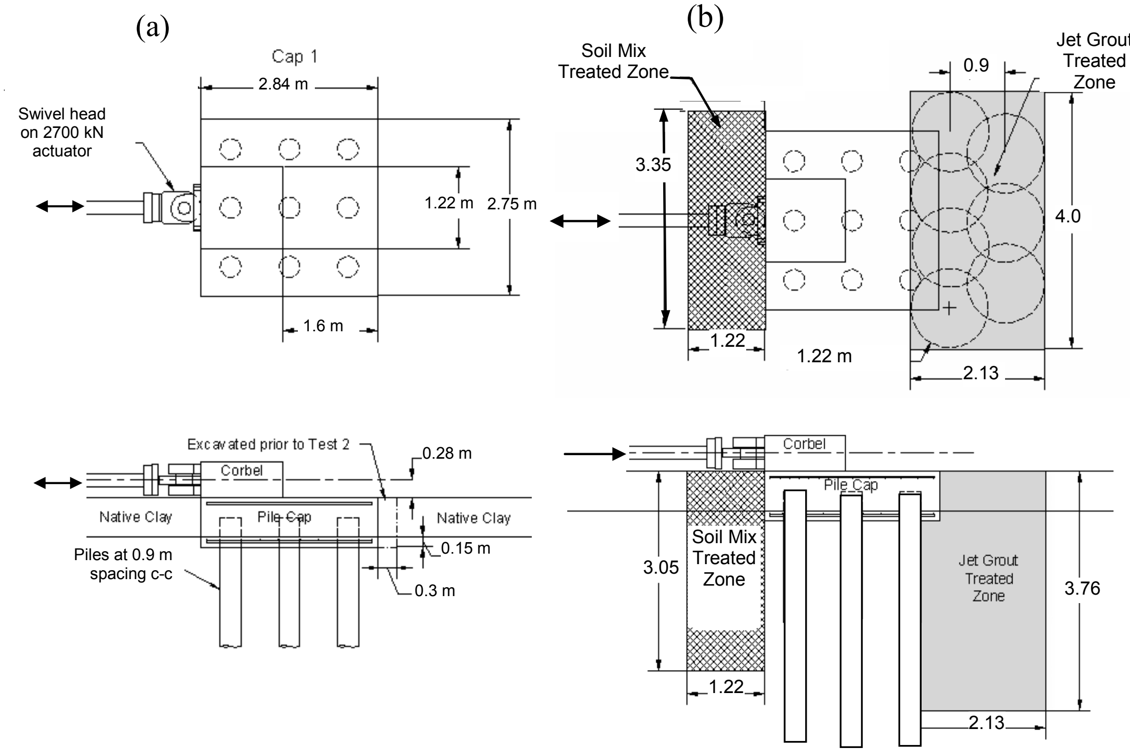

Figure 2 Plan and profile drawings of the pile group (a) in virgin clay before treatment and (b) after construction of soil mixed and jet grout walls.

Pile Group Testing Sequence

Plan and profile drawings showing the layout of the pile group for Tests 1 and 2 are provided in Figure 2(a). Tests 1 and 2 were performed to provide a baseline of the lateral load behavior of the pile caps in virgin soil conditions prior to any soil treatment. Test 1 was conducted by pulling cap 1 to the left using the actuator while the untreated native soil was in place to the top of the pile cap. Following Test 1, the pile cap was pulled back to zero deflection, but after the actuator load was released some residual deflection remained.

Prior to Test 2, the soil immediately adjacent to the opposite side of the pile cap was excavated to create a 0.3-m wide gap between the pile cap face and the adjacent soil as shown in Figure 2(a). This excavation eliminated passive force against the pile cap for the subsequent test. After excavation was complete, which required less than an hour to accomplish, Test 2 was carried out by pushing the pile cap to the right using the actuator. The testing was performed using the same procedure described previously. Test 2 was designed to define the passive force vs. deflection curve provided by the unsaturated clay against the pile cap.

Prior to additional testing, a "soil mixed wall" was constructed on the left side of the pile group as shown in Figure 2(b). The soil mixed wall was 3.05 m deep, 3.35 m wide and extended 1.22 m beyond the front face of the pile cap. In addition, 7 jet grout columns were installed on the opposite side of the cap to create a treated zone 3.66 m deep and 4.0 m wide which extended 2.13 m beyond the face of the pile cap as shown in Figure 2(b). In contrast to the soil mixed wall, which extended vertically below the edge of the cap, the jet grout technique allowed the treated zone to extend under the pile cap and impinge on the front row of test piles. For Test 3, the actuator pulled the pile cap to the left. Comparisons between this test and Test 1 in virgin clay make it possible to determine the improvement provided by the soil mixed wall. For Test 4, the soil mixed wall was excavated to the base of the pile cap and the pile cap was once again pulled to the left. This was done to help evaluate the resistance provided by passive force on the pile cap. Finally, Test 5 was performed by pushing the pile cap to the right with the actuator to evaluate the improvement in resistance provided by the jet grout wall. Comparison between Test 3 and 5 also provides some idea of the increased improvement provided by treating the soil immediately adjacent to the piles.

SOIL IMPROVEMENT CONSTRUCTION

Construction of Jet Grout Wall

Plan and profile views of the jet grout column wall on one side of the pile cap are shown in Figure 2(b). A total of seven 1.22-m diameter jet grout columns were installed in two rows to create the wall. Prior to jet grouting, the excavation in which the pile cap was located was temporarily backfilled with soil so that jet grouting could extend from the top of the pile cap to a depth 3 m below the base of the pile cap. Four of the columns were installed near the edge of the pile cap so that the grout column could extend under the pile cap and impinge on the front row of piles. The columns were spaced at 0.9 m intervals to create a 0.3 m overlap between the columns. The centers of an additional three columns were located 0.9 m behind the first row so that they would overlap with the first row. As can be seen in Figure 2(b), the grout columns formed a soilcrete wall with a depth of 3.66 m and a width of 4 m which extended about 2.13 m beyond the face of the pile cap.

To form the grout columns, a single hole double fluid jet grouting technique was employed and each of the columns was constructed with the same installation parameters. The jet grout drill head was initially advanced to the base of the treatment zone, 3 m below the pile cap, using water jets and a drilling bit at the bottom of the drill rod. Subsequently, the drill head was rotated and pulled upwards at a rate of one rotation per 30 mm of lift, while cement slurry was injected at a pressure of 41.4 MPa, and a flow rate of 340 liters/min from the inner orifice of the drill nozzle. Concurrently, compressed air was injected from the outer orifice of the drill nozzle to form a protective shroud around the slurry jet which improves the erosive capacity of the cement slurry jet. The grout slurry mix had a specific gravity of 1.52, which is equivalent to a 1:1 water to cement ratio by weight. Throughout the jet grouting process, the flow rates, pressures, pull rate, drill rod rotation rate and specific gravity were controlled by a computerized system which monitored and recorded these parameters. Based on these parameters the jet grout columns would be expected to be about 400 kg/m^3 or about 20% by weight.

The unconfined compressive strength of the soilcrete produced by the jet grouting process was evaluated using wet grab samples along with core samples. Although there was significant scatter to the data, which is typical for soilcrete columns installed using jet grouting, there was a trend of increasing strength with curing time. Two weeks after jet grouting, the mean compressive strength of the wet grab samples had increased to about 3000 kPa and after four weeks it had increased to about 4500 kPa. These strength gains are typical for jet grouting applications (Burke, 2004). However, the average strength from two cored samples was about 3170 kPa, which is about 30% lower than the strength obtained from the wet grab samples. The strength from the core samples is likely more representative of in-situ conditions because the jet grouting process does not achieve the same quality of mixing which is achieved by the hand mixing process employed with the wet grab samples.

Construction of Soil Mixed Wall

Because of the small size of the wall, economics did not permit the mobilization of a dedicated soil mixing rig to the site. Instead, a procedure was devised to produce a volume of soil with a compressive strength typical of that produced by soil mixing. The native soil was first excavated to a depth of 1.5 m below the top of the cap using a trackhoe bucket. The excavation was then filled to the top of the cap with jet grout spoils from the opposite side of the pile cap. Afterwards, the remaining intact soil from 1.5 to 3 m below the top of the cap was progressively excavated with the excavator bucket and mixed with the jet grout spoils. Mixing was accomplished by repeatedly stirring the native soil and grout spoil until the consistency of the mixture became relatively homogeneous and no large blocks were obvious in the mixture. This process required approximately 10 to 15 minutes of mixing and provided a 1 to 1 ratio of soil to grout spoil mixture.

As indicated previously, the grout used in the jet grouting procedure was designed to have a specific gravity of approximately 1.52, which is the equivalent of a 1 to 1 water to cement ratio by weight using normal type I cement. The cement content per volume of jet grout slurry was computed to be about 420 kg/m^3. Mixing the jet grout slurry with the underlying clay at a 1 to 1 ratio by volume reduced the cement content of the resulting soilcrete wall to approximately 210 kg/m^3. This corresponds to about 10% cement by weight. Six core samples obtained from the soilcrete wall indicate that the mean compressive strengths were 870 and 965 kPa after 28 and 60 days of curing, respectively. This strength gain is consistent with previous experience for soil mixed walls (Terashi, 2003). The strength of the soilcrete produced by soil mixing is only about 30% of the strength obtained with jet grouting.

TEST RESULTS AND ANALYSIS

Figure 3 presents plots of the load-displacement curves for cap 1 during Test 1 (virgin clay) and Test 3 after the mass mix soil improvement. With the soil mix wall in place, the pile cap resisted 2013 kN compared to the 1253 kN resisted by the pile cap in the virgin clay at a displacement of 40 mm. This represents an increase of 63% in the lateral resistance provided by the pile cap. It is also interesting to evaluate the increase in initial stiffness due to the mass mixing. Prior to treatment, the secant stiffness of the load-displacement curve at a displacement of 3 mm was 140 kN/mm while after soil mixing the stiffness increased to about 230 kN/mm. This represents an increase in stiffness of about 65%.

The increase in lateral resistance shown in Figure 3 can be explained if it is assumed that the soil mixed wall essentially moves as a block as the pile cap pushes it laterally. The increased lateral resistance can then be reasonably computed based on (1) passive force on the back of the wall, (2) the shear force on the side of the wall and (3) the shear force on the base of the wall using the undrained strength profile. This calculation was made using an average undrained shear strength of 50 kPa in the upper 0.76 m of the profile and an average undrained shear strength of 15.5 kPa in the zone from 0.76 m to 3 m based on the soil strength testing as shown in Figure 1. This approach can account for all of the measured increased lateral resistance. With this approach, no increase in lateral resistance was attributed to pile-soil interaction, because the wall does not extend to the face of the piles which are 0.3 m behind the face of the pile cap.

Figure 4 provides plots of the load-displacement curves for cap 1 during Test 2 in virgin clay and Test 4 after the mass mix wall construction. In contrast to Tests 1 and 3, in these two tests the soil adjacent to the cap was excavated to the base of the cap. Because the soil had been previously loaded, the load-displacement curve for the pile cap with the mass mix wall is actually lower than that for the pile cap in virgin clay due to gap formation at small deflections. However, as displacement increased to the maximum previous displacement, the load-displacement curve appeared to follow the load-displacement curve for the pile cap in virgin clay with little apparent increase. This result is consistent with the mechanisms for increase resistance discussed previously. Because the soil adjacent to the pile cap had been excavated, the pile cap no longer pushed the soil mixed wall laterally, therefore, no increase in lateral resistance was produced. This also confirms the notion that little or no increase in resistance was produced by soil-pile interaction.

Figure 3 also provides a comparison of the load-displacement curves for the pile cap during Test 1 (virgin clay) and Test 5 after construction of the jet grout wall on one side of the cap. Unfortunately, because the load capacity of the actuator was reached, the measured load-displacement could not be extended beyond about 18 mm. However, at that displacement, the pile cap with the jet grout wall resisted 2700 kN compared to the 950 kN resisted by the pile cap in the virgin clay. This represents an increase of about 1750 kN or 184% in the lateral resistance provided by the pile group. The load-displacement curve for test 5 has been extrapolated based on the slope of the curve and tests on similar pile groups treated with jet grouting (Adsero 2008). The extrapolated curve is shown with a dashed line in Figure 3. At a displacement of 38 mm, the jet grout wall is estimated to have increased the lateral resistance to 3200 kN relative to 1253 kN value in the virgin clay. This represents an increase in resistance of 1950 kN or 155%. It is also important to evaluate the increased stiffness due to the mass mixing. Prior to treatment, the secant stiffness of the load-displacement curve at a displacement of 2.5 mm was 130 kN/mm while after jet grout treatment the stiffness had increased to 460 kN/mm. This represents an increase in stiffness of about 260%.

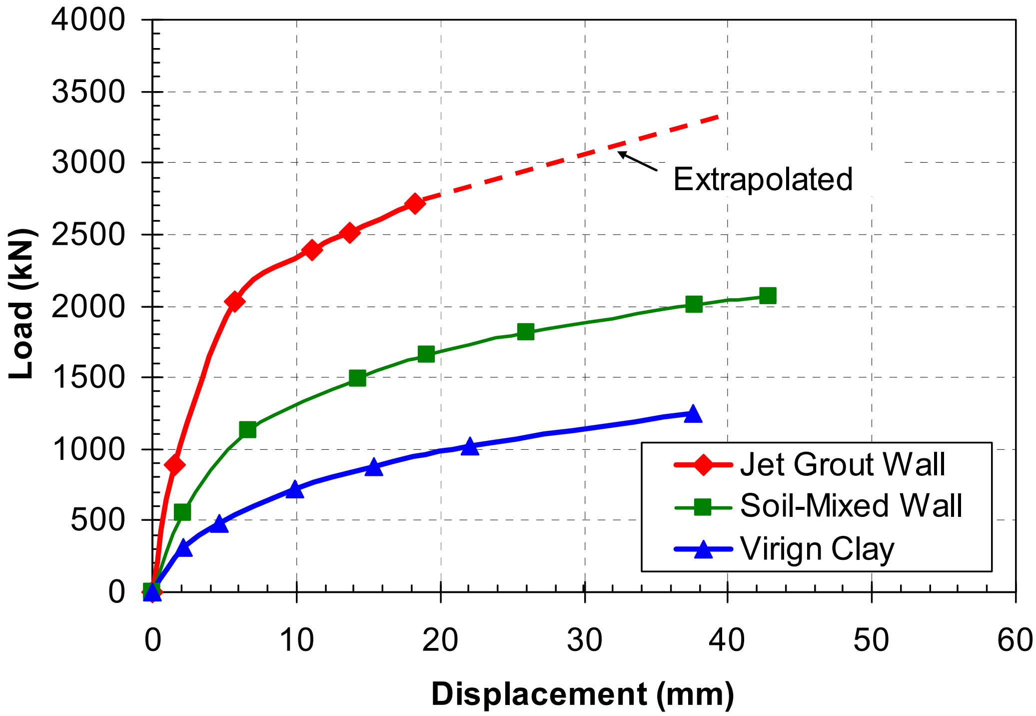

Figure 3. Comparison of load-displacement curves for the lateral load tests on the pile cap in native clay (Test 1), after construction of the soil mixed wall (Test 3), and after construction of jet grout wall (Test 5) with soil against pile cap.

As was done for the soil mixed wall, the lateral resistance of the soilcrete block produced by jet grout treatment was computed by adding the passive force on the back of the block to the shear forces on the sides and base of the block. This approach accounts for 1500 kN or about 75% of the measured increase in lateral resistance. Therefore, the additional 25% of resistance must be a result of more complex soil-pile interaction between the piles and the jet-grout strengthened soil. This result is consistent with the fact that the jet grouting procedure improved the soil under the pile cap and immediately adjacent to the piles thereby increasing the pile-soil resistance. As noted previously, the soil-mixing technique, which did not treat the soil under the pile cap, provided no increase in soil-pile resistance. The observed increase in resistance due to pile-soil interaction and movement of the soilcrete block itself is currently being evaluated using finite element modeling techniques.

It should be noted that the load-displacement curves in Figure 3 after construction of the soil-mixed and jet grout wall represent re-loading conditions.

Previous experience in similar soil deposits indicates that peak loads will be 7 to 10% lower than observed during virgin loading for the second loading to the maximum previous displacement shortly after previous loading (Rollins et al 2006, Snyder 2004). However, at displacements less than the maximum value more significant decreases could be expected. Despite this fact, the load-displacement curves in Figure 3 do not exhibit any concave upward shape which would indicate the presence of gaps around the piles. This result suggests that the soft clay likely had time to "squeeze" back around the piles prior to subsequent load tests. In addition, for the test involving jet grouting, the construction process produced a mixture of soil and cement which likely eliminated gaps around the front row of piles.

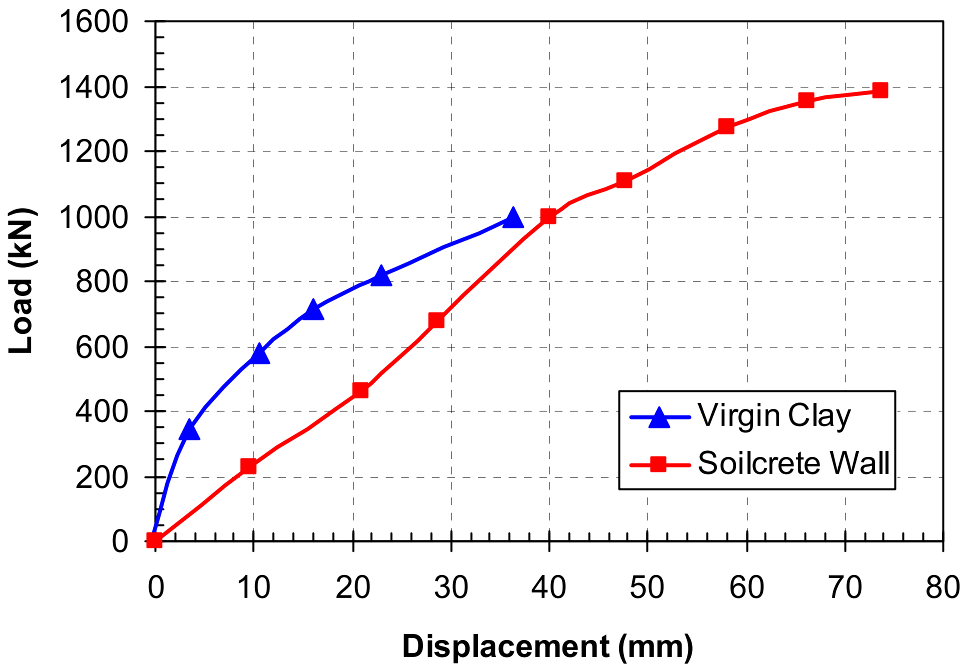

Figure 4. Load-displacement curves for the lateral load tests on the pile cap in native clay (Test 2) and the test after construction of a soil mixed wall (Test 4), after excavation of soil to the base of the pile cap.

CONCLUSIONS

1. Mass mixing with a cement content of approximately 200 kg/m^3 (10% by weight) was able to increase the compressive strength of a soft, plastic clay from a value between 15 to 50 kPa to an average of 970 kPa while jet grouting with a cement content of about 400 kg/m^3 (20% by weight) produced an average strength of 3170 kPa. These strength gains are consistent with previous experience.
2. Construction of a mass mixed "soilcrete" wall (3.05 m deep, 1.22 m wide, and 3.35 m long) adjacent to an existing pile cap (2.74 m square and 0.76 m deep) increased the lateral resistance and initial stiffness by about 60%. Construction of a jet grout "soilcrete" wall (3.66 m deep, 2.13 m wide and 4.0 m long) adjacent to

the other side of the pile cap increased the lateral resistance by 155% and the initial stiffness by 260%
3. Analysis and subsequent testing, after excavation of the mass mixed wall to the base of the pile cap, indicates that essentially all of the increased resistance was due to passive resistance and side/base shear against the soil mixed wall as the pile cap pushed the wall laterally. No appreciable increase in lateral resistance could be attributed to soil-pile interaction.
4. For the jet grout wall, resistance due to passive force and shear on the sides and base of the soilcrete wall could only account for about 75% of the increased lateral resistance while about 25% of the increased resistance was provided by increased soil-pile resistance.
5. The increase soil-pile resistance for the jet grouting case is likely due to the fact that the jet grout technique could treat the soil underneath the pile cap and immediately adjacent to the piles while the soil mixing technique could not.
6. Soil mixing and jet grouting provide the opportunity to significantly increase the lateral resistance of existing pile group foundations with relatively little investment of time, effort, and expense relative to constructing additional piles.

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