Ground Improvement for Increasing Lateral Pile Group Resistance

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ABSTRACT

Lateral load tests were performed on a full-scale pile cap in untreated clay along with pile groups involving (a) excavation and replacement with sand backfill, (b) a soilcrete wall along the side of the pile group, and (c) a jet grouted zone below the pile cap. The average compressive strength of the soft, plastic clay increased from an average of 50 kPa to an average of about 1000 kPa with soil mixing (10% cement) and to 3000 kPa with jet grouting (20% cement). Excavation and replacement only increased resistance by about 20%; however, the soil mixed wall increased resistance by 60%, and jet grouting increased resistance by 160%. For the soil mixed wall, essentially all of the increased resistance can be explained due to passive pressure and side/base shear against the soil mixed wall. However, for the jet grout treatment, additional resistance can also be attributed to increased structural resistance of the composite soilcrete volume under the cap. 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 the addition of more piles.

INTRODUCTION

The lateral resistance of pile groups is often critical to the design of bridges and high-rise structures subject to seismic, wind, wave and landslide forces. 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. Furthermore, an expanded pile cap or connecting beam is often required to structurally connect the new piles to the existing pile group. While this approach provides the required lateral resistance, it may 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 and thereby increase the lateral resistance of the pile group. The improved zone could potentially be relatively shallow because the lateral resistance of piles is typically transferred within 5 to 10 pile diameters. For example, the soil around the periphery of the foundation could be relatively easily improved for an existing foundation as illustrated in Fig. 1(a) using a variety of soil improvement techniques. Alternatively, the soil under the foundation could be improved for a new foundation or even for an existing foundation with a technique such as jet grouting (see Fig. 1(b)). Improving the soil under the foundation would have the potential for producing greater

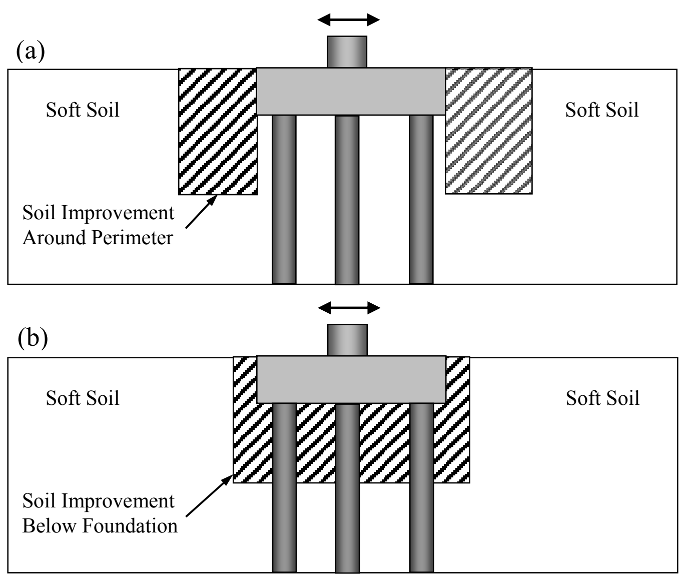

Fig. 1. Illustrations showing methods of using soil improvement (a) around the perimeter of an existing foundation or (b) below the foundation to increase lateral resistance

increases in lateral resistance because the improvement would reach interior piles. In addition, the process of creating a cemented “soilcrete” zone around pile foundations could potentially produce a zone which would behave like a reinforced “superpile” with increased structural stiffness.

Although soil improvement techniques have the potential for being more cost-effective and reducing construction time, relatively few tests have been performed to guide engineers in evaluating the actual effectiveness of this approach. In addition, numerical models to evaluate this approach have not been validated. This paper describes several full-scale lateral load tests on a nine pile group where ground improvement techniques were employed to increase the lateral resistance of pile foundations. In one case, soft clay was excavated and replaced with compacted sand. In a second case, a soil mixed wall was constructed along one side of the pile group and in a third case jet grouting was used to create a volume of soilcrete around a pile group. Tests were also performed on a pile group in untreated clay to provide a control on the results.

SOIL CONDITIONS AT TEST SITE

A generalized soil boring log at the test site is provided in Fig. 2. 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 to 25 as shown in Fig. 2. 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.

The undrained shear strength is also plotted as a function of depth in Fig. 2. 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 the correlation equation

$$s_u = (q_c - \sigma)/N_k \qquad (1)$$

where q_c is the cone tip resistance, σ is the total vertical stress, and N_k is a coefficient which was taken to be 15 for this study. The undrained shear strength obtained from Eq. (1) is also plotted versus depth in Fig. 2 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.

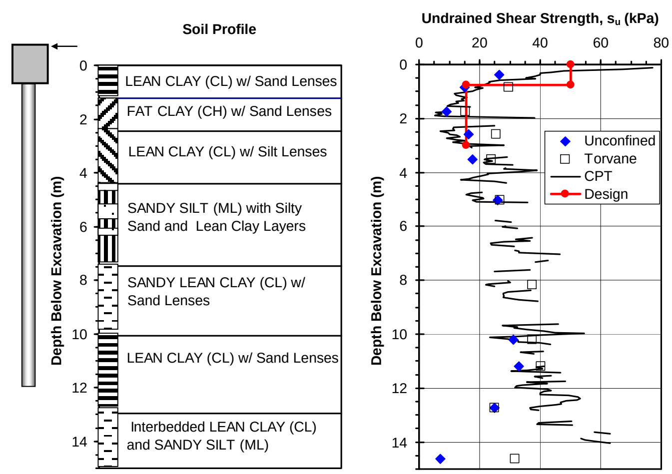

Fig. 2. Soil profile and undrained shear strength profile for the test site.

PILE GROUP PROPERTIES

The pile groups 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. 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 (57 ksi) 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 each pile cap. This 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 and 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. Steel reinforcing mats were placed in the top and bottom of each cap.

A corbel 0.55 m tall and 1.22 m wide was constructed on top of each cap to allow the actuator to apply load above the ground surface without affecting the soil around the pile cap.

TESTING PROCEDURE

The lateral pile group load tests were conducted using one or two 2700 kN hydraulic actuators to apply load to the pile group. Another pile group or groups provided a reaction for the applied load. In all cases, the reaction pile group or groups were located 10 m away from the test pile group to minimize 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 this cyclic loading at each increment, the pile group was pulled back to the initial starting point prior to loading to the next higher displacement increment.

PILE GROUP TEST LAYOUT

Pile Group in Untreated Virgin Clay

Plan and profile drawings showing the layout of the pile group in untreated clay for Tests 1 and 2 are provided in Fig. 3(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. At the completion of 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 face of the pile cap was excavated by hand to create a 0.3-m wide gap between the pile cap face and the adjacent soil as shown in Fig. 2. 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 provide an indication of the passive force provided by the unsaturated clay soil against the pile cap.

Pile Group with Compacted

Plan and profile drawings showing the layout of the pile group with compacted fill are provided in Fig. 3(b). Test on this pile

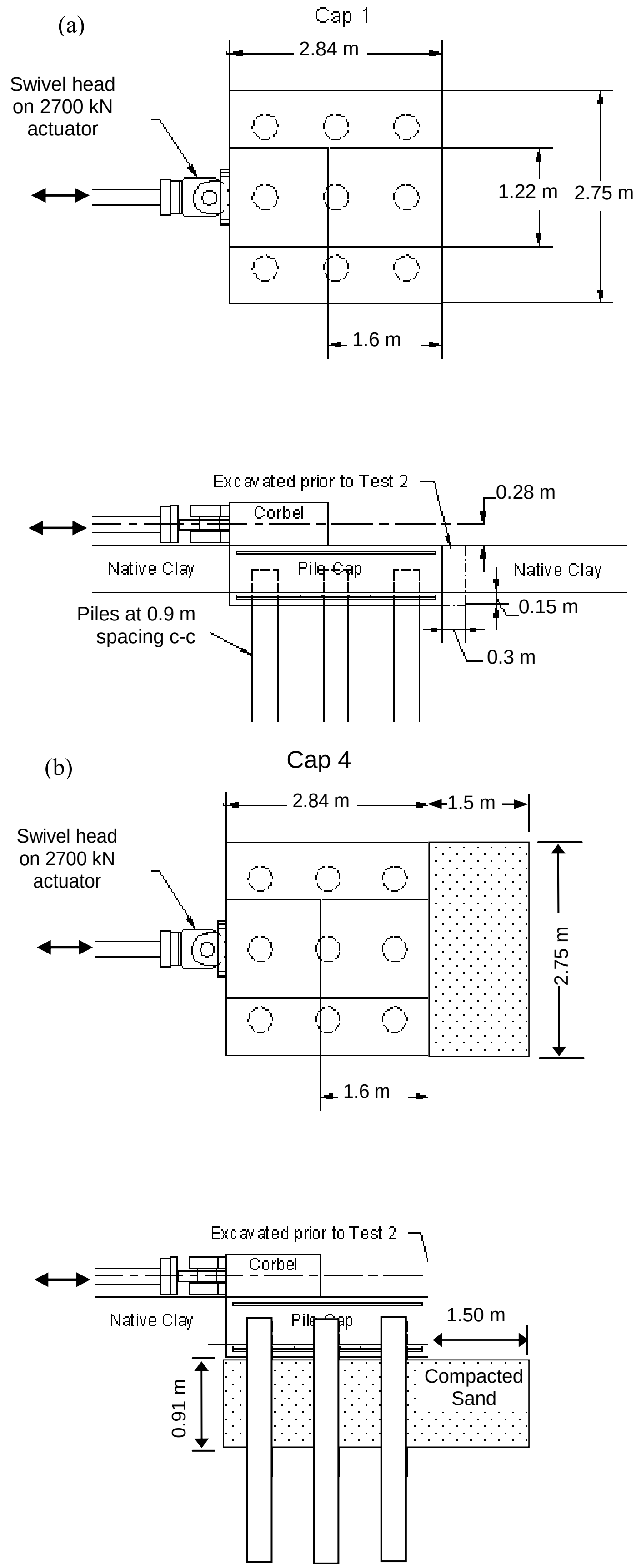

Fig. 3. Plan and profile drawings of pile groups (a) in untreated virgin clay and (b) with compacted sand..

group were designed to determine the increased strength that could be provided by excavating the soft clay and replacing it with compacted sand. Prior to pile driving, clay was excavated to a depth of 1.90 m and replaced with compacted fill up to the base of the pile cap. Clean concrete sand, meeting ASTM C-33 specifications, was used as the backfill material. The sand was compacted in 150 to 200 mm lifts using a hydraulic plate compactor attached to the end of a trackhoe. Based on nuclear density measurements, the sand was compacted to an average in-place dry density of 16.33 kN/m^3 which is 93.7% of the modified Proctor density (γ_{dmax}=kN/m^3). Plans originally called for excavation and replacement to a greater depth; however, caving of the soft clay precluded deeper excavation. When the piles were installed, the ground heaved and, in order to maintain the correct pile cap thickness, approximately 0.23 m of backfill had to be removed, leaving approximately 0.91 m of sand under the cap. Lateral load tests were performed in both directions. The sand fill extended beyond the cap face on one side to evaluate the increased pile-soil resistance from extending the sand fill. Comparison with tests 1 and 2 allow a determination of the increased resistance for sand backfill.

Pile Cap with Soil Mix Wall

Plan and profile drawings of the pile group with a soil mix wall are provided in Fig. 4. 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 applied to produce a volume of soil with a compressive strength and consistency 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. The excavation was then filled to the top of the cap with jet grout spoils from an adjacent test area. 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.

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 past experience for soil mixed walls (Terashi, 2003).

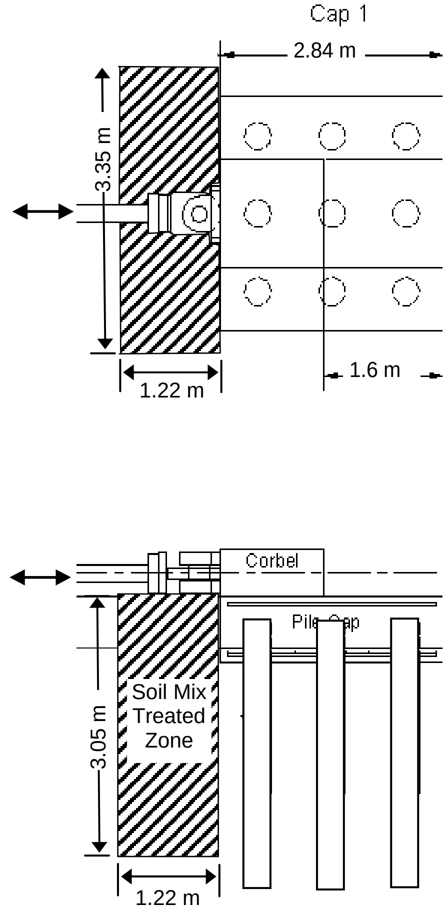

Fig. 4. Plan and profile drawings of pile group with soil mixed treated wall on one side.

Pile Cap with Jet Grout Treatment

Plan and profile views of the jet grout columns around pile cap 2 are shown in Fig. 5. A total of eight 1.5-m diameter jet grout columns were installed beneath and around the pile cap. Four of the columns were installed at the periphery of the pile cap while an additional four were installed through the cap itself as shown in Fig. 5. During construction of the pile cap and corbel, four 0.15 m diameter PVC pipes were placed in the pile cap between the rebar to provide access for the jet grout drill rods. For retrofit projects these access holes would have to be drilled through the pile cap. The target diameter of the jet grout columns was 1.5 m. The jet grout columns were spaced at approximately 0.9 m center-to-center left to right and at 1.5 m center-to-center from top to bottom. This likely produced a 0.6 m overlap of the columns in the direction of loading with no expected column overlap perpendicular to the direction of loading. As can be seen in Fig. 5, nearly the entire volume of soil beneath the pile cap was treated to a depth of 3 m below the bottom of the pile cap. In addition, the grout treatment extended about 0.9 m beyond the front and back ends of the cap and somewhat beyond the cap on the top and bottom sides.

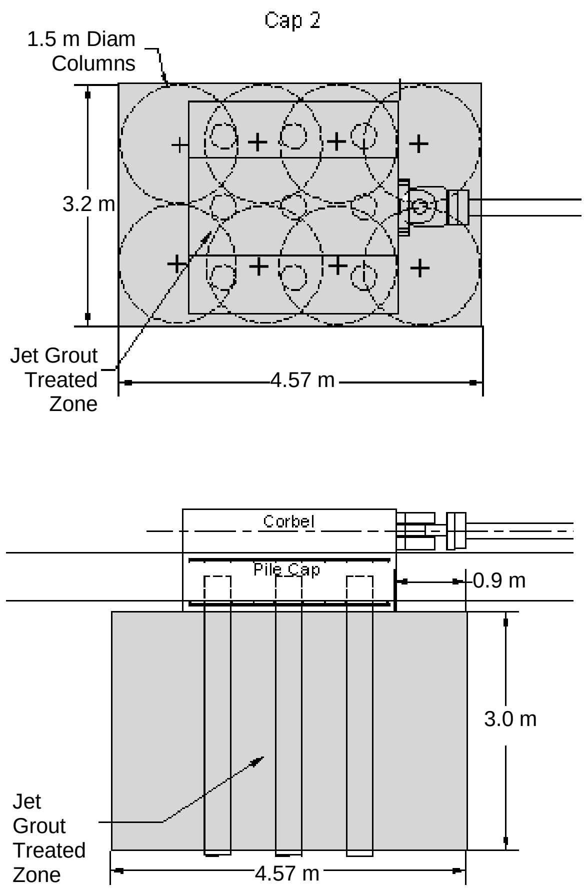

Fig. 5. Plan and profile view of pile group with jet grout treatment under the pile cap.

A single hole double fluid jet grouting technique was employed to form the grout columns and each of the columns was constructed with identical 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 located at the bottom of the drill rod. Subsequently, the drill head was rotated and pulled upwards at a constant rate, while cement slurry was injected at a specified pressure and flow rate 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 to improve 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 also monitored and recorded these parameters. These parameters are summarized in Table 1. Based on the column diameter, flow rates, pull rates and rotation rates, the cement content for the jet grout columns would be expected to be about 400 kg/m^3. It can be seen that the pull rate is greater than the rotation speed. Thus, one rotation of the high pressure nozzle occurred for each 30 mm lift.

Table 1. Summary of jet grout treatment parameters.

Column Length	3.66 m
Estimated Column Diameter	1.5 m
Grout Specific Gravity	1.52
Grout Pressure	41.37 Mpa
Grout Flow Rate	340 Liters/min
Rotation Speed	7 rpm
Pull Rate	20 cm/min

The unconfined compressive strength of the soilcrete produced by the jet grouting process was evaluated using wet grab samples as well as core samples. Although there was significant scatter to the data, which is typical for soilcrete columns installed using jet grouting, there is a trend of increasing strength with curing time. Prior to treatment, the mean compressive strength of the untreated clay was only 40 to 60 kPa. Two weeks after jet grouting, the mean compressive strength of the wet grab samples had increased to about 3000 kPa; after four weeks the strength had increased to about 4500 kPa. These strength gains are typical for jet grouting applications (Burke, 2004). 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 and is attributable to the poorer mixing produced by the jet grouting process relative to the hand mixing employed with the wet grab samples.

TEST RESULTS

<u>Pile Group in Untreated Virgin Clay</u>

Fig. 6 presents plots of the load-displacement curves for pile cap 1 in virgin clay before excavation (Test 1) and after excavation (Test 2) of the soil immediately adjacent to the front face of the pile cap. A comparison between the two curves indicates that the difference, attributable to passive resistance on the pile cap, is approximately 220 kN. The full passive force develops after a displacement of about 20 mm or 2.5% of the cap height.

Based on the measured passive force (P_p) the average undrained shear strength (s_u) of the upper 0.76 m of the soil profile was back-calculated using the equation

$$P_p = 0.5\gamma H^2 B + 2 s_u H B \qquad (2)$$

based on Rankine theory for undrained conditions where γ = total unit weight of the clay = 18.37 kN/m^3, H = depth of the pile cap = 0.76 m, B = width of the pile cap = 2.74 m. Based on this back-analysis, the undrained shear strength in the upper 0.76 m of the soil was found to be about 50 kPa. This shear strength is higher than that measured by the unconfined compression testing, but within the range predicted by the correlation with the CPT cone resistance as shown in Fig. 2.

For test 2 where the soil has been excavated adjacent to the cap, the lateral resistance was provided exclusively by soil-pile resistance. At a displacement of 37 mm the lateral pile group resistance was about 1000 kN or 111 kN per pile.

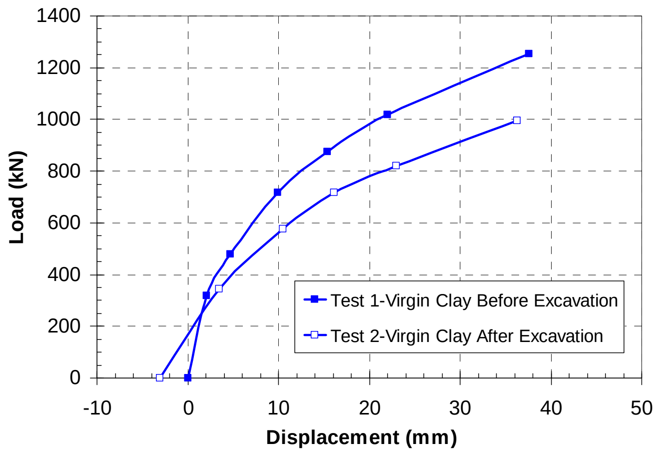

Fig. 6. Load versus deflection curves for the pile group in untreated virgin clay before and after excavation of soil against pile cap.

Pile Group in Compacted Sand

Fig. 7 shows plots of the load-displacement curves for pile group 1 during Test 2 in virgin clay and for pile group 4 with compacted fill below the cap as shown in Fig. 4. The pile group with compacted fill is being loaded to the right in Fig. 4. In both of these tests the soil immediately adjacent to the pile cap was excavated so that the lateral resistance is only provided by pile-soil resistance. As shown in Fig. 7, the placement of the relatively thin compacted sand layer typically increased the lateral resistance of the pile group by about 18%. At a displacement of about 40 mm the increased resistance is nearly 180 kN. This increase in lateral resistance can only be attributed to increased soil-pile resistance because there was no soil adjacent to the pile cap.

Assessment of the lateral load test in the other direction was complicated by issues associated soil relaxation following the previous tests and the presence of the native clay against the pile cap. Nevertheless, the test results suggest that the increased resistance was only slightly lower with 0.3 m of sand in front of the pile relative to 1.8 m of sand in front of the pile.

Greater improvement could potentially have been achieved if the compacted fill could have extended deeper; however, this would have required flatter excavation slopes to prevent caving and more backfill material, which would increase the cost. Finite element studies conducted by Weaver and Chitoori (2007)) suggest that most of the benefit from compacted fill around a pile occurs for fill materials extending five pile diameters below the ground surface. In this case the fill extended about three pile diameters.

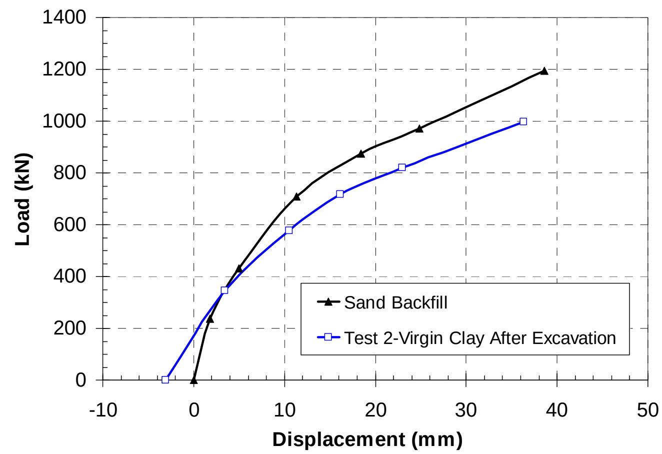

Fig. 7. Load versus deflection curves for the pile groupn with limited capacted granular backfill relative to the group in untreated virgin clay.

Pile Group with Soil Mixed Wall

Fig. 8 presents plots of the load-displacement curves for pile group 1 during Test 2 in virgin clay and the test after construction of the soil mixed wall shown in Fig. 4. With the soil mixed wall, 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 62.5% in the lateral resistance provided by the pile group. It is also interesting to evaluate the increase in initial stiffness due to the mass mixed wall. 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 230 kN/mm. This represents an increase in stiffness of about 65%.

It should be noted that the load-displacement curve in Fig. 8 after construction of the soil mixed wall represents 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 2008, Snyder 2004). However, at displacements less than the maximum value more significant decreases could be expected. Despite this fact, the load-displacement curves in Fig. 8 do not exhibit any concave upward shape which would indicate the presence of gaps around the piles. This suggests that the soft clay likely had time to "squeeze" back around the piles prior to the 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.

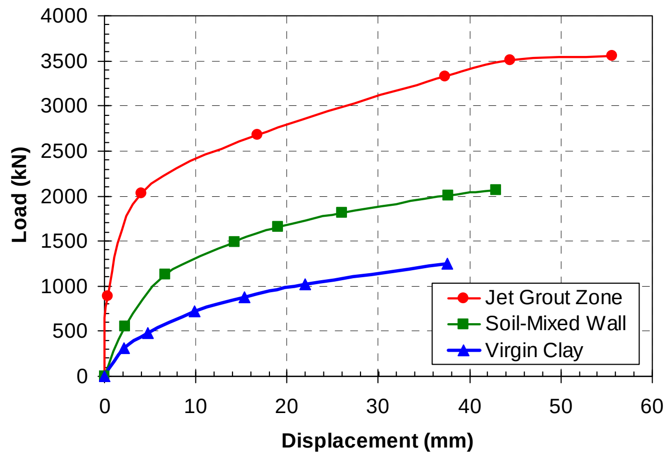

Fig. 8. Load vs. deflection curves for the pile group in untreated virgin clay, with soil-mixed wall at edge of cap, and jet grout treatment below cap around piles.

<u>Pile Group with Jet Grout Treatment</u>

Fig. 8 also provides a comparison of the load-displacement curves for pile group 1 during test 1 (virgin clay) and the pile group after the jet grouting treatment shown in Fig. 5. With the jet grout improvement, the pile cap resisted 3475 kN compared to the 1253 kN resisted by the pile cap in the virgin clay at a displacement of 38 mm. This represents an increase of about 2200 kN or 177% in the lateral resistance provided by the pile group. It is also important to evaluate the increased stiffness due to the jet grout. Prior to treatment, the secant stiffness of the load-displacement curve at a displacement of 3 mm was 140 kN/mm while after jet grout treatment the stiffness increased to 700 kN/mm. This represents an increase in stiffness of about 400%.

SIMPLIFIED ANALYSIS PROCEDURES

<u>Soil Mixed Wall</u>

The increased lateral resistance of the soil mixed wall shown in Fig. 4 can be explained if it is assumed that the wall moves as a block as the pile cap pushes it laterally. The increased lateral resistance can be computed based on (1) passive force on the back of the wall, (2) the adhesive force on the side of the wall and (3) the adhesive force on the base of the wall using the undrained strength profile. These components and their values are illustrated schematically in Fig. 9. The calculation of these forces was made using the back-calculated 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 as shown in Fig. 2.

Passive force was computed using the Rankine method with Equation 2. Based on the undrained shear strength back-calculated for the upper 0.76 of the soil profile, the passive force on the soilcrete wall would produce 271 kN, while passive resistance on the wall from 0.76 to 3 m depth would be expected to produce an additional 470 kN. Therefore, passive force would produce a combined force of about 740 kN on the wall. Adhesive side shear on the two sides of the soilcrete wall would be expected to contribute a total of 169 kN, while shear at the base of the wall would likely produce an additional 71 kN of force. These shear forces are simply obtained by multiplying the average undrained shear strength within a depth range by the area of the wall involved. Combining these forces, this approach can account for 978 kN of force which is equal to the difference in resistance for pile cap with the soil mixed wall relative to the pile group in virgin clay without soil against the wall (Test 2). 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.

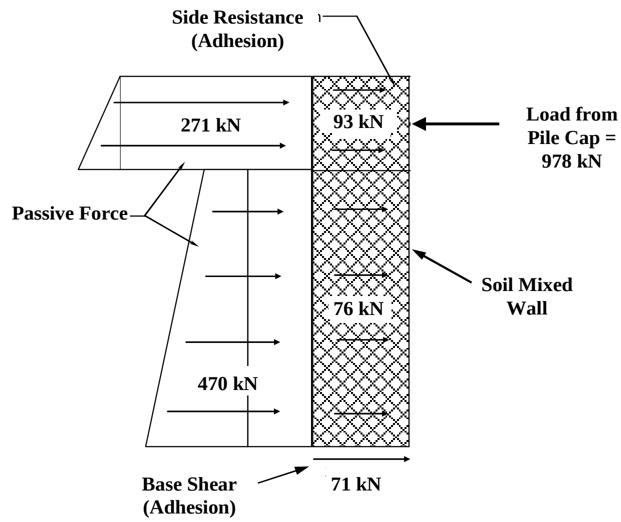

Fig. 9. Free-body diagram of the soil mixed wall showing the resistance to force from the pile cap provided by (a) passive force on the wall, (b) adhesive force on the side of the wall and (c) adhesive force on the base of the wall.

The passive force-displacement curve was computed using the hyperbolic curve approach proposed by Duncan and Mokwa (2001) where the curve is defined by the ultimate passive force and the initial stiffness. The initial stiffness was computed based on the geometry of the pile cap and the elastic modulus of the soil. For the cohesive soil at the test site the initial undrained elastic modulus, E_i , was estimated based on the equation

$$E_i \approx \frac{15{,}000 \cdot s_u}{PI(\%)} \tag{3}$$

where PI is the plasticity index in percent (≈25). The ultimate resistance was assumed to develop with a movement equal to 1.5% of the wall height. The computed passive force-displacement curve is also plotted in Fig. 10 relative to the measured force-displacement curve. At displacements greater than about 6 mm, the computed curve generally trends parallel to the measured curve. Passive force eventually provides about 73% of the measured increase in lateral resistance.

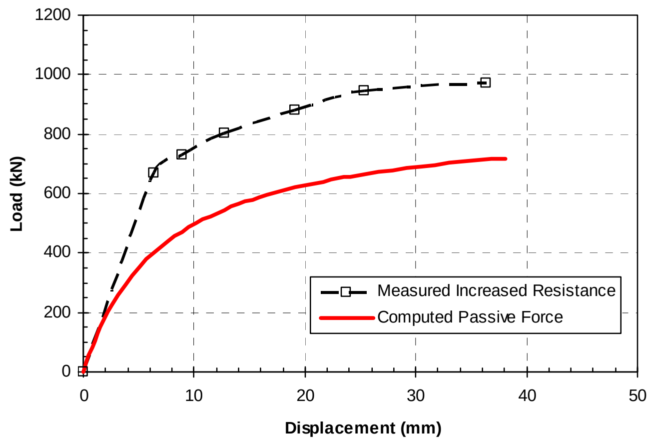

Fig. 10. Measured increased load vs. displacement curve relative to computed passive force vs. displacement curve using simple model for soil mixed wall.

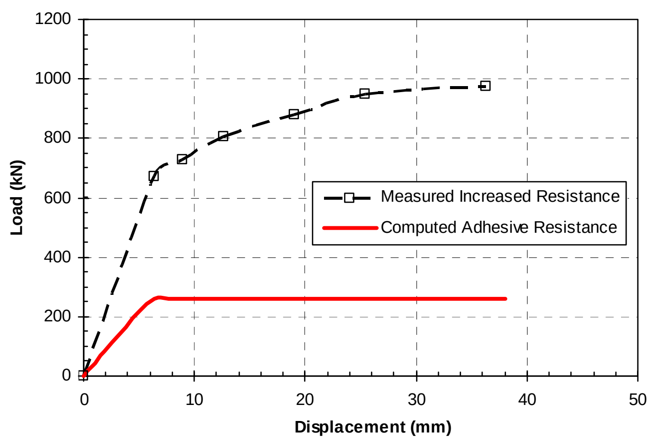

Fig. 11. Measured increased load vs. displacement curve relative to computed adhesive shear force vs. displacement curve using simple model for soil mixed wall.

The computed shear resistance versus displacement curve was computed by simply assuming that the shear resistance mobilized linearly and was fully mobilized at a displacement of 5 mm. This is consistent with observations from many investigators that side resistance is typically mobilized with small displacements on the order of 2.5 to 6 mm. The adhesive shear force-displacement curve is plotted in Fig. 11 in comparison with the total measured increase in lateral resistance. Adhesive shear force accounts for about 27% of the measured increase in lateral resistance.

The total lateral resistance-displacement curve was computed by summing the resistance due to both passive force and adhesive shear force on the soilcrete wall at each deflection level. The computed curve is compared with the measured increase in lateral resistance in Fig. 12 and the agreement is excellent. The excellent agreement strongly indicates that there was very little increase in lateral pile resistance due to the presence of the soilcrete wall in front of the piles despite the fact that the piles were only 0.3 m behind the soilcrete wall.

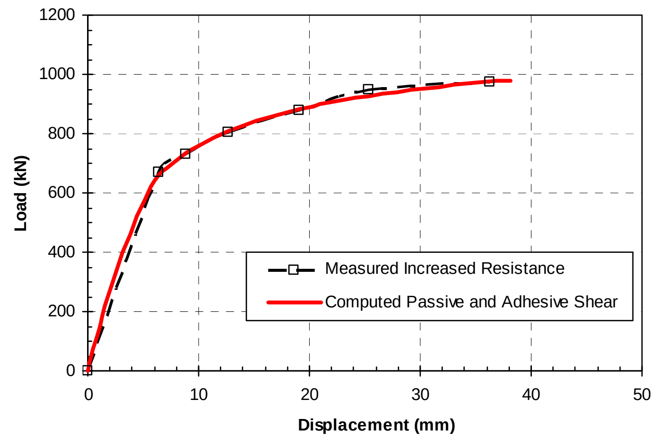

Fig. 12. Measured increased load vs. displacement curve relative to computed passive force and adhesive shear force vs. displacement curve using simple model for soil mixed wall.

Jet Grout Treatment

The same simplified analysis procedure used to compute the increased lateral resistance provided by the soil mixed wall was also employed to evaluate the increased resistance provided by the soilcrete block produced by jet grouting. Using this simplified analysis, passive force would provide 733 kN, side shear would provide 433 kN and base friction would provide 266 kN for the mean geometry and strength conditions. In contrast to the soil mixed wall, the adhesive force in this case is nearly as large as the passive force. This results from the longer length of the jet grout block relative to the thinner soil mixed wall.

As discussed previously, adhesive shear forces were assumed to fully mobilize at a displacement of 5 mm while displacements of something more than 50 mm would be required to develop the passive resistance with the hyperbolic model. These curves were then combined to produce the total computed force-displacement curve for mean conditions shown in Fig. 13. The increased lateral force-displacement curve measured during the testing is also shown in Fig. 13 for comparison. In this case, the computed force is only about 65% of the measured resistance.

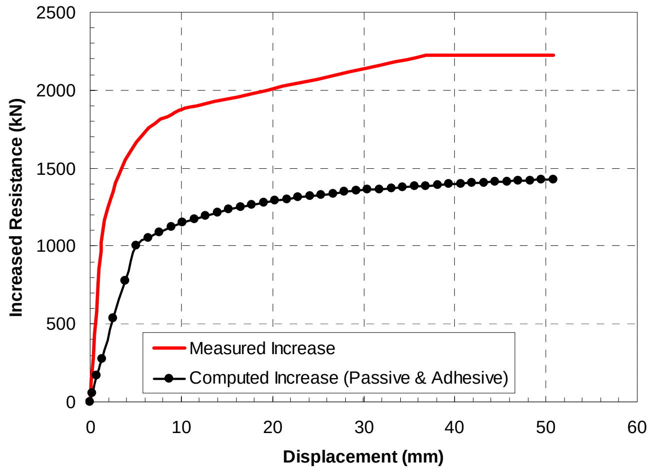

Fig. 13. Measured increased load vs. displacement curve relative to computed passive force and adhesive shear force vs. displacement curve using simple model for jet grout zone.

In addition to increased soil resistance, a large part of the increased lateral resistance could be a result of increased structural stiffness (EI) due to the composite of soilcrete and piles in the jet grout zone as illustrated in Fig. 14. To evaluate this factor, simple LPILE analyses were performed using a simplified equivalent single pile approach.

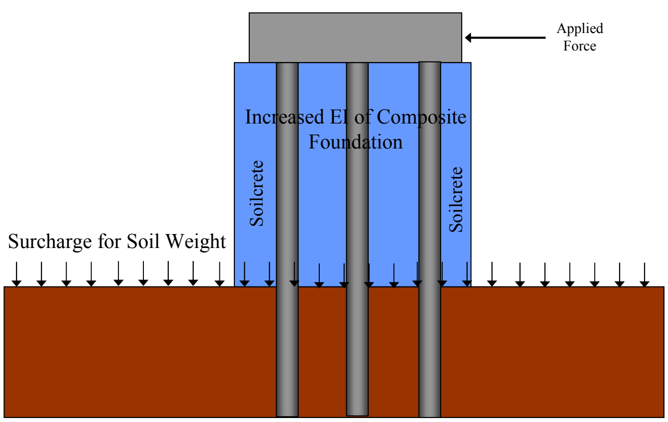

Fig. 14. Simple model to account for increased lateral resistance owing to increased structural stiffness of the composite soilcrete and pile group within the jet grout zone.

The EI for the soilcrete was combined with that of the nine pipe piles to provide a combined EI. The lateral resistance was then evaluated assuming that there was no lateral soil resistance on the side of the jet grout section. However, the vertical stress due to the weight of the soil around the pile group was applied to the underlying soil. The combined EI for the piles in the underlying soil was summed and the soil resistance was considered using the sum of the p-multipliers for the pile group as suggested by Juirnarongrit and Ashford (2006). The resistance provided by the increased EI was then combined with the resistance provided by passive force and adhesive soil resistance to obtain the total resistance.

The total computed force vs. displacement curve is plotted relative to the measured curve in Fig. 15. The agreement between measured and computed response is relatively good with this approach but somewhat conservative at smaller displacements. Some degree of conservatism is normally desirable for a simplified design model.

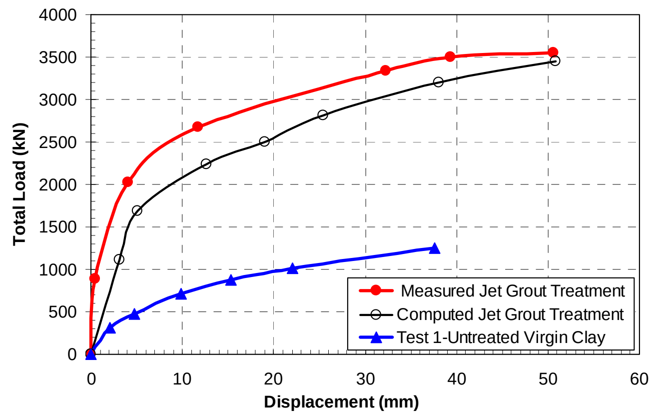

Fig. 15. Comparison of computed and measured load-displacement curve for pile group with jet grout treatment along with curve for untreated virgin clay.

COST CONSIDERATIONS

A complete cost assessment is beyond the scope of this paper; nevertheless, some rough assessments are possible. Based on the lateral load test in untreated soil (Test 2), the piles in the group carried an average load of about 111 kN at a displacement of about 40 mm. Therefore, 2 additional piles would provide more additional resistance (226 kN) than the 180 kN of increased resistance provided by excavation and replacement with compacted sand. Furthermore, an additional 7 piles would be necessary to provide the additional 760 kN produced by the soil mixed wall and 18 piles would be necessary to produce the 1950 kN of increased resistance provided by the jet grout treatment. In addition, a larger pile cap would be required for pile groups with additional piles. Considering the relatively modest increase in resistance provided by the compacted fill option, economics might not favor this approach. However, based on typical unit costs, the soil mixing and jet grouting alternatives would be significantly less expensive than the piling alternative neglecting mobilization costs (Adsero 2008, Herbst, 2008). Even considering mobilization costs, which are sometimes higher for jet grouting than pile driving, the total cost would still have been lower for the jet grouting alternative. Of course, mobilization costs become less important for larger projects, making jet grouting more cost-effective in these cases.

CONCLUSIONS

1. Excavation and replacement of about 1 m of clay with compacted sand (94% relative compaction) led to an 18% increase in lateral resistance of the pile group relative to the pile group in untreated soil.
2. 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 40 to 60 kPa to an average of 970 kPa while jet grouting with a cement content of about 400 kg/m^3 produced an average strength of 3170 kPa. This result is consistent with previous experience.
3. 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 65%.
4. The increased lateral resistance for the pile group with a soil mix wall can be explained with a simple model which accounts for passive resistance behind the soil mixed wall and adhesive shear resistance along the side and base of the wall as the pile cap pushed the soil mixed wall laterally. No appreciable increase in lateral resistance could be attributed to soil-pile interaction.
5. Construction of eight 1.5 m diameter jet grout columns around the nine pile group increased the lateral pile group resistance to 3475 kN relative to the 1253 kN resistance for the pile group in untreated virgin clay. This represents an increase in lateral resistance of about 180%.
6. Jet grouting treatment of the pile group also increased stiffness from 140 kN/mm to 700 kN/mm, an increase of 400%.
7. About 65% of the increase in lateral resistance for the jet grout treatment can be accounted for by passive force and shear resistance on the treated soilcrete block around the pile group. Preliminary analyses suggest that the additional resistance can largely be explained by an increase in the structural stiffness (EI) of the composite pile group and soilcrete block.
8. Soil improvement technique, such as 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 approaches that rely on adding additional structural elements.

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