USE OF JET GROUTING TO INCREASE LATEARAL PILE GROUP RESISTANCE IN SOFT CLAY

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ABSTRACT :
Lateral load tests were performed on a full-scale pile cap in clay before and after construction of a soilcrete wall on one side of the cap using seven 1.22-m m diameter jet grout columns to a depth of 3.76 m. Jet grouting with a cement content of about 400 kg/m^3 (20% by weight) increased the average compressive strength of a soft, plastic clay from 40 to 60 kPa to an average of 4500 kPa. The lateral resistance was increased by 1950 kN or 155% and the initial stiffness was increased by 400%. About 75% of the increased resistance could be accounted for by passive pressure and side/base shear on the jet grout mass; however, the remaining 25% increase must be due to the interaction between the piles and the strengthened soil. Jet grouting provides a method to significantly increase the lateral resistance of pile group foundations at costs much lower than typical structural approaches.

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

1. INTRODUCTION

The lateral resistance of pile groups in soft clay is often critical to the seismic design of bridges and high-rise structures. 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 provided 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 and thereby increase the lateral resistance of the pile group. The improved zone could be relatively shallow because the lateral resistance of piles is typically transferred within 5 to 10 pile diameters. This relatively simple approach has the potential of being more cost-effective and reducing construction time, but almost no 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 validated. To provide basic test data, full-scale lateral pile group load tests were performed on a nine pile group before and after treatment with jet grouting.

2. SOIL CONDITIONS

A generalized soil boring log at the test site is provided in Figure 1. The depth is referenced to the top of the excavation 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.

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 test or 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. The undrained shear strength was also computed from the cone tip resistance using the correlation equation

$$s_u = (q_c - \sigma)/N_k \tag{1}$$

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 Figure 1 and the agreement with the strengths obtained from the Torvane and unconfined compression tests is reasonably good. Nevertheless, there is much greater variability. The drained strength in the interbedded sand layers is not plotted. The CPT data, as well as 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 is typical of a soil profile with a surface crust that has been overconsolidated by desiccation as is the case in this situation.

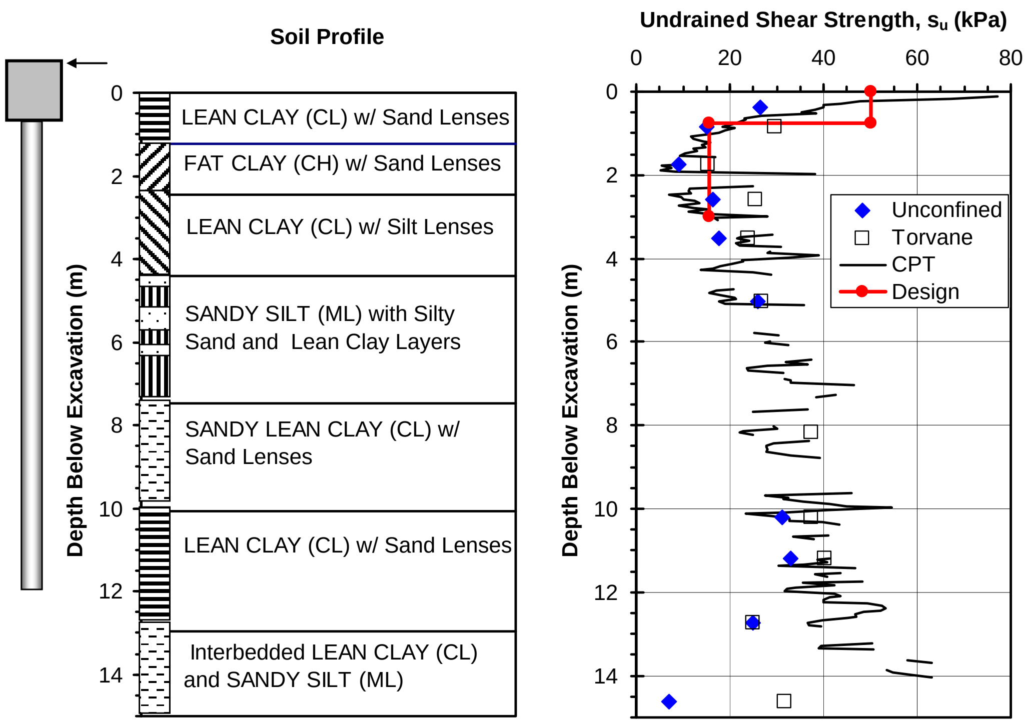

Figure 1 Soil profile and undrained shear strength profile for the test site.

3. PILE GROUP CHARACTERISTICS

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. The tests piles were 324 mm OD pipe piles with a 9 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 each 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 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.

4. TESTING PROCEDURE AND SEQUENCE

Lateral pile group load tests were conducted using a 2700 kN hydraulic actuator to apply load to the pile groups. A similar pile group provided a reaction for the applied load. The reaction group was 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 small displacement cycling at each increment, the pile group was pulled back to the initial starting point prior to loading to the next higher displacement increment.

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 treatment. Test 1 was conducted by pulling the pile cap 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 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 provide an indication of the passive force provided by the unsaturated clay soil against the pile cap.

Prior to Test 3 seven 1.22-m diameter jet grout columns were constructed on one side of the pile group as shown in Figure 2(b). The jet grout columns extended 0.76 m above the base of the pile cap to the ground surface and to a depth of 3.0 m below the base of the pile cap and. The jet grout columns created a soilcrete block adjacent to the pile group extending to the first row of piles with dimensions of approximately 2.13 m x 4.0 m in plan. The actuators then pushed the pile cap to the right. Comparisons between Test 3 and Test 1 in virgin clay make it possible to determine the improvement provided by the jet grout treatment.

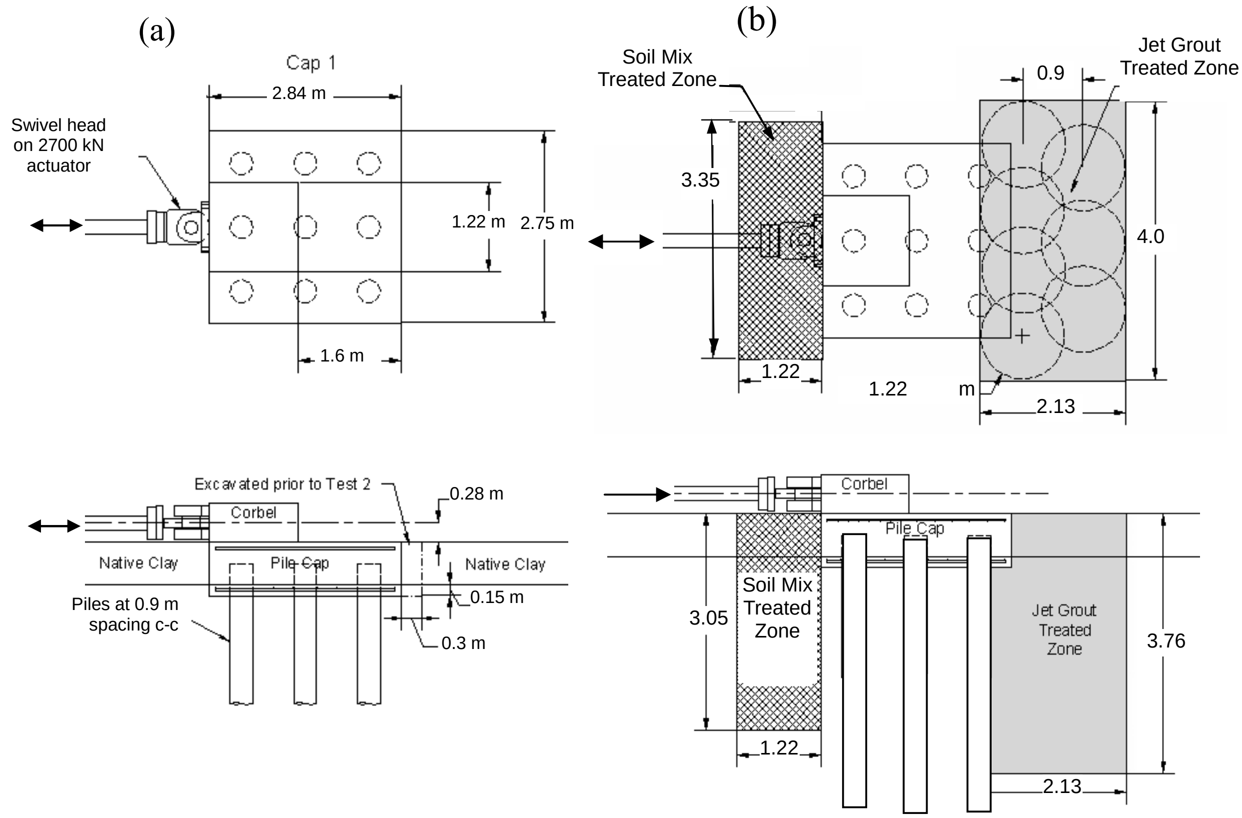

Figure 2 Plan and profile drawings of the pile group (a) in virgin clay before treatment and (b) after jet grout treatment.

5. JET GROUTING PROCEDURE

Plan and profile views of the jet grout column wall on one side of the pile cap are shown in Figure 2. 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, the grout columns formed a soilcrete wall with a width of 2.13 m, a length of 4.0 m and a depth of 3.76 m.

A 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 and 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 and 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 nozzles occurred for each 30 mm lift.

Table 1 Summary of jet grout treatment parameters.

Column Length	3 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.

6. LOAD TESTING RESULTS AND ANALYSIS

Figure 3 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

$$P_p = 0.5\gamma z^2 B + 2 s_u z B \tag{1}$$

based on Rankine theory for undrained conditions where γ = total unit weight of the clay = 18.37 kN/m^3, z = 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 tip resistance as shown in Figure 1.

Figure 3 also provides a comparison of the load-displacement curves for the pile cap during Test 1 (virgin clay) and Test 3 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. At a displacement of 18 mm, 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 has been extrapolated based on the slope of the curve and tests on similar pile groups treated with jet grouting (Adsero, 2008) and the curve is shown as a dashed curve in Figure 3. At a displacement of 38 mm, the jet grout wall is estimated to have increased the lateral resistance to 3200 kN from the 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 140 kN/mm while after jet grout treatment the stiffness increased to 700 kN/mm. This represents an increase in stiffness of about 400%.

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 calculation 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 based on the soil strength testing as shown in Figure 1. This approach can account for 1500 kN or about 75% of the increase in lateral resistance. The additional 25% of resistance must, therefore, be a result of more complex soil-pile interaction between the piles and the jet-grout strengthened soil. This interaction is currently being evaluated using finite element modeling techniques.

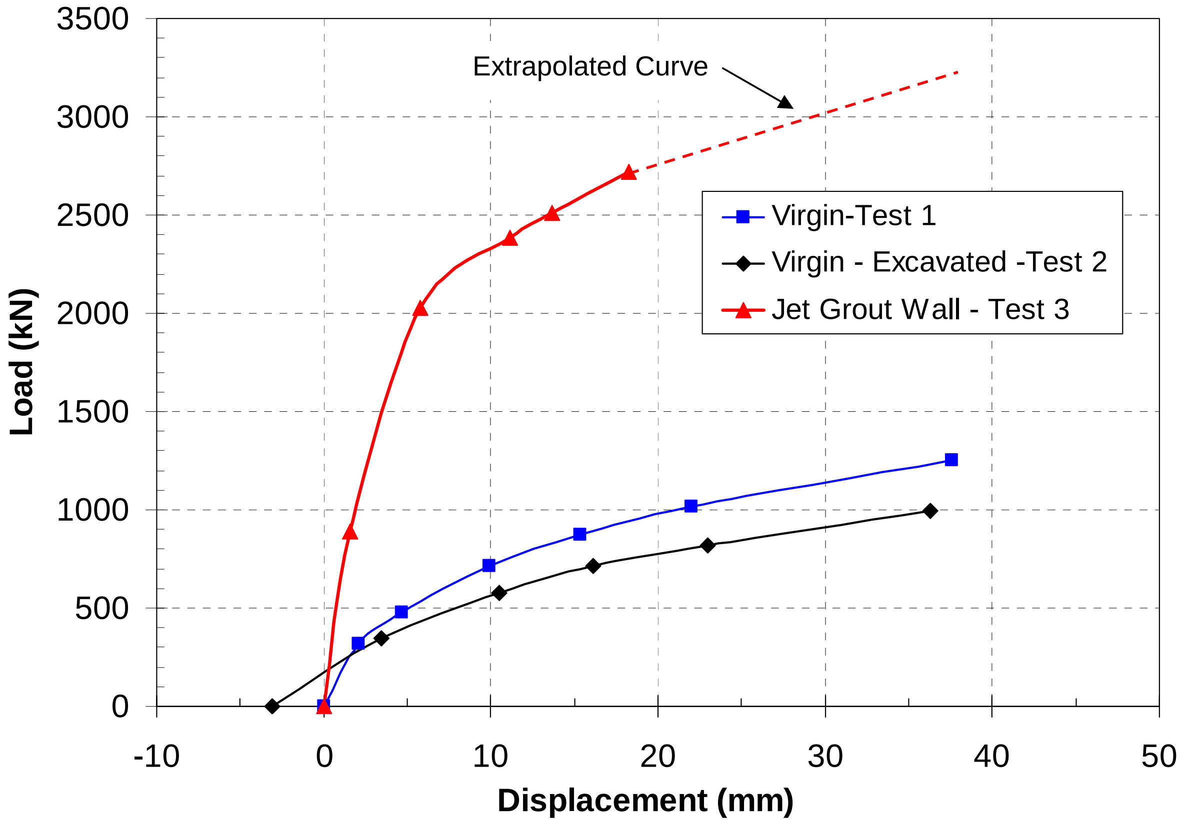

Figure 3 Lateral load-displacement curves for pile groups in virgin clay before and after excavation of the soil adjacent to the pile cap along with curve for clay treated with jet grout columns.

7. 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 113 kN at a displacement of 40 mm. Therefore, an additional 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 the pile group. Based on typical unit costs, the jet grouting alternative would be significantly less expensive than the piling alternative neglecting mobilization costs (Adsero, 2008). Even considering mobilization costs, which are typically 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.

8. CONCLUSIONS

1. Jet grouting with a cement content of approximately 400 kg/m^3 (20% 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 4500 kPa. This result is consistent with previous experience.
2. Construction of seven 1.22-m diameter jet grout columns on one side of the nine pile group increased the lateral pile group resistance to 2700 kN relative to the 950 kN resistance for the pile group in untreated virgin clay at a displacement of 18 mm. This represents an increase in lateral resistance of 184%.
3. Jet grouting treatment of the pile group also increased stiffness from 140 kN/mm to 700 kN/mm, an increase of 400%.
4. About 75% of the increase in lateral resistance can be accounted for by passive force and shear resistance on the treated soilcrete block around the pile group, while an additional 25% must be a result of increased soil-pile interaction.
5. Jet grouting provides an opportunity to significantly increase the lateral resistance of existing pile group foundations at a cost that is economically viable with alternatives such as deep foundations.

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