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Full-Scale Testing of Blast-Induced Liquefaction Downdrag on Auger-Cast Piles in Sand

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Full-Scale Testing of Blast-Induced Liquefaction Downdrag on Auger-Cast Piles in Sand

Joseph Erick Hollenbaugh

A thesis submitted to the faculty of
Brigham Young University
in partial fulfillment of the requirements for the degree of

Master of Science

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ABSTRACT

Full-Scale Testing of Blast-Induced Liquefaction Downdrag on Auger-Cast Piles in Sand

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Master of Science

Deep foundations like auger-cast piles and drilled shafts frequently extend through liquefiable sand layers and bear on non-liquefiable layers at depth. When liquefaction occurs, the skin friction on the shaft decreases to zero, and then increases again as the pore water pressure dissipates and the layer begins to settle, or compact. As the effective stress increases and the liquefiable layer settles, along with the overlaying layers, negative skin from the soil acts on the shaft. To investigate the loss of skin friction and the development of negative skin friction, soil-induced load was measured in three instrumented, full-scale auger-cast piles after blast-induced liquefaction at a site near Christchurch, New Zealand.

The test piles were installed to depths of 8.5 m, 12 m, and 14 m to investigate the influence of pile depth on response to liquefaction. The 8.5 m pile terminated within the liquefied layer while the 12 m and 14 m piles penetrated the liquefied sand and were supported on denser sands. Following the first blast, where no load was applied to the piles, liquefaction developed throughout a 9-m thick layer. As the liquefied sand reconsolidated, the sand settled about 30 mm (0.3% volumetric strain) while pile settlements were limited to a range of 14 to 21 mm (0.54 to 0.84 in). Because the ground settled relative to the piles, negative skin friction developed with a magnitude equal to about 50% of the positive skin friction measured in a static pile load test. Following the second blast, where significant load was applied to the piles, liquefaction developed throughout a 6-m thick layer. During reconsolidation, the liquefied sand settled a maximum of 80 mm (1.1% volumetric strain) while pile settlements ranged from 71 to 104 mm (2.8 to 4.1 in). The reduced side friction in the liquefied sand led to full mobilization of side friction and end-bearing resistance for all test piles below the liquefied layer and significant pile settlement. Because the piles generally settled relative to the surrounding ground, positive skin friction developed as the liquefied sand reconsolidated. Once again, skin friction during reconsolidation of the liquefied sand was equal to about 50% of the positive skin friction obtained from a static load test before liquefaction.

Keywords: liquefaction, downdrag, auger-cast piles, pile load test, settlement, drilled shafts
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1 INTRODUCTION

1.1 Problem Statement

Deep foundations are typically used to support bridge and high-rise structures when weak or liquefiable soils are encountered. Deep foundations can bypass liquefiable layers and bear in more competent strata at depth. Dead and live loads imposed on the pile foundation are typically resisted by positive skin friction acting on the side of the pile and by end-bearing resistance at the toe of the pile. However, when liquefaction occurs in a layer along the pile, settlement of that layer, and the associated movement of the soil above it, could exceed the settlement of the pile. If this is the case, the liquefied layer and the layers above it slide down relative to the pile leading to negative skin friction along that length of the pile, as shown in Figure 4.1-1.

Negative skin friction acting on the pile creates a “dragload” on the pile in addition to the permanent pile head load. The neutral plane is the depth where the settlement of the pile equals the settlement of the soil and also where the load in the pile is the greatest. Below the neutral plane, the positive skin friction and end-bearing pressure provide upward resistance which decreases the load in the pile. The elevation of the neutral plane is found by trial and error such that the applied load plus negative friction equals the positive friction plus end-bearing resistance. In addition, the end-bearing resistance mobilized must be consistent with the settlement of the pile toe. Thus, the location of the neutral plane creates a force equilibrium based on the soil settlement and the pile settlement.
In contrast to non-liquefiable layers, where the negative skin friction might simply be equivalent to the positive skin friction, the negative skin friction in liquefiable layers immediately following liquefaction is likely to be a very small fraction of the pre-liquefaction value or perhaps zero. However, as the earthquake induced pore pressures dissipate in the liquefiable layer, the skin friction at the pile-soil interface is likely to increase. Therefore, the negative skin friction which ultimately develops in liquefied layers might be related to the rate of pore pressure dissipation and the increase in effective stress.

In the absence of test results, some investigators have used theoretical concepts to predict the behavior of piles when subjected to liquefaction induced dragloads. Boulanger et al. (2004) defined negative skin friction in the liquefied zone in terms of the effective stress during

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Figure 1.1-1 Relationship between liquefaction induced settlement, positive and negative skin friction, and neutral plane

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reconsolidation, but concluded that the negative skin friction could be assumed to be zero with little error in the computed pile force or settlement. Fellenius and Siegel (2008) applied their “unified pile design” approach which was developed for downdrag in clays, to the problem of downdrag in liquefied sand, once again assuming that negative skin friction in the liquefied zone would be zero. They conclude that liquefaction above the neutral plane would not increase the load in the pile owing to the development of dragload under long-term static conditions prior to liquefaction.

To understand better the development of negative skin friction on piles in liquefied sand and the resulting pile response, a field testing program was undertaken using three 610 mm (24 inch) diameter continuous flight auger-cast concrete piles. Controlled blasting was employed in two tests first without load on the piles, and then with load on the piles. The blasts were intended to liquefy a 10 m layer of sand along the length of the pile. The axial load distribution along the length of the pile due to negative skin friction was measured after liquefaction. This report describes the test program, the test results, and implications for design practice based on analysis of the test results.

Piles must be analyzed to determine that the structural capacity of the pile is not exceeded and that the pile settlement is acceptable. In making these assessments, the dragload should be added to the dead load on the pile (Fellenius 1996, 2006). As discussed previously, the maximum pile load will occur at the neutral plane and the neutral plane is the location where the soil settlement and pile settlement are equal. Liquefaction induced soil settlement can be estimated using SPT or CPT based procedures (Tokimatsu and Seed 1987, Ishihara et al. 1990, Zhang et al. 2002).
1.2 Research Objectives and Scope

The objective of this research was to measure the magnitude of the skin friction on auger-cast piles during and after blast-induced liquefaction. Three instrumented, full-scale auger-cast piles were constructed at a test site where blast-induced liquefaction would occur. First, while the piles had no applied load, blasts caused liquefaction through a 9 m layer, causing negative skin friction in the piles. Approximately 6 weeks later, a static load test was performed on each of the piles to gather information about available skin friction and end-bearing. Immediately following the static load test, another blast caused liquefaction in a 6 m layer while the piles were loaded. The piles settled more than the surrounding soil, so no negative skin friction developed, but the magnitude of the liquefied positive skin friction was very similar to that of the negative liquefied skin friction from the first blast. Thus, via these three tests, a recommendation for the magnitude of negative or positive skin friction in liquefied sands is presented. Other considerations on the mechanics of end-bearing, installation process, and design are presented.

1.3 Outline of Report

This report contains six chapters. The first chapter contains a review of the current literature and design approach for liquefied skin friction. The second chapter explains the test layout for pile installation, instrumentation, and blasting procedures and the geotechnical setting and site characterization. The third chapter explains the results of the static load test and expounds on the soil-pile interactions related to determining the effect of liquefaction. The fourth and fifth chapters explain the soil-pile interactions during the period following the first blast, in which the piles had no applied load, and the second blast, in which the piles had significant applied load. The sixth and final chapter offers a summary and conclusions about liquefied skin friction on drilled shafts.
LITERATURE REVIEW

2.1 Overview

There have been several publications regarding the evaluation of skin friction against piles, typically driven piles, under static conditions, but very limited research has been published about the value of skin friction during or after a liquefaction event. There is some disagreement in these publications regarding the appropriate approach for the design of piles, governing failure mechanisms, and other considerations, which will be discussed.

2.2 Current Research

Fellenius (2008) has presented several ideas related to downdrag, some of which are related to liquefaction-induced downdrag. One of the important ideas presented by Fellenius is that all driven piles in soft and loose soils will experience dragload even in static conditions. He suggests the idea that the negative skin friction develops around segments of the pile in soft or loose soils because the pore pressure build up around the pile during installation and later dissipation after installation. Therefore, the pile will develop a neutral plane, where the settlement of the pile and the soil profile are equivalent, and where the maximum load in the pile will be. Also, there will be positive skin friction below this neutral plane, and end-bearing mobilization associated with the actual static pile settlement. The difficulty with some of these assumptions is that there must be a cumulative profile settlement, but in this case it is only along the pile shaft.
Figure 2.2-1 shows an example of the static load distribution of a pile in these conditions. The case studies associated with this idea were typically long-term, static conditions (Fellenius 2006). A typical cumulative settlement profile is developed from compaction of some layer. However, these dragload forces seem to be developed by the settlement of a small radius of soil directly around the pile due solely to installation of the pile. Therefore, the assumption is that as the soil around the pile moves downward, dragload starts to cause the pile to settle. The end-bearing responds by developing force beyond what it developed under the applied static load. From the base up to the neutral plane, the pile settles more than the soil, and there is positive skin friction. From the neutral plane to the surface, the soil settles more than the pile, and there is negative skin friction. It is as if the settlement were occurring throughout the spread of the layers, but it is not.
Static load tests performed in association with this report do not show any static dragload. It is possible, however, that settlements around the pile could occur over a longer period of time. It is also possible that they will not occur because there is no driving involved in the installation of an auger-cast pile, and therefore no increase in pore water pressure.

Fellenius presents some other ideas that are useful in the discussion of pile design, assuming that the static condition is like that portrayed in Figure 2.2-1. Figure 2.2-2 shows how the pile might react if liquefaction occurs in a layer above the neutral plane. There will be a temporary loss of friction, in this case negative friction, in the layer that liquefies. According to Fellenius, the decrease in dragload associated with this reduction in negative skin friction causes the neutral plane to move downward. However, the implications for the settlement suggest that this is not true, and that Figure 2.2-2 is not accurate. Because there is no movement by the pile or by the soil below the neutral plane, the neutral plane should not move down. A lower neutral plane would mean that the pile settles less. Rather, the neutral plane remains at the same depth and the same positive friction exists below it. The reduction in dragload then translates directly into a reduction of end-bearing, and although end-bearing is also movement dependent, no movement is actually occurring. Thus, the end-bearing would be less than it had mobilized previously. Rebounding upward movement of the soil under the pile base because of this reduction in end-bearing is not likely to create the upward movement of the pile sufficient to cause a section of the skin friction to switch from negative to positive and therefore cause the neutral plane to move downward. Regardless, this situation is not critical. And, eventually, it is reasonable to assume that the liquefied layer would re-mobilize friction, probably negative as Fellenius suggests, which would raise the neutral plane back to its previous depth.
The other case considered by Fellenius is when the liquefied layer occurs in an isolated layer beneath the neutral plane. Apparently, according to Figure 2.2-3, the liquefied layer is able to produce friction. This is not explicitly stated, but is suggested in the drawing. However, there is some ambiguity in the drawing around the liquefied zone. To suggest that the neutral plane goes to the bottom of the liquefied layer, with no friction in that layer, is to say there is a neutral zone (the liquefied layer) rather than a neutral plane, at least until friction begins to develop. As the pore pressures dissipate over time, the outcome is that the liquefied layer produces dragload, and that settlement is occurring in and above the liquefied layer. The increasing soil settlement lowers the neutral plane, which at the same time increases the toe-resistance and therefore the pile settlement. These two forces again come into equilibrium, and Fellenius suggests that the neutral plane will move to the bottom of the liquefied layer, in other words, that the additional settlement throughout

Figure 2.2-2 Load vs. depth for a driven pile with liquefaction above the neutral plane, Fellenius (2008)
the liquefied layer exceeded the additional pile settlement. This is not necessarily true because the pile settlement may exceed the incremental settlements at the bottom of the liquefied layer, causing positive skin friction near the bottom of the liquefied zone, and thereby moving the neutral plane into the liquefied zone. Figure 2.2-3 demonstrates this interaction according to Fellenius. In any case, the conservative, governing scenario is that in which liquefaction occurs below the neutral plane and causes dragload to the lower boundary of the liquefied layer, and increased settlement.

![Figure 2.2-3 Load vs. depth for a driven pile with liquefaction below the neutral plane, Fellenius (2008)](image)

Some ambiguity remains in regards to how much dragload is created in the liquefied zone, if any at all. In the case with liquefaction above the neutral plane, there was no dragload from the liquefied layer, but there apparently is dragload when the liquefied layer is below the neutral plane. We can assume that these layers settle equally, but this settlement only affects the neutral plane when it is below the neutral plane. As is clear from the examples, changing the equilibrium in the
pile due to liquefaction is complex and it is important to understand how dragload might develop in liquefied layers.

The AASHTO LRFD Bridge Design Specifications (AASHTO, 2014) says little about the design of piles in consideration of liquefaction. Essentially, the pile is to be designed with load and resistance factors such that the positive friction in the pile and end-bearing at the toe can resist the applied load and any potential dragload. It does not specify by what mechanism this dragload would occur, whether it is a long-term development as Fellenius suggests, a consolidation-related development, or liquefaction-compaction mechanism. In any case, the dragload is to be applied down to the bottom of the lowest settling layer. There are two large flaws with this simplified method, both of which are briefly address by Fellenius, and which will be expounded on here.

First, using factored loads and resistance is fundamentally inaccurate. The ties between positive and negative skin friction, the neutral plane, the end-bearing, and the settlement necessitate using unfactored design. Factoring the loads creates false settlements, and false neutral planes, and gives an incorrect description of how the pile will react. Rather, safety can be increased by lowering the design neutral plane (decreasing settlements).

The second flaw is that this very simple AASHTO method does not provide information about the anticipated settlement. It is quite possible for settlement to occur below the neutral plane (the pile would still be settling more than the layers below the neutral plane). The relative settlements are an integral part of this interaction of the soil and the pile, and must be considered.

Boulanger and Brandenberg (2004) commented more specifically on the magnitude and development of dragload against the shaft. They describe an equilibrium that adjusts through time, rather than an equilibrium based on “end-of-consolidation” conditions. This is known as the modified neutral plane solution. Their method accounts for changes over time in excess pore
pressure, the sand compressibility, and the dependence of friction on these two parameters. The main difference in this article is the conclusion that soil settlements may not equal pile settlements at the neutral plane at the end of consolidation. The pile is likely to have settled much less than the soil at the neutral plane because the neutral plane moves upward through the time of consolidation. Looking at one short time increment, the settlement of the pile equals the settlement of the soil at the neutral plane. However, because the neutral plane was moving from low to high, the soil at its finally location was experiencing high settlements the entire duration of consolidation, whereas the neutral plane experienced incrementally higher settlements (those of the deeper layers in which it originated were smaller). They conclude that pile settlement will be small if the upward resistance from the pile and soil below the liquefied layer are larger than the applied service load and dragload from the crust. They also conclude that downdrag loads from within the liquefied layers do not need to be included when evaluating the potential for pile settlement.

Rollins and Strand (2006) induced liquefaction in a 7 m layer of sand surrounding a 324 mm steel pipe pile using an array of small explosive charges. Pore pressure transducers in the sand indicated an excess pore pressure ratio of 1.0 and that liquefaction induced settlement exceeded 11 inches. Strain gauges on the test pile indicated that skin friction on the pile in the liquefied layer, which were positive prior to liquefaction, quickly decreased to approximately zero following liquefaction. However, as excess pore pressure dissipated and the sand settlement relative to the pile, the negative skin friction increased to approximately 50% of the positive skin friction prior to liquefaction, as shown in Figure 2.2-4. These results suggest that skin friction in liquefied sand should not be considered to be zero but that the magnitude of skin friction against steel piles might be proportional to the average excess pore pressure during reconsolidation, or 50% of the typical non-liquefied values.
Stringer and Madabhushi (2013) performed centrifuge tests and measured changes in magnitude in both positive and negative skin friction against piles groups with a focus on the influence of pile caps that bear on the soil and change the axial loading of the piles. Specifically, they note that as a liquefied layer starts to settle, the skin friction in that layer switches from positive to zero and then to negative friction. They also discuss the influence of hydraulic conductivity of different layers upon the development of dragload.

Knappett and Madabhushi (2008) also performed centrifuge tests and commented particularly on the changes in end-bearing with an increases in excess pore pressure in the bearing stratum. They conclude that increases in excess pore pressure in the bearing layers result in a loss of end-bearing capacity during and immediately following an earthquake event. They also
measured dragload on several piles and found the average magnitude of dragload to be very similar to that measured by Rollins and Strand (2006).

For most of these publications, design methods and dragload magnitudes were measured in driven steel piles. There is no question that downdrag occurs, but there is some question as to when it develops in relation to a liquefaction event or otherwise, and how to predict the magnitude of dragload. It would also be helpful to have some comparison to the performance of auger-cast piles, a type of drilled shaft, in relation to downdrag.
3 SITE CHARACTERIZATION AND TEST LAYOUT

3.1 Geotechnical Site Conditions

The test site was located in Avondale near the Avon River in Christchurch, New Zealand, as shown in Figure 3.1-1. This area experienced significant liquefaction settlement (0.3 to 1.0 m) during the Christchurch earthquake sequence in 2010-2011 and most homes in the area had been condemned. In connection with this study, site characterization, consisting of cone penetration tests (CPT), standard penetration tests (SPT), shear wave logging, and undisturbed sampling, was performed by Tonkin and Taylor in association with the Earthquake Commission in New Zealand. The compression and shear wave velocity measurements were made by Brady Cox (University of Texas) using a cross-hole seismic cone penetrometer.

Prior to installation of the test piles, two CPT soundings were performed with a piezocone at the location of the future test pile to define the soil profile. An SPT boring was also performed within about 3 meters of the piles after pile installation. The SPT was performed using the sonic drilling system, and some photographs of the SPT are included in Figures 3.1-2, 3.1-3, and 3.1-4. The SPT testing involved an automatic hammer delivering 85% of the theoretical free-fall energy. Corrections to obtain \((N_{ij})_{60}\) and \(q_{c1}\) were made using the procedures outlined by Youd et al. (2001). Plots of the CPT and SPT test results and are provided in Figure 3.1-5.

In general, the soil profile consists of three major units. The top unit, approximately 1.5 m thick, consists of sandy silt. The second unit, approximately 9 m thick (from 1.5 to 10.5 m depth),
consists of poorly graded medium-dense clean sand. The third unit, at least 6 m thick (10.5 to 14.6 m depth) consists of inter-bedded layers of medium-dense clean sand and dense clean sand. The measured fines contents for each unit are also shown in Figure 3.1-5. In unit 1, the fines were greater than 50%, but within unit 2 the fines content was typically between 2% and 8.5%. Groundwater fluctuated with tides but was typically about 1.5 m below the ground surface during the pile load tests. However; p-wave velocity testing conducted by Cox and Roberts (personal communication) suggest that the soil was not fully saturated until a depth of 2.5 to 3 m.

Figure 3.1-1 Avondale, Christchurch, New Zealand near the test site location
Figure 3.1-2 Photograph of extracted sonic drilling borehole sample
Figure 3.1-3 Photograph of sonic borehole machinery
Figure 3.1-4 Photograph of a typical profile sample of very fine saturated sand
Relative density \((D_r)\) in percent was estimated from the cone tip resistance measurements using the correlation equation developed by Kulhawy and Mayne (1990) assuming young normally consolidated, clean sand conditions.

\[
D_r = \left( \frac{q_{c1}}{310} \right)^{0.5}
\]

The relative density was also estimated based on SPT blow count using the equation

\[
D_r = \left[ \frac{(N_1)_{60}}{60} \right]^{0.5}
\]

from Kulhaway and Mayne (1990). The relative density profile from both the CPT and SPT are shown in Figure 3.1-5 and the agreement is very good. The average relative density is approximately 60% and 70% in units 2 and 3, respectively.

\(I_c\) values greater than 2.6 are generally considered to indicate non-liquefiable cohesive soil (Youd et al. 2001). \(I_c\) values from the CPT measured 2.6 only at depths of 0 m to 0.1 m and from 10.9 m to 11 m but the intervals where this occurred are negligible. The average \(I_c\) value in unit 1 was 2.1.

Shear wave velocity measurements show an average shear wave velocity \((V_{s1})\) of 190 m/s and 250 m/s in units 2 and 3, respectively, as shown in Figure 3.1-5. Based on correlations developed by Andrus and Stokoe (2000) this would indicate that unit 2 was susceptible to liquefaction while unit 3 would not be expected to liquefy.

Seismic dilatometer testing was also performed at the site near SPT location, and the resulting data are shown in Figure 3.1-6, and were provided by Sara Amoroso (Istituto Nazionale di Geofisica e Vulcanologia, L’Aquila in Italy, personal communication).
Figure 3.1-5: CPT, SPT, Vs, and fines content, and generalized soil profiles for the test site.
Figure 3.1-6 Material index, constrained modulus, horizontal stress index, shear wave velocity, coefficient of lateral earth pressure, corrected shear wave velocity, and relative density as found through seismic dilatometer measurements.
3.2 Test Pile Properties and Instrumentation

The test piles were constructed with lengths of 8.5 m, 12 m, and 14 m, and consisted of high-strength concrete with a nominal diameter of 610 mm and a steel reinforcing cage extending the full length of the shaft. The reinforcing cages consisted of six 25 mm diameter bars with a spiral bar having a pitch of 30 cm (12 in). A photograph of one of the cages is shown in Figure 3.2-2. Test cylinders indicate that the average compressive strength of the concrete was approximately 38 MPa (5500 psi). The piles were installed using a continuous flight auger with a hollow tube center. As the auger was withdrawn, concrete was pumped through the hollow-tube to the bottom of the auger. Once the auger was removed, the reinforcing cage was lifted vertically and lowered into the wet concrete with a vibratory hammer. This process is shown in Figures 3.2-3 and 3.2-4. Using this installation method, it is typical for the volume of the concrete pumped to exceed the anticipated volume of the hole by about 30%. This was case on all three of the holes for this test. The reason for this common excess is unknown.

Prior to installation, each of the reinforcing cages were instrumented with strain gauges at approximately 1.5 m intervals along the pile length to a depth of about 0.5 m from the bottom of the pile. Each electrical resistance gauge was careful installed on a 0.45-m (18-inch) length of reinforcing bar known as a “sister bar” which was then tied to the reinforcing cage. At each depth interval, two strain gauges were applied to the pile at 180° spacing around the circumference of the test pile, as shown in Figure 3.2-1 and 3.2-2.

To determine the cross-sectional area of the shaft versus depth, each test pile was also instrumented with four thermal gauge wires developed by Pile Dynamics, Inc. These thermal gauges, spaced at 30 cm along each wire, as shown in Figure 3.2-5, use the heat of concrete curing to indicate the pile shape throughout the length of the pile. Figure 3.2-6 shows the thermal gauge
recording units which remain attached throughout the curing period. In contrast to a cross-hole sonic approach, the thermal system can define the geometry of the shaft beyond the diameter of the reinforcing cage. Thermal methods for shaft integrity have been shown to be a reliable method for estimating the shaft shape (Mullins and Kranc, May 2007). The shaft radii (in inches) as a function of depth (in feet) for each test pile are presented in Figure 3.2-7 along with an average curve. A similar chart, from the thermal imaging software, is shown in Figure 3.2-8. The general consistency of the radii from each wire relative to the average indicates that the pile diameter is relatively consistent with depth with no significant anomalies present. As shown in Figure 3.2-7, the diameter of the shafts typically fluctuates between 0.69 and 0.79 m. For the 8.5 and 14 m piles, the diameter is about 15 to 20% greater than the design diameter of 0.61 m (24 in.) while the diameter of the 12 m shaft is quite close to the specified diameter.

Figure 3.2-1 Photograph of a sister bar tied to the reinforcing cage
Figure 3.2-2 Photograph of the instrumented reinforcing cages used in the three auger-cast piles
Figure 3.2-3 Photograph of the auger pulling soil out as it is drilled
Figure 3.2-4 Photograph of inserting one of the cages into the concrete mix
Figure 3.2-5 Photograph of the spacing of thermal gauges tied to the reinforcing cage
Figure 3.2-6 Photograph of the thermal gauge recording units
Figure 3.2-7 Shaft diameters as recorded by the thermal gauges
3.3 Test Layout and Instrumentation

Plan and profile views of the layout of the test piles relative to the blast holes and instrumentation are shown in Figure 3.3-1 and 3.3-2. The test piles were surrounded by a ring of blast holes having a radius of 5.0 m. The piles were 2 m from the center of this ring and 2.3 m from one another. Eight blast holes were distributed equally around the circumference of the ring at 45°, with an offset of 22.5° for a second blast, as shown in Figure 3.3-2. In each blast hole, 1.1 and 2.7 kg charges were located at depths of 4.0 and 8.5 m, respectively, below the ground surface as shown in Figure 3.3-2. The eight explosive charges at 8.5 m were detonated sequentially.

Figure 3.2-8 Shaft diameters as displayed in thermal gauge software, 8.5m left, 12m center, 14m right
followed by the eight charges at 6.4 m, each at 300 millisecond intervals. This configuration was selected after a pilot blast liquefaction trial which indicated that these charges would liquefy the soil out to a distance of about 8 m from the center of the blast ring.

Figure 3.3-1 Plan view of the test site
The ground settlement as a function of depth was monitored using a “Sondex” profilometer located at the center of the test area as shown in Figure 3.3-2. The profilometer consisted of a flexible corrugated pipe with stainless steel rings attached at approximately 0.6 m intervals along its length. Assuming that the corrugated pipe moved along with the surrounding soil, a probe was lowered down a PVC access pipe inside the corrugated pipe to monitor the change in elevation of the steel rings to indicate change in elevation of the soil at each measurement level. The generation and dissipation of excess pore pressure during the blasting process was monitored using six pore pressure transducers, or piezometers.
The transducers were capable of withstanding maximum blast pressures of 20.7 MPa (3000 psi) associated with the blast but were also capable of recording residual pore pressures to an accuracy of about 1.4 kPa (0.2 psi).

Piezometers were installed at depths of 2.75, 4.85, 6.8, 9.7, 12.8, and 15.85 m below the ground surface as shown in 3.3-2. The piezometers were typically located about 1 m from the center of the blast ring. Additional details regarding the installation procedures are provided elsewhere (Rollins et al. 2005, Ashford et al. 2004).

The first test to be performed at the site was a blast-liquefaction test in which there was no load on the piles. Almost two months later, a static load test was performed, and the following day another blast-liquefaction test was performed with significant load still on the piles. However, the tests are presented in a different sequence: static load, first blast, second blast. This order is helpful in presenting and understanding the results of the tests.
4 STATIC LOAD TEST

4.1 Overview

Prior to liquefaction blasting with loaded piles, but after liquefaction blasting without load on the piles, static load testing was performed. The intent of the static load testing was to provide information on the actual capacity of each pile in side resistance prior to liquefaction and end-bearing capacity. As with all deep foundations, there is considerable uncertainty about the axial capacity of auger-cast piles. The pile group was loaded using a rectangular steel frame (3 m x 4.5 m) and 272 metric tons (600 kips) of steel and concrete block weights, as shown in Figures 4.1-1 4.1-2, and 4.1-3. The rectangular frame rested on the three piles with a load cell installed between the frame and each pile. This is shown in more detail in Figure 4.1-4. Pile head settlement for each test pile was monitored by three, string potentiometers attached to separate independent reference frames for each pile. Weights were added to the frame and distributed in a manner to concentrate loads first towards the 8.5 m pile and then towards the 12 m and 14 m long piles, as shown in Figure 4.1-3. Loads were typically held in place for 3 to 5 minutes before the next load was applied. This testing approach was adopted to maximize load-displacement data regarding all test piles within the constraints of the project budget. A conventional load test on one test pile, with a load frame and reaction piles, would have expended the budget while providing data on only one test pile. In this case, the weights were sufficient to fully mobilize the resistance of the 8.5 m pile,
but only provided load-deflection data to about 12 mm (0.5 inch) deflection for the 12 and 14 m piles.

Figure 4.1-1 Plan drawing of the loading frame

Figure 4.1-2 Photograph of the loading frame positioned over the piles
Figure 4.1-3 Static load test on the 8.5 m pile showing eccentric loading
4.2 Results from Static Load Test

Load deflection curves from the static load test for all three of the piles are shown in Figure 4.2-1. The 8.5 m pile was loaded to failure at 125 metric tons (276 kips) and settled 33 mm (1.3 in) under the maximum load. This displacement represents nearly 5% of the measured pile diameter which is typically adopted as the displacement necessary to develop ultimate end-bearing resistance in U.S. practice (O’Neill and Reese, 1999). For the final load increment, the load-displacement curve was nearly vertical suggesting that a plunging type failure was imminent. Because this was a load controlled test, no further load increments were applied to the 8.5 m pile.
based on safety considerations and the load on this pile was actually reduced prior to additional loading of the other piles.

The 12 and 14 m piles were loaded to 127 metric tons (280 kips) each and both settled 12 mm (0.45 in). At this displacement, the ultimate side resistance for both piles would likely be almost fully mobilized (Kulhawy and Hirany, 1989; Hirany and Kulhawy, 2002). In contrast, at this displacement the mobilized end-bearing resistance was likely only 30% to 40% of the ultimate resistance defined by displacement equal to 5% of the pile diameter (Reese and O’Neill, 1988). Despite the fact that the 14 m pile was bearing in denser sand, had a larger base diameter, and a larger shaft surface area than the 12 m pile, the two load deflection curves were nearly identical.

Figure 4.2-1 Load deflection curves and predictive load deflection curves
This result indicates that the unit side resistance and end-bearing resistance for the 14 m pile, as constructed, are actually less than for the 12 m pile.

The method of installation of auger-cast piles may cause differences in the concrete quality, the soil density, side friction, or the shaft shape at the toe of the pile. For example, when the 14 m pile was constructed, the trap door on the auger failed to open during the initial penetration of the auger. Therefore, the auger was extracted by reversing the rotation, the trap door was reset, and the hole was re-drilled to the desired depth where the concrete was injected. This re-drilling process, which was occurred for the 8.5 m pile, apparently reduced the unit side resistance and end-bearing resistance.

To attach the load frame to the test piles and provide a level seat for the load cell, it was necessary to drill four holes into the top of each test pile and insert steel dowels into the piles, as shown in Figure 4.1-4. During this process, the lead wires for the strain gauges on the 8.5 m pile were damaged. Although it initially appeared that damage was of limited extent, subsequent analysis of strain gauge readings indicates that they are inaccurate and unreliable. Therefore, plots from the static load test and from the second blast test showing loads within the 8.5 m pile will not be presented.

Plots of load in the pile versus depth are useful in evaluating load transfer associated with side shear and end-bearing. To convert strain in the pile to load in the pile, \( P \), the following equation is used

\[
P = \varepsilon E_s A
\]

(3)

where \( \varepsilon \) is average strain, \( E_s \) is the secant modulus, or modulus of elasticity, and \( A \) is the cross-sectional area of the pile. Average strain was measured by two strain gauges on the rebar cage in
the concrete and cross-sectional area was measured using the average shaft diameter at the depth of the strain gauge from the thermal gauges as discussed previously.

In contrast to steel piles where the elastic modulus is constant, the elastic modulus for a reinforced concrete shaft is non-linear and decreases with strain (or load) level. For the large stress range imposed during a static load test, the difference between the initial and final moduli can be substantial and must be correctly represented. Fellenius (2001) proposed a procedure to compute the modulus as a function of strain based on the applied pile head load and measured strain on a pile in-situ. This procedure is based on the fact that, after the side shear on a shaft is fully mobilized and no longer changes with loading, the change in stress divided by the change in strain at the plane of the strain gauges is equal to the modulus of the shaft at that load or strain (Hayes and Simmons, 2002).

Experience suggests that the strain level must be greater than at least 50 με in order for the side shear to be largely mobilized so that computed modulus is not affected. As the strain level increases the modulus becomes less affected and the best-fit line at higher strains can be extrapolated back to lower strain levels.

The tangent modulus, $M_t$, at a given strain can be found using the equation,

$$M_t = \frac{\Delta P}{A(\Delta \varepsilon)}$$

(4)

where $\Delta P$ is some small change in applied load, $A$ is the cross-sectional area of the pile, and $\Delta \varepsilon$ is the change in strain resulting from the change in load. For the 14 m pile, a plot of $M_t$ against the strain level is displayed in Figure 4.2-2. Note that the scatter in the modulus is from deep gauges at low strains where side resistance has not yet been fully mobilized, as explained previously, and can be neglected. To partially deal with the scatter, the trend line is based only on those points where strain exceeds 75 με. As explained by Fellenius (2001), the trend line is correlated with the
secant modulus of the concrete, which is the modulus used to convert strain in the concrete to load in the concrete. Thus, the secant modulus, $E_s$, is estimated by the equation

$$E_s = 0.5A\varepsilon + B$$

(5)

where $A$ is the slope of the tangent modulus line, $\varepsilon$ is the strain in microstrain, and $B$ is the y-intercept of the tangent modulus line. For the 14 m pile, this equation becomes

$$E_s = 0.5(-0.051)\varepsilon + 19$$

(6)

Equation (6) was used for the 14 m pile when converting strain data into load data. A similar equation was used for the 12 m pile, but the slope used was -0.031.

Figure 4.2-2 Variation of tangent modulus with strain based on measurements from the 14 m pile
Based on the static load testing, plots of the measured load in the pile versus depth for both the 12 and 14m piles are provided in Figure 4.2-3. Separate curves are plotted for approximately 20% increments of the maximum static load in each case. In each of these plots, the load at the top of the pile was measured by the load cell, while load in the pile was interpreted from the strain gauge readings. As applied load increased, the slope of the load versus depth curves eventually reached a maximum as side resistance became mobilized. In addition, as the applied load increased, the end-bearing defined by the load at the bottom of the curve gradually increased.

To provide a framework for investigating the measured side friction and end-bearing resistance, the measured values were compared with the resistance computed by the FHWA equation for drilled shafts capacity (O’Neill and Reese, 1999). The FHWA approach has often been used as a point of reference in understanding the behavior of auger-cast piles (Coleman and Arcemont, 2002). However, the unit side resistance of auger-cast piles has often been found to be lower than that for drilled shafts in sand. A complete understanding of the capacities of the test piles actually requires an analysis of the no-load blast test, the loaded blast test, and the static load test combined so that the results can be interpreted consistently.

According to the FHWA method, unit skin friction, $q_s$, is given by the equations

$$q_s = \beta \sigma'_{v}$$

(7)

$$\beta = 1.5 - 0.135z^{0.5}$$

(8)

where $z$ is the depth in feet to the center of a layer and $\sigma'_{v}$ is the vertical effective stress. $\beta$ is dimensionless coefficient which represents the value of $K\tan\delta$, where $K$ is the lateral earth pressure coefficient and $\delta$ is the interface friction angle between the pile and surrounding soil.
Figure 4.2-3 Progression of load in the 12 and 14 m piles during the static load test
The total side resistance, $Q_{s\text{-max}}$ is given by the equation

$$Q_{s\text{-max}} = \sum q_{si} A_{si}$$  \hspace{1cm} (9)

where $q_{si}$ and $A_{si}$ are the unit side resistance and the surface area or the shaft, respectively, for each layer $i$ along the length of the shaft.

$Q_{b\text{-max}}$ is generally assumed to occur at a settlement equal to 5% of the base diameter, $B$, and can be estimated by

$$Q_{b\text{-max}} = 0.6 \times N_{60} \times A_{b}$$  \hspace{1cm} (10)

where $N_{60}$ is the SPT N value obtained with a hammer energy of 60% within a depth range of $1B$ to $2B$ below the base of the shaft and $A_{b}$ is the area of the base of the shaft.

The FHWA equations (7) and (8) estimate unit skin friction values which are much higher than that measured in the static load test. Zelada and Stephenson (2000) reported that the O’Neill and Reese (1999) method might often overestimate the friction capacity of auger-cast piles. They reported that the ratio of measured ultimate friction capacity to the capacity predicted by O’Neill and Reese (1999) was 0.92 on average with a standard deviation of 0.41. By trial and error, the skin friction predicted by the FHWA method was scaled down to produce improved agreement with the measured skin friction. As shown in Figure 4.2-4, the best agreement with measured load versus depth curves was obtained using scaling factors of 70% and 55% of skin friction anticipated by the FHWA equations for the 12 and 14 m piles, respectively. Both piles had settled about 12 mm (0.45 in.) at the time of these measurements. Using these reduced skin friction values, the predicted load versus depth curves are in remarkably good agreement with measured curves at the maximum static load when side friction should have be close to fully mobilized. These results suggest that, for whatever reason, the maximum skin friction on the piles is about 60% of the FHWA values. However, it is still unclear how much end-bearing has been mobilized since the
piles have not settled 5% of the diameter, which is about 35 mm (1.4 in.). Also note that the last
gauge on the 12 m pile did not function properly so that a direct measurement of end-bearing
resistance was not available.

The method discussed above was first presented by Reese and O’Neill in 1988, and was
later update with minor revisions (O’Neill and Reese 1999) and the FHWA recently adopted a new
equation (Brown et al., 2010) based on Kulhawy and Chen (2007). This updated method for drilled
shafts is centered on the effective friction angle of the soil, and is developed as follows:

\[
\beta \approx (1 - \sin \phi') \left( \frac{\sigma'_{v}}{\sigma'_{p}} \right)^{\sin \phi'} \tan \phi' \leq K_p \tan \phi' \tag{11}
\]

where \(\sigma'_{v}\) is the vertical effective stress, \(\sigma'_{p}\) is the effective vertical preconsolidation stress, and \(\phi'\)
is given by

\[
\phi' = 27.5 + 9.2 \log[(N_1)_{60}] \tag{12}
\]

This method will not be discussed further in this report, but the resulting curves, scaled by
the same percentages as the earlier FHWA curves, are shown in Figure 4.2-4. At depth, the earlier
FHWA method fits the static load curves better than the later method, but they are very
comparable. Therefore, the earlier method, not the later, will be used and referenced for the
remainder of the discussions in this report. Table 4.2-1 shows how the O’Neill and Reese (1999),
Kulhawy and Chen (2007) and the Zelada and Stephenson (2000) methods compare to those values
measured in the static load test. These were also compared to the Neely (1991) and Coleman and
Arcement (2002) methods, but the resultant curves did not reasonably match the measured curves
with scaling. However, beta values from Coleman and Arcement (2002) are about 60% of the
values predicted by the FHWA method from a depth of 9 to 25 m (30 to 60 ft).
Figure 4.2-4 Measured load versus depth in comparison with curves as a percentage of load predicted by the old and updated FHWA equations for drilled shaft design
Table 4.2-1 Percentage of side resistance values to match measured values from the static load test

<table>
<thead>
<tr>
<th>Method</th>
<th>12 m</th>
<th>14 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old FHWA</td>
<td>70%</td>
<td>55%</td>
</tr>
<tr>
<td>New FHWA</td>
<td>70%</td>
<td>55%</td>
</tr>
<tr>
<td>Zelada and Stephenson</td>
<td>80%</td>
<td>65%</td>
</tr>
</tbody>
</table>

O’Neill and Reese (1999) developed Q-Z and T-Z curves which describe how side and end-bearing resistance are mobilized as a function of drilled shaft settlement, respectively. However, some modifications to the curves seemed appropriate for auger-cast piles in this study. For example, in the case of the T-Z curve, rather than reaching the maximum $q_s$ at a settlement of 1% of the shaft diameter as recommended by O’Neill and Reese (1999) (a displacement of about 7 mm or (0.25 inch) for these piles), the T-Z curve was scaled to cause the maximum $q_s$ to occur at a shaft displacement of 0.5 inches. A 0.5 inch displacement has been found to be the most appropriate value for mobilization of side friction on drilled shafts based on analyses of pile load tests (Kulhway and Hirany 1989, Hirany and Kulhawy 2002). This T-Z curve is shown in Figure 4.2-5.

![Figure 4.2-5 T-Z curve for predicting mobilization of $Qs_{max}$ as a function of deflection of auger-cast piles. Full resistance is assumed to be mobilized at a deflection of 0.5 inches.](image-url)
The Q-Z curve used to estimate the mobilization of the maximum end-bearing resistance, \( Q_{b_{-max}} \), is also shown in Figure 4.2-6 and is directly based on the recommendation of O’Neill and Reese (1999). Although the Q-Z curve shown in Figure 4.2-6 indicates that the end-bearing resistance continues to increase beyond deflections of 5% of the based diameter to values as high as 10% of the base diameter, the results of the load test on 8.5 m shaft suggest that end-bearing resistance may actually plateau or increase much more gradually for these auger-cast piles. The T-Z and Q-Z curves in Figure 4.2-5 and 4.2-6, respectively, were used to compute the simultaneous development of the side and end-bearing resistance and create a computed load-deflection curve for each pile as shown in Figure 4.2-7.

Figure 4.2-6 Q-Z curve used in prediction mobilization of \( Q_{b_{-max}} \) vs. deflection for the auger-cast piles (O’Neill and Reese, 1999)
To produce these plots, $Q_{b\text{-}\text{max}}$ and $Q_{s\text{-}\text{max}}$ values are required. For the 8.5 m pile, $Q_{b\text{-}\text{max}}$ was calculated with equation (10) as a value of 71 metric tons (157 kips). The $Q_{b\text{-}\text{max}}$ values for the 12 and 14 m piles were not the values determined by equation (10) because these values would have significantly overestimated the measured load-deflection curves from the static testing. Instead, $Q_{b\text{-}\text{max}}$ values were taken as 35 and 18 metric tons (78 and 40 kips) for the 12 and 14 m piles, respectively. These values are based on load versus depth measurements from the second blast, which will be discussed subsequently. $Q_{s\text{-}\text{max}}$ values were taken as 60% (129 kips or 58 metric tons), 70% (224 kips or 102 metric tons), 55% (255 kips or 116 metric tons) of the values estimated by equations (7) and (8) for the 8.5, 12, and 14 m piles, respectively. These scaling factors for the 12 and 14 m piles are consistent with back-calculated values which matched the load vs depth curves from the static load tests. The scaling factor of 60% for the 8.5 m pile was found by trial
and error, but it is also generally consistent with the scaling factors for the 12 and 14 m piles. It is possible that the lower skin resistance in the 8.5 and 14 m piles was the result of some difficulties during installation. As explained previously, when the 8.5 and 14 m piles were constructed the trap door on the auger failed to open during the initial penetration of the auger. Therefore, the auger was extracted by reversing the rotation, the trap door was reset, and the hole was re-drilled to the desired depth where the concrete was injected. This may have caused a slightly lower skin resistance than that of the 12 m pile, and this lower skin resistance matches the load-deflection curves quite well. Generally, the computed load-displacement curves are in remarkably good agreement with the measured curves for all the three test piles, although there is still some uncertainty at larger deflections for the 12 and 14 m piles.
5 FIRST BLAST-INDUCED LIQUEFACTION TEST – NO LOAD

5.1 Overview

The explosive charges were detonated one at a time at approximately 0.20 second intervals beginning with the eight 2.7 kg charges at 8.5 m depth followed by the eight 1.1 kg charges at 4.0 m depth. The blasts caused liquefaction between depths of 4 and 14 m which produced settlement in the area around the blast ring of approximately 30 mm. Liquefaction is defined as $r_u = \frac{\Delta u}{\sigma'_o} = 1.0$ where $\Delta u$ is the excess pore pressure above static and $\sigma'_o$ is the initial vertical effective stress. The 8.5, 12, and 14 m piles settled 18, 14, and 21 mm, respectively, but there is no data to determine how quickly those settlements may have occurred. The final vertical displacements of the piles were measured using a total station and ground settlement was measured using Laser Terrestrial Scanning. These measurements were taken about 30 minutes after the blasting.

Figure 5.1-1 shows the maximum blast-induced pore pressures and a reference line of effective vertical stress based on a unit weight of 18.84 kN/m$^3$ (120 lbs/ft$^3$), the approximate unit weight throughout the profile. Liquefaction appears to have occurred within a depth range of between 4 and 12 m.
Time histories of the dissipation of the excess pore pressure ratio for each of the six piezometers following blasting are presented in Figure 5.1-2. Immediately following blasting the excess pore pressure ratios at depths of 4.9, 6.8, 9.7 and 12.8 m were about 1.0 while the ratios at depths of 2.8 and 15.9 m were about 0.4 each. The deeper soil did not liquefy because it was further from the blast charges and has a higher relative density. The soil at 2.75 m may not have liquefied because of the higher fines content or partial saturation as suggested by p-wave velocity measurements performed by Brady Cox and Julia Roberts (personal communication) at the University of Texas. The pore pressures dissipated more quickly as the depth increased, indicating that the sand reconsolidated from the bottom upward. Excess pore pressure ratios were less than 0.3 after about 20 minutes. Figure 5.1-3 shows a photograph of a large sand boil produced by the blasting which is a characteristic feature of liquefaction. Upward flow from the high excess pore
water pressure carries sand grains to the surface as the water pressure dissipates. Figure 5.1-4 shows a map of the sand boils produced around the test site during the first blast.

Figure 5.1-2 Dissipation of excess pore pressure ratio for the first blast test with no load on the piles
Figure 5.1-3 Photograph of a sand boil at the test site formed as a result of blast-induced liquefaction

Figure 5.1-4 Map of sand boils (hatched) at the test site formed as a result of blast-induced liquefaction
5.2 Blast-Induced Settlement from the Blast with No Load

Figure 5.2-1 provides color contours of ground surface settlement, pile settlement and locations of sand boils based on the laser scanning survey. According to the laser scanning, the ground surface settlement at the location of the profilometer was approximately 30 to 40 mm as shown in Figure 5.2-1. A plot of the soil settlement vs. depth profile obtained from the profilometer is provided in Figure 5.2-2. Unfortunately, the excess pore pressure ratios suggest settlements deeper than 9 m while the profilometer only provided data to a depth of 9 m. In Figure 5.2-2, the profilometer data for the first blast has been shifted to match an average surface settlement of 30 mm, but the shape of the curve is the only reliable information. It original measured surface settlements of 140 mm, which was clearly incorrect, so the laser scanning was used as the reference point for shifting the values down. From the slopes of the profilometer data, the settlement profile was nearly constant from the ground surface to a depth of about 5 m and then decreased roughly linearly below that depth.

Figure 5.2-3 shows the factor of safety associated with the methods used in Figure 5.2-2 for predicting settlement. The methods included here are primarily for reference, and it is noted that the Tokimatsu and Seed (1987) method matches best the profilometer from the blast with load, which will be discussed in more detail later (Zhang and Robertson 2002, Ishihara 1990). The Zhang and Robertson (2002) method was performed using the software “CLiq”. Note that profilometer readings for the second blast are included. Possible reasons for the differences between the profilometer readings for the first and second blasts will be discussed later. Also note in Figure 5.2-1 that the pile settlements were greater than the ground surface settlements for all three piles.
Figure 5.2-1 Contours of ground surface settlements and pile head settlements based on laser scanning after the first blast. Labels include pile head settlements for each pile.
Figure 5.2-2 Settlement profiles for the two blasts, and three predictive methods
Figure 5.2-3 Factor of safety against liquefaction
5.3 Load in Piles vs. Depth with No Pile Head Load

Because the ground settlement was greater than the pile settlement, negative skin friction would be expected to develop for this set of test piles. Plots of the load in each pile as a function of depth are provided in Figure 5.3-1 for the conditions 60 minutes after blasting when liquefaction induced settlement was completed. These plots also show the load in the pile (dashed lines) that would be anticipated if 50% of the skin friction found in the static load test, approximately 30% of that estimated by the FHWA method, developed along the pile length. Because no pile head load is applied, any load in the piles is induced by negative skin friction or dragload above the neutral plane and positive skin friction below the neutral plane. Thus, the neutral plane is visible in each of the plots as the point where the load in the pile begins to decrease. The neutral plane depth increased with increasing pile lengths, suggesting that the pile settlement decreased as the pile length increased. The neutral planes for the 8.5, 12, and 14 m piles are at depths of 5.5, 8.25 and 8.8 m, respectively. The steeper slopes of these load vs depth curves, which imply reduced post-liquefaction skin friction, suggest that the soil liquefied to a depth of about 13 m which correlates with the measured excess pore pressure ratios of nearly 1.0 down to 12.8 m.

Based on the strain gauges within unit 1 at the top of each pile, the skin friction from the surface down to a depth of about 2.5 m is approximately 0.5 ksf and is about the same as the pre-liquefaction value obtained from the static load tests. These results suggest that liquefaction did not occur within unit 1 which is a fine-grained layer or within the zone of partial saturation to a depth of about 2.75 m. This result correlates well with the pore pressure measurements at 2.75 m where the excess pressure ratio was limited to about 0.4.
There is a section of the 14 m pile where the load in the pile quickly increases and decreases between depths of 6 and 10 m. Consistency in the two strain gauge readings at each depth suggests that these are accurate values. There may have been an unusually dense section of soil at this depth, as suggested by the CPT profile, but it apparently did not prevent the soil around the 8.5 and 12 m piles within the depth range from liquefying. The skin friction shown at this depth is double what

Figure 5.3-1 Measured load versus depth in the piles (solid lines) along with predicted curves (dashed lines) assuming skin friction of about 50% of measured positive skin friction from the static load test. The neutral plane is shown in each plot with a horizontal line separating negative skin friction above it, and positive skin friction below it.
the FHWA method would predict based on equation (7). However, the section from 10 to 14 m still follows the values anticipated by 30% of the FHWA equation. Some cone penetrometer testing shows thin, sporadic, extremely dense material at these depths with cone resistance of over 30 MPa. The fact that the skin friction is still negative, but shows very high skin friction values, merely suggests that the soil is moving downward relative to the pile, not that it liquefied. Because the soil from 10 to 14 m liquefied, everything above it settled. However, the depths at which the soil settled less than the pile still show positive skin friction. So, the neutral plane, where the soil settlement matches the pile settlement, appears to be located in that dense layer, with high values of skin friction above and below the neutral plane in the dense material, and low values of skin friction above and below the neutral plane in liquefied areas.

It is possible that the tight arrangement of piles may have allowed stress-arching to develop in the soil between the piles which could have prevented the full amount of settlement from manifesting itself at the ground surface. Laser scanning showed that the ground surface settlement was actually less in the area of the piles than in the area outside them. Assuming that load spreads into the soil radially and that each pile has an area of influence of maximum radius of 1.1 m before they would begin to overlap, the weight of the soil between the piles is accounted for and can be calculated. This is shown as the maximum soil weight in Figure 5.3-2 and is an upper bound for the amount of soil-induced load in the piles. However, if the soil experiences arching only in the space between the piles, the weight of the soil held up by each pile would be much less, and is shown as the minimum soil weight in Figure 5.3-2. Thus, the piles were holding sufficient load in the upper layers to suggest that they were supporting the soil and reducing settlement.
The neutral planes in the piles could also give some insight into the settlement of the profile. The neutral planes for the three piles are at depths of 5.5, 8.25 and 8.8 m (18, 27 and 29 ft) for the 8.5, 12, and 14 m piles, respectively, as seen in Figure 5.3-1. In theory, the settlement of the soil at the neutral plane should equal the pile head settlement because the elastic compression of the pile is negligible for the loads applied. The pile head settlements for the 8.5, 12 and 14 m piles were 18, 14, and 21 mm, respectively. Assuming that the soil settlement decreased with depth,
as suggested by the profilometer data, pile head settlement would have been expected to be lower for 14 m pile because the neutral plane was deeper. Unfortunately, the settlement of the 14 m pile is not consistent with this theory. Having the deepest neutral plane, as seen from the strain in the pile, it should have the least settlement. However, if arching was reducing settlement near the surface, the settlement at depth could have been somewhat higher than in the upper layers.

An analysis of the development of the load in the pile through time suggests that the skin friction increases as the excess pore pressure decreases. Figure 5.3-3 shows how the skin friction increases through time as a percentage of the expected values from the static load test. The unit side friction, $q_s$, divided by the pre-liquefaction side friction from the static load test can be estimated using the equation,

$$\frac{(q_s)_{\text{post-liquefaction}}}{(q_s)_{\text{pre-liquefaction}}} = 11ln(t)$$

(13)

where $t$ is the time since initial liquefaction in minutes and $(q_s)_{\text{pre-liquefaction}}$ is equal to approximately 60\% of the FHWA value from equation (7).

Once excess pore pressure had dissipated and settlement had stopped, the load vs. depth curve in the previously liquefied zone developed a negative slope as shown in Figure 5.3-1. The negative slope indicates that negative skin friction had developed in this zone and was applying dragload to the pile. As the pore pressures dissipated and effective stresses increased, the skin friction at the pile interface also increased as predicted by equation (7) and produced a dragload of about 73, 126, and 300 metric tons (33, 57, and 136 kips) in the 8.5, 12, and 14.5 m piles, respectively. This is the load which induced the pile settlements, though positive side resistance removed much of that load before the end of the pile.
Shown in Figure 5.3-4 are the theoretical Q-Z curves for end-bearing in cohesionless soil for each of the piles. The mobilization curves in Figure 5.3-4 were developed using Q-Z curves by O’Neill and Reese (1999) as discussed previously. The three curves indicate theoretical mobilization of end-bearing with settlement, while the three corresponding points show the actual mobilization for each pile, however, in this case the 12 and 14 m piles have a $Q_{b-max}$ different from the FHWA equation, as will be explained in the discussion of results from the second blast test. For each of the test piles, the measured settlement exceeded the settlement which would be predicted to mobilize the measured end-bearing. The measured end-bearing was 15%, 70% and 82% of the resistance that would be anticipated based on the static load and second blast test results in non-liquefied conditions for the 8.5, 12 and 14 m piles, respectively. This suggests that the anticipated end-bearing was reduced as a result of the excess pore pressures developed by the blasting. For the 8.5 m pile base of the pile was well within the liquefied zone and the 85% decrease in resistance is understandable. For the 12 and 14 m piles, the pile toe was closer to the

Figure 5.3-3 Increase in $q_s$ over time as percentage of static load friction values

- Measured Values
- $11 \times \ln(t)$
lower boundary of the liquefied layer where dissipation would have been more rapid. In addition, the denser sand below the base of the deeper shafts likely dilated, reducing excess pore pressure more rapidly. As a result, the reduction in end-bearing resistance for these piles was much smaller, on the order of 20% to 30%. The peak excess pore pressure ratio at 12.8 m was 0.9, but quickly drops to 0.4 at 15.8 m. The general loss in end-bearing resistance due to increases in pore pressure is consistent with other tests. (Knappett and Madabhushi, 2008).

![Figure 5.3-4 End-bearing mobilization, or Q-Z curves, for the piles and measured value comparisons](image)

Figure 5.3-4 End-bearing mobilization, or Q-Z curves, for the piles and measured value comparisons
6 SECOND BLAST-INDUCED LIQUEFACTION TEST – APPLIED LOAD

6.1 Overview

The purpose of the second blast was to evaluate the liquefaction-induced reduction in skin friction and end-bearing of the piles after they had already mobilized positive skin friction. This test was performed one day after the static load test. The weights on the frame were repositioned prior to blasting to increase the static factor of safety for the various piles and prevent the assembly from tipping due to potentially uneven settlements in the piles. The photograph in Figure 6.1-1 shows the configuration of the loading immediately prior to blasting, where the piles had loads of 44, 112, 118 metric tons, (96, 246 and 259 kips) for the 8.5, 12 and 14 m piles, respectively.

The ultimate capacities of these piles have been discussed previously and were used to predict load deflection curves as shown in Figure 4.2-1. The side resistance and end-bearing capacities for each of piles, along with the pile head load applied during the second blast and the associated factor of safety, are all listed in Table 6.1-1. The factor of safety for the shorter pile was kept higher relative to the longer test piles to prevent excessive settlement of shorter pile and toppling of the weights on the load frame. Note that the low factors of safety in the 12 and 14 m piles is tied to end-bearing mobilization, and therefore settlement. A low factor of safety would lead to relatively higher settlements with an increase in load.
Table 6.1-1 Ultimate pile capacities, applied loads, and factors of safety during the second blast

<table>
<thead>
<tr>
<th>Pile</th>
<th>Q_s</th>
<th>Q_b</th>
<th>Q_total</th>
<th>Load</th>
<th>F.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.5 m</td>
<td>129 k</td>
<td>157 k</td>
<td>286 k</td>
<td>96 k</td>
<td>2.98</td>
</tr>
<tr>
<td>12 m</td>
<td>224 k</td>
<td>78 k</td>
<td>302 k</td>
<td>246 k</td>
<td>1.23</td>
</tr>
<tr>
<td>14 m</td>
<td>255 k</td>
<td>40 k</td>
<td>295 k</td>
<td>259 k</td>
<td>1.14</td>
</tr>
</tbody>
</table>

The explosive charges for the second blast were set up with the same configuration as in the first blast and the detonation sequence was also essentially the same as in the first blast. Detonation of the explosive charges caused liquefaction between depths of 4 and 7 m as shown in Figure 5.1-1 and the area of the blast ring settled approximately 80 mm. The 8.5, 12, and 14 m piles settled 103, 104, and 71 mm, respectively.
6.2 Excess Pore Pressure Generation and Dissipation

Time histories of the dissipation of the excess pore pressure ratio for each of the six piezometers following blasting are presented in Figure 6.2-1. Immediately following blasting the excess pore pressure ratios at depths of 2.75, 4.85, 6.8 9.7, 12.8 and 15.85 m were 0.5, 0.8, 0.9, 0.7, 0.6, and 0.3, respectively. The extent of liquefaction was much lower in this blast, and Figure 5.1-1 suggests that no liquefaction occurred below a depth of between 8 m or 9 m. Even so, excess pore pressure ratios below this depth were likely high enough to produce significant reduction in shear strength and induce settlement. As in the first blast, the pore pressures dissipated more quickly at depth, indicating that the sand reconsolidated from the bottom upward. However, in contrast to the first blast, the rate of dissipation was much faster for the second blast. For example, excess pore pressure ratios were less than 0.3 after about 10 minutes in the second blast whereas this required about 20 minutes to occur in the first blast. This observation is consistent with the idea that the thickness of the liquefied zone for the second blast was thinner than for the first blast. One might speculate that the second blast caused less liquefaction because of increased soil stiffness from compaction caused by the first blast. It seems unlikely considering the past, repetitive seismic activity of the Christchurch area. It was more likely caused by the fact the first blast test was performed simultaneously with the testing of other foundations types, none of which are related to this report, within 10 meters of the auger-cast piles. The second blast test was performed in isolation, so the relative energy applied to the site was likely lower.
Figure 6.2-1 Dissipation of excess pore pressure ratio for the second test blast with applied load on the piles

6.3 Blast-Induced Settlement from the Blast with Applied Load

Figure 6.3-1 provides color contours of the ground and pile settlement following the second blast obtained from the laser scanning results. These contours indicate that the ground surface settlement at the location of the profilometer was approximately 80 mm. A plot of the soil settlement vs. depth obtained from the profilometer is provided in Figure 6.2-2. Also shown in Figure 6.3-2 is the profilometer data from the first blast, and the Tokimatsu and Seed (1987) estimates for liquefaction-induced settlements. The profilometer from the second blast matches very closely with the curve from the Tokimatsu and Seed method.

Also shown in Figure 6.3-1 are the pile settlements of 103, 104, and 71 mm (4.1, 4.1 and 2.8 in.) for the 8.5, 12, and 14 m piles, respectively. The large pile settlements are due to the applied load being larger than the combined positive friction below the liquefied zone and the static end-bearing.
Because 12 m pile settled more than the surrounding soil, positive skin friction would be expected to occur along the length of the pile. Based on the settlement profile from the profilometer shown in Figure 6.3-2, the expectation would be to see negative skin friction in the 14 m pile from 0 to 4 m and positive friction below this level. However, this did not occur. The ground surface settlement contours shown in Figure 6.3-1 indicates that the 8.5 and 14 m piles settled about as 

Figure 6.3-1 Contours of ground surface settlements and pile head settlements based on laser scanning after the first blast. Labels include pile head settlements for each pile.
much as the soil around them. The measured loads in the piles, as discussed later, shows that these piles still experienced positive skin friction through their entire length. Thus, the discrepancy in the profilometer likely came from the difference between settlements at the profilometer location and settlements at the pile locations. The settlement in the interval from 3 to 9 m is equal to 1.1% volumetric strain.

Figure 6.3-2 Settlement profiles for the two blasts, and three predictive methods
Because the test piles are settling more than the surrounding ground they are incapable of holding up the ground between the piles by means of arching as likely occurred during the first blast. As a result, the settlement of the ground between the piles is unrestricted and better represents the settlement that would occur around an isolated test pile. Without these arching effects, the measured settlement of the ground from the profilometer in the second blast is about the same as predicted based on the Tokimatsu and Seed approach as shown in Figure 6.3-2. Another piece of evidence supporting the notion of arching during the first blast is the fact that liquefaction induced settlement was less for the first blast (30 mm) than for the second blast (80 mm) even though the thickness of the liquefied zone was smaller from the second blast than from the first blast. Figure 6.3-3 shows ground surface settlements radially from the center of the blast ring, or pile group, as measured using a string-and-ruler system. This matches Figure 6.3-1 reasonably well.

Figure 6.3-3 Ground surface settlements shown radially from the center of the blast right, or pile group
6.4  **Loads in Piles vs. Depth with Pile Head Load**

Plots of the load in each pile as a function of depth are provided in Figure 6.4-1 for the conditions 45 minutes after blasting. These plots also show the load in the pile that would be anticipated if 30% of the positive skin friction estimated by the FHWA equation were developed in areas where the soil liquefied.

In Figure 6.4-1 there is a visible change in the skin friction at a depth of about 7 m (23 ft). The dotted line shows 30% of the skin friction from the FHWA equations above a depth of 7 m, and 60% of the skin friction values from the FHWA equations below a depth 7 m. Thus, in agreement with the measured excess pore pressures ratios, the profile did not appear to liquefy below a depth of about 7 or 8 m. This depth is shown by the horizontal line in Figure 6.4-1. The increase in resistance at a depth of 7 m for the 14 m pile also corresponds to the layer where increased resistance was observed for the first blast, as shown in Figure 5.3-1, although it is not quite as high. One might argue that, despite not having liquefied, the soil immediately below 8 m had a high excess pore pressure ratio (0.75 at 10 m and 0.6 at 13 m) and therefore should have affected the seismic compaction and post-liquefaction skin friction. If such occurred, was not significant and is not discernible in Figure 6.4-1.

Using the skin frictions values from Figure 6.4-1, the load vs. depth in the 8.5 m pile can be predicted, and is shown in Figure 6.4-2. The end-bearing associated with this prediction is around 30 kips and is about 20% of $Q_{b\text{-max}}$ predicted from the static load test. This is consistent with the first blast test and is likely the result of the pile bearing within the liquefied zone.
Figure 6.4-1 Load in the piles after the second blast showing resistance in liquefied and non-liquefied sections

- 30% qs from FHWA
- 60% qs from FHWA
It is important to note the end-bearing mobilized in Figure 6.4-1. The end-bearing in this instance was not affected by liquefaction although excess pore pressure did develop. For the measured pile settlements of 104 and 71 mm (4.0 and 2.8 in.), both of which are greater than 5% of the shaft diameter (about 35 mm for the shafts involved), the maximum end-bearing values should have been mobilized. Therefore, the measured end-bearing values of 78 and 40 kips (35 and 18 metric tons) for the 12 and 14 m piles, respectively, were selected as the $Q_{b\text{-max}}$ values as discussed previously. (See discussion of Figure 4.2-1). This assumes that the end-bearing
resistance is flat or slightly inclined beyond a displacement equal to 5% of the base diameter, which assumption is supported by the plunging type behavior of the 8.5 m shaft during static loading, as shown in Figure 4.2-1. These $Q_{b\text{-max}}$ values were determined by extrapolating the load vs. depth curves in Figure 6.4-1 to the bottom of the pile in each case. The anticipated end-bearing values from equation (10) are 169 and 281 kips for the 12 and 14 m piles. Therefore, the measured end-bearing values for the 12 and 14 m piles were only 46 and 14%, respectively, of the anticipated end-bearing resistance.

There are at least two possible reasons that the end-bearing is so much smaller than anticipated. First, there may be variability in the soil profile, but blow counts would have to have been very low to match the end resistance shown in Figure 6.4-1, and therefore this explanation seems rather unlikely. Second, the method of installation of auger-cast piles may cause differences in the concrete quality, the soil density, or the shaft shape at the toe of the pile. For example, as previously discussed, when the 14 m pile was constructed the trap door on the auger failed to open during the initial penetration of the auger. Therefore, the auger was extracted by reversing the rotation, the trap door was reset, and the hole was re-drilled to the desired depth where the concrete was injected. This process apparently left a relatively loose layer at the bottom of the shaft which likely explains the low end-bearing resistance.

Another point of interest is the possible increase of end-bearing immediately following the second blast during the time in which there was a decrease positive friction in the liquefied layer. This decrease should cause an increase in end-bearing as the pile settles, and later the end-bearing would decrease as the positive friction in the liquefied layer developed. Supposing that there was no positive friction in that layer immediately after the blast, there would be approximately a 50 and 60 kip increase in the 12 and 14 m piles, respectively. If this were the case, the $Q_{b\text{-max}}$ values
would actually be 58 and 45 metric tons (128 and 100 kips) for the 12 and 14 m piles. An alternate
of Figure 5.3-4 is shown below in Figure 6.4-3 for the case just described.

This implies that the end-bearing of the 12 and 14m piles during the first blast were 38 and
33% of the anticipated mobilized end-bearing, respectively. This is a decrease from the 70 and
82% values previously described, and is more aligned with the excess pore pressure ratios
measured at those depths in the first blast. This increase in maximum end-bearing would also affect
the shape of the load deflection curves shown in Figure 4.2-7, and these alternate curves are shown
in Figure 6.4-4. This possible change in the maximum end-bearing does not have a significant
impact on the anticipated load-deflection curves.

Figure 6.4-3  Alternate end-bearing mobilization, or Q-Z curves, for the piles and measured value
comparisons from the first blast
The consistency of the interpreted static load-displacement curves, the load vs depth for the first blast, and the load vs depth curves for the second blast provides increased assurance regarding the validity of the values selected for $Q_{b,max}$, $Q_{s,max}$, and the percentage (50%) of positive skin friction which develops in liquefied sand. It is a rather difficult to make an interpretation from any of these tests independently, but the consistency across all three tests creates a strong argument.

Figure 6.4-4  Alternate load deflection curves against predicted load deflection curves for a potentially higher end-bearing value for the 12 and 14 m piles
7 CONCLUSIONS

7.1 Summary

Because of the complexity of the interaction between positive and negative skin friction (soil settlement vs. pile settlement), neutral plane location, and end-bearing mobilization, information from a combination of tests is needed to understand the soil-pile interactions. The primary conclusions of this report are related to the magnitude of post-liquefaction skin friction on augercast piles.

7.2 Conclusions

For the piles without applied pile head load, ground settlement was more than the pile settlement. As the liquefied layer settled owing to dissipation of excess pore pressures, the increased effective stress allowed negative skin friction to progressively increase at the sand-pile interface. At the end of consolidation, the average negative skin friction was roughly equal to 50% of the positive skin friction obtained from the static load test, rather than zero as assumed in some approaches.

For the piles with applied pile head load, the applied load was large enough and the end-bearing resistance was low enough that the piles settled relative to the surrounding soil when the pile came to equilibrium. Therefore, the skin friction was all positive along the length of the pile even within the liquefied layers. Once again back-analysis indicates that the side friction in the
liquefied layer after reconsolidation was approximately 50% of the positive skin friction measured from the static load test.

For both the negative and positive skin friction cases after reconsolidation of the liquefied sand the measured side friction was approximately 50% of the side friction in the non-liquefied sand. This 50% value is relatively consistent for all three test piles. In addition, the 50% value is consistent with previous negative skin friction measurements reported by Rollins and Strand (2006) for a steel pile after blast induced liquefaction. The consistency of these results strongly suggests that skin friction in liquefied layers is not zero as has been assumed but should be considered to be about 50% of the pre-liquefaction skin friction in assessing dragload and pile settlement following liquefaction. This redundancy, both in the two blast tests from this report, and that of Rollins and Strand (2006) suggests that the magnitude of post-liquefaction skin friction is a function of the relevant mechanics and not the site conditions.

Liquefaction-induced settlement was approximately 80 mm (3.15 in) which equates to a volumetric strain of about 1.1% and is consistent with predictions based on the Tokimastu and Seed (1987) approach.

Negative friction can rapidly change from negative to positive friction if the pile settles more than about 12 mm (0.5 inch), which is the displacement necessary to mobilize skin friction. In this series of tests, the piles quickly developed positive skin friction during the static load test after having developed negative friction in the first blast test.

For auger-cast piles, the end-bearing and side resistance values can be highly dependent on the construction quality. For the test piles investigated in this study the unit side resistance was typically 50% to 70% of the unit side resistance predicted by the FHWA method for drilled shafts in sands.
There is a reduction in end-bearing capacity related to significant excess pore water pressures in the bearing layer. This is consistent with findings by Knappatt and Madabhushi (2008) and can be a source of significant pile settlement immediately following an earthquake event.

It is quite difficult to predict settlement behavior beyond settlements of 5% of the base diameter. It would be unwise to rely on any resistance mobilized at settlements beyond 5%B. Therefore, the pile should have enough resistance from side friction and end-bearing below the liquefied zone to prevent excessive settlement of the pile toe considering the applied load on the pile and negative skin friction.
REFERENCES


