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# Liquefaction Induced Negative Skin Friction from Blast-induced Liquefaction Tests with Auger-cast Piles

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## ABSTRACT

Blasting was used to induce liquefaction around auger-cast piles extending to 8.5, 12, and 14m below ground at a site in Christchurch, New Zealand. Liquefaction led to negative skin friction and pile settlement. The depth to the neutral plane increased as the pile length increased. Skin friction following liquefaction was compared to pre-liquefaction values based on static load tests. Negative skin friction in the non-liquefied soil was equal to the positive skin friction. Contrary to common design assumptions, the negative skin friction in the liquefied sand was not zero. As excess pore pressure dissipated, the increased effective stress allowed negative skin friction to increase. After consolidation, the average negative skin friction was roughly equal to 50% of the positive skin friction which agrees with previous full-scale tests with driven steel piles. The average unit side resistance for the auger-cast piles was typically 50% to 70% of the unit side resistance predicted by design FHWA equations for drilled shafts.

## Introduction

Frequently, deep foundations extend through potentially liquefiable sand layers near the ground surface and bear on more competent layers at depth. However, when liquefaction occurs, negative skin friction and associated settlement may occur. Many design procedures assume that negative skin friction in the liquefied layer will be zero and use this value to evaluate the consequences of negative friction and pile settlement. However, a full-scale field test indicates negative skin friction after liquefaction may be 50% of the pre-liquefaction skin friction (Rollins and Strand 2006). To investigate the loss of skin friction and the development of negative skin friction, soil-induced load was measured in three instrumented full-scale piles after blast-induced liquefaction at a site in Christchurch, New Zealand. The three test piles consisted of 0.6 m diameter Auger-Cast Piles (ACP) extending 8.5, 12, and 14 m below the ground surface. The piles were instrumented with strain gauges to determine load vs depth in each pile.

## Geotechnical Site Conditions

The test site was located in Avondale near the Avon River in Christchurch, New Zealand. 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, standard penetration tests, shear wave logging, and undisturbed sampling, was performed by Tonkin and Taylor in association with the Earthquake Commission in New Zealand.

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Prior to installation of the test piles, two CPT soundings were performed to define the soil profile. An SPT boring was also performed using the sonic drilling system. The SPT testing involved an automatic hammer delivering 85% of the theoretical free-fall energy. Corrections to obtain $(N_1)_{60}$ and $q_{c1}$ were made using the procedures outlined by Youd et al (2001). Plots of the CPT and SPT test results are provided in Figure 1 along with estimates of relative density obtained using correlations proposed by Kulhawy and Mayne (1990).

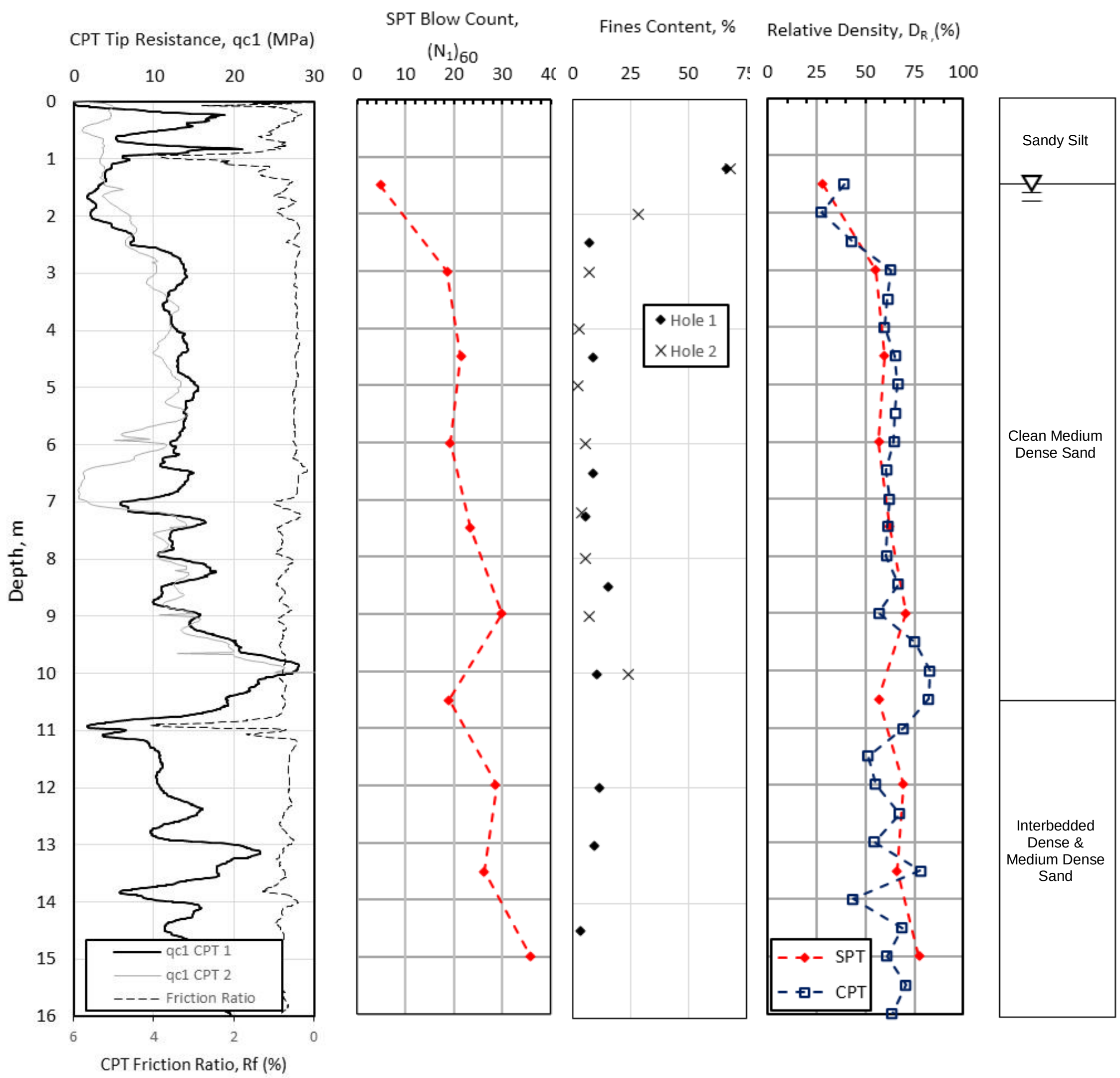


Figure 1. Profile showing CPT cone tip resistance, SPT blow count, fines content, relative density and generalized soil profile for the test site.

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 16 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 1. 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.

## Layout of Test Piles and Instrumentation

Plan and profile views of the layout of the test piles relative to the blast holes and instrumentation are shown in Figures 2 and 3. The test piles were auger-cast piles constructed with lengths of 8.5 m, 12 m, and 14 m, and consisted of high-strength concrete ($f'_c$=38 MPa) 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). To determine load in the pile vs depth, 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. To determine the cross-sectional area of the shaft versus depth, each test pile was also instrumented with four thermal gauge wires. These gauges were spaced at 30 cm along each wire and used the heat of concrete curing to indicate the pile diameter throughout the length of the pile.

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° as shown in Figure 3. 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. The eight explosive charges at 8.5 m were detonated sequentially 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. 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.

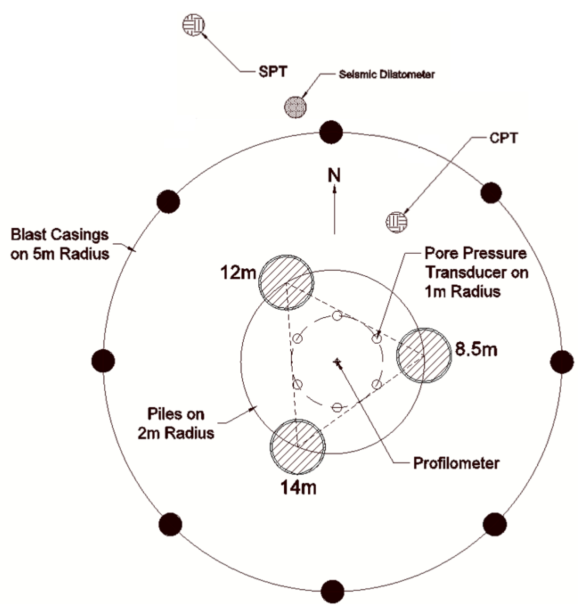


Figure 2. Plan view of test piles, blast holes and instrumentation.

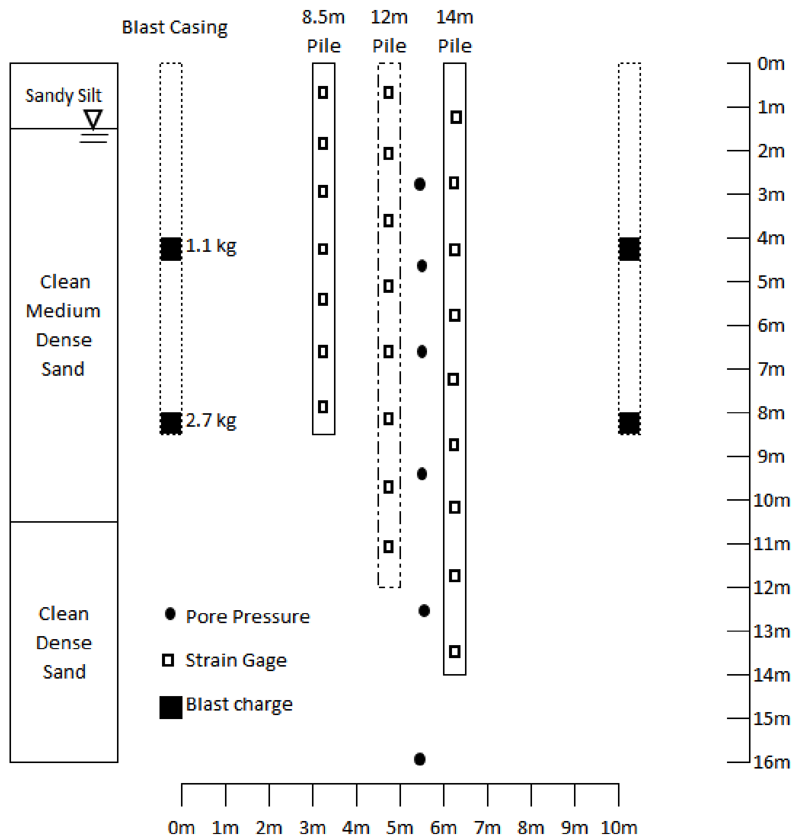


Figure 3. Elevation view of test piles, blast charges, and instrumentation relative to the soil profile.

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 Figure 3. 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).

## Static Load Testing

Though presented here for clarity, static load testing was performed on the three test piles after the subsequent blast test to provide information on the actual side resistance and end-bearing in a non-liquefied state. As with all deep foundations, there is uncertainty about the axial capacity of auger-cast piles. The piles were loaded using a rectangular steel frame and 272 metric tons of weights. The rectangular frame rested on the three piles with a load cell installed between the frame and each pile. 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 piles.

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 deflection for the 12 and 14 m piles which was still sufficient to fully mobilize side friction (Hirany and Kulhawy, 2002). End-bearing versus displacement (Q-z) curves proposed by O'Neill and Reese (1999) suggest that mobilized end-bearing resistance was about 50% of ultimate capacity defined by displacement equal to 5% of the pile diameter.

Based on the strain during the static load test, plots of the load in the pile versus depth for both the 12 and 14m piles were developed as shown in Figure 4. No plots are provided for the 8.5 m pile because lead wires for the strain gauges were damaged during the installation of dowel bars for the support of the load frame. To provide a basis for investigating the measured side friction and end-bearing resistance, the measured values were compared with equations for drilled shaft side friction specified by the US Federal Highway Administration (FHWA) (O'Neill and Reese 1999, Brown et al. 2010). These FHWA equations have often been used as a point of reference in understanding the behavior of auger-cast piles; however, the unit side resistance of auger-cast piles has typically been lower than that for drilled shafts in sand. As expected, the FHWA equations predict unit skin friction higher than interpreted from strain in the static load tests.

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, the best agreement with measured load versus depth curves was obtained using scaling factors of 70% and 55% of skin friction predicted by the FHWA equations for the 12 and 14 m piles, respectively. Both piles had settled about 12 mm at the time of these measurements so skin friction should have been fully mobilized (Hirany and Kulhawy, 2002). Using these reduced skin friction values, the predicted load versus depth curves are in remarkably good agreement with measured curves. Additional analyses to fit the load-settlement curve for the 8.5 m pile suggest that ultimate side friction for that pile was approximately equal to 60% of the value predicted by FHWA equations. These results suggest that on average, for whatever reason, the maximum skin friction on the auger-cast piles is about 60% of the FHWA values.

## Blast Liquefaction Testing

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. Measurements of the excess pore pressure versus depth relative to the initial vertical effective stress indicate that liquefaction ($r_u = \Delta u/\sigma'_o = 1.0$ ) occurred between depths of 4 and 14 m which produced settlement in the area around the blast ring of approximately 30 mm. Liquefaction was confirmed by the presence of several sand boils around the test area.

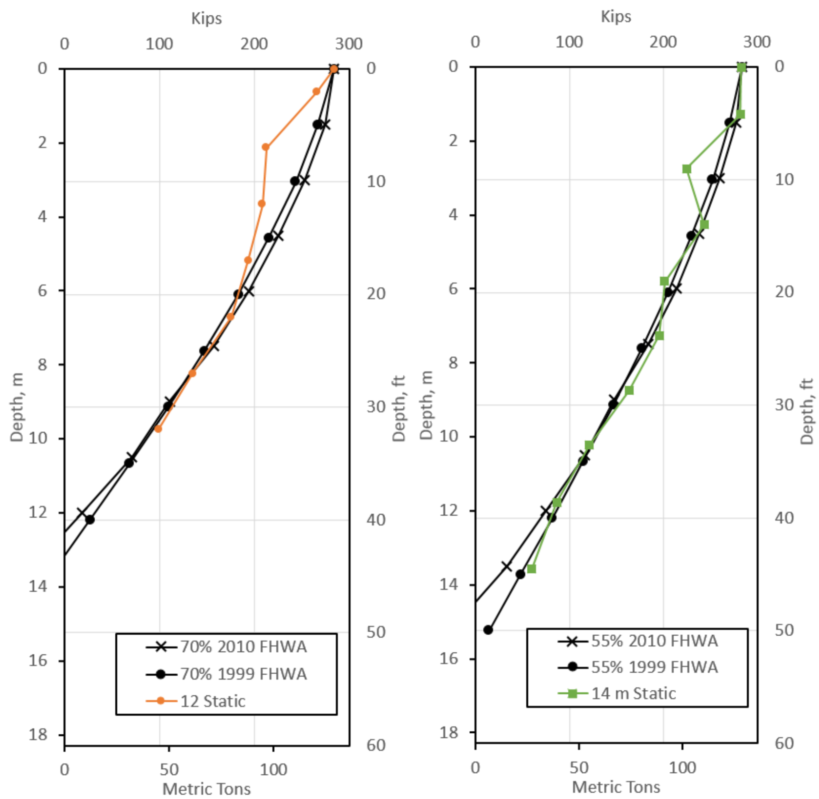


Figure 4. Interpreted pile load versus depth curves from static load tests in comparison with curves as a percentage of load predicted by 1999 and 2010 FHWA equations for drilled shaft design.

The 8.5, 12, and 14 m piles settled 18, 14, and 21 mm, respectively. Because the ground settled more than the piles, negative skin friction developed in each case. Plots of the load in each pile as a function of depth interpreted from the strain gauge readings are provided in Figure 5 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. Positive skin friction reduces pile load 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. Theoretically, the neutral plane is also the depth where pile and soil settlement are equal. 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 depth to the neutral plane increases as the length of the pile increases suggesting that the pile settlement decreased as the pile length increased. However, variations in pile settlement from this pattern are likely due to differences in soil settlement profiles across the area. 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 24 kPa and is about the same as the non-liquefied 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 only about 0.4.

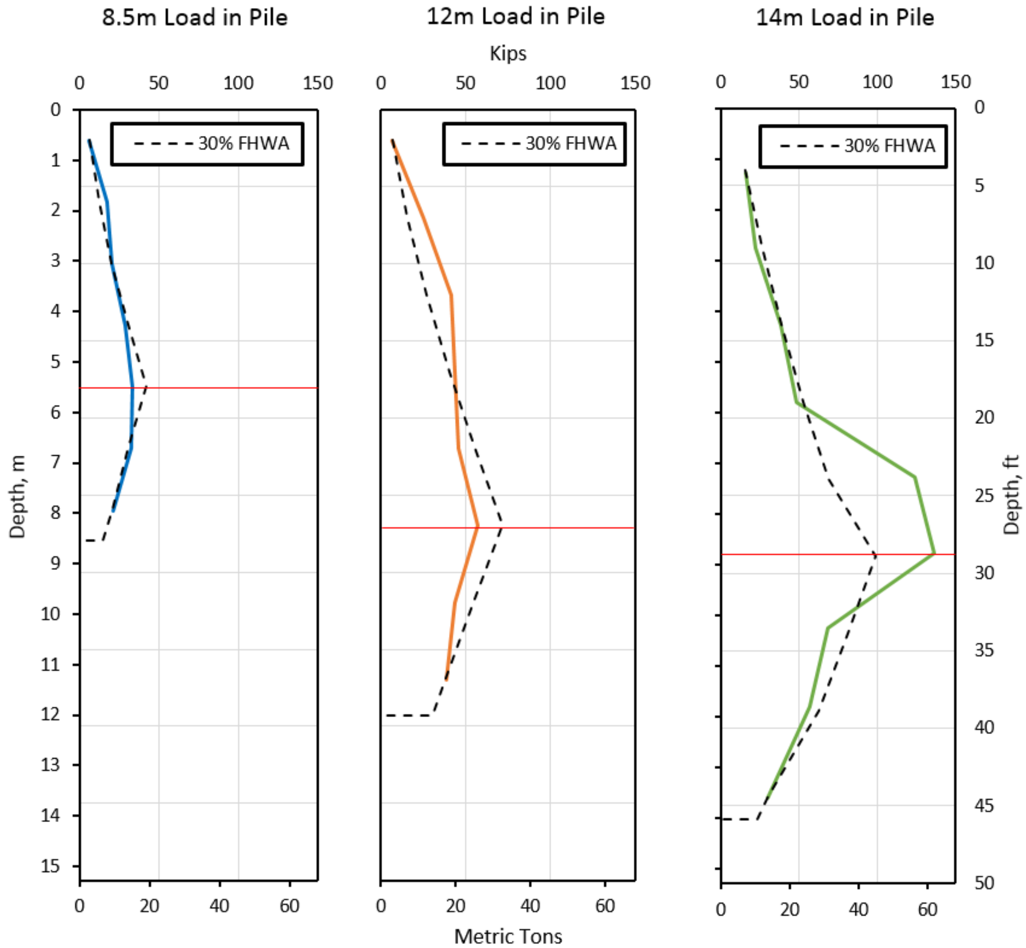


Figure 5 Interpreted pile load versus depth curves (solid lines) following blast liquefaction along with predicted curves (dashed lines) assuming skin friction equal to 50% of measured positive skin friction from the static load test or 30% of FHWA drilled shaft skin friction. The neutral plane is shown in each plot with a horizontal line separating negative skin friction above from positive skin friction below.

There is a length of the 14 m pile where the load in the pile quickly increases and decreases between depths of 6 and 10 m. There may have been an unusually dense layer 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 this depth range from liquefying. The load transfer in this depth interval indicates that skin friction is double what the FHWA method would predict. However, the section from 10 to 14 m still follows the values anticipated by 30% of the FHWA equation (≈50% pre-liquefaction friction). Some CPT results show tip resistance in this depth range between 15 and 30 MPa where liquefaction might not have occurred. The fact that the skin friction is still negative, but shows very high skin friction values, merely indicates that the soil is moving downward relative to the pile, not that it liquefied. Because the soil from 10 to 14 m liquefied, layers 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 skin friction in the dense sand, and low skin friction above and below the neutral plane in liquefied areas.

## Conclusions

Blast induced liquefaction led to negative skin friction and pile settlement. The depth to the neutral plane increased as the pile length increased. 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 before liquefaction obtained from the static load test, rather than zero as assumed in some approaches. This result is consistent with a previous full-scale blast liquefaction test on driven steel piles (Rollins and Strand, 2006). The consistency of these results strongly suggests that skin friction in liquefied layers is not zero but should be considered to be about 50% of the pre-liquefaction skin friction in assessing dragload and pile settlement following liquefaction. 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 US FHWA for drilled shafts in sands.

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