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Downdrag on tapered concrete piles during full-scale blast-induced liquefaction test

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ABSTRACT: Deep foundations often extend through potentially liquefiable sand layers near the ground surface and bear on more competent layers at depth. When liquefaction occurs, the skin friction in the liquefied layer would be expected to decrease to some negligible value, but as the liquefiable layer settles, negative skin friction could potentially develop around the pile in this layer as effective stress increases. To investigate the loss of skin friction and the development of negative skin friction, axial load tests were performed on an instrumented full-scale tapered pile before and after blast-induced liquefaction at a site in Italy that was affected by liquefaction following the 2012 Emilia earthquakes. The test pile was a 16.5 m long concrete pile with a diameter of 0.52 m at the head tapering to 0.26 m at the toe. Following blasting, liquefaction developed within a 6-m thick sand layer below a clay surface layer resulting in significant settlement. Skin friction in the liquefied layer initially dropped to near zero. However, as the liquefied sand reconsolidated, negative skin friction became equal to about 50% of the pre-blast ultimate positive skin friction. Negative skin friction in the overlying non-liquefied clay layer was only 80% of the pre-blast ultimate positive skin friction. This is likely due to the surrounding soil moving slightly away from the tapered pile as the soil settled vertically downward. Despite significant ground settlement, pile settlement was relatively small because of the resistance provided by the toe of the pile.

KEYWORDS: Pile downdrag, liquefaction-induced dragload, blast liquefaction testing, liquefaction, negative friction, tapered piles.

1 INTRODUCTION

Deep foundations often extend through potentially liquefiable loose to medium dense sand layers and bear on more competent layers at depth as shown in Fig. 1(a). Prior to liquefaction, the applied pile head load, P, is transferred to the soil through upward (positive) skin friction, Q_s and the load in the pile decreases as shown in Fig. 1(b). The load at the base of the pile is carried by toe resistance, Q_b, which requires some settlement to develop as illustrated by the toe resistance vs. settlement (Q_b-z) curve in Fig. 1(c). When liquefaction occurs, skin friction in the liquefied layers is expected to decrease to near zero and many design procedures use this value to evaluate the consequences of skin friction loss and pile settlement as shown in Fig. 1(b). The reduction in positive skin friction in the liquefied layers leads to an increase in load at the toe of the pile and mobilization of the increased toe resistance leads to additional pile settlement as shown in Fig. 1(c and d).

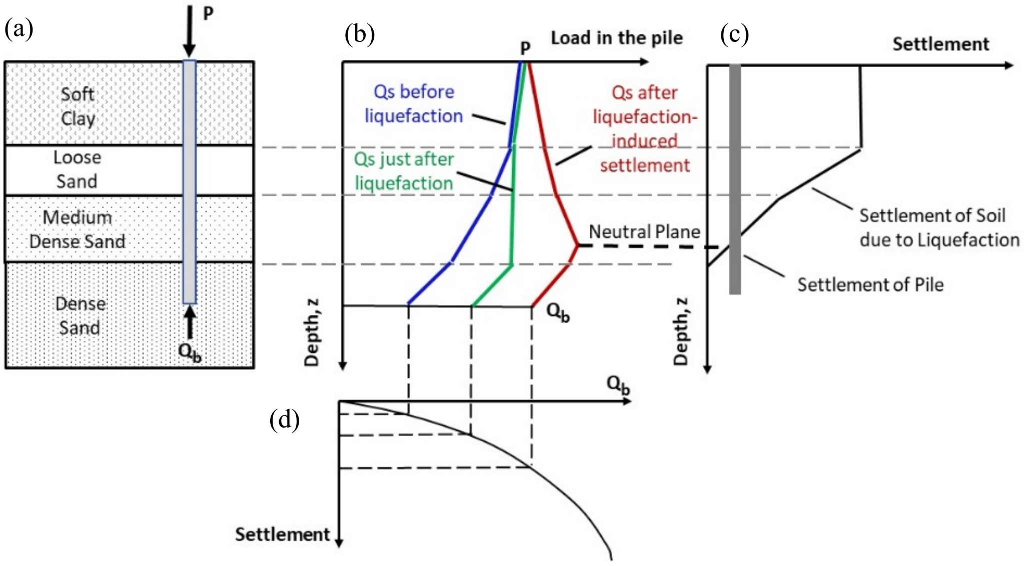

Figure 1. Plot showing (a) soil profile around pile, (b) load in the pile vs. depth, (c) settlement vs. depth, and (d) e resistance (Q_b) vs. settlement before and after liquefaction-induced settlement.

As the earthquake-induced pore pressures dissipate in the liquefiable layer and settlement occurs, downward (negative) skin friction (dragload) develops at the pile-soil interface in the clay layer above the liquefied layer, increasing the load in the pile (see Fig. 1(b)). In addition, the skin friction at the pile-soil interface in the liquefied layer is likely to increase as the excess pore pressure decreases. Therefore, the negative skin friction that ultimately develops in the liquefied layers will likely be higher than zero and will induce even greater load in the pile. The increased negative skin friction in the liquefied and non-liquefied layers further increases the required toe resistance (see Fig. 1(c)) and leads to additional pile settlement as shown in Fig. 1(d).

The neutral plane, shown in Fig. 1, represents the depth where pile settlement and soil settlement are equal. Negative friction develops above the neutral plane and positive friction develops below the neutral plane. As a result, the maximum load in the pile occurs at the neutral plane. The location of the neutral plane is normally obtained by trial and error so that the pile head load, P, plus the negative skin friction above the neutral plane is equal to the positive skin friction and toe resistance, Q_b below the neutral plane.

Some investigators have used theoretical concepts to predict the behavior of piles when subjected to liquefaction induced drag loads. For example, Boulanger and Brandenberg (2004) defined negative skin friction in the liquefied zone in terms of the effective stress during 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 the Unified Design of Piles approach that 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 also conclude that liquefaction above the neutral plane would not increase the load in the pile based on the concept that negative friction would already be present prior to liquefaction.

To understand better the development of negative skin friction on piles in liquefied sand and the resulting pile response, several full-scale pile downdrag tests have been performed using blast-induced liquefaction. Blast-induced liquefaction was first used to investigate the lateral resistance of piles in liquefied sands (Weaver et al. 2005 and Rollins et al. 2005) and has become widely used to investigate ground improvement strategies (Wentz et al. 2015; Rollins et al. 2021; Gallagher et al. 2007). Typically, these blast liquefaction tests have shown that the negative skin friction following reconsolidation of the liquefied sand is between 40 and 60% of the ultimate positive skin friction in these layers before liquefaction (Rollins and Strand 2006; Rollins and Hollenbaugh 2015; Rollins et al. 2018; and Kevan et al. 2019). However, the downward skin friction in non-liquefied layers is approximately the same as the ultimate positive skin friction. Very little relative displacement (2 to 5 mm) is required to fully mobilize negative friction.

Centrifuge tests to investigate downdrag have indicated that negative friction in liquefied sand layers was equal to 100% of the ultimate positive friction after reconsolidation (Sinha et al. 2022). These results suggest that the 50% reduction of negative skin friction in blast tests may result from destruction of aging effects and sand microstructure during liquefaction that does not occur in centrifuge tests with unaged sand.

All previous downdrag liquefaction tests have been performed with cylindrical piles with a constant pile diameter versus depth. However, for tapered piles, where the pile diameter decreases with depth, the soil might settle away from the pile as the soil reconsolidates following liquefaction. This could lead to a reduction in the negative skin friction induced on the pile and might be a reasonable approach for mitigating dragload effects. To investigate the effect of a tapered pile on the downdrag behavior following liquefaction, we conducted a full-scale test on a 17-m long tapered pile. Liquefaction was induced using controlled blasting while pile head load was being applied by a hydraulic jack. This paper summarizes the test procedures and the basic results that were obtained.

2 SOIL PROFILE AND PILE PROPERTIES

A generalized soil profile for the test site is provided in Fig. 2(a). The profile consists of a clayey to sandy silt to a depth of about 7 m underlain by loose to medium dense silty sand. Groundwater was at 4 m. A single tapered pile was driven to a depth of 16.5 m with a hydraulic hammer along with two cylindrical reaction piles located about 3 m on either side of the test pile. The test pile had a diameter of 0.51 m at the pile head and 0.26 m at the toe, while the cylindrical reaction piles had a diameter of 0.5 m. The piles were made of a hollow reinforced concrete section that allowed the insertion of a steel reinforcing bar instrumented with vibrating wire strain gauges at approximately 1.5 m intervals along the length of the pile. Following insertion, the hollow interior was grouted. Results from CPT and DPT tests provided in Fig. 2(b) and (c), before and after pile driving, show increases in q_t and K_D indicating increased density, strength and lateral earth pressure in the silty sand following pile driving.

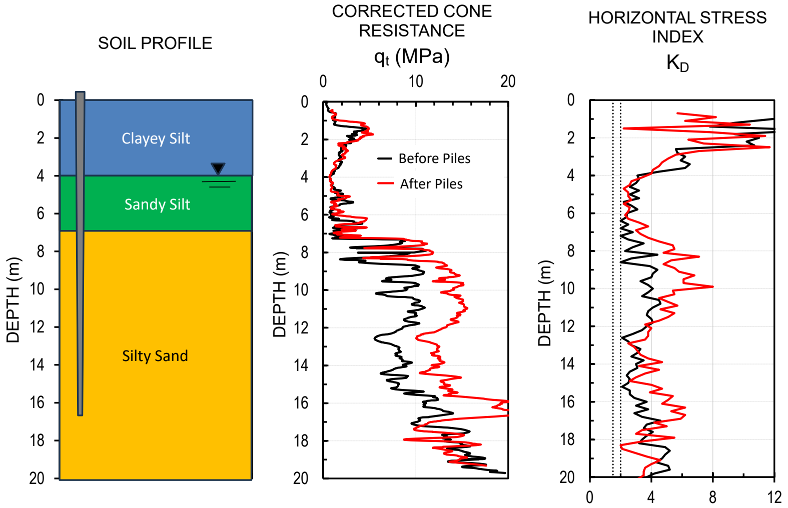

Figure 2. Generalized soil profile, CPT corrected cone resistance, q_t(MPa), and DMT horizontal stress index, K_D.

3 STATIC LOAD TESTING

About a month after pile driving, a steel reaction frame was attached to the two reaction piles and ahydraulic jack reacted against the frame to apply axial compressive force to the test pile. Pile head load was applied in approximately 240 kN intervals and pile head deflection was measured with an independent reference frame. The test was terminated at a load of about 2700 kN where the capacity of the reaction frame was reached. Fig. 3 provides a plot of the pile head load-deflection curve obtained from the static load test along with the ultimate load interpreted using the Davisson method. Because the pile capacity exceeded the capacity of the frame, some extrapolation was required to obtain an ultimate static pile resistance of about 2800 kN. The load in the pile vs. depth is plotted for the maximum load increment in Fig. 4. Side resistance accounted for about 2275 kN or 84% of the total pile resistance while extrapolation of the load vs. depth curve to the pile toe indicates an toe resistance of about 525 kN at the maximum load.

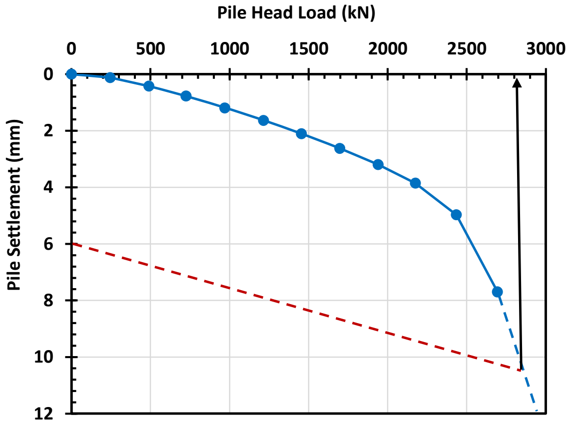

Figure 3. Measured pile head load-deflection curve from static load test on the tapered test pile.

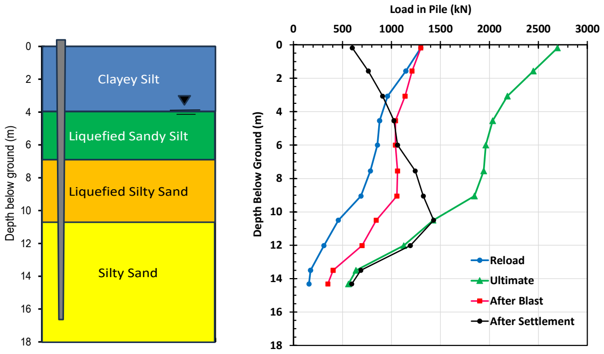

Figure 4. Idealized soil profile at test site along with load versus depth curves for the test pile for different conditions before and after blast-induced liquefaction

The static pile resistance was also calculated using a nearby CPT sounding (see Fig. 2b) with a recently developed unified CPT approach proposed by Lehane et al. (2022). This method is an attempt to produce a single consensus design equation based on best practices from four CPT approaches specified by the API code (2011). This unified approach, which is based entirely on load tests involving cylindrical piles, predicts an ultimate pile resistance of only about 1620 kN, which is far below the measured pile resistance. However, research has indicated that tapered piles can produce significantly more skin friction than cylindrical piles (Nordlund 2011). Italian practice typically increases skin friction by a “conicity” correction factor of 1.5 for tapered piles to account for this increased resistance. In this case, a correction factor 2.2 would be needed to match the higher measured capacity for this tapered pile with a 1.5 cm/m taper.

4 CONTROLLED BLASTING DOWNDRAG TEST

Following the static load test, the pile head load was released and just prior to the blast test a static load of 1300 kN was re-applied to the pile head. This represents a load with a factor of safety of 2.15. The load vs. depth curve after re-application of the 1300 kN load is shown in Fig. 4 and this curve is also in good agreement with the load vs. depth curve from the initial static load test at this load level.

The blast sequence consisted of the detonation of two charges within each of eight blast holes distributed uniformly around a circle with a radius of 7 m. The explosive in each hole consisted of a 0.5 kg charge at a depth of 7 m with another charge of 2.0 kg at a depth of 11 m. The charges were detonated sequentially with a 1000 millisecond delay between detonations. Following detonation of the charges, strain was recorded with a high-speed data acquisition system. Pile and ground settlement were measured with a digital autolevel and with drones using structure from motion technology.

Shortly after the completion of the blasting sequence, the load vs. depth curve from about 4 m to 9.5 m became nearly vertical indicating liquefaction with skin friction close to zero (see Fig. 4b) This reduction in skin friction led to increased mobilization of toe resistance and pile settlement. Pile head settlement gradually reduced the load applied by the hydraulic jack at the pile head load to about 600 kN when soil settlement was nearly complete and the load vs. depth curve reached equilibrium. At this point, as shown by the load vs. depth curve in Fig. 4(b), negative skin friction extended from the ground surface to a depth of about 10.5 m, including within the liquefied zone. Based on load in the pile, the neutral plane was at 10.5 m and positive skin friction developed below this depth.

Negative skin friction in the upper cohesive layer was about 80% of the ultimate positive skin friction measured during the static load test. This reduction in skin friction may be a result of the cohesive soil settling slightly away from the pile due to the pile taper as hypothesized previously.

The negative skin friction in the liquefied layer varies somewhat versus depth. For the zones within the liquefied layer from 4 to 6 m and 7.5 to 10.5 m the average negative skin friction is 53% of the ultimate positive skin friction. This value (53%) is consistent with several previous blast liquefaction tests where the average negative skin friction was 40 to 60% of the ultimate positive skin friction (Rollins and Strand 2006; Rollins and Hollenbaugh 2015; Rollins et al. 2018, and Kevan et al. 2019). However, the average negative skin friction for the entire liquefied layers was about 67% of the ultimate positive skin friction. It appears that the sand in the zone from 6 to 7.5 m did not fully liquefy or the silt content increased resistance.

These results indicate that the tapered pile was not successful in reducing the negative skin friction at the pile-soil interface in liquefied sand. One explanation for this behavior is that the liquefied soil could simply flow towards the pile while in a liquefied state and then exert a similar negative friction to that observed with a cylindrical pile during reconsolidation. In contrast, the undrained shear strength of the surface layer could potentially maintain a slight gap at the soil-pile interface and reduce the negative skin friction.

The positive skin friction below 10.5 m was nearly the same as that for the maximum load in the static load test which is consistent with observations from previous pile downdrag tests (Rollins and Strand 2006; Rollins and Holenbaugh 2015;

Rollins et al. 2018, and Kevan et al. 2019). This result is contrary to what would be expected if the reduction in skin friction in the liquefied layer were simply a result of decreased vertical effective stress owing to arching in the overlying layer during reconsolidation as has been hypothesized by Orozco-Herrera et al (2024). If the vertical effective stress was reduced in the layer above the liquefied layer by arching in the overlying layer, it would also be reduced in the soil below the liquefied layer and lead to a similar reduction in skin friction; however, this was not the case.

The negative skin friction that developed following liquefaction and subsequent reconsolidation significantly increased the load in the pile above the applied load. In fact, the load at the toe of the pile was close to the maximum that which developed during the static load test. After reconsolidation, the soil around the single pile had settled approximately 2.7 cm while the pile head settlement was only 1.07 cm. Therefore, the pile was successful in reducing the pile head settlement relative to that of the surrounding ground despite liquefaction-induced dragload. Of course, this behavior could change with the pile head load and actual conditions would need to be evaluated to determine pile performance.

5 CONCLUSIONS

Based on the results from the static and blast-induced liquefaction tests, the following observations and conclusions have been made:

1. The measured ultimate pile resistance from the static load test on the tapered concrete pile was significantly higher than would be expected using recent CPT-based design methods based on load tests on cylindrical piles. In this case, the computed skin friction would need to be multiplied by a factor of 2.2 to produce agreement with the measured ultimate pile resistance, which is higher than the 1.5 factor typically employed in Italian practice. Increased skin friction on tapered piles is consistent with previous research reported by a number of researchers.
2. Following blasting, the skin friction in the liquefied zone was essentially zero; however, as excess pore pressures dissipated and the sand settled around the test pile, negative skin friction developed in both the overlying cohesive soil and the liquefied zone.
3. In the cohesive surface layer, the negative skin friction was about 80% of the ultimate positive friction during the static load test. This result suggests that the cohesive soil may have settled away from the pile sufficiently to reduce the negative skin friction.
4. In the liquefied zone following pore pressure dissipation and reconsolidation, the average negative skin friction was 53% of the ultimate positive skin friction during the static load test for most of the zone. This result is consistent with previous tests where the negative skin friction following reconsolidation was typically 40 to 60% of the positive ultimate skin friction before liquefaction. This result suggests that a tapered pile does not reduce negative friction in liquefied sand, presumably because the liquefied sand can simply flow towards the pile during liquefaction and reconsolidation. The average negative skin friction over the entire liquefied zone was 67% of the positive ultimate value prior to liquefaction.
5. In the non-liquefied sand below the liquefied zone, the positive skin friction was approximately the same as the ultimate positive skin friction measured in the static load test. This result indicates that the 50% reduction in skin friction in the liquefied layer was not due to decreases in the vertical effective stress from arching during reconsolidation.
6. Although liquefaction induced settlement in the sand around the pile was 2.7 cm, the pile head settlement was only 1.07 cm.

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