NCHRP12-116
PROPOSED AASHTO SPECIFICATIONS FOR DESIGN OF PILES FOR DOWNDRAG

Prepared for

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Transportation Research Board

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Kyle Rollins
Brigham Young University
Provo, Utah
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Task 1. Literature Survey

1.1 Studies on single piles in compressible clays

Negative skin friction is side resistance mobilized as the adjacent soil moves downward relative to a pile. Drag force (or drag load) is the axial compressive force that develops within a pile due to negative skin friction. Testing of full-scale instrumented piles to study the magnitude and development of negative skin friction dates back to the 1960s and early 1970s. Fellenius (2006) presented details of many early studies and summarized their important findings.

Bjerrum and Johannessen (1965) monitored single, steel piles driven in clay in Norway and were the first to show the development of a neutral plane—that is, the location along the pile where the load applied to the pile head plus the accumulated negative skin friction is in equilibrium with the positive resistance below (shaft and toe resistances). The neutral plane development was confirmed by Bjerrum et al. (1969), Endo *et al.* (1969), Fellenius and Broms (1969), Fellenius (1972), and Fellenius (1998).

Figure 1-1 shows the distributions of forces and settlement with depth from data published by Endo *et al.* (1969) based on in three test piles in clay after 672 days of soil settlement. Combining the test data of distributions of axial pile force and of settlement with depth, shows that the depth of the neutral place, the force equilibrium, is also the depth where the settlement of the pile is the same as the settlement of the soil, the settlement equilibrium. The latter governs the settlement of the pile head, *i.e.*, the settlement of the foundation supported by the pile. This means that when determining the neutral plane from an analysis of force distribution and correlating this to a settlement analysis, the settlement of the pile head (downdrag) introduced by the settling soil (general subsidence) can be established. Adding sustained load to the pile head will eventually reduce the depth to the neutral plane and vice versa. All other conditions being identical, a stiffer toe response will increase and a softer response will reduce the depth to the neutral plane. A change in the depth of the neutral plane will only moderately affect the maximum axial load in the pile. The pile-toe force-displacement response is a critical factor aspect for calculating the pile settlement resulting from drag force.

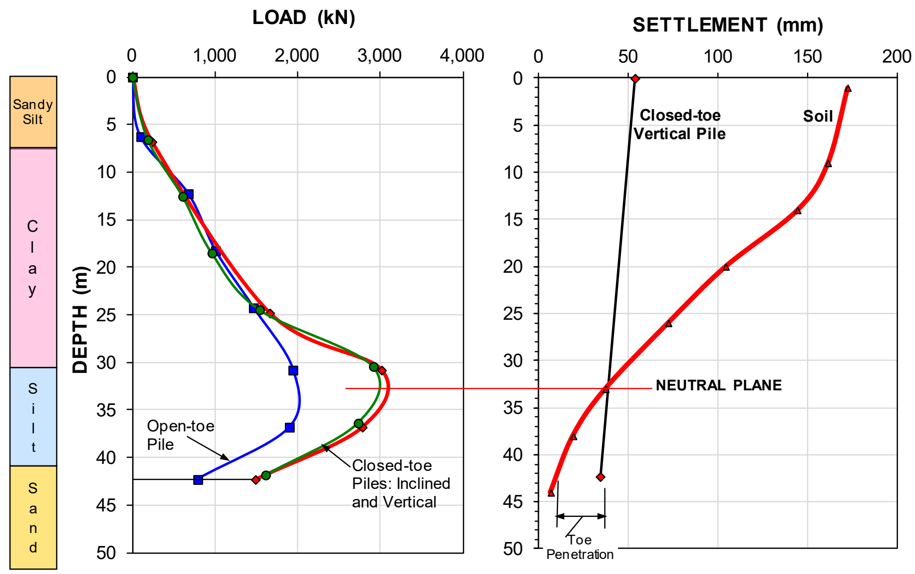

Fig. 1-1. Distribution of force in the three full-length test piles and of soil settlement 672 days after start [Data from Endo et al. (1969)].

Bozozuk and Labrecque (1969), Bozozuk (1970; 1972; 1981), and Bozozuk *et al*. (1972) reported long-term measurements on a 49 m long, 320 mm diameter, closed-toe floating, pipe pile installed in Berthierville, Quebec, Canada. Following placement of a highway fill embankment, the test pile was driven through the fill embankment and into the native clay. The axial forces were monitored for ten years after driving and a static loading test was then performed. Over time, negative skin friction developed from the ground surface to the neutral plane and positive friction developed below the neutral plane as shown in Figure 1-2. Following the application of static load at the pile head, positive skin friction developed from the ground surface downward until pile failure occurred. Thus, the development of negative skin friction did not reduce the compressive geotechnical resistance of the pile.

Interpretation of the data prior to the static load test indicated that the shaft resistance was governed by effective stress, that the accumulated negative skin friction was equal in magnitude to the accumulated positive shaft resistance and mobilized toe resistance (which was small), and that the shaft resistance was a function of the horizontal stress. Furthermore, the mobilization of shaft shear occurred as a result of only about 4 mm (or 0.15 in) movement between the pile and the surrounding soil. Following static loading, the positive skin friction above the original neutral plane was approximately equal in magnitude to the negative skin friction that had previously developed in this zone.

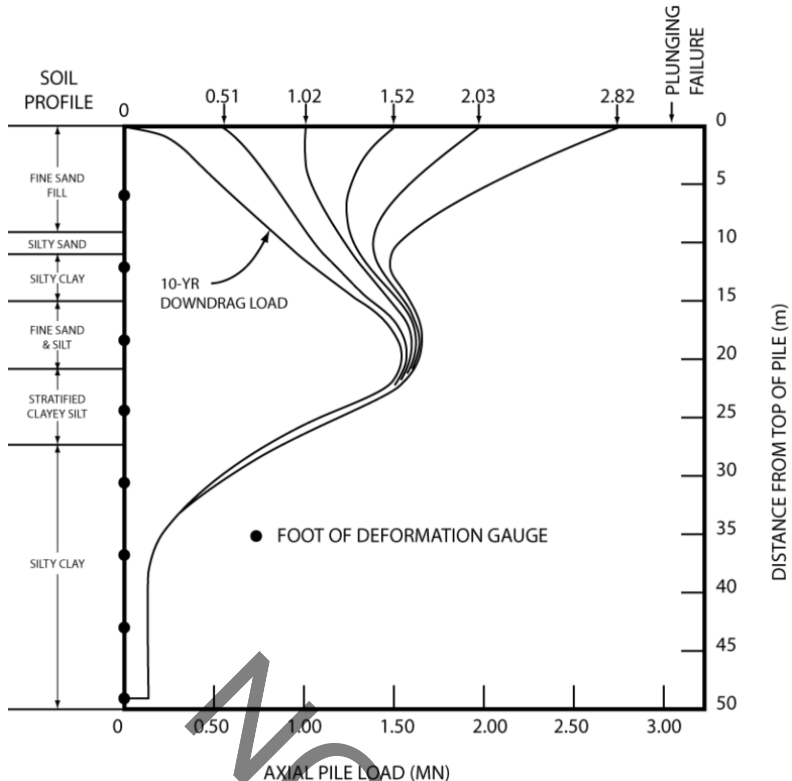

Fig. 1-2. Load distribution in test pile at various levels of applied load (from Bozozuk 1981).

Rollins and Sears (2008) measured drag force in 40 cm (16 in) OD steel pipe piles at two bridge abutments in Salt Lake City, Utah prior to fill placement, during fill placement and consolidation settlement, and during subsequent construction of the bridge superstructure. The piles were driven through 15 to 18 m (48 to 60 ft.) of compressible clays and into a silty sand bearing layer. Negative friction developed in both test piles as a result of consolidation settlement. In one case, structural loads from the bridge superstructure were applied to the piles prior to the completion of consolidation settlement. As structural dead loads were applied, drag force decreased to some extent, due to the development of positive friction from the pile head downward. However, within a short time, the continuing consolidation settlement brought the drag force back to about its original value and increased the total pile load at the neutral plane by the amount of the applied pile head load. In the other case, structural loads from the bridge were applied to the pile after the completion of consolidation settlement. Once again, positive skin friction developed from the pile head downward but the increase in pile load at the neutral plane was only about 50% of the applied dead load as shown in Figure 1-3. Pile load at the neutral plane increased very little over 400 days of observation following completion of the bridge. Unfortunately, static load tests and ground settlement versus depth profiles were not obtained for these abutment pile tests.

Walker *et al*. (1973) installed two 760 mm diameter, 22 m long, open-toe pipe piles into an interbedded sand, silt, and gravel profile. One of the test piles was coated with bitumen along its entire length. After completion of pile driving, fill was placed in the area of the two piles. Over the next 238 days, a large drag force developed in the uncoated pile due to settlement induced by the fill. In contrast, the coated pile attracted insignificant negative drag force. The ground settlement measured at the ground surface was only about 25 mm (1 inch) during the monitoring period. The pile head settlement was small and corresponded to the compression of the pile due to the axial load.

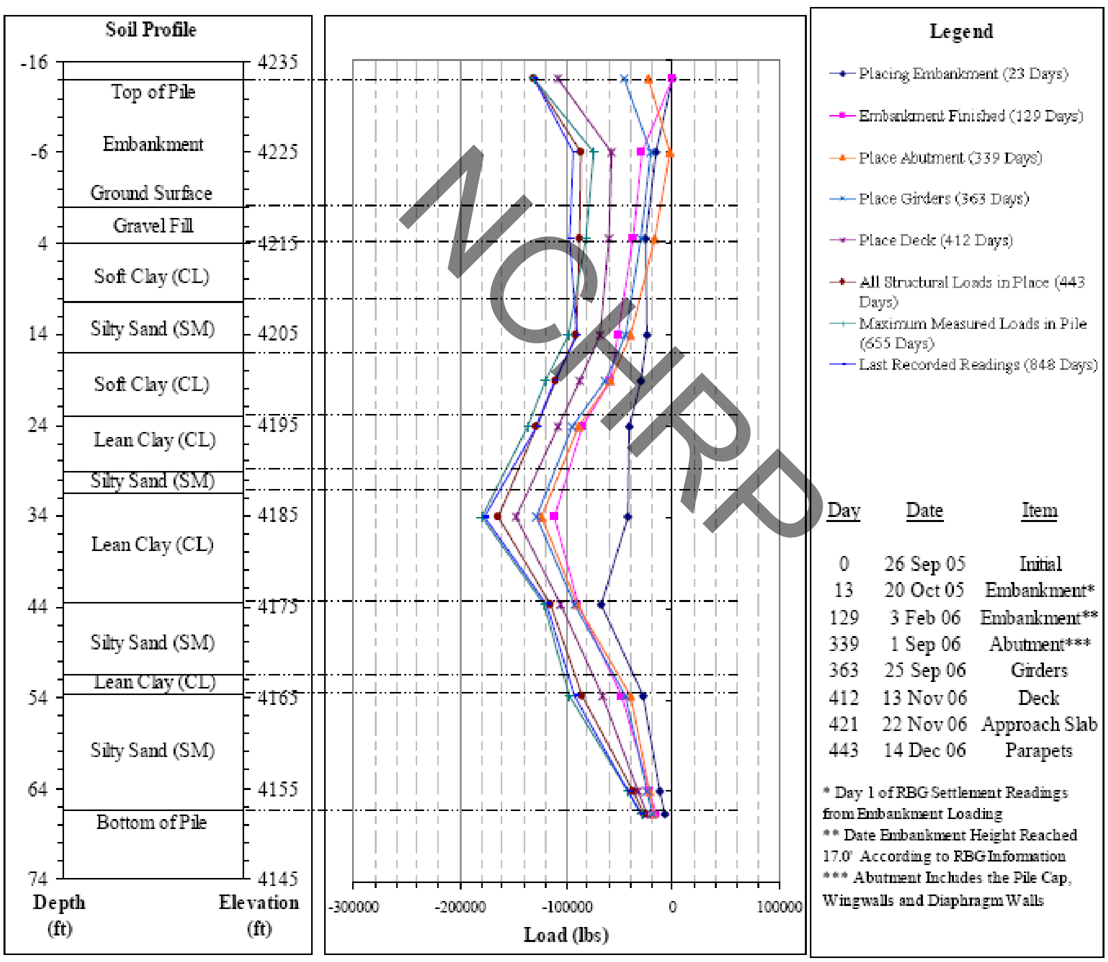

Fig. 1-3. Axial load distribution in bridge abutment pile following consolidation settlement from approach fill placement and subsequent application of dead load from construction of bridge superstructure. Positive skin friction develops from the pile head downward during dead load application after completion of primarily consolidation settlement (Rollins and Sears, 2008).

Since the 1960's and early 1970's, many other studies have been published addressing the development of negative skin friction on single piles in settling ground due to a variety of causes (Garlanger 1974, Auvinet and Hanell 1981, Clemente 1979; 1981, Keenan and Bozozuk (1985), Leung *et al.* 1991, Indraratna *et al*. 1992, Rollins and Strand 2006, Vijayaruban et al. 2015, Fellenius *et al.* 2015, Muhunthan *et al.* 2017, Elvis 2018). Only a few studies involve piles without an obvious cause for ground subsidence. Fellenius (2001) presented an analysis of instrumented bored piles that were dynamically tested and illustrated the presence of drag force. Altaee et al. (1992) instrumented a precast concrete pile that exhibited substantial drag force due to the effects of installation. Siegel and McGillivray (2009) monitored a single cast-in-place pile in Rincon, Georgia, USA with zero load applied to the pile head and no external cause of ground settlement and determined that negative skin friction fully mobilized over a period of 58 days from installation. Vipulanandan et al. (2007) monitored strain in a cast-in-place pile in dense sand for a time period of 7 days after installation at a site in Texas USA and observed compressive strain increasing with depth with zero load at the pile head.

Okabe (1977) monitored strain-gage instrumented, 600 mm diameter, 43 m long, pipe piles driven through silty clay and silt with silty sand. A fill was placed on the ground over a vast area of the site and pumping of water at depth lowered the pore pressures at depth. The axial force distribution in one single pile increased continually and a neutral plane developed at about 40 m depth. After the first 3 months, the maximum drag force was about 4,000 kN and after 4.5 years, the maximum force had increased to about 7,000 kN. An identical second test pile was also monitored and showed a similar response. However, after 1.5 years, a 700 kN load was applied to the pile head and after an additional two months, the applied load was raised to 1,700 kN. Both load applications reduced the drag force at first, but it re-developed over the next 2 months. The final depth to the neutral plane was 30 m and the maximum drag force was 3,000 kN.

Fellenius (1988) identified changes in effective stress during reconsolidation of the soil after pile construction as a possible cause of negative skin friction development. Fellenius (1969), Leung at al. (1991), and Leifer (1992) showed that a transient (live) load will not coexist with a drag force.

While much of the drag force research and associated literature focuses on measured drag force on single vertical piles in settling ground, attention has also been given to other fundamental pile response (Nishi and Esahi 1982, Burland and Starke 1994, Lam *et al.* 2013, Lucarelli *et al*.

2014, Tan and Fellenius 2016, Siegel and Lucarelli 2017), and specifically, inclined piles (Takahashi 1985, Sawaguchi 1989, Davisson 1993, and pile groups (Lee et al. 2001; 2002, Fellenius 2017).

In summary, the results of the many field tests show that negative skin friction on single piles can accumulate to a very large drag force, that the distribution of negative skin friction is proportional to the effective stress, and that the relative movement between pile and soil required to fully mobilize shaft resistance can be insignificantly small. For example, substantial drag force was observed where the ground surface settlement rate was as little as 1 to 2 mm (0.04 to 0.08 in) per year.

1.2 Studies on pile groups in compressible clays

Okabe (1977) also monitored the force development of four piles in a pile group comprised of 38 piles placed in an octagonal formation. All piles were connected with a concrete raft about 300-mm thick. No load was applied to the raft. The weight of the raft was almost 8,000 kN, that is, about 200 kN/pile. The pile spacing was 1.7 pile diameter and the footprint ratio was 22%. The free distance between the piles was about the same value as the raft thickness. One of the four monitored piles was along the perimeter and the other three were interior piles. Figure 1-4 shows the layout of the pile group and the axial force distributions measured in the monitored piles after 1,040 days.

The response of the perimeter pile was very similar to that of an adjacent single pile. Because the raft prevented the pile from being dragged down by the soil subsidence, a tensile force of about 600 kN developed at the pile head, which lowered the neutral plane causing the entire length of the perimeter piles to be affected by negative skin friction.

The axial load distribution for the interior piles was neither affected by negative skin friction nor positive shaft resistance; the pile head load went unaltered to the pile toe region. The 14 perimeter piles did not support the weight of the raft but actually added load to the 24 interior piles. Note that the pile head load measured for the interior piles was about 600 kN/pile as opposed to the about 200 kN/pile weight of the raft. The reason for the difference was that the interior piles had to resist the load portion of the 14 perimeter piles as well as the tension force due to drag force.

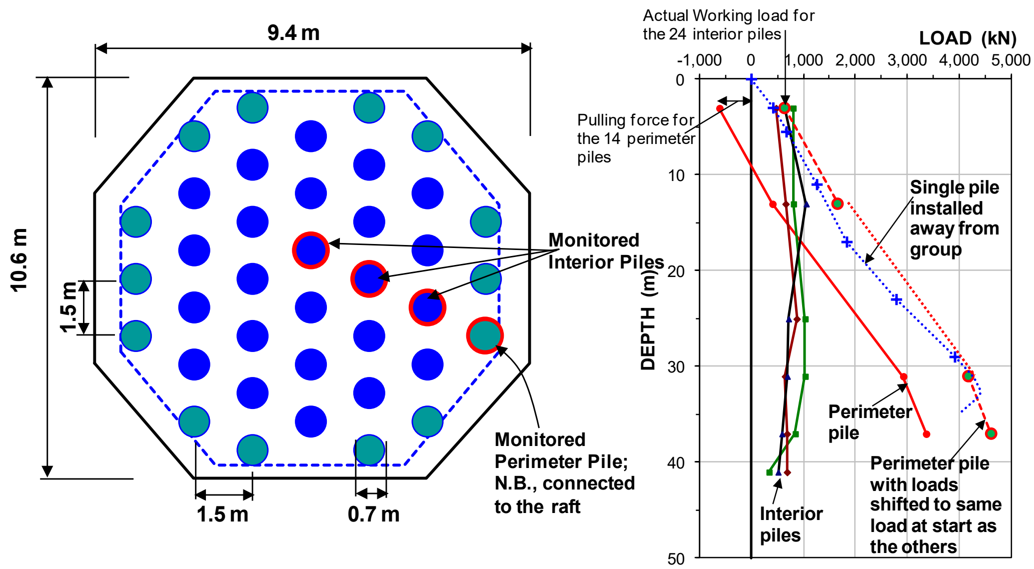

Fig. 1-4. Layout of pile group and distribution of axial force in the four test piles and in a single test pile away from the group. (After Okabe 1977)

Russo and Viggiani (1995) and Mandolini *et al.* (2005) monitored forces in perimeter and interior piles of a group of 144 piles mandrel-driven, then concrete-filled, steel pipe piles, 406-mm diameter, 48 m long, uniformly distributed in a 10.6 m by 19.0 m raft in a rectangular configuration supporting a bridge abutment during and following construction of a cable-stayed bridge over the Garigliano River in Southern Italy; constructed in 1991-94. The soil profile consisted of about 10 m of clay on about 10 m of dense sand underlain by soft clay deposited at about 48 m depth on a very dense sand and gravel bed. The pile c/c distance was 1.2 m (3.0 pile diameters). The footprint ratio was 9%. The clay was undergoing small regional subsidence. Figure 1-5 shows the load distributions in corner, side, and interior piles during and after the construction. At the end of the 500-day construction period, the interior piles resisted 60% of the attributed to the corner piles. After the bridge had been constructed, two trends in the distribution of pile loads can be seen: the load on the interior piles increased and the load on the corner and side piles decreased, while the total load on all of the piles increased by 10%. After completion of construction, the regional subsidence developed negative skin friction along the perimeter piles, causing their response to the raft load to become less stiff, thus, reducing their ability to take on load from the raft. The load on the corner piles reduced significantly and the load on the side piles reduced slightly, while the load on the interior increased as the load was transferred to the interior piles. Moreover, the increase is also due to accumulated negative skin friction (drag force) on the perimeter piles that added load to the total load on the raft, which was then supported by the interior piles.

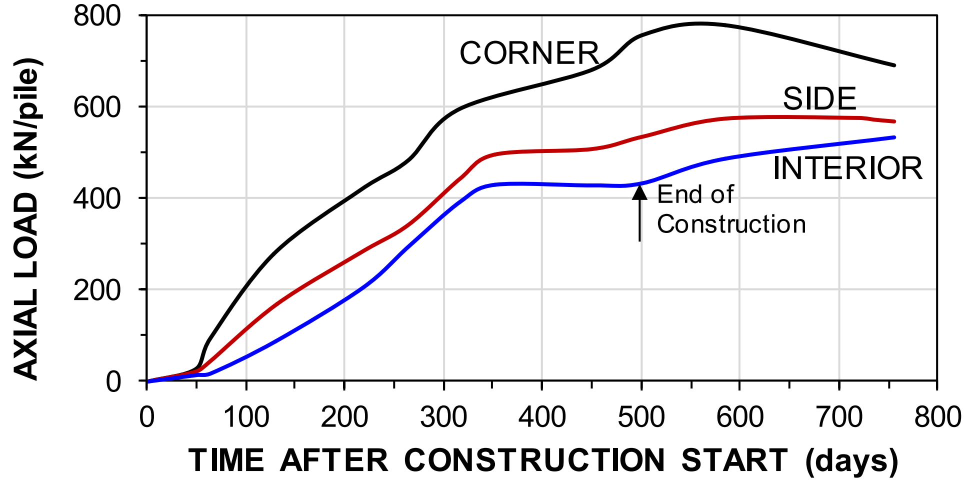

Fig. 1-5. Measured axial load during and after construction (data from Russo and Viggiani 1995).

Inoue *et al.* (1969) presented a case history of monitoring settlement of a three-story building with a plan dimensions of 15 m by 100 m founded on 500 mm diameter open-toe pipe piles driven through sand and silty clay to bearing within a sand layer at about 35 m below the ground surface. The piles had more than adequate geotechnical resistance to support the building. Two years after construction, about 150 mm of differential settlement across the pile foundation was noticed to have occurred. Measurements during the following two years showed about 200 mm additional settlement. The settlement was downdrag due to pumping of water in the bearing soil below the neutral plane. The building could not be repaired but had to be demolished.

Luna *et al.* (2015) instrumented and monitored rock-socketed micropiles that were installed in a circular group of ten per cap for an elevated roadway in mountainous terrain in eastern Tennessee USA. The results showed drag force within a year after installation. Budge and Dasenbrock (2011) and Budge *et al.* (2015) monitored instrumented piles at five sites in Minnesota USA including in-service piles within a group. Negative skin friction and a distinct neutral plane developed at several of the Minnesota test sites.

1.3 Applying the information to piled foundation design in compressible clays

On the basis of long-term pile monitoring data, Fellenius (1984; 1988) concluded that essentially all piles will progress toward equilibrium where the sustained applied load, if present, and the cumulative negative skin friction (i.e., drag force) will act downward and be opposed by the positive shaft resistance and mobilized toe resistance.

Fellenius (1984; 1988; 2004; 2006; 2017; 2018; 2019) developed a method for design analysis of single pile and small pile groups called the Unified Method. The method correlates the force and settlement distributions and the force-movement response of the pile toe to determine the depth to the neutral plane. The method makes use of the dependency of force equilibrium on the toe force and the dependency of settlement equilibrium on the toe movement. The location of the neutral plane is iteratively adjusted until the pile head load and drag force are in equilibrium with the positive friction and mobilized end-bearing. In addition, the settlement of the pile toe, based on the neutral plane settlement, must be consistent with the movement necessary to develop the required toe resistance for force equilibrium.

The Unified method is illustrated in Figure 1-6. The force distribution graph shows the development of axial force in the pile at three events: (1) after pile installation just before the supported structure has been placed on the pile applying a sustained load, and (2) just after completion of the supported structure, and (3) the long-term distribution.

The settlement graph shows the distribution soil settlement (N.B., settlement below the pile toe level is not shown). The graph also shows the pile toe penetration for the pile-toe force indicated in the first graph and as a function of the pile-toe load-movement relation shown below the graph. The distribution of pile settlement is shown with the pile compression for the axial load added to the pile-toe penetration. When the pile-toe force in the load-distribution diagram matches the pile-toe penetration in the settlement diagram per the particular q-z function for the pile toe, the neutral plane determined as force or by settlement equilibrium will be at the same depth and the loop shown as a dashed line will have closed. The q-z function is the pile-toe load-movement response as determined experimentally in a static loading test or by a theoretical analysis pertinent to the pile and site.

The two-graphs in Figure 1-6 demonstrates that a stiffer pile response corresponds to a small pile settlement, while conversely, the softer the pile response (notably the pile toe response) results in a larger the pile settlement. It is obvious that whether a pile is acceptable or not acceptable as a foundation unit depends on the settlement response, not on the magnitude of maximum axial force—provided that the maximum axial force can still be accepted structurally by the pile.

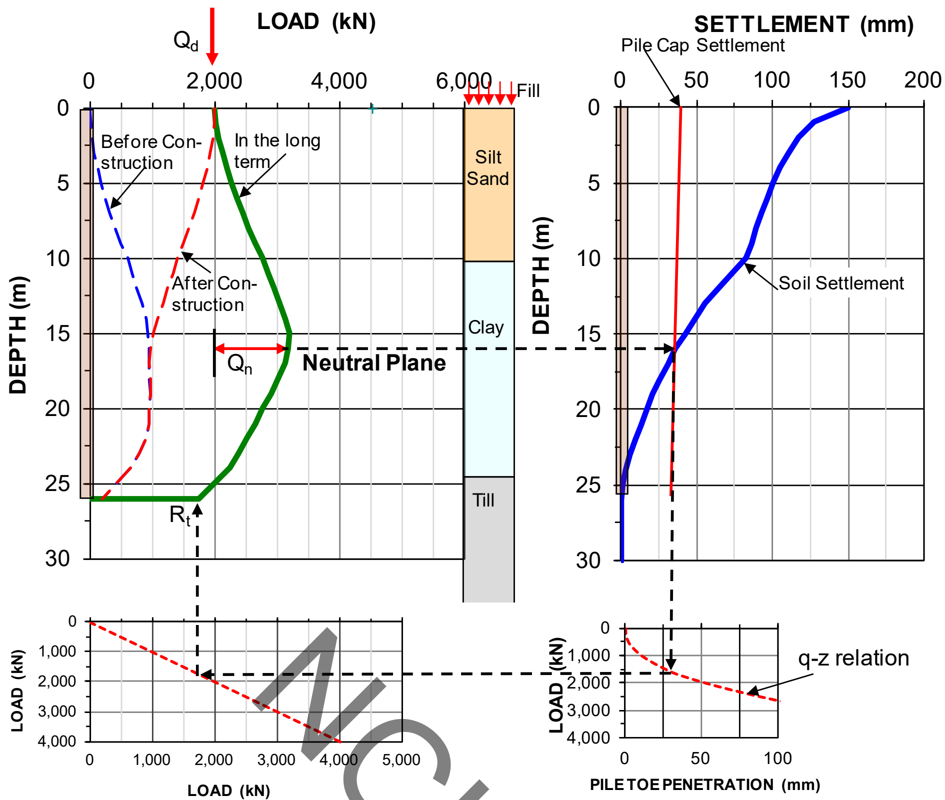

Fig. 1-6. How force and settlement distribution combined with pile-toe force-movement determine the depth to the neutral plane and, eventually, the pile settlement.

The Unified Design has been adopted by several prominent geotechnical manuals including the Canadian Foundation Engineering Manual (2006), the Australian Piling Standard (2009), and the Hong Kong Foundation Design and Construction Manual (Hong Kong Geotechnical Office, 2006). The Unified Design of Piles adapted to the LRFD framework (Siegel *et al.*, 2013 and 2014) is included in the FHWA's Design and Construction of Driven Pile Foundations (Hannigan et al., 2016). A large number of other design approaches have been proposed including Long and Healy (1974), Alonso et al. (1984), Briaud and Tucker (1996), Eurocode (2004), Davisson (1993), Singapore Code of Practice (2003), Poulos (2008), Wang and Brandenberg (2013), and AASHTO (2014).

A synthesis of the published literature indicates that it is generally accepted that a thin bitumen coating can be sufficient to substantially decrease the negative skin friction, that is, the shaft resistance in either positive or negative direction (Fellenius 1979, Clemente 1979; 1981, Briaud and Tucker 1996, Khare and Gandhi 2007).

Fellenius (2019) and Fellenius et al. (2019) confirmed the Russo and Viggiani (1995) case history in a Plaxis analysis indicating that interior piles in a wide pile group (from four or more piles in a row) transfer a load applied to the pile head to the soil starting at the pile toe level, whereas perimeter piles essentially start mobilizing the soil from the ground surface similar to single piles. Thus, an interior pile has minimal negative skin friction and the positive shaft resistance develops near the pile toe level. Perimeter piles will not only appear stiffer than interior piles at first loading and, thus, be receiving more than their share of the average pile load. If installed at a site with subsiding soils, they will be subjected to negative skin friction and downdrag that, depending on the rigidity of the raft, will transfer load to the interior piles—both portion of the applied load and portion of the drag force.

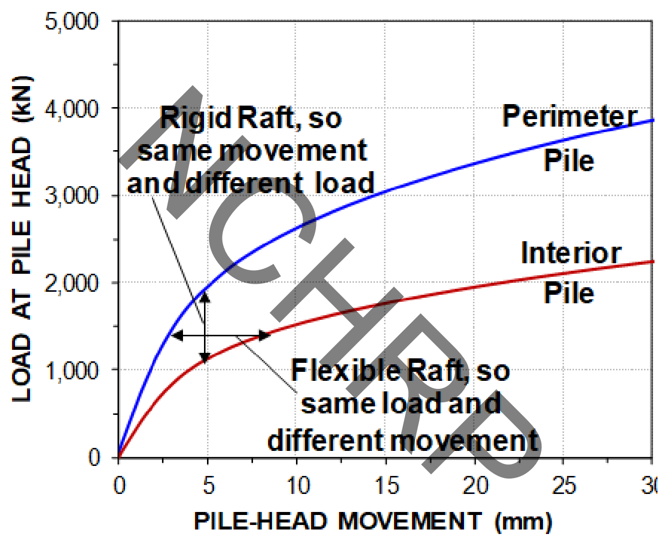

Fig. 1-7. Load distribution for a perimeter pile and an interior pile in a wide pile group.

The movement of the raft (the pile heads) under an absolutely rigid raft will move equally, but have quite different loads, whereas piles under a flexible raft will have the same load but the interior piles will move more. This is illustrated in Figure 1-7.

If negative skin friction develops, the load received by perimeter piles will reduce and be transferred to the interior piles for a rigid or flexible pile cap. The effect is important for the design of the actual pile cap. Through rational analysis, it can be shown that negative skin friction does not decrease the ultimate geotechnical resistance (pile bearing capacity). The drag force and the applied sustained load concern only the internal compressive axial strength.

2.1 Studies on single piles in following liquefaction-induced settlement

In contrast to non-liquefiable layers, where the negative skin friction is typically found to be equivalent to the positive skin friction, the negative skin friction immediately following liquefaction is likely to be a very small fraction of the pre-liquefaction value or perhaps zero. Nevertheless, as the earthquake-induced pore pressures dissipate in the liquefiable layer and settlement occurs, the skin friction at the pile-soil interface is likely to increase. Therefore, the negative skin friction which ultimately develops will likely be higher than zero and may depend on the dissipation rate 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 drag loads. Boulanger et al. (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, Rollins and his co-workers have conducted a number of full-scale field tests involving blast-induced liquefaction as summarized in Table 1. Blast-induced liquefaction was first used to investigate the lateral resistance of piles in liquefied sands (Ashford *et al*. 2005, Rollins *et al*. 2005) and has become widely used to investigate a number of ground improvement strategies (Wentz *et al*. 2015, Rollins *et al*. 2004, Ashford *et al*. 2000).

Rollins and Strand (2006) conducted a full-scale load test using a 324 mm diameter steel pipe pile driven to a depth of 21 m in Vancouver, Canada. As shown in Figure 1-8, the soil profile with a water table at 3.5 m consisted of non-liquefiable soils to a depth of about 5 m underlain by loose liquefiable silty sand with a relative density of about 40% to a depth of 15.m The loose silty sand was underlain by a sand with a relative density of 50 to 60%.

Table 1 Summary of blast-induced liquefaction downdrag tests.

Site Location	Pile Type	Soil Profile	Reference
Vancouver, Canada	**Driven Steel Pipe** 12.75" diameter 70 ft long	20 ft of cohesive soil over loose clean sand (Dr=40%)	Strand & Rollins (2006) Rollins *et al.* (2018)
Christchurch, New Zealand	**Three Augercast Piles** 20" diameter 28 ft long 39 ft long 46 ft long	5 ft of cohesive soil over medium silty sand (Dr = 60%)	Rollins and Hollenbaugh (2015) Rollins *et al.* (2018)
Mirabello, Italy	**Bored piles** 10" diameter 50 ft long	20 ft of cohesive soil over 10 ft sandy silt and 60 ft of silty sand	Rollins *et al.* (2019), Amoroso et al. (2017)
Turrell, Arkansas	**Three Driven Piles** H pile (H14 x 117) 92 ft long Pipe pile (18" diam.) 78 ft long 18" square PSC pile 74 ft long **Three Drilled Shafts** 4 ft dia., 90.5 ft long 6 ft dia., 70 ft long 4 ft dia. 92 ft long	30 ft of cohesive soil over silty sand and sandy silt	Kevan *et al.* (2019) Ishimwe *et al.* (2018)

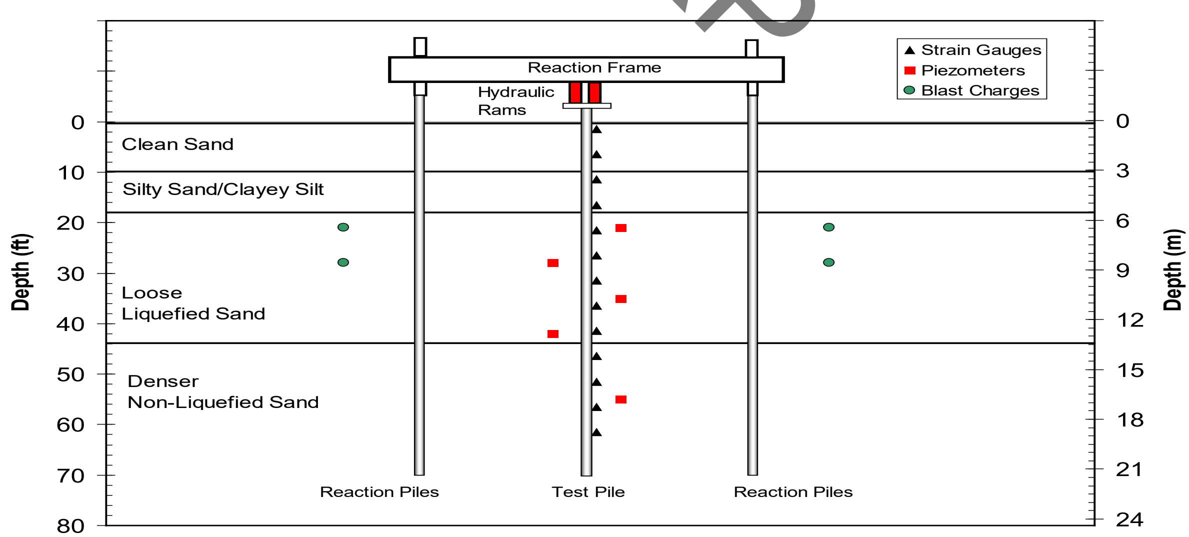

Fig. 1-8. Schematic drawing showing soil profile at Vancouver, Canada test site along with test pile, reaction piles, pore pressure transducers, strain gauges and blast charge locations

Reacting against the load frame, the hydraulic jacks initially applied a load of 536 kN which was about 50% of the ultimate pile resistance based on the Davisson criteria. On the basisi of pore pressure transducer measurements, detonation of the sequence of explosive charges produced liquefaction from about 5.5 m to 13 m. Reconsolidation of the liquefied sand produced 27 cm of settlement or about 3% volumetric strain, similar to what would be expected for liquefaction produced by an earthquake.

Figure 1-9 provides a summary of the load in the pile versus depth before liquefaction, immediately after liquefaction, and at the completion of pore pressure dissipation. Prior to blasting pile head load was transferred to the surrounding soil primarily by side friction. At the onset of blasting, the test pile settled slightly so that the load applied by the hydraulic jacks dropped by 156 kN at the top of the pile. When this 156 kN load was re-applied, this load was resisted by positive skin friction from the top downward in the upper section of the pile. It should be noted that the total measured skin friction from the ground surface to a depth of 6 m immediately prior to blasting was approximately 166 kN. Therefore, the redevelopment of positive skin friction due to this applied load appears to be reasonable. The load of 536 kN was maintained throughout the remainder of the test by adding hydraulic fluid to the jack as the pile began to settle and relieved the load. This apparently maintained the positive friction in the upper 6 m of the pile. This result indicates that it would be desirable to apply dead load to the top of the pile in future tests to avoid the complication of re-application of pile head load.

Following liquefaction, load transfer within the liquefied zone dropped to near zero and the load originally carried by positive skin friction liquefied in this zone was transferred to the lower end of the pile where liquefaction had not developed. As a result, at the base of the liquefied zone the load in the pile increased by 130 kN after blast induced liquefaction and the pile settled about 4.5 mm as a result of mobilization of skin friction and end-bearing in the underlying sand layer.

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 1-9. The negative slope indicates that negative skin friction had developed in this zone and was applying drag load to the pile. The drag load produced during reconsolidation was approximately one-half of the positive skin friction force prior to liquefaction. As the pore pressures dissipated and effective stresses increased, the skin friction at the pile interface also increased and produced a drag load of about 100 kN (22.5 kips). This load was again transferred to the sand below the liquefied zone with a resulting additional pile settlement of about 2.5 mm or a total pile settlement of 7 mm.

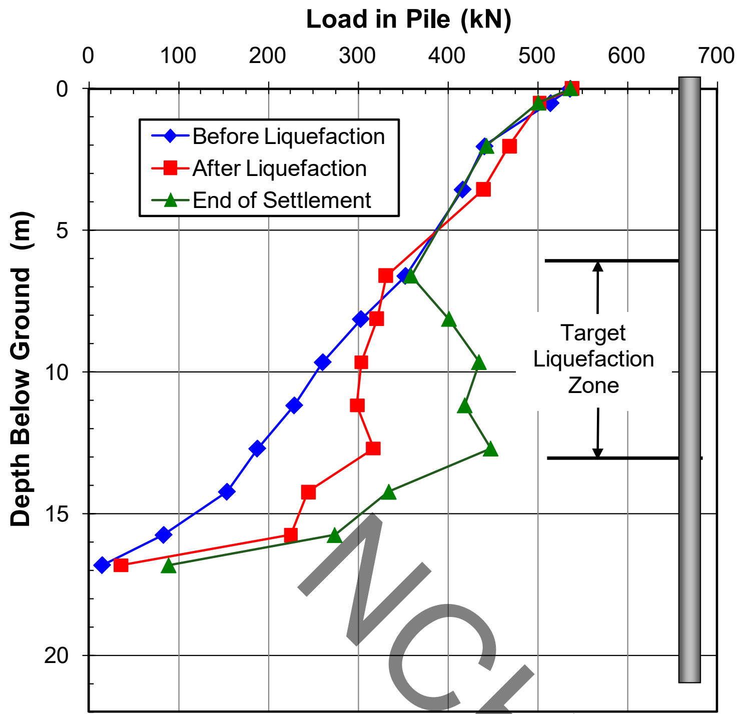

Fig. 1-9. Pile load vs. depth curves before blasting, immediately after blasting and after settlement of the liquefied layer at Vancouver, Canada (Rollins and Strand, 2006).

Rollins *et al*. (2015) report results of additional blast liquefaction tests on three 60 cm diameter continuous flight auger piles in Christchurch, New Zealand. The three test piles were installed in a triangular arrangement at 2 m center-to-center spacing to depths of 8.5, 12, and 14 m, respectively. In these tests, the soil profile consisted of a 1.5 m thick layer of sandy silt underlain by poorly graded medium-dense clean sand to a depth of 10.5 m. This layer was in turn underlain by inter-bedded layers of medium-dense to dense clean sand.

Two blast-induced liquefaction downdrag tests were performed on the piles to evaluate their performance with and without applied static load. In the first blast test here was no load applied to the piles. Detonation of a sequence of small explosive charges liquefied a layer of sand from the water table at 1.5 m to a depth of about 13 m. Ground settlement was approximately 4 cm immediately around the group but higher beyond it. Because the ground settled more than the piles (1 to 2 cm), 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 1-10 for the conditions 60 minutes after blasting when liquefaction induced settlement was completed. Because no pile head load is applied, any load in the piles is induced by negative skin friction or drag load above the neutral plane. Clearly, the negative skin friction is not zero at the end of consolidation. The neutral plane is visible in each of the plots as the point where the load in the pile begins to decrease. Because the neutral plane in each case is located within the liquefied layer, rather than at the bottom of the liquefied layer as suggested by some design procedures, positive skin friction below the neutral plane is also occurs within the liquefied zone as reconsolidation occurs. The depth to the neutral plane increased as the length of the pile increased suggesting that the pile settlement decreased as the pile length increased.

About one month after the initial blast induced liquefaction tests, static load tests were performed on each pile using dead weights (Rollins and Hollenbaugh, 2015). Figure 1-10 also presents dashed lines showing the load in the pile assuming 50% of the average positive skin friction found in the static load test along the pile length were liquefaction occurred. Because the neutral plane is located within the liquefied zone, both negative and positive skin friction are reduced by 50% in the computations. Agreement with the measured curves is generally very good and confirms the reduced skin friction value obtained from the test in Vancouver.

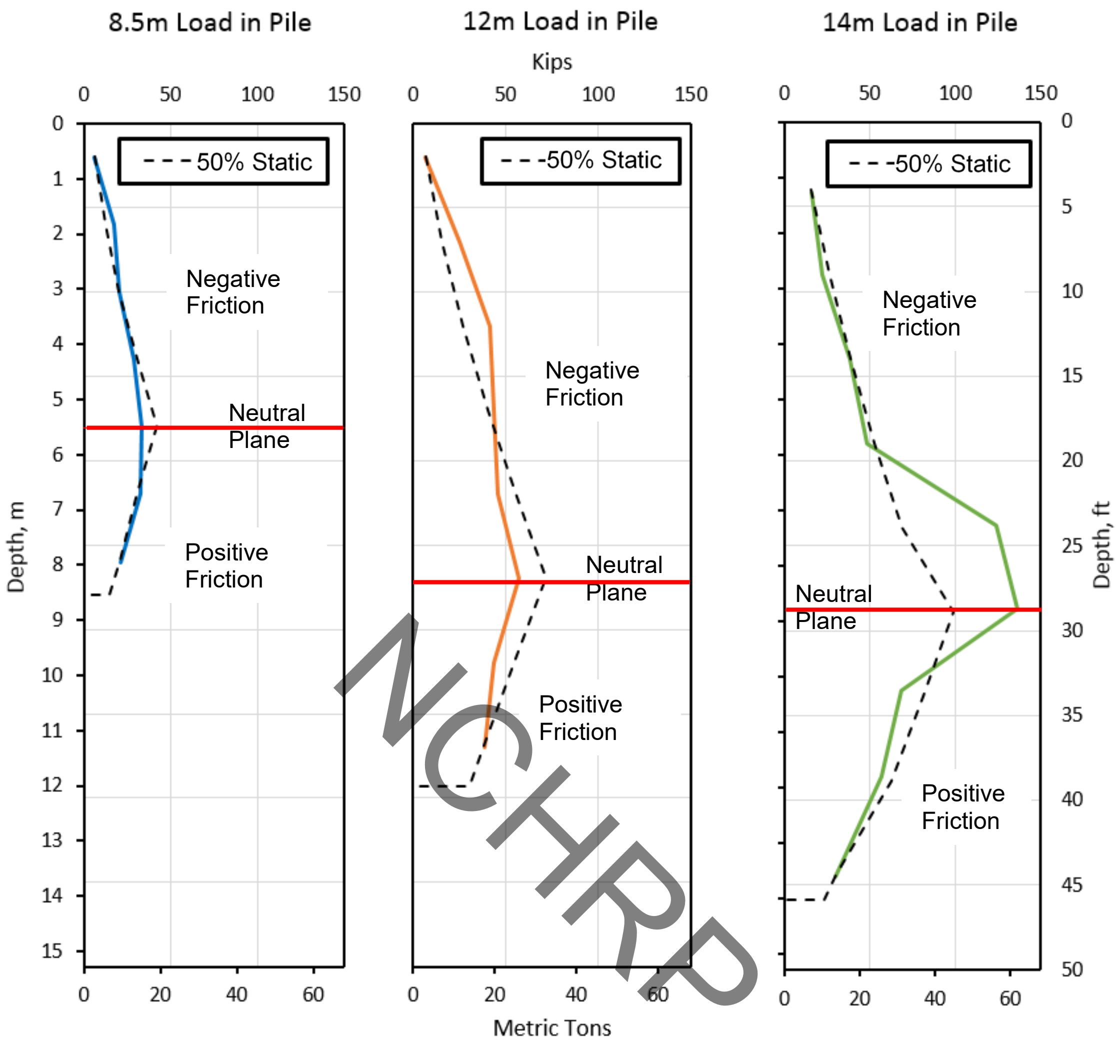

Fig. 1-10. Interpreted pile load versus depth curves (solid lines) following blast liquefaction along with predicted curves (dashed lines) assuming skin friction equal to 50% of measured average positive skin friction from the static load test. The neutral plane is shown in each plot with a horizontal line separating negative skin friction above from positive skin friction below (Rollins et al. 2018).

Following the static load tests, a total of 300 tons of dead load was distributed among the test piles prior to a subsequent blast-induced liquefaction downdrag test. The load carried by each test pile was measured by a load cell on the top of each pile. In this test, liquefaction developed from a depth of 3 to 7 m below the ground surface. Because of the load on the piles they settled more than the surrounding ground and positive skin friction developed even within the liquefied layers. Skin friction within the non-liquefied layers was roughly the same as that measured prior to liquefaction, while skin friction in the liquefied zones immediately after reconsolidations was about 40% of the pre-liquefaction skin friction.

Rollins *et al*. (2019) and Amoroso *et al*. (2017) describe results from a blast-induced downdrag test conducted on a 25-m diameter mircropile at a test site in Mirabello, Italy, where liquefaction was observed in the M_w 6.1 Emilia Romagna earthquake in 2012. As shown in Figure 1-11, the soil profile consists of about 6 m of non-liquefiable cohesive soil underlain by a 2-m thick sandy silt layer and a 10-m thick sand layer. The test pile extended to a depth of 17 m but was not loaded.

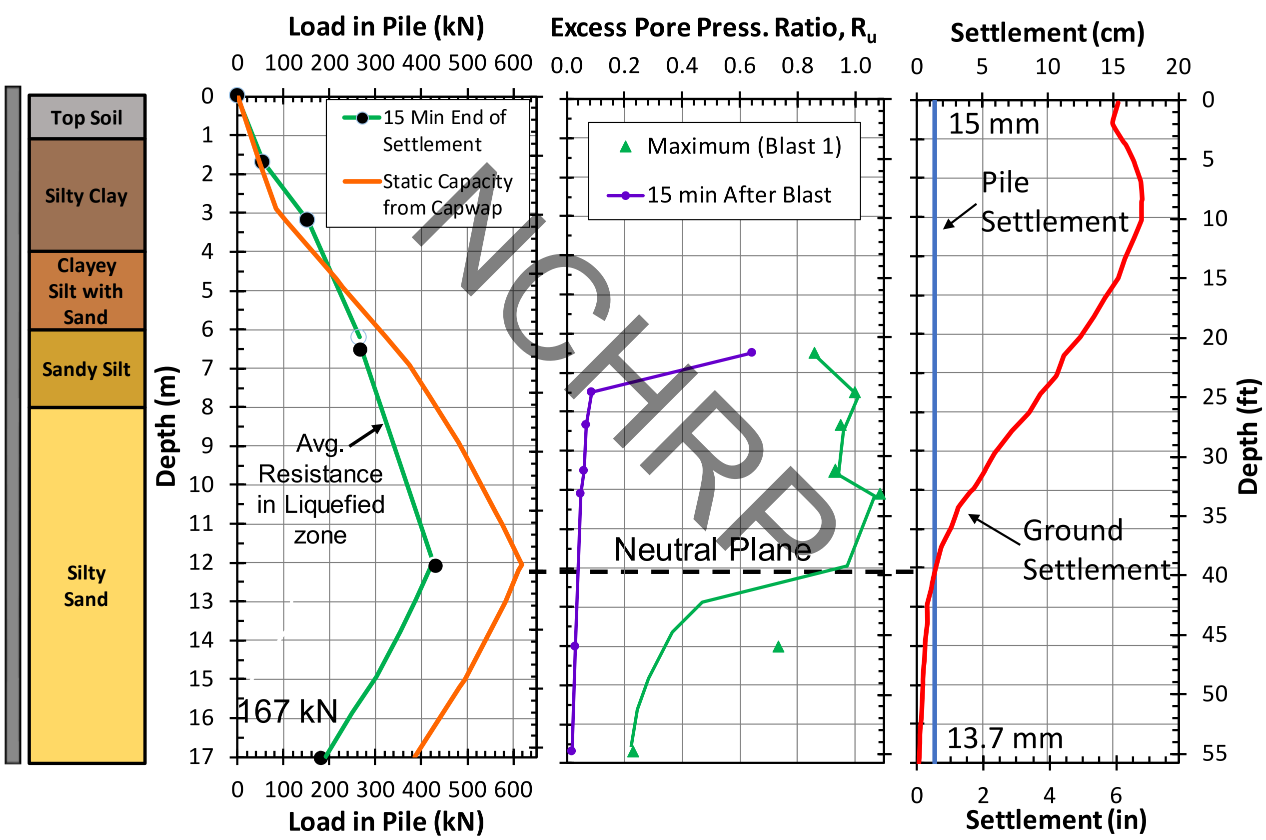

Fig. 1-11. Plots showing soil profile, load in the pile, and excess pore pressure ratio, along with soil and pile settlement following blast induced liquefaction. Rollins et al. (2019).

The blasting sequence liquefied a layer from about 6 to 13 m, resulting in about 15 cm of settlement at the ground surface although the test pile only settled about 15 mm. A Sondex settlement profilometer was used to record settlement with depth as show in Figure 1-11. As excess pore pressure dissipated and the sand reconsolidated, the liquefied layers from 6 to 12 m settled about 11.5 cm (≈ 2% volumetric strain). The cohesive surface layer largely settled as a block on top of the liquefied layer and settlement below 12 m was relatively minor (less than 1.5 cm). The volumetric strain in the liquefied zone produced by blasting is consistent with that expected in an earthquake based on predictive equations developed by Ishihara & Yoshimine (1992) and Zhang *et al*. (2002).

During re-consolidation of the liquefied soil, negative skin friction developed from the ground surface to the neutral plane at a depth of about 12 m where pile settlement and soil settlement were equal as shown in Figure 1-11. The negative skin friction in the non-liquefied layers was similar to the positive skin friction prior to liquefaction; however, the average negative friction in the liquefied layers was only about 50% of the positive skin friction based on CAPWAP measurements without liquefaction. Significant end-bearing resistance was mobilized at the toe of the pile as a result of dragload that produced a settlement equal to about 4% of the pile diameter at the toe even without any load at the top of the pile.

Kevan *et al*. (2019) and Ishimwe *et al*. (2018) report results from blast-induced downdrag tests conducted on three driven piles and three drilled shafts at a test site near the Mississippi River in Turrell, Arkansas. The driven piles consisted of a H pile (HP14x117), a 46-cm (18-inch) diameter pipe pile and a 46-cm (18-inch square) pre-stressed concrete pile. The drilled shafts consisted of two 4-ft diameter shafts and one 6-ft diameter shaft. The test piles were loaded using a pile cap and steel beams.

The soil profile consisted of a 9 m of non-liquefiable clay underlain by liquefiable silty sand and sand layers, underlain by a dense sand layer as shown in Figure 1-12. Three separate blast tests were performed involving one shaft and one driven pile and typical results are provided in Figure 1-12. Liquefaction was typically induced within the 5-m thick silty sand layer and elevated pressure extended into the underlying sand. Liquefaction produced ground surface settlements of about 75 to 100 mm or a volumetric strain of about 1.0 to 1.5%.

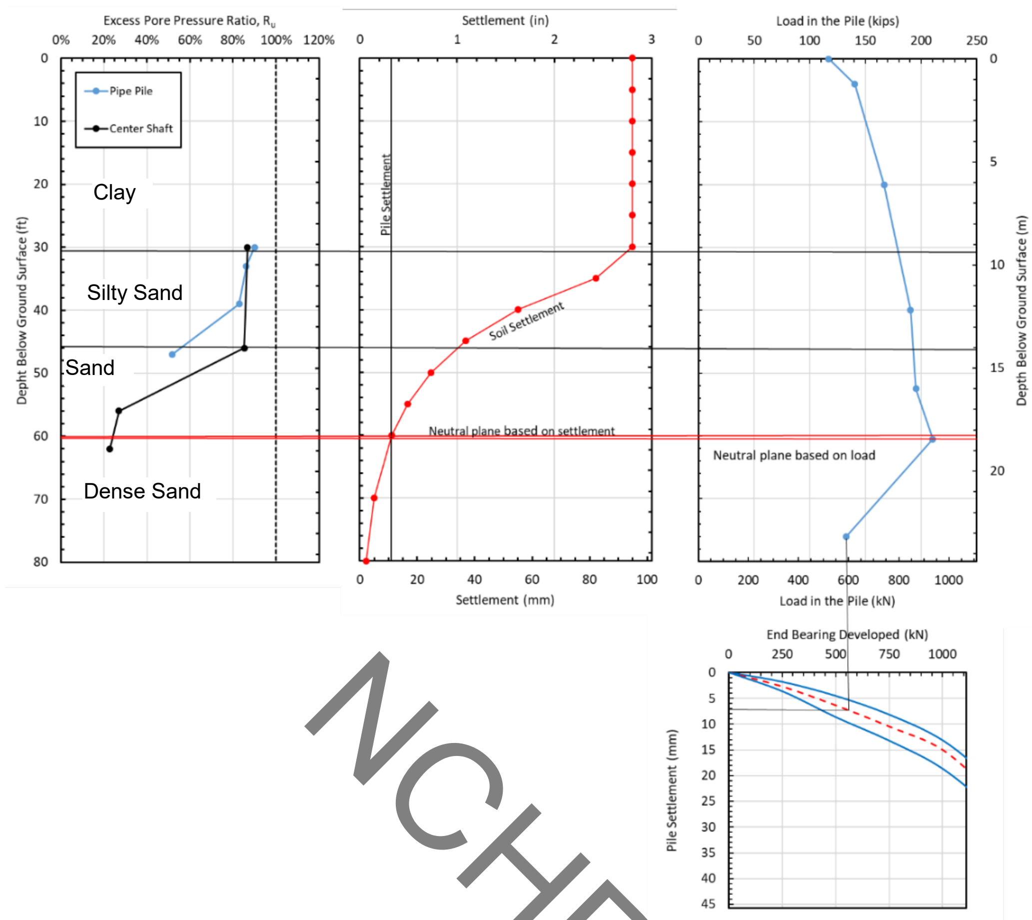

Fig.1-12. Plots showing excess pore pressure ratio, pile and soil settlement, and load in the pile vs. depth along with toe (end-bearing) resistance vs. settlement curve for the steel pipe pile (Kevan et al. 2019).

For the pipe pile test shown in Figure 1-12, the neutral plane, where the pile settlement equals the ground settlement, was located at a depth of about 18.3 m and this point also corresponded with the maximum load in the pile. The average side resistance in the liquefied soil layers following reconsolidation was calculated as 38% of the static resistance prior to liquefaction. In contrast, the side resistance in the non-liquefied clay was within 4% of the pre-blast resistance in the clay layer but about 20% higher in the sand layer below the liquefied zone. Similar results were obtained for the other test piles and shafts.

Although significant negative skin friction was developed along the length of the deep foundations and liquefaction induced settlement was substantial, measured settlement of the deep foundations was normally within acceptable levels for the pile head loads involved. The measured pile settlement was generally consistent with the neutral plane concept obtained by balancing applied pile head force and negative friction with the positive friction and displacement-compatible toe resistance after reducing skin friction in the liquefied layers.

2.2 Conclusions regarding liquefaction-induced downdrag

1. In liquefied soils, negative and positive skin friction after liquefaction and reconsolidation was 40 to 55% of the skin friction before liquefaction.
2. In non-liquefied soils, negative friction was equal to positive friction.
3. These results are generally consistent for all available tests and suggest that this may be a typical result
4. The depth to the neutral plane increased (and pile settlement decreased) as pile length increased
5. In general, the neutral plane was not located at the base of the liquefied layer.
6. Measured pile settlement was generally consistent with neutral plane concept after balancing applied pile head force and negative friction with the positive friction and displacement-compatible toe resistance.

Task 2. Synthesis of Literature Review

After more than 50 years of full-scale field studies in many parts of the world, the response of single piles and narrow pile groups to soil subsidence is generally well understood. However, many details to consider in the foundation design are poorly expressed in textbooks and standards and much effort is needed to bring the necessary information out to the engineering practice. This is likely a result of the fact that no single test case history has provided a complete record of pile performance, rather, results from a variety of separate case histories have been required to produce a general design procedure.

For example, some field case histories have shown that a static neutral plane can develop after pile installation even prior to placement of an approach fill and subsequent consolidation settlement. In contrast, other case histories did not observe this phenomenon, but may not have been instrumented sufficiently or monitored for a sufficient time prior to fill placement to make the observation. As a result, confusion and uncertainty exist regarding this point. Likewise, in some case histories a pile head load has been applied following the development of negative skin friction and the development of a neutral plane. This loading shows that positive skin friction develops from the pile head downward as the pile settles more than the surrounding soil until the full positive side friction and toe resistance are mobilized. However, in many case histories no pile head load is applied following the development of negative skin friction above the neutral plane therefore, they provide no information about pile resistance at the ultimate state.

Lastly, some case histories do not provide the soil settlement profile relative to the pile settlement profile, while others do not have a load test that provides the distribution of side resistance and toe resistance or the movement necessary to develop the toe-resistance. The absence of these key measurements make it difficult to confirm the accuracy one analysis procedure relative to another.

To supplement the existing field test data, we believe it would be desirable to conduct a full-scale downdrag test on an instrumented test pile at a bridge abutment in which the pile is driven into a compressible clay profile prior to construction of the approach fill. This instrumented test pile would then provide data on load distribution: (a) prior to fill placement, (b) during consolidation settlement after fill placement, and (c) during application of dead load from the bridge superstructure. This test would provide a case history documenting pile performance during the typical sequence of load experienced by an abutment pile. Ideally, a static load test or bidirectional load test would be performed on an adjacent test pile to define the toe resistance-deflection relationship and the distribution of load in the pile at the ultimate state. In addition, pile driving analyzer (PDA) measurements would be made during pile driving so that a CAPWAP analysis could provide complimentary side resistance and toe resistance data. Strain measurements along the length of the pile would define load in the pile and ground settlement would be measured with depth. A test such as this would provide all the information needed to evaluate competing analysis procedures and would provide a well-documented case history for calibrating numerical models that could then be used for parametric studies.

With regards to liquefaction-induced downdrag and negative skin friction on single piles, a growing set of test data has been accumulated to provide a basic framework for understanding pile behavior for this condition. However, a couple of important issues remain unresolved. First, the analysis method suggested by Fellenius and Siegel (2007) assumes that a static neutral plane develops for any pile such that negative skin friction above the neutral plane need not be considered should liquefaction occur. In contrast, tests conducted by Rollins and his co-workers (Rollins and Strand 2006; Rollins and Hollenbaugh, 2015; Kevan *et al.* 2019) have not shown the development of a static neutral plane prior to blast-induced liquefaction. It is presently unclear from the available data whether or not essentially all piles will develop negative skin friction and a static neutral plane as previous research has primarily focused on piles in settling ground. Different initial assumptions can have a significant effect on computed pile load and settlement.

Secondly, field test results to date have indicated that the average negative skin friction in liquefied layers following dissipation and reconsolidation is approximately 50% of the positive skin friction prior to liquefaction. However, there is considerable uncertainty regarding whether or not the full positive skin friction can again be relied upon after some time period (e.g. one month) once the micro-structure of the sand has had a chance to recover. For example, shear wave velocity tests often show a reduction in velocity immediately following blast-induced liquefaction followed by an increase with time (Amoroso et al. 2017). This issue could be readily investigated by performing static pile load tests before liquefaction and then again a month or so after the liquefaction

In contrast to our understanding of single pile behavior during downdrag loading, the response of larger (wider) pile groups is less well researched and understood. It is also more complex, as it involves additional aspects, such as pile cap rigidity, whether the subsidence is due to lowering of groundwater table or to placing of fill, manner and sequence of construction, and location of a pile within the pile group, for example, a perimeter pile location relative to an interior pile location within the group. Unfortunately, the cost and logistical difficulties associated with testing and instrumenting a pile group are relatively high and fewer tests are available as a result. Therefore, we feel that it would be desirable to conduct downdrag tests on a group of piles at a compressible clay site and at a liquefiable sand site in connection with this study if the budget can be accommodated. Ideally, the pile groups should have a minimum of two interior piles, which could be produced with a 13-pile group, along with a companion single pile for comparison purposes. A static load test would be performed on the single companion single. Strain gauges would need to be monitored long-term on three to four piles in each group and ground settlement would need to be measured versus depth inside and outside the group. For the liquefiable sand site, ground settlement could be induced by blasting whereas a limited amount of fill would likely be required to induced settlement for the pile group in clay. Group tests would provide important case histories with which to evaluate competing analysis techniques and they would be particularly valuable in calibrating numerical models which then be used to conduct "virtual" load tests with variations in pile type, cap type and geometries.

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