Static response of a group of 13 piles tested simultaneously

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ABSTRACT. Static loading tests to plunging "failure" were performed on a single pile and a group of 13 piles in a loose to compact silty sand. The piles were 300-mm diameter, 9.5 m long, pressure-grouted, bored piles with an expanded pile toe, a 400 mm wide Expander Body (EBI). Each pile had a bidirectional cell placed just above the EBI at 8.3 m depth. The tests comprised a bidirectional test on the piles with the pile group cells activated via a common pressure pump, thus, ensuring that equal load was applied to all piles, allowing each pile to move individually. The pile group responded as a pier with shaft resistance acting mostly along the perimeter piles or along the pier circumference rather than as 13 single piles. A rigid cap was then constructed over the piles and a head-down test performed. Also in the head-down test, the developed resistance was mainly from the perimeter piles.

1. INTRODUCTION

As a part of the 3rd Bolivian International Conference on Deep Foundations, May 2017, a comprehensive pile testing programme was undertaken at the Bolivian Experimental Site for Testing Piles, B.E.S.T. The main objective of programme was to compare the results of static loading tests on single piles constructed using different methods at a site where the geotechnical conditions would be documented by detailed investigations, using state-of-the-art testing and interpretation methods. The geotechnical conditions and the details on the piles and testing arrangement were presented by Fellenius and Terceros (2017). All the B.E.S.T. field investigations records and results of static loading tests on single piles are available for online downloading at the conference web site per the following link:
http://www.cfpbolivia.com/web/page.aspx?refid=157.

2 Soil Profile

The site investigation at the B.E.S.T. site, notably the CPTU sounding results, showed the soil profile to consist of essentially two soil layers: an upper 6 m thick layer of loose silt and sand on compact silty sand. The CPTU pore pressure measurements indicated a groundwater table at or near about 0.5-m depth and a hydrostatically distributed pore pressure. Figure 1 shows a diagram compiling borehole information, SPT *N*-indices, and the CPTU cone stress, q_t, to 14 m depth. The CPTU was pushed to 22 m depth and Figure 2 shows all the results.

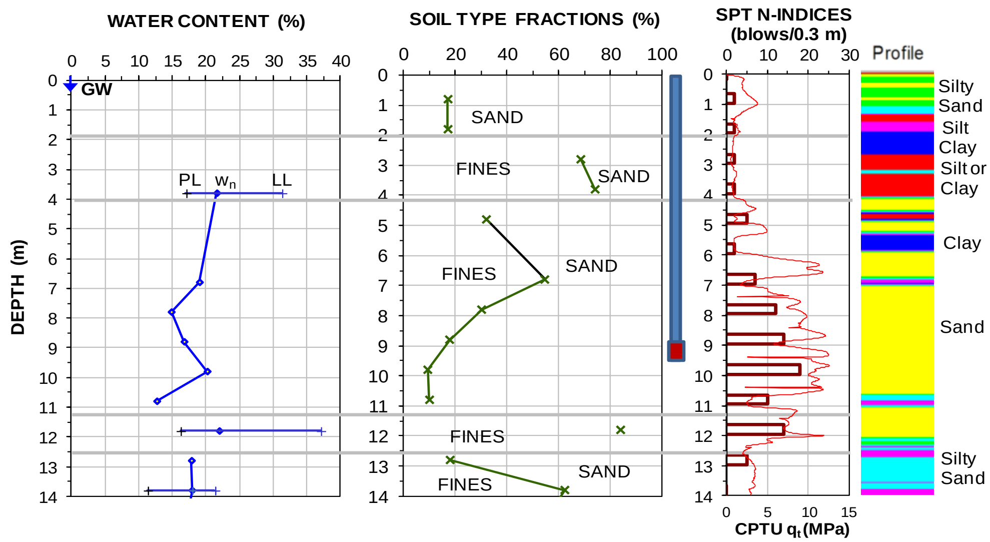

Fig. 1 Soil profile at BH-E2.

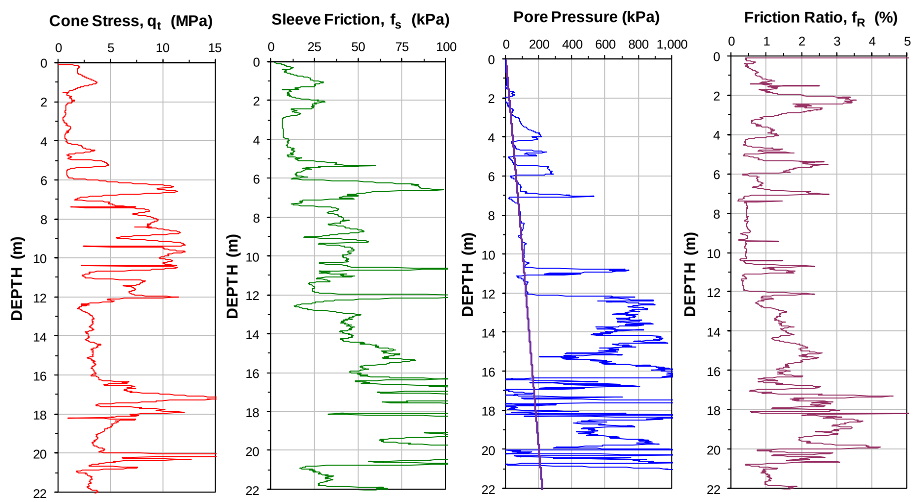

Fig. 2 CPTU sounding diagram, CPTU E2.

3 Test Piles

The pile group contained 13 piles. All were 300-mm diameter, pressure-grouted, Full Displacement Piles (FDP) constructed to 9.5 m depth on April 20-21, 2017. For reference to the group test, a single test pile, the same in all respect to the group piles, was installed (March 15, 2017), 5.0 m away from the nearest group pile and 3.5 m away from its nearest reaction pile. All piles were equipped with a bidirectional cell (BD) consisting of a 200 mm high and 80 mm wide hydraulic jack installed centered in each test pile at 8.3 m depth (lower end of the BD). All the BDs in the group piles were connected to a common pump, i.e., all BDs were acted upon by equal magnitude of load. Immediately below the BD, the piles were equipped with a (prior to expansion) 1.2-m long Expander Body, EBI612, that was expanded (pressurized with grout) after construction of the shaft (an EB equipped with post-grouting arrangement is termed EBI). Twenty-four hours after the expansion, the soil underneath the EBI was post-grouted to recompress the soil below the EBI. The purpose of the post-grouting was to ensure a large soil toe stiffness below the pile toe when activating the BD to test the shaft above the BD..

The Expander Body and its use is described by Terceros and Massarsch (2014). Figure 3 shows a photo of the EBI and BD (placed immediately above the EBI) before insertion into the pile shaft. The BD was connected to the pile reinforcement cage. The separation of the BD and cage from the EBI was designed to occur at the bottom of the BD.

Figures 4 and 5 show photos of an exhumed EBI and one expanded in air (models EBI612 and EBI815, respectively, with the numbers indicating the diameter of a freely expanded unit. Figure 4 shows a photo of an exhumed EBI (EBI612) with post-expansion grout material and an in-air expanded EBI (EBI815). The grout volumes and injection pressures used for the expansion of the EBIs of the 14 test piles were very similar for the piles. The averages were 102 L and 4.0 MPa, respectively.

Fig. 3 Photo of the EBs and BD before pile construction.

Fig. 4 Exhumed EBI.

Fig. 5 In-air expanded EB

The piles were installed at a 3 pile-shaft diameter, center-to-center spacing, 0.90 m in the configuration shown in Figure 6. The spacing corresponds to an 11 % total footprint ratio (nominal total pile cross section area at the ground surface over group footprint area).

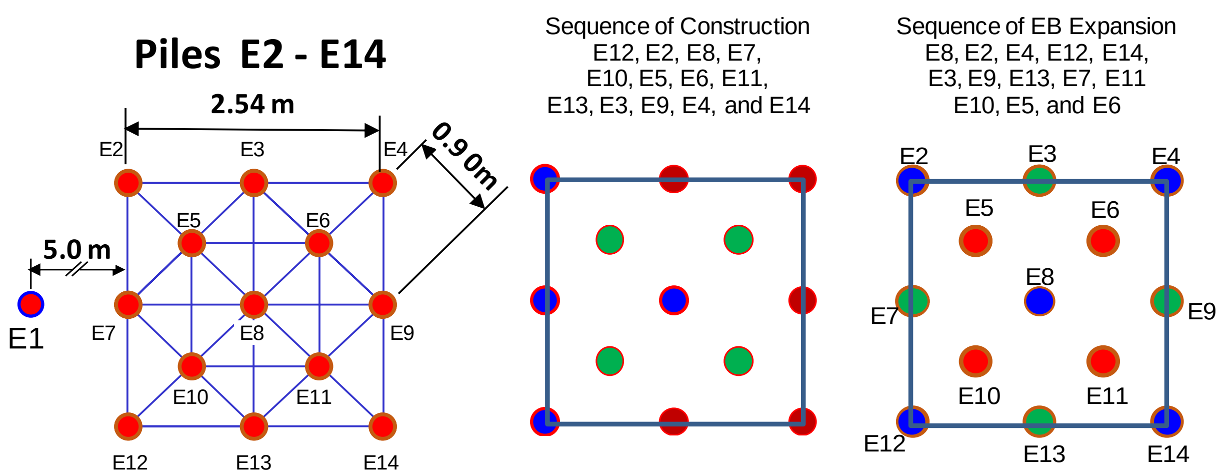

Fig. 6 Pile locations and sequence of construction and EB expansion.

The test piles were instrumented with one set of vibrating wire gages and three sets of electrical resistance gages. As mentioned in the report on the test results of the many other single test piles at the B.E.S.T. site, the vibrating wire gages did not provide reliable records due to difficulties in recording the data. Moreover, it was found that the EBI-expansion created an axial tension in the test piles that left them with considerable residual force (locked-in force) of inconsistent magnitude and distribution. Interpretation and presentation of the strain-gage measurements are therefore not included in this paper.

Although the primary purpose of equipping the test piles with the EBI was to provide reaction force for the BD in forcing the pile length above the EBI to move upward, telltales had been included to measure the downward movement of the EBI to enable studying also the BD force versus downward movement of the EBI. Unfortunately, in all test piles, when the EBI expanded, the telltale connection of the downward telltale broke and no records of the downward movement of the EBI were obtained. The telltale measuring the upward movement of the BDI upper plate functioned, however. Thus, the comparison between that movement and the pile head movement provided a measure of the pile shortening over the 8.3-m distance between the pile head and the BD base.

4 Test Programme

The test programme—on the single pile (E1) and the 13 group piles (E-Group)—comprised three phases. Phase 1 was by performing a bidirectional test on the piles. Phase 2 was by head-down test—via a rigid pile cap cast on the pile group—and Phase 3 was a repeat of Phase 1 with the pile heads of the group still connected to the rigid pile cap.

Pile E1

Phase 1	Bidirectional test, April 27, 2017
Phase 2	Head-down test, BD passively recording. April 2017

E-Group

Phase 1	Bidirectional test, April 27, 2017
Phase 2.1	Head-down test, BD free-draining. January 2018
Phase 2.2	Head-down test, BD closed and passively recording, January 2018
Phase 2.3	Unloading pile head; BD closed and passively recording, January 2018
Phase 2.4	No load at pile head, BD unloading, January 2018
Phase 3.1	No load at pile head, BD loading, January 2018
Phase 3.2	No load at pile head, BD unloading , January 2018

Phase 1 loading tests were performed by engaging the BD cell to push the pile shaft upward with the EBI providing the necessary reaction force. The BD cells in the group piles were connected to a common pump to have all group piles, Piles E2 - E14, activated with equal load per pile. The pile heads were unrestrained and the pile head movements were measured individually. Figure 7 shows a photo of the arrangement for recording the individual pile-head movements of the E-Group. The measurements included monitoring the ground surface movements at 10 points along the perimeter and in the interior area of the group. The test schedule, same for the single pile, Pile E1, and the group piles, Piles E2 - E14, consisted of applying equal load increments of 50 kN/pile every 10 minutes and recording the movement of the pile head until it became excessive.

Fig. 7 Photo of frame for recording pile movements at Phase 1 test on Piles E2 - E14.

The Phase 2 head-down test was performed by jacking against reaction piles placed at a pile surface-to-surface distance of 2.0 m or more away from the nearest test pile, Figure 8 shows the locations of piles and reference beam support for the E-Group. The reaction piles were installed after completion of the tests on Pile E1.

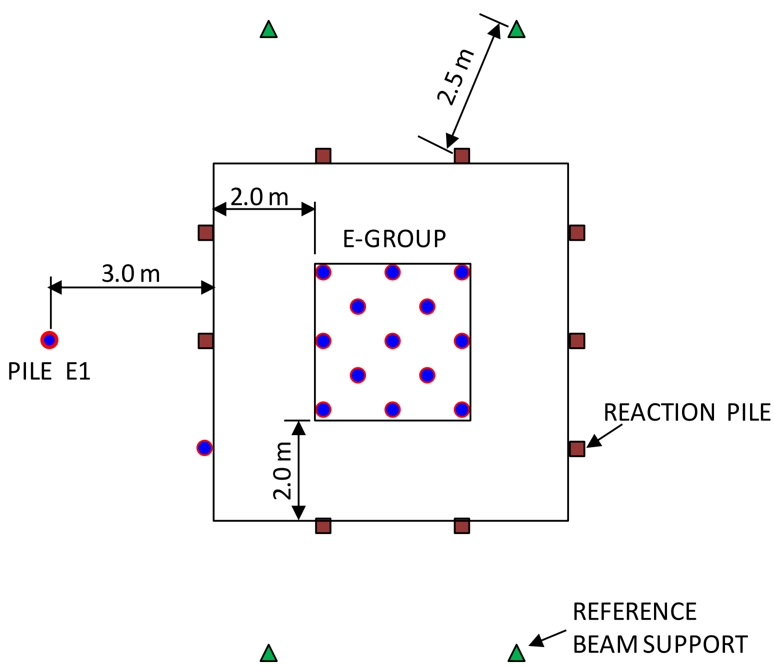

Fig. 8 Locations of test piles, reaction piles, and reference beam support at the E-Group.

After performing the Phase 2 test on the single pile, it was realized that the unknown distribution of residual force in the pile made it difficult to determine the true axial force at the BD. Therefore, Phase 2 test on the pile group was started with the BD free-draining. Phase 2.1 comprised measurements until 6 mm of pile-head movement had developed and Phase 2.2 followed, for which phase only positive direction shaft resistance was considered to act along the pile. Thus, when then connecting the BD to transmit the axial load to the pile toe, the BD would passively measure the load reaching the BD and the difference between the load applied at the pile head and the load a the BD would be the shaft resistance.

To arrange for the Phase 2 head-down test on the pile group, a rigid, rebar-reinforced pile cap with a 3.5 m square base and 1.8 m height was cast on the group piles. Before forming and casting the cap, a 0.5 m compacted fill was placed on the ground over a 7.5 m square area centered around the test pile. The fill sides sloped 1(V):3(H), leaving a 0.5 m wide horizontal area around the pile cap. Figure 9 shows a photo of the Phase 2 set-up.

Fig. 9 Photo of set-up for head-down test on the E-Group.

4 Results of Field Tests

4.1 Phase 1

PILE E1. The measurement comprised imposing load in the BD and recording the upward movement of the pile. The compression of the pile as measured between the BD upper plate and the pile head was generally very small; <1 mm at the maximum test load. The maximum strain measured at 5 m depth was about 100 $\mu\varepsilon$.

Figure 10 shows the measured Phase 1 load-movement of Pile E1 upper pile length and the Phase 2 head-down test on the pile. The pile response to the applied BD load was quite stiff with very small movement, about 1 mm, measured for the first about 500 kN applied load. This, indicates the presence of residual compression force in the pile at the start of the test. Beyond 600 kN load and 2 mm movement, the response went into plunging. The figure also shows the result of fitting an effective stress analysis and a hyperbolic t-z function with a function coefficient of 0.0099 ($1/Q_{100\ \%}$) to the measured curve. An effective stress proportionality beta-coefficient, ß, was applied to 1.0 m pile elements, ranging from *ß* = 0.4 at the ground surface to *ß* = 2.0 at 6 m depth, which value was then kept to the BD depth. The UniPile5 software (Goudreault and Fellenius 2014) was used for this back-calculation.

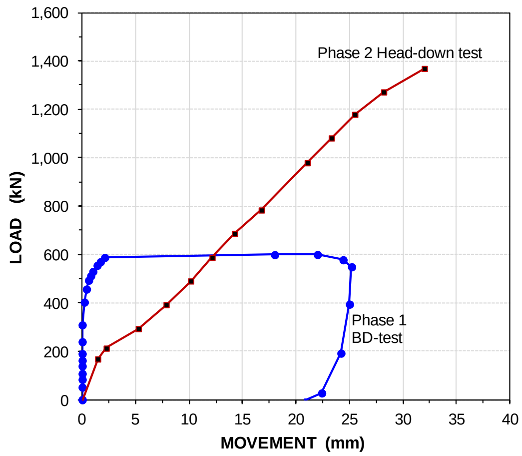

Fig. 10 Pile E1: Phase 1 BD-test and Phase 2 head-down test.

PILES E2 - E14. The static loading test on the pile group, Piles E2 - E14 started on April 28, 2017, applying the same schedule of loading as used for the Phase 1 single pile test. Nine load increments into the test, it was realized that the data collector was not recording properly and this triggered an unloading of the test to restart with a functioning collector. Unfortunately, on reaching zero BD load, the cells were let to drain further, which allowed the remaining force in the pile to contract the BD at free-draining, i.e., no measurement of force. The test was then restarted three days later from assumed zero BD load and with resetting of all displacement gages. Figure 11 shows the load/pile response of the group piles versus time applied comprising the now two-part loading sequence, Phases 1a and 1b. The load-movement measured for Pile E1 is included for reference. Shortly after re-starting the test (Phase 1b), it was found that the pump had sprung a leak preventing it from maintaining pressure. The BDs were then unloaded to almost zero pressure, whereupon the manifold was locked from further release of hydraulic pressure and the pump was repaired and the test re-started (Phase 1b). No similar records of force and movements were made for the single-step unloading of the BD in Phase 1a. The Phase 1b records are plotted with a starting movement equal to the last reading of Phase 1a.

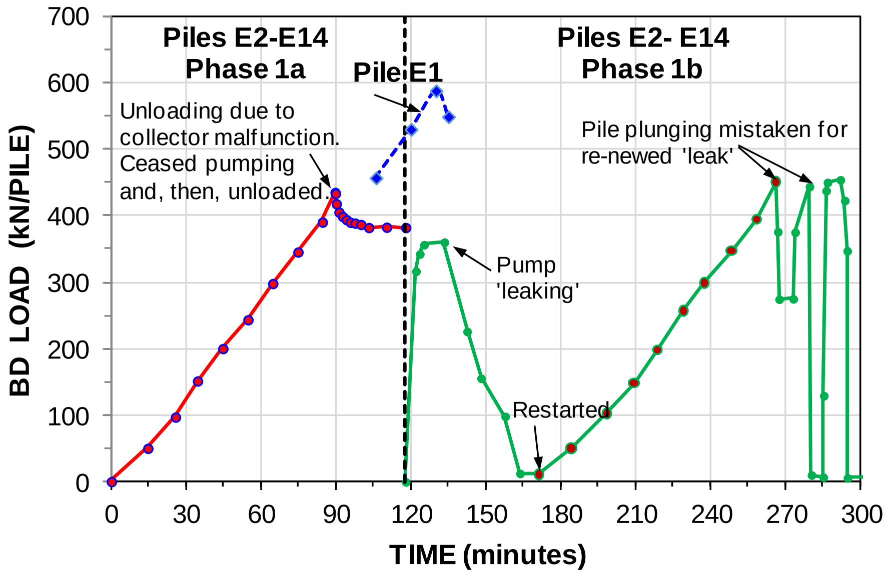

Fig. 11 Phases 1a and 1b, Loading sequence for Pile E1 and Piles E2 - E14.

Figure 12 shows the load-movements of Piles E2 - E14 and, for reference, that of Pile E1.

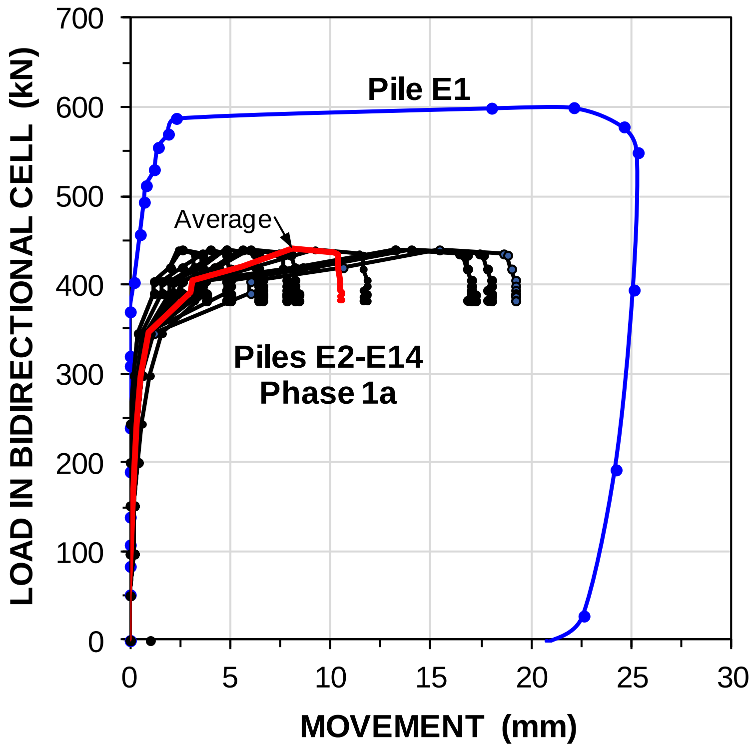

Fig. 12 Phase 1a Piles E2 - E14 and Phase 1 Pile E1: Pile head upward load-movements.

Figure 13 shows the full Phase-1 load-movements of Piles E2 - E14. The response of the 13 piles was very similar to that of the single pile: initially quite stiff and plunging after just a few millimetre of movement. However, the plunging load per pile was only about 70 % of the plunging load measured for Pile E1—435 kN versus 600 kN.

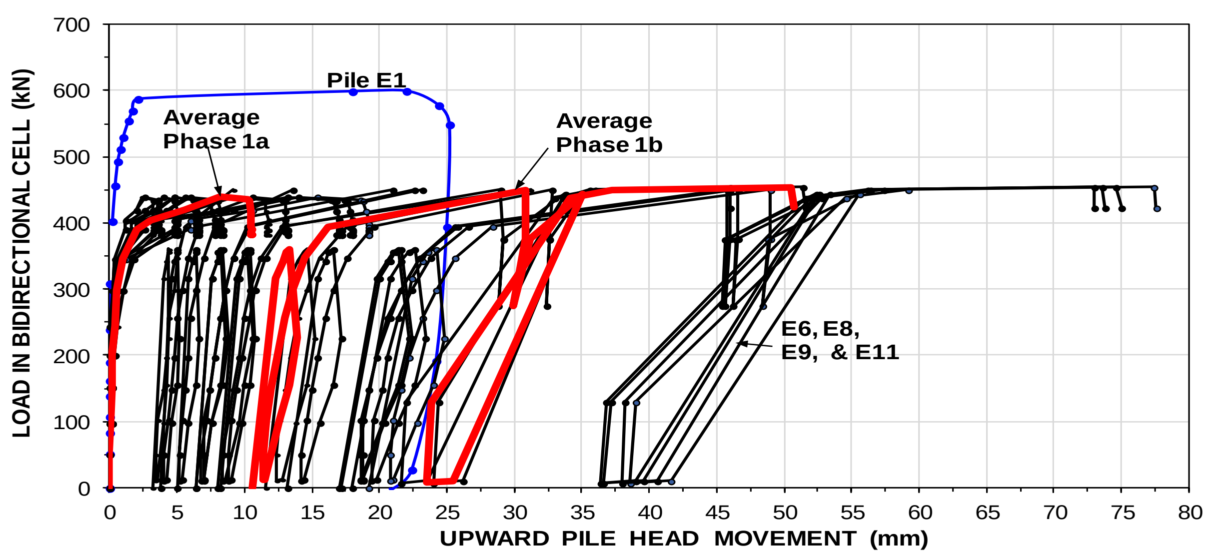

Fig. 13 Pile E1 and Piles E2 - E14. Pile head upward load-movements.

Figure 14 shows the upward movement of all pile heads, including the net movements after Phase 1b unloading (limited to the four piles indicated to minimize the interference of the curves). Both graphs include the average of all pile head movements measured in Phase 1b for the maximum load. The measurements showed that the center pile moved about three times more than the perimeter pile.

Figure 15 shows the ground-surface upward movements across the diagonals (including net movements after Phase 1b unloading) and average (at maximum load) ground surface heave, as measured in ten anchors for Phase 1b. The average (at maximum load) upward movement of the pile heads (c.f., Figure 13) is repeated for reference. The "zero" measurement is the start of the Phase 1b, the net effect of Phase 1a is not known. It would seem that the pile heads moved a bit more than the soil surface. However, this is misleading because the pile head movements in Phase 1a in unloading were not recorded and, as mentioned, the Phase 1a ground surface measurements were not recorded by the data collector. That is, the previous movements of the pile heads and the ground surface at the start of Phase 1b are unknown. The trend of the ground surface movements and that of the pile heads are the similar. In unloading after Phase 1b maximum load, the unloading movements were about the same for the pile heads and the ground surface. It would seem that the pile heads and ground surface moved more or less in unison.

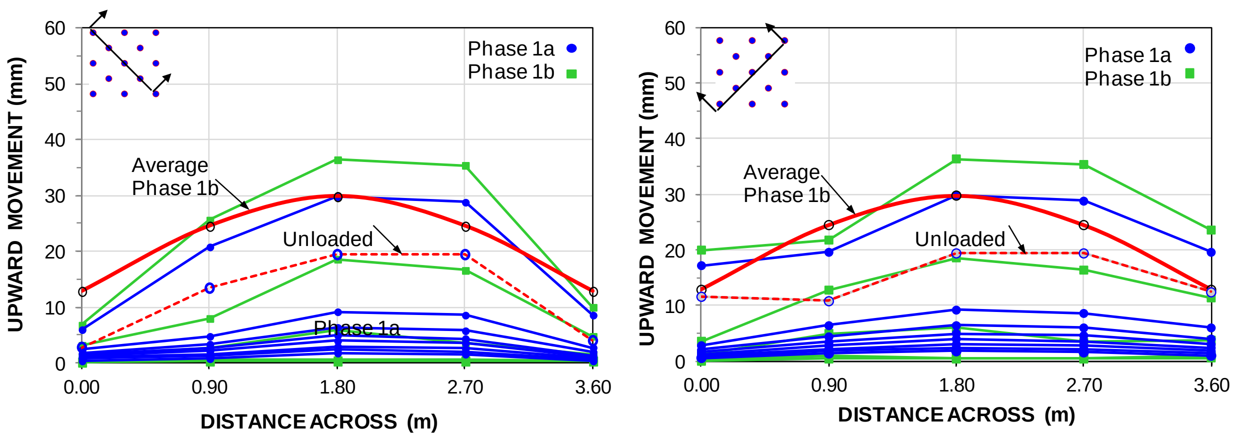

Fig. 14 Upward movement of the pile heads along the diagonals, Phases 1a and 1b.

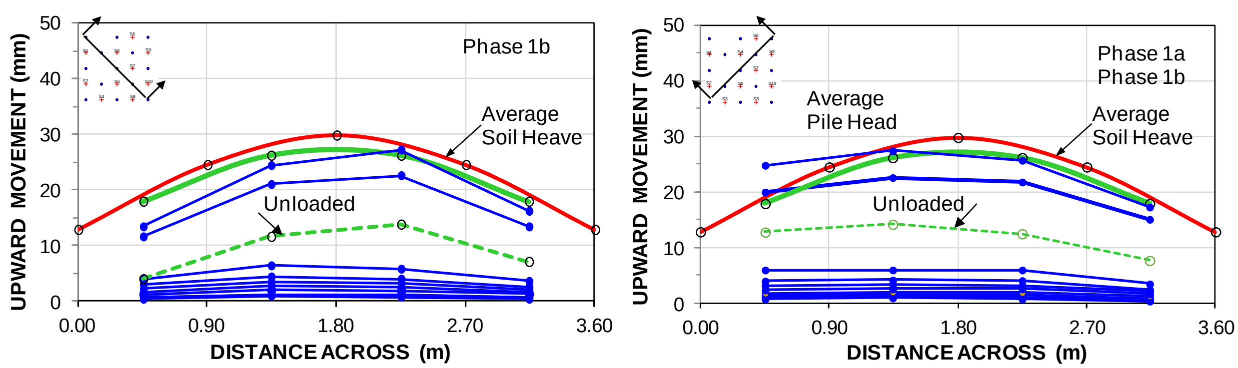

Fig. 15 Upward movement of the ground surface within the pile group. Phase 1b

The Phase 1 test on the single pile, Pile E1, showed plunging response for a 600-kN load. In contrast, the plunging load for the E-group piles was 435 kN/pile (the total load for the 13 piles was 5,655 kN). Thus, if assuming that each of the eight perimeter piles provided the same plunging resistance (600 kN) as the single pile, the contribution from the interior piles would be only 170 kN/pile.

The total weight of the "equivalent pier", was 580 kN (2.54^2 times the effective stress at the bottom of the BD = $\sigma'_{8.3m}$ = 90 kPa). Thus, the net total load was 5,075 kN and if carried in full by the perimeter piles, the load would have been 634 kN/pile—similar to the Pile E1 plunging load.

The net total load divided by the E-group total circumferential area (84 m^2) is 60 kPa, which represents the average unit shaft resistance for the "equivalent pier". The 60 kPa is smaller than the 75 kPa for the single pile.

Either the resistance was only from the 8 perimeter piles with the soil mass inside the “pier” being “there just for the ride” with no shaft resistance developing along the 5 interior piles, or the resistance to upward pushing of the pile group (the “pier”) acted along the “pier” circumference with smaller unit shaft resistance mobilized for soil-to-soil contact (<60 kPa) than for the pile-to-soil contact (75 kPa). The difference could be due to the fact that the piles were pressure-grouted.

Figure 16a and 16b show photographs of piles and ground taken after Phase 1b unloading and show cracks in the ground surface around a corner pile and from pile to pile along the perimeter of the group. The observations suggest that the pile group moved in unison encompassing piles and in-between soil.

Fig. 16a Photographs of cracks in ground surface at a corner pile and at two side piles.

Fig. 16b Photographs of cracks in ground surface stretching from one pile toward the next along the side of the group.

4.2 Phase 2

PILE E1. The Phase 2 head-down test on Pile E1 comprised adding 50-kN increments every 10 minutes. When the load reached 1,400 kN, the pile head broke and the test was over (c.f., Figure 10).

PILES E2-E14. The load-movement measured for the E-Group (Phase 2 head-down and Phase 3 BD tests) on the pile group is shown in Figure 17. The records shown are the load applied to the pile cap divided by the number of test piles (13) and the measured BD load (per pile). At the maximum load (910 kN/pile), the test was terminated because the pile-head movement started to increase coinciding with signs of yielding of the reaction system.

It is interesting to see that after complete unloading of the pile head, significant force was maintained in the pile between the BD and the pile head. In the subsequent unloading of also the BD, only a very small increase of movement (downward) occurred.

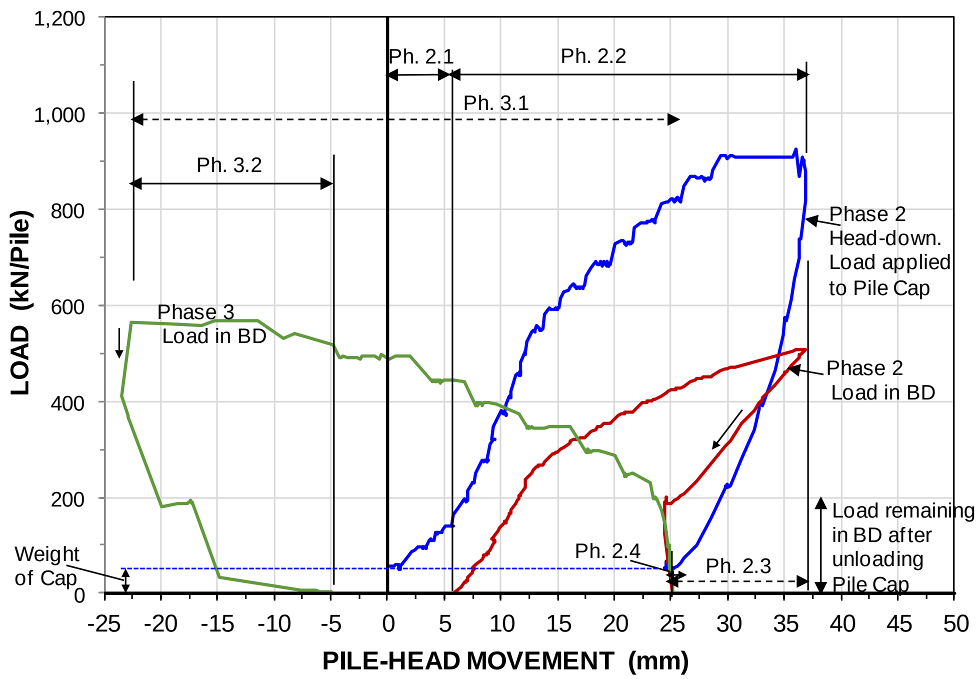

Fig. 17 E-Group: Pile-head load-movement records during Phases 2 and 3.

Figure 18 shows shaft resistance versus pile-head movement for the three test phases of Piles E2-E14. The results indicate that the maximum shaft resistances, which indicated a plastic response, was equal in all three phases. However, the movement to mobilize the plastic response in Phases 2 and 3 was much larger than in Phase 1, which is likely due to the fact that the movement directions were reversed between the tests.

Phase 2 test results of the E-Group are particularly interesting. Because of the rigid pile cap, all pile heads movement were equal. Figure 19 shows the load-movements of the individual piles together with the average load-movement of the perimeter and interior piles, as well as the average of all piles. For equal load, the interior piles moved much more than the perimeter piles, much as they had done when, in Phase 1, the pile heads were unrestrained and the load, same for all piles, was induced from the BD (c.f., Figures 13 and 14). For equal movement, the load on the perimeter piles was much larger than on the interior piles, that is, the perimeter piles mobilized a much larger resistance for equal movement.

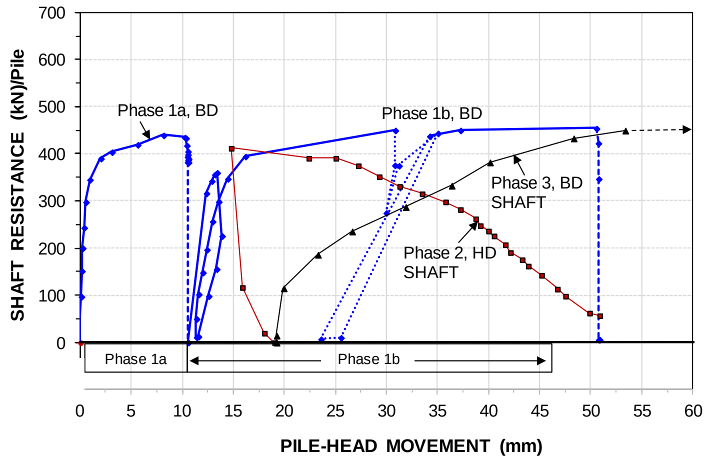

Fig. 18 Shaft resistance measured in Phases 1 through 3.

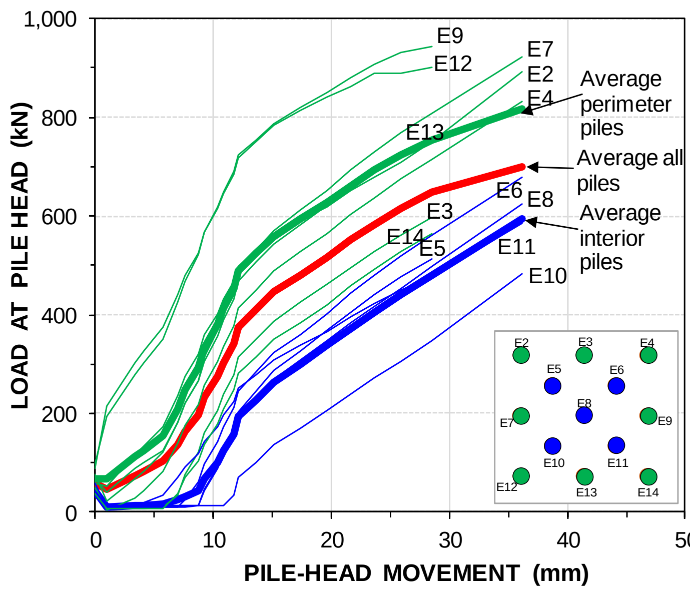

Fig. 19 E-Group Phase 2 Loads at the pile head; individual and averages.

Figure 20 compares the load-movements of the total load applied to the pile cap, the average pile-head loads, the shaft resistance between pile head and BD level, and the average contact load. The average contact load is the difference between the total load and the pile-head load. The average shaft resistance was determined as the total load (load/pile) minus the BD-load/pile. When assuming that the average axial strain of compacted fill under the raft would be the same strain as in the in the piles (≈500 με), the contact load correlated to an about 200-MPa E-modulus of the fill—a very high value even when considering that the sand is compacted and confined.

6 Conclusions

In Phase 1 bidirectional test, plunging mode developed at very small movements for both the single pile as well as for the E-Group piles. The results of the test on the pile group indicated that the eight perimeter piles carried the load with the five interior piles not experiencing resistance to the upward loading. This was supported by the emergence of cracks around the pile group indicating that soil and piles had moved more or less in unison.

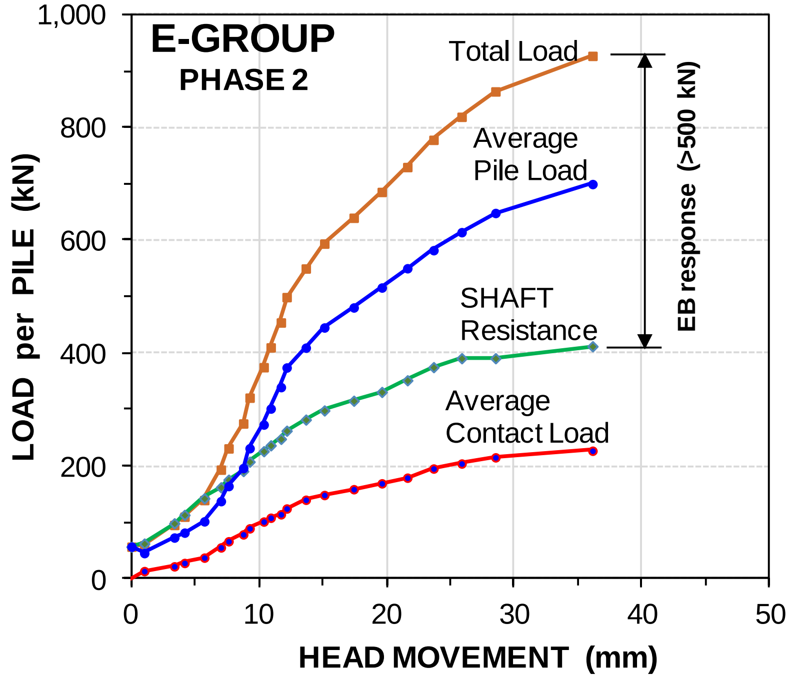

Fig. 20 E-Group Phase 2; Loads vs. movements.

An effective stress analysis of the response to the maximum test load on the single pile corresponded to ß = 0.4 at the ground surface increasing linearly to ß = 2.0 at 6 m depth, which value was then kept the same to the BD depth. The 2.0-value is large and probably a result of the pile diameter being larger than the nominal value and that the FDP construction process and the pressure-grouting had increased the horizontal stress against the pile.

In both Phases 1 and 2, the interior piles moved considerably more than the perimeter piles for the same applied load. Phase 2 measurements showed that the perimeter piled mobilized considerably more shaft resistance than the interior piles.

When assuming that the average axial strain in the piles would be the same strain as in the compacted fill under the raft, the contact load correlated to a 650-MPa E-modulus of the fill.

The Expander Body provided a considerably enhanced pile-toe resistance thus, enabling the BDs to push the piles as planned; The enhanced resistance was a key aspect of the study.

7. References

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