Example of the Unified Design procedure for use in the September 8, 2014, short course on in Brazil

Introduction

The Unified Design procedure involves two main steps. The first is verifying that the loads from the supported structure—dead plus live; sustained plus transient—are adequately safe considering the pile capacity. The second is verifying that the settlement of the piled foundation will be adequately smaller than the maximum value the structure will accept (there will always be settlement) before the structure is inconvenienced or, in the extreme, damaged. This step also verifies that the maximum load in the pile (occurring in the long-term at the neutral plane) is safe considering the axial strength of the pile.

The two-step procedure will be illustrated in an analysis of a simple example based on normally occurring conditions of load and soil profile. The analysis is then developed into a more complex case by introducing "complications" often encountered in practice.

Basic Example[1)]

A two-storey warehouse will be constructed with a 50 m by 150 m footprint. The building outside walls, the upper floor, and the roof will be supported on columns exerting a 3,200 kN dead load (800 kN/pile) and placed about 15 m apart. No appreciable live load is anticipated. There will be no inside walls in the ground floor. Each column foundation will include four, 300-mm diameter, driven precast prestress concrete piles placed at a c/c of 1.5 m. Pile testing at the site has established the pile length to be 20 m. The ground floor will be placed level with the ground surface and consists of asphaltic concrete. The ground-floor loading will amount to 50 kPa and is considered to be a uniformly distributed sustained load.

The structural engineer indicates that a 50-mm differential settlement between the columns will be acceptable for the structure.

A site investigation involving boreholes with split-spoon sampling, CPTU soundings, Shelby tube sampling, and laboratory studies has been performed. Presenting the detailed results of the investigation is beyond the purpose of this example and here only a summary will be provided. It must be remembered, however, that this is a crucial part of the foundation design. Often, a site investigation is planned according to expectation of shallow foundations. The investigation is therefore geared to establishing the conditions of the upper soil layers. Then, when the results show the necessity of a piled foundation, the site investigation is only too often not extended to provide the required information for the deep foundation design. Happily, this is not the case of the example.

The investigation has shown that the soil profile consists of 2.5 m of loose to compact sand on 12.5 m of compressible, slightly preconsolidated, marine clay deposited on dense to very dense sand at a depth of 15 m. The dense sand is at least 15 m thick and is underlain by a veneer of very dense glacial till on bedrock. The groundwater table is located at a 3.0-m depth and the pore pressure distribution is hydrostatic.

The upper loose to compact sand layer has a total density of 2,000 kg/m^3 and a void ratio of 0.67.
The marine clay layer has a total density of 1,600 kg/m^3 and a void ratio of 1.76.
The lower dense sand layer has a total density of 2,200 kg/m^3 and a void ratio of 0.39.

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1) All the case data are available in the file " Demo Example 10 - Unified Design.Unipile5". The link is: http://www.UniSoftltd.com/uploaded/file/UniPile5_Examples.zip

The compressibilities (Young moduli) of the upper and lower sand layers are estimated to be 40 MPa (j=1; m=400) and 100 MPa (j=1; m=1,000), respectively, applicable to a linear response to load ('elastic' condition). Oedometer tests on the marine clay have established that C_c = 0.63 and C_{cr} = 0.063, which, when coupled with the void ratio values, show that the compressibility of the clay in terms of Janbu modulus numbers is m = 10 and m_r = 100, or, in terms of MIT compression ratio coefficients, CR = 0.23 and RR = 0.023. The preconsolidation margin in the clay is 40 kPa (corresponds to OCR = 1.8 at the upper clay boundary and to OCR = 1.4 at the lower clay boundary).

The analysis of the pile response to load will be performed in an effective stress analysis which requires the input of the appropriate ß-coefficients. We assume that a static loading test has been performed at the site on a pile similar, but not necessarily identical, in size, length, and construction. The following ß-coefficients and unit toe resistance were used to produce the loading test curves of this "virtual test": for the shaft resistance in the upper sand, the clay, and the lower sand, they are 0.40, 0.25, and 0.60, respectively, and the unit toe resistance is 6 MPa (540 kN) at a 30-mm toe movement. In an actual test, the coefficients and unit toe resistance would have been established in a back-analysis of the test results.

Moreover, the t-z and q-z functions for the pile response have also been established: For both sand layers, the shaft resistance response can be modeled by the Exponential function with the exponent equal to 0.5, which essentially describes an elastic-plastic response for the pile element with the shift from 'elastic' to plastic response occurring at a movement between 5 mm for 90 % of ultimate to 10 mm for 100 %. Thus, the analysis benefits from a well-defined ultimate shaft resistance. No such ultimate resistance exists for a pile toe, of course. The pile toe response of the example pile is assumed to follow a Ratio function with an Exponent of 0.63. This exponent means that a doubling of the toe force results in a trebling of the toe movement, i.e., the mentioned 6-MPa at 30-mm movement corresponds to a 3 MPa unit toe resistance at 10 mm movement. Figure 1 shows the results of the static loading test. Note that the results do not include any influence of residual load. Moreover, the shaft resistance shows neither strain-softening nor strain-hardening—a rather unusual load response.

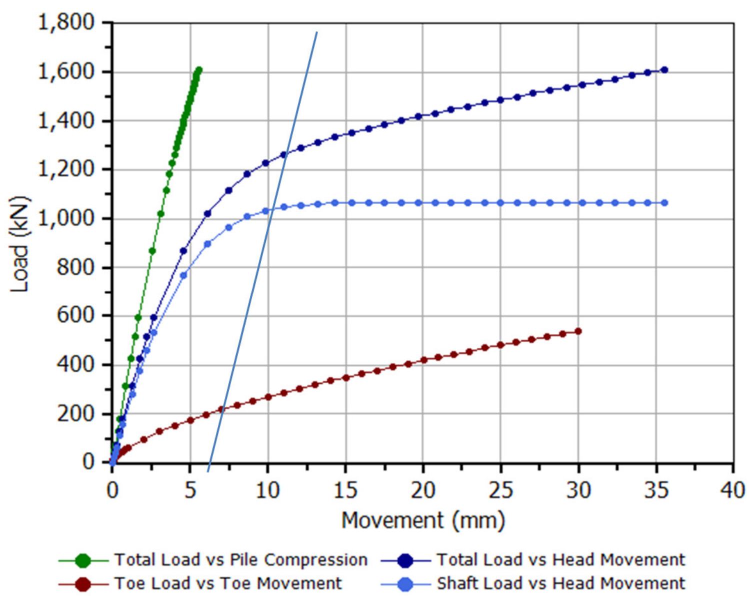

Fig. 1 Load-movement curves established in a virtual static loading test on the 20 m long, 12-inch square concrete pile at initial condition

Step 1 — Pile Capacity

In choosing a unit shaft resistance ß-coefficient, it is usually understood that, in an effective stress analysis, the unit shaft resistance determined by the coefficient is the ultimate resistance, i.e., the shaft capacity (as, for that matter, would be the case also for a total stress analysis with direct input of unit shaft resistance). Although ultimate toe resistance does not exist, the 'toe capacity' is here assumed—defined—to be the toe load that generates a 30-mm toe movement, as stated above.

When the mentioned parameters are used in a simple spread-sheet calculation, the distribution with depth of effective stress along the pile is easily obtained. Then, by multiplying the effective stress at each depth with the ß-coefficient, adjusting for pile circumferential area, and accumulating the values along the pile, the pile capacity is quickly determined. For the condition immediately after the warehouse has been constructed ("initial condition"[2)] before any floor loads have been placed in the building, the total pile capacity is 1,607 kN (R_{shaft} = 1,067 kN; R_{toe} = 540 kN), which means that the factor of safety is 2.01. If, as in the assumed case, if established by a static loading test, this factor of safety is normally considered adequate.

Note, however, that there are many methods for determining the pile capacity from a static loading test, usually from the pile-head load-movement curve. With regard to the results shown in Figure 1, capacity can be interpreted to range from a low of about 1,200 kN through 1,607 kN, the maximum load applied (capacity must never be extrapolated from a loading test). That is, the capacity interpreted from the test may not agree with the capacity summed up from the pile elements and, vice versa, the average shaft resistance back-calculated from a capacity determined from the pile-head load-movement curve of a static loading test will not necessarily be that determined from the particular pile elements.

For the piled foundation design, it is not important that the capacity will increase with time after the consolidation of the clay from the floor load is completed—the design must consider the case of the least capacity. However, the consolidation will result in a settlement of the soil, primarily of the compressible clay layer, which will impose downdrag (settlement). It is very important that the settlement be assessed in the design. This is the conditions addressed in Step 2.

Step 2 — Load Transfer, Settlement, and Downdrag

When the warehouse is constructed, the transfer of the 800-kN/pile dead load will be associated with some downward movement of the pile as well as 'elastic' shortening. As the ultimate shaft resistance (1,067 kN) is larger than the dead load, it will not be fully mobilized along the lower portion of the pile. However, some load will reach the pile toe and cause a small pile toe movement. It is realistic to assume that the lowest pile element will penetrate a few millimetre, maybe 5 mm for the example case, into the soil. The pile shortening due to the 800-kN pile-head load will be about 3 mm. Some load-transfer movement will be caused by the load from the 4-pile group compressing the soil layers below the pile toe. The effect of the load transfer above the pile toe is covered by the shaft resistance shear-movement relation. Vertical stress distribution along the pile shaft can be assumed not to cause measurable settlement near the piles because of the reinforcing (stiffening) contribution of the piles. The calculation of the settlement of the soil below the pile toe level can be made assuming that the load acts on an equivalent footing placed at the pile toe. The compression of the soils below the pile group toe level will

[2)] The "initial condition" calculates capacity before any loads, fills, and other effects pertaining to the "final condition" are placed at the site.

cause an about 3 mm additional settlement[3)]. The total pile settlement immediately after the construction of the warehouse will thus amount to about 10 mm. As for most structures, these immediate "settlements" can normally be considered inconsequential and be disregarded.

When the floor load is placed, consolidation plus a small contribution due to compression of the sand layers will result in a long-term settlement of the ground amounting to about 150 mm at the ground surface and about 25 mm at the pile toe level[4)]. The resulting downdrag on a piled foundation located in the center of the structure footprint causes a 26-mm calculated piled foundation settlement for a neutral plane at 12.6 m depth. However, this location presupposes that the total pile toe penetration is 30 mm, but the calculation shows that the net penetration due to the downdrag is only about 20 mm. Considering the initial about 5-mm toe penetration, the calculation needs to be adjusted to a smaller toe force (5,500 kPa), which results in a raising the neutral plane to 12.0 m depth and shows a 32-mm pile settlement due downdrag. The settlement values are about the same. However, at other sites and for other conditions, not matching the toe resistance to the toe penetration may result in significant error in the estimated downdrag magnitude. Moreover, calculations using different unit toe resistance values will show that the results are very sensitive to the input values. This supports the general advice that piles in settling soils, where downdrag is expected, should aim for a neutral plane to be below the settling soil layer, that is, the design may have to aim for a larger toe resistance and larger capacity than otherwise considered necessary.

A column foundation located near the outer wall or at the corner will settle appreciably less than one in the center. The long-term differential settlement will be about half the total, say, about 20 mm[5)].

The maximum axial load will be about 1,300 kN, which is well within the acceptable considering the pile axial strength.

It would seem that the warehouse foundation is adequate both with regard to capacity and settlement.

Realistic Shaft Resistance t-z Function

The case is a "textbook" case, meaning that it is rather simpler than most real life cases. The most obvious simplification is the assumption of elastic-plastic shaft resistance in the clay layer. Shaft resistance in clay is usually strain-hardening in firm to stiff clay. In soft clay, as in the current case, it would be strain-softening. The t-z function representative for the case would therefore normally be the Hansen or the Zang functions. Figure 2 shows the results of a simulated loading test for the t-z function in the clay layer input as a Zang function with the "a-coefficient" equal to 0.0125, which indicates that if the ultimate shaft shear is mobilized at a 5-mm movement, then, at a 30-mm movement the shaft shear

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[3)] Change, temporarily, the equivalent raft area to 9.5 x 9.5 m (pile group input; see the Red Book) to reflect the load-spreading along the pile shaft as 20% of the pile length and run the analysis. The results table for settlement will display the soil settlement at the pile toe depth due to the load on the pile group plus sustained floor load.

[4)] Return the input of the equivalent raft area to 6.5 x 6.5 m (pile group input reflecting the load-spreading from the neutral plane to the pile toe 20% of the pile length below at about 12.5 m depth; see the Red Book), adjust, temporarily, the toe resistance input to 5,500 kPa (to consider the pile toe load for the actual pile toe penetration) and run the analysis. The results table for settlement will display the soil settlement at the pile toe depth, as well as the pile settlement.

[5)] The settlement for a pile located along the outer wall or at a corner of the footing can be calculated by indicating new coordinates for the pile and the pile group (Pile Data input), e.g., x =25 m and y = 75 m for the middle of a long side and x = 0.75 m and y = 0.75 m for a corner pile in the pile cap. This calculation will also demonstrate the sensitivity of the calculations to the input preconsolidation margin, which in a real case might trigger a reassessment of the margin.

has reduced to about 75 % of the ultimate[6]. The effect of this is that the maximum load (at 30-mm pile toe movement) has reduced by 200 kN, to 1,422 kN from 1,607 kN. The capacities determined by other methods have been reduced similarly. For example, the Offset Limit reduces from about 1,350 kN to about 1,300 kN. Note however, that the assumed ultimate shaft resistance (per the input ß-coefficient) is still 1,607 kN.

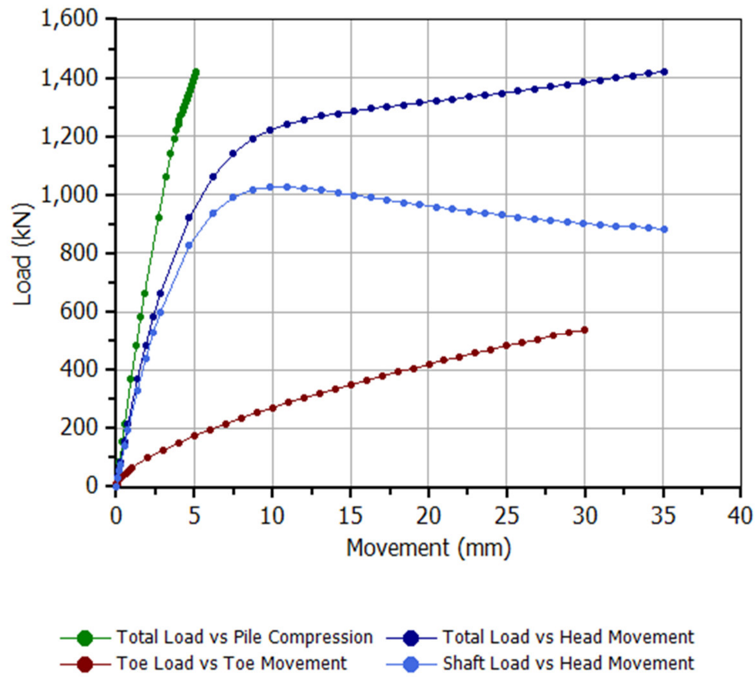

Fig. 2 Load-movement curves established in a virtual static loading test for strain-softening

Bidirectional test

Most static loading tests are uninstrumented and the only records taken are those at the pile head: the pile-head load-movement. Such tests (called "head-down tests") provide very little useful information. Of course, one can deduce a pile capacity from the curve by some definition or other. But what that capacity actually represents is highly ambiguous and of little use for assessing the pile foundation's response to load. Much of the uncertainty can be removed by placing strain-gage instrumentation at a few depths in the pile. However, this significantly increases the costs of the test and strain-gage records require evaluation as to reliability and accuracy of the records, as well as resolving confusion about what the measured strains actually represent. The best way to obtain useful records of the pile response in a test is to perform a bidirectional test, that is placing a hydraulic jack at suitable depth and perform the test by simultaneously pushing the upper portion of the pile upward and the other downward.

Figure 3 shows the virtual results of a bidirectional test for the same pile, load, and soil conditions as the head-down test shown in Figure 1. The bidirectional cell is placed at 17 m depth (to represent a desired emphasis of the response of the lower pile length in the lower dense sand).

Usually, the bidirectional cell is placed at the presumed "balance level" so as to ensure that both the upward and downward load-movement responses are obtained. When planning a test, it is a good rule to

[6] Just change the assigned t-z function for the clay to the preprepared option "Shaft in clay -- strain-softening".

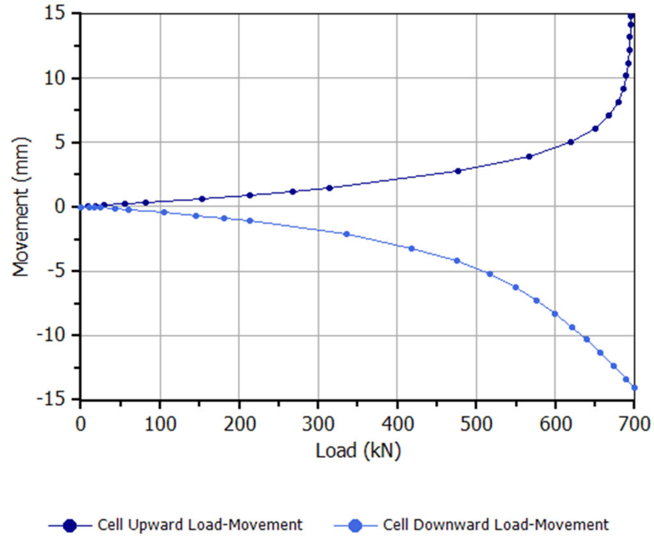

Fig. 3 Load-movement curves established in a virtual bidirectional loading test with the cell placed at 17 m depth[7) 8)]

"err" on the side of the upward response being the larger, because, then, the test will establish the downward response, which is the most important part of a test. If more information on the response of the upward length is desired, one can always carry out a simple tension test. No equally simple and low-cost enhancement test exists for the downward response.

Residual load

Normally, at the start of a static loading test, load already exists in the pile. This load is called "Residual Load". It is almost always present in a driven pile after the driving. It may also occur in a bored pile, because residual load can develop during the time elapsed from the construction (installation, driving, casting) to the testing. Residual load is caused by the soil having moved down relative to the pile, or the pile moved up (as in a swelling soil). These movements can be very small and yet develop considerable residual load. The relative movement is resisted by negative direction shear forces in the upper portion of the pile and by positive direction shear forces along the lower portion. The mechanism is similar to the build-up of a drag force.

The residual load is difficult to measure directly. It may affect significantly, and often to a hard-to-determine degree, the load distribution in a test pile, when measured, for example, by strain gages at selected depths. Less realized is that the presence of residual load will also affect the pile-head load-movement curve. For example, if the subject warehouse pile would be affected by residual load along the upper sand and the clay amounting to, say, about 400 kN, which would have to be resisted by a residual load in the dense sand of equal amount, then, a static loading test would result in the load-movement curves shown in Figure 4. A capacity interpreted from the pile-head load-movement curve in Figure 4 would be about 200 kN larger than that interpreted from Figure 1.

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7) The curves are shown in the manner of a conventional test report, i.e., without adjustment for pile weight and pore pressure.

8) The graph is cut-off at equal upward and downward loads, which is the case for an actual bidirectional test. However, UniPile calculates the upward and downward results independently and stops the calculation at the specified toe movement, which can result in different upward and downward movements. That the upward and downward movements for the maximum load are equal (almost) here is a coincidence.

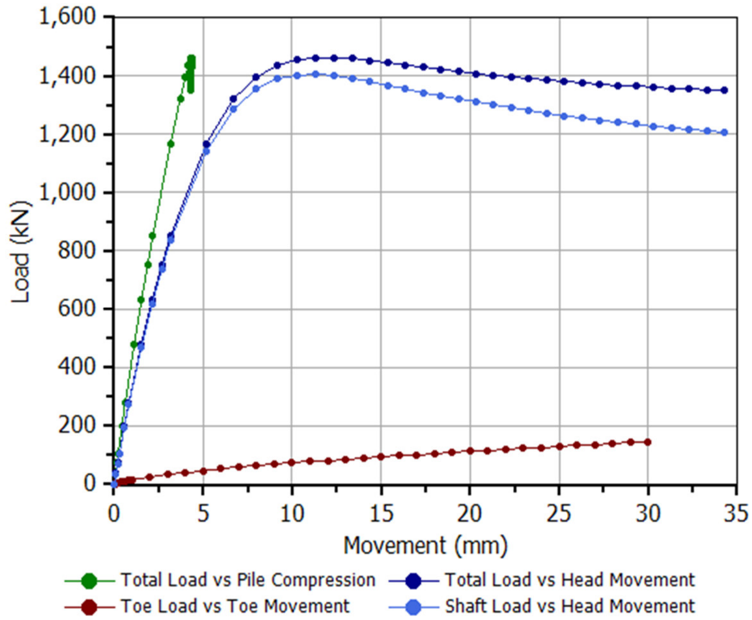

Fig. 4 Load-movement curves established in a virtual test on a pile with residual load

When not certain whether or not the test pile is affected by residual load, which capacity value should one rely on? And, if the pile would have been strain-gage instrumented to separate shaft and toe resistance, which shaft and toe resistance distributions would be believed, those of Figure 1 or of Figure 4?

The bidirectional test has the advantage that the load measured by the cell is the true load in the pile. This is why a bidirectional test on a strain-gage instrumented pile often shows a discrepancy between the cell load and the strain-gage load near the cell level. The cell load is the correct values and the load difference can be used to assess the residual load in an instrumented pile.

Future Complications

The example is taken from the real world and modified from records of a warehouse constructed in the 1960s near Milwaukee, WI. Starting in the mid-1980s, the City constructed a sewage facility that required extensive underground construction and rock tunneling. As a consequence, the phreatic height of the groundwater in the lower sand layer reduced considerably, say, by about 5 m at the example site[9]. This lowering of the pore pressures in the ground has resulted in a regional settlement and downdrag for many foundations. For the subject example, this can easily be quantified by simply changing the final pore pressure distribution to a 7-m head as opposed to the original 12-m head at the lower boundary of the clay layer. The clay will show a downward pore pressure gradient. The calculations for this condition show that the settlement of the piled foundations will increase. The simultaneous effect of increasing the pile capacity is irrelevant to the case (assuming that the loads from the structure are unchanged).

The main effect of the lowering of the groundwater table will be on the settlement of the clay layer, which will increase from about 120 mm to about 300+ mm. The settlement of the lower sand layer will increase by about 4 mm. The calculated additional downdrag on the piles will be about 10 mm, after including the fact that the toe resistance will increase due to the increased pile toe penetration, which will result in a lowering of the neutral plane. The drag force is of no consequence as the piles are not long.

[9] Change the Final Pore Pressure/Profile Type to Non Hydrostatic (already in the file) and run the analysis to model the effect of the groundwater table lowering.

The settlement of the floor will not be even and it will be necessary to adjust the height by adding additional asphalt concrete. This will introduce about 5 to 10 kPa additional floor loads, which will cause increased settlement and downdrag.

It is quite possible that over the life of a building, the area around it will be raised by placing fill. This fill will renew the consolidation settlement and cause additional downdrag. It is also quite possible that the use of the warehouse will change to include an increased floor load. The original design should consider all implications and produce a sufficiently conservative foundation design[10].

Furthermore, the building might be close to an excavation, a river bank, a dock, or other feature affecting the distribution of effective stress under the warehouse to a larger or smaller extent. In the future, a new building may be constructed adjacent to one side of the first building. All such features are easy to incorporate into an analysis and there is no reason for omitting them.

[10] The file includes a load option for increasing floor load and for placing fill around the warehouse. Note, no fill will be placed inside the building. It is necessary to adjust the floor load to reflect this. If no new floor load is placed, then, input a negative stress equal to the fill stress as the "New Floor Load to account for the fact that the warehouse is a "hole" in the fill.