Large Diameter Long Bored Piles in the Mekong Delta

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ABSTRACT *Static loading tests, O-cell tests, were performed on two long, strain-gage instrumented, bored piles in HoChiMinh City, Vietnam, where a series of twelve apartment towers were to be constructed. The test piles were constructed to 76 and 91 m depth and tested to maximum O-cell loads of 10 and 18 MN, respectively. For both piles, the O-cell level was placed at a depth of about 20 % of the pile length above the pile toe. The soil profile consisted of very soft organic clay to about 10 to 15 m depth underlain by firm to stiff clayey soil to about 25 to 45 m depth. Hereunder, the soil consisted of a compact to dense sandy silt. Neither of the tests was able to fully engage the shaft resistance of the piles above the O-cell level, but did so below the O-cell level. Back-calculation of the load distributions determined from the strain-gage measurements showed the shaft resistance, even where fully mobilized, to be very small: the beta-coefficient applied in an effective stress analysis was only about 0.13 to 0.14. The evaluations of shaft resistance development showed a maximum shear resistance to occur after a movement of only 3 to 4 mm, after which the response became plastic and strain-softening. The toe resistance was very low because the construction had left soil debris at the bottom of the drilled hole. Ongoing regional settlement leads to concerns about the possibility for the production piles to have a similarly low toe resistance. This would locate the neutral plane of the shorter piles in settling soil and create a downdrag situation for the piled foundation.*

KEYWORDS: Mekong Delta, O-cell test, strain-gage evaluation, bored piles, shaft resistance, settlement, beta coefficient.

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INTRODUCTION

Vietnam is integrating into the world economy at an increasing rate, causing a rapid growth in the urban population and necessitating extensive investment in new urban housing and infrastructure. The geology in these areas is characterized by thick deposits of soft deltaic soil. The project consists of a series of apartment towers, depicted in Figure 1, located in HoChiMinh City, Vietnam, in the Mekong River basin near the Saigon River. This paper presents the results of static loading tests on two instrumented bored piles, applying the O-cell method (Osterberg 1989; 1998) with a single-level O-cell placed 15 and 20 m above the toe of the respective piles (Fugro Loadtest 2010). The test results are presented and discussed as to the quality and use for the design of the piled foundations. The main foundation issues for the project are pile capacity and downdrag primarily caused by ongoing regional settlement.

SOIL PROFILE

As is typical for the HoChiMinh City general area, the site profile consists of an upper organic soft clay, which is highly compressible and normally consolidated. Figure 2 shows this layer to be about 10 m thick at the subject site and to be underlain by soft to firm, normally compressible clay to about 28 m depth. Hereunder, the soil consists of firm sandy clay. Between about 28 m though 40 m depth, the soil consists of clayey sandy silt, followed by about 8 m of clay and silt. From about 50 m depth, the soil consists of a sandy silt, usually considered as the bearing layer for deep foundations in the area.

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Figure 1. Photo of architect's model of the Sunrise City project.

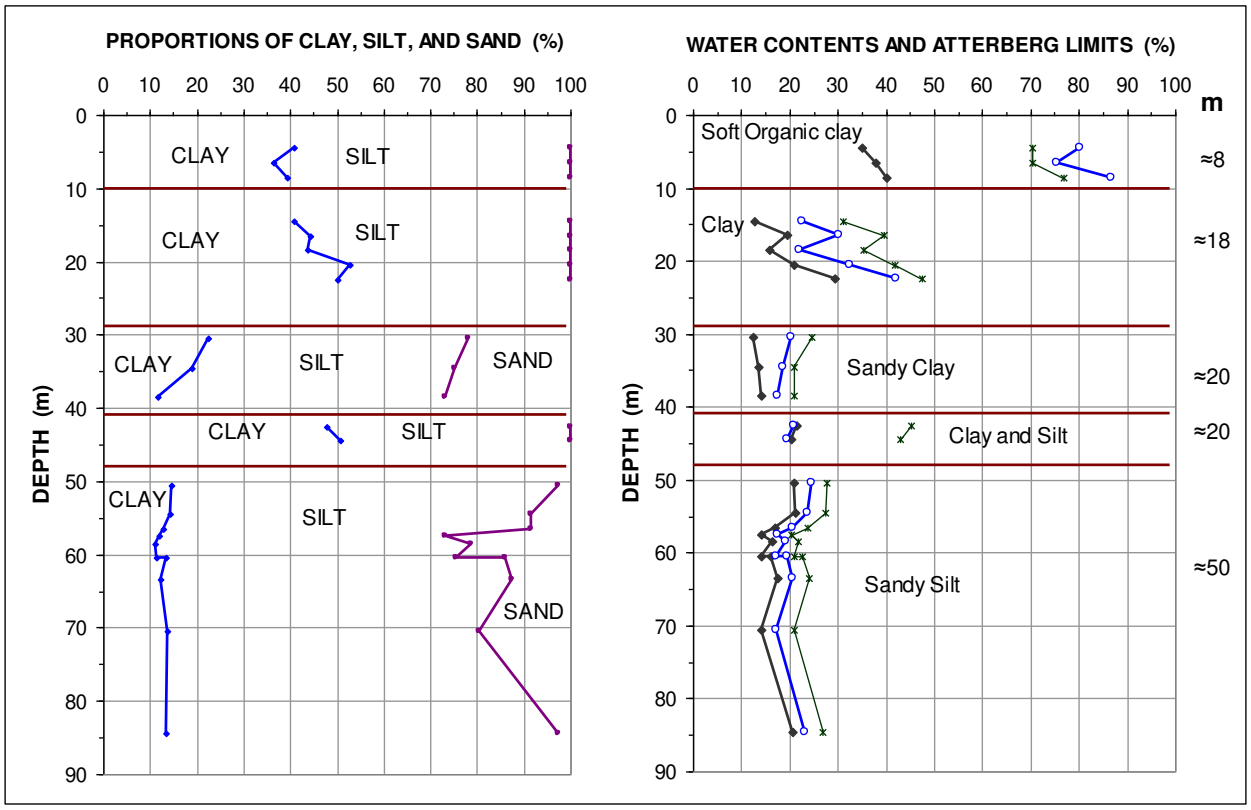


Figure 2. Soil profile showing grain size distributions and consistency parameters. The "m-values" in the column to the right are Janbu modulus numbers.

Standard penetration tests (SPT) gave N-indices ranging from about 10 to 13 blows/0.3m in the 30 to 48 m depth range, gradually increasing to about 30 blows/0.3m at 80 m depth. Below 80 m, the soil is very dense (SPT index >50 blows/0.3m). The site investigation did not include CPTu soundings.

The compressibility of the soil layers is indicated by the "m values" listed to the right of the graph. The "m" denotes the Janbu modulus number—$m = 2.3(1+e_0)/C_c$—detailed by Janbu (1967; 1998), Canadian Geotechnical Society (1992), and Fellenius (2012a). The area is affected by a small regional settlement in the soft clay layer.

The Saigon River level fluctuates seasonally by about 3 m. The groundwater table lies near the ground surface, becoming slightly artesian with depth. The firm to stiff clay and clayey silt has a virgin modulus number of approximately 20, and it is, typically, slightly preconsolidated with a preconsolidation margin of about 20 kPa and a Janbu recompression modulus number, m_r, of about 150 to 200. The dense sandy silt is moderately compressible—the modulus number is about 50. However, this value is lower than expected for the dense to very dense soils, and it is likely that the soil samples were disturbed by the sampling.

THE PILES

Two bored piles, Piles TBP-1 and TBP-2, with 1,500 and 1,800 mm diameter, respectively, were constructed in October 2010 to 76 m and 91 m depths, respectively, using bentonite slurry. The piles were arranged with a single O-cell level at 60 m and 71 m depths, respectively (Figure 3), and the loading tests were carried out 30 days after concreting. The test piles were instrumented with pairs of vibrating-wire strain gages at 11 and 14 gage levels in Piles TBP-1 and TBP-2, respectively. The towers are to be supported, typically, on groups of about 88 piles, 1,500 mm diameter, over an area of about 35 by 35 m. The footprint ratio, pile total area to foundation area, is 13 % and the average pile center-to-center (c/c) spacing is 3.75 m (2.5 pile diameters).

TEST RESULTS

The O-cell load-movement records from the tests on Piles TBP-1 and TBP-2 are shown in Figures 4 and 5, respectively. Neither test mobilized fully the shaft resistance along the length of pile above the O-cell level. The upward movements of Piles TBP-1 and TBP-2 shaft were small: 4 mm and 3 mm, respectively. In contrast, the downward movements were large, 140 mm and 120 mm respectively, at the maximum load. The maximum test load was the load applied when the limit of the O-cell opening was reached.

In order to determine the load values from the strain-gage measurements, knowledge of the pile "elastic" modulus is required. The modulus of concrete is not constant, but is a linear function of the induced strain, reducing with increasing strain. It is best determined directly from the test data according to the "incremental stiffness" or "tangent modulus method" (Fellenius 1989; 2001; 2012a; 2012b). Figures 6 and 7 show the incremental stiffness values at the various gage levels of Piles TBP-1 and TBP-2. The largest strain is about 250 με, which is smaller than the desirable range. (For best accuracy, the largest strain introduced in the test should be larger than about 400 με). The gage records at GL 4, GL 5, and GL 8 for Pile TBP-1 and GL 6 for Pile TBP-2 showed the gages to malfunction and were therefore disregarded in the analysis. The GL 1 values may be affected by the proximity of the gage level to the pile toe. The scatter in the figures is because the shaft resistance went into a strain-softening regime at movements larger than about 4 mm, which will be discussed below. No strain dependency was apparent and the best evaluation is a constant stiffness (EA) of 44 GN and 63 GN, respectively. These stiffness values correlate to the same E modulus, 25 GPa, for the nominal cross section areas.

The evaluated stiffnesses were used to calculate the load corresponding to each strain value. The distributions of load in the piles for each applied O-cell load are shown in Figure 8. Records of the malfunctioning gages (GL 4, GL 5, and GL 8 in TBP-1 and GL 6 in TBP-2) were disregarded. For each pile, the buoyant pile weight was subtracted from the loads calculated for the gage levels above the O-cell level. The arrow in each figure indicates where about 4 mm downward movement was reached, beyond which the shaft resistance below the O-cell degraded. For Pile TBP-2, GL 1 suggests a much smaller toe load than measured for Pile TBP-1 and at the same time a much larger shaft resistance between GL 2 and GL 1 than measured between GL 3 and GL 2. This is inconsistent and, therefore, the records from GL 1 in Pile TBP-2 are considered unreliable.

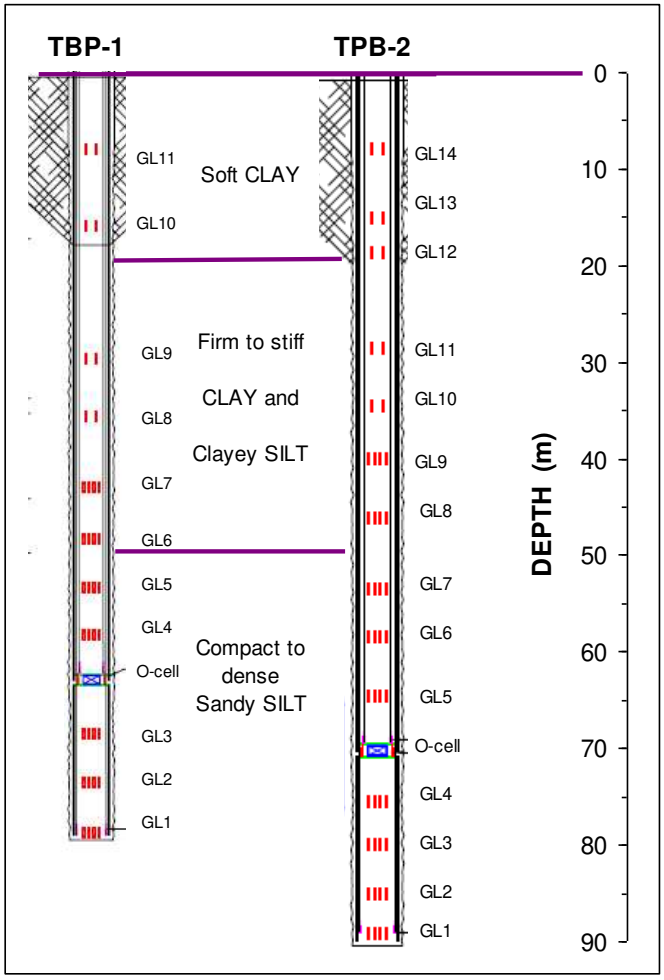


Figure 3. Schematics of the O-cell test piles (GL: Gage level).

Figure 9 shows the average unit shaft resistance as calculated from the evaluated loads by dividing the difference in load between the gage levels with the shaft area between the gage levels. The curves show that, for both piles, ultimate shaft resistance along the length below the O-cell level occurred already at a downward movement of about 3 to 4 mm. At that movement, when the next increment of load was applied, the downward movement suddenly increased by 56 and 36 mm, in Piles TBP-1 and TBP-2, respectively. It is probable that a peak resistance had been reached and the large movement readings represent post-peak values of shaft resistance.

The maximum values of the unit shaft resistance evaluated between the O-cell and Gage Level 1 for both test piles showed an average unit shaft resistance of about 90 kPa. When correlated to effective overburden stress, the Bjerrum-Burland proportionality coefficients—beta-coefficients (Burland 1973)—for Piles TBP-1 and TBP-2 are 0.14 and 0.13, respectively, representing post-peak conditions. Pre-peak values would seem to have been about 25 % larger. The values are smaller than expected. It is probable that the construction process left a filter cake between the concrete and the soil and, therefore, the shear movement occurred in the filter cake.

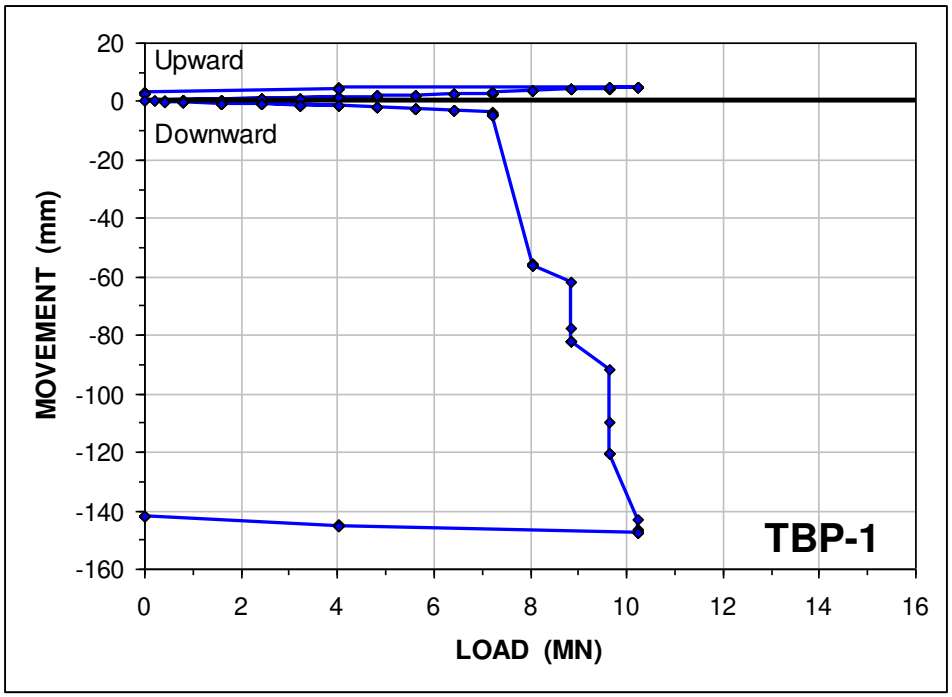


Figure 4. Load-movement measured for Pile TBP-1.

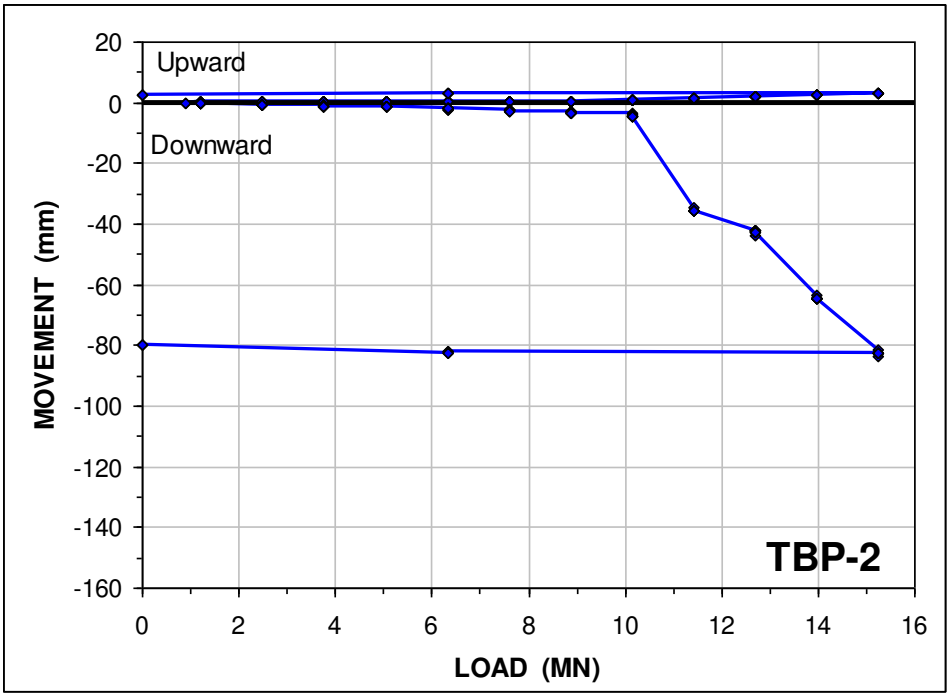


Figure 5. Load-movement measured for Pile TBP-2.

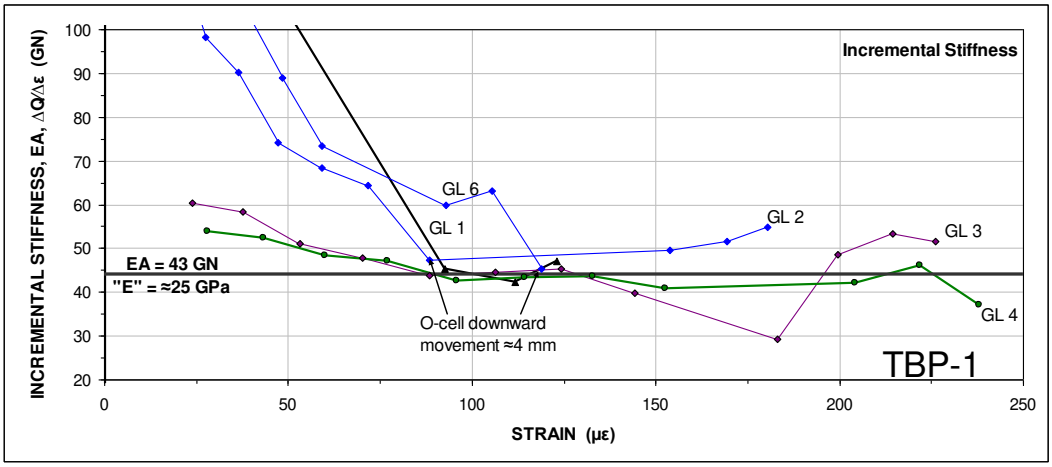


Figure 6. Incremental stiffness plot of the strain-gage measurements from Pile TBP-1.

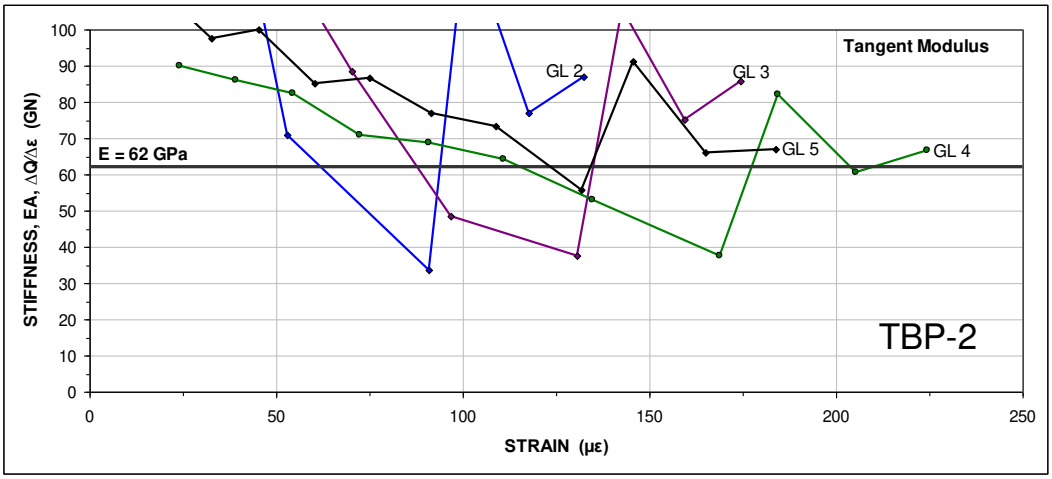


Figure 7. Incremental stiffness plot of the strain-gage measurements from Pile TBP-2.

For Pile TBP-1, the pile toe load is the load determined by gage level immediately (0.5 m) above the pile toe, GL 1. For Pile TBP-2, the toe load was determined by assuming that the unit shaft resistance between the pile toe and GL 2 was the same as between GL 2 and GL 3 and subtracting this from the load determined for GL 2. By dividing the toe loads with the nominal cross sectional area of the respective pile, the pile unit toe resistance acting on the test piles was established. Figure 9 shows the so-determined toe loads versus the pile toe movements.

It would have been easy to conclude from the downward movement of the O-cell plate, that an ultimate toe resistance had been established in the test, that is, a plastic response to the increasing load could be surmised. However, it is a misconception that ultimate pile toe resistance can develop in sand (Fellenius 1999; 2012a). It is well-established that pile toe response to load is by gradual increase of resistance with increasing movement, and the toe response is a function of the stiffness of the soil below the pile toe, as is demonstrated in Figure 10 for the test piles.

Considering that the pile toes were located in very dense soil, the pile toe stiffness is very low. To clarify the pile toe conditions, two other piles were constructed nearby using the same techniques (i.e., bucket excavation of the soil maintaining the hole with bentonite slurry and tremie pouring of the concrete) and equipped with a coring tube to about a meter above the pile toe. Coring through the remaining part of the pile and into the material below the pile toe revealed that a few inches thick layer of slurry and debris between the concrete and the soil had been left behind at the pile toe. Therefore, we consider the measured pile toe movements to be more due to compression of the slurry-debris layer than to compression of the intact soil below the pile toe.

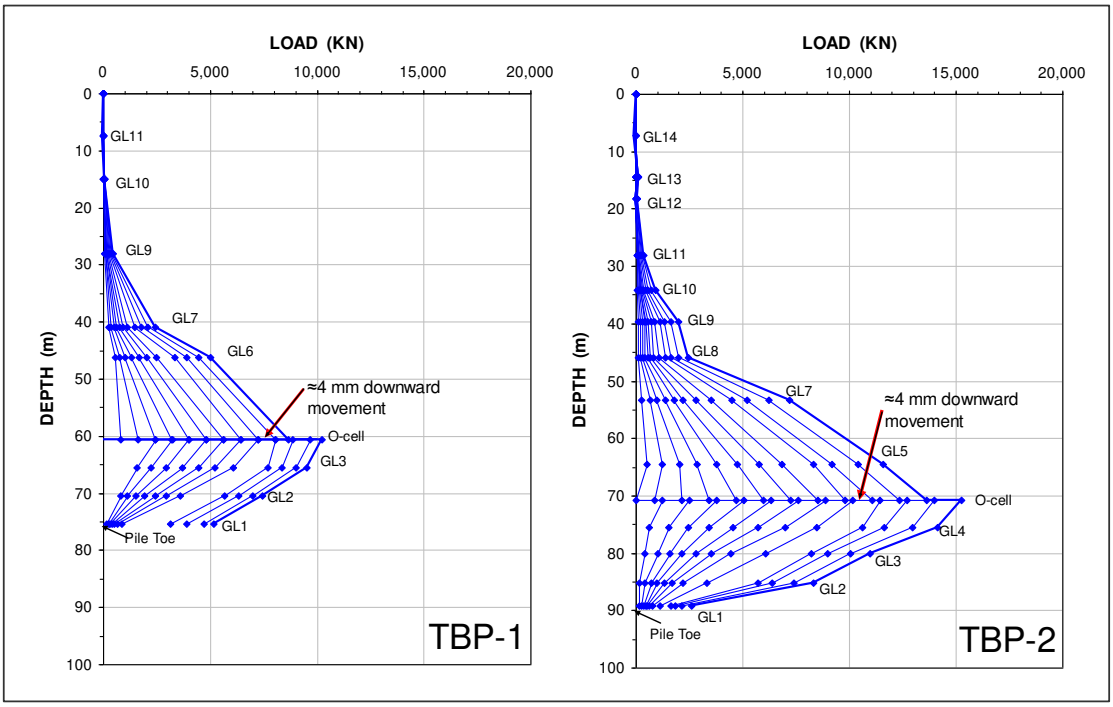


Figure 8. Load distribution in Piles TBP-1 and TBP-2 during the tests.

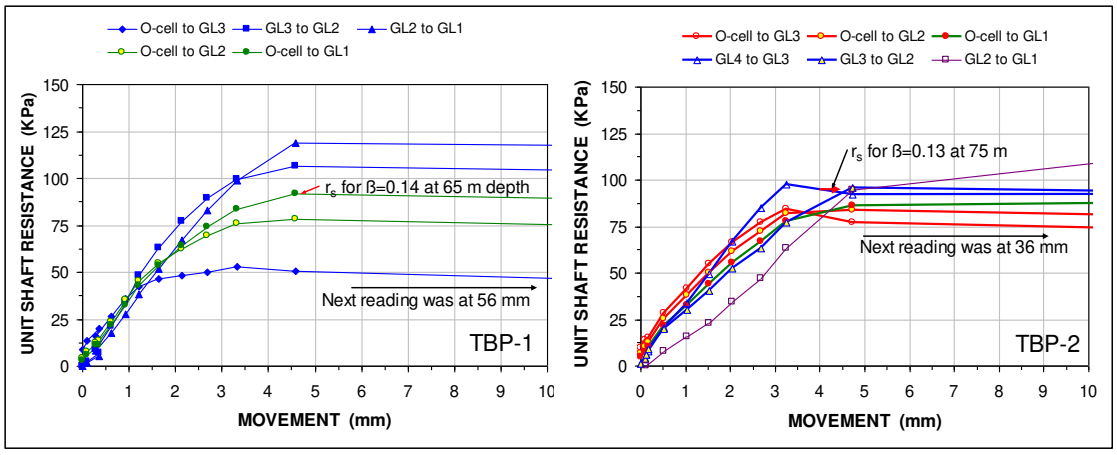


Figure 9. Unit shaft resistance along Piles TBP-1 and TBP-2.

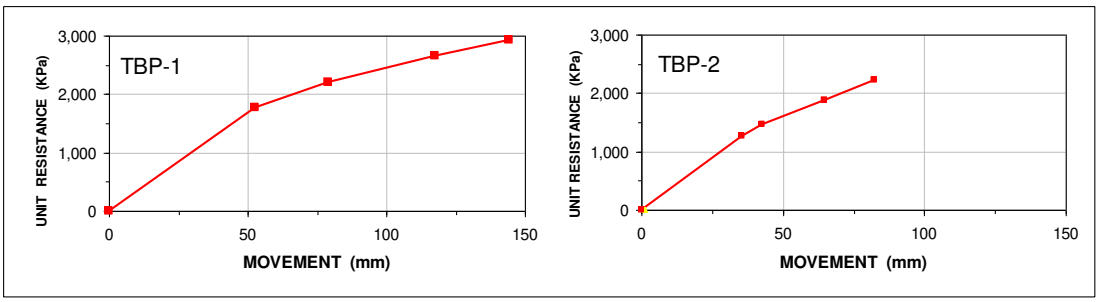


Figure 10. Unit toe resistances for Piles TBP-1 and TBP-2.

The O-cell load distribution curves can be turned into "equivalent head-down load distribution curves", as illustrated in Figure 11. These curves are produced by mirroring, "flipping", the upward O-cell curve from the buoyant O-cell load. The maximum pile-head test load is then twice the maximum O-cell load minus the buoyant weight of the length of the pile above the O-cell. The load distributions in Figure 10 are supplemented with the distributions determined in an effective stress calculation (ß-analysis) fitting the calculated distribution (dashed curves) to the measured. Beta-coefficients (ß) of 0.12 to 0.14 were applied to the soil layers above and below the O-cell and showed to provide the fit. The analyses were made using the UniPile software (Goudreault and Fellenius 1999). The separation between measured and calculated curves above about 30 m depth represents the fact that the shaft resistance above 30 m depth was not fully mobilized in the O-cell tests.

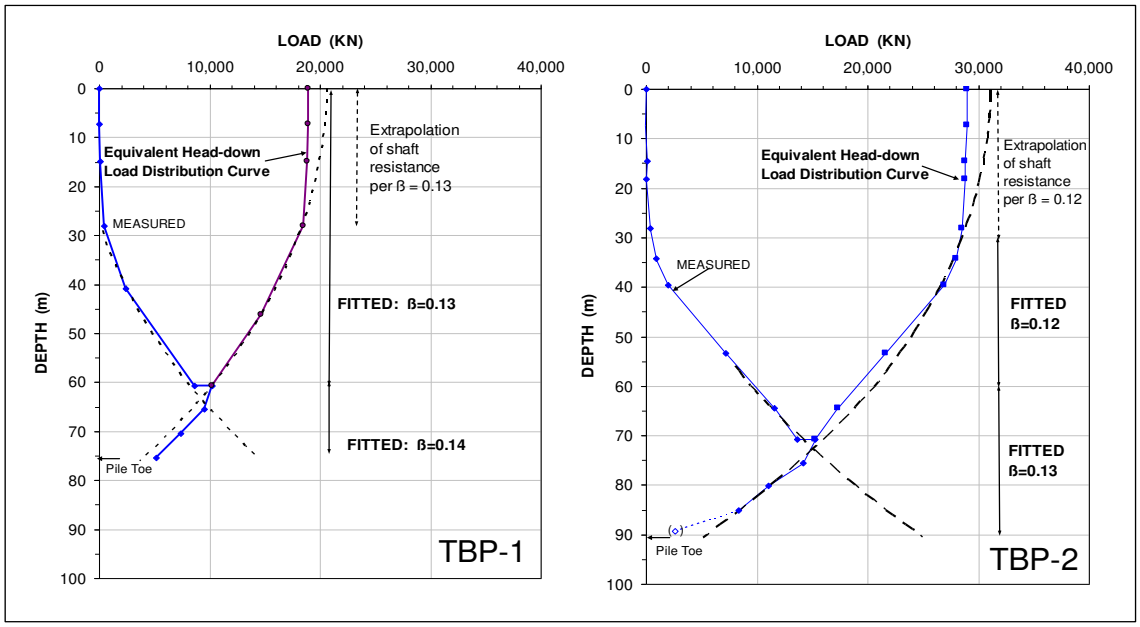


Figure 11. O-cell load distribution and its equivalent head-down distribution for Piles TBP-1 and TBP-2.

Most reports submitted by testing companies include a load-movement graph showing a conversion of the O-cell test data to an "equivalent head-down load-movement curve". The conversion is achieved by adding the O-cell loads measured at equal movement values upward and downward and plotting the sum of the loads versus the movement until the maximum upward or downward movement is reached. This plot is supplemented with compensation for the larger pile compression imposed in a head-down test as opposed to the O-cell test. Figure 12 shows the routine construction for the subject tests as plotted from data in the report (Fugro Loadtest 2010). The maximum value is the combined load at the maximum of the two measured movements, in this case, the maximum upward movement. Sometimes, the curve is extrapolated beyond the maximum of the smallest of the upward or downward movements, either by assuming, conservatively, that the load for continued movement is constant, or, alternatively, by extrapolating the trend of the smallest movement.

An alternative construction of the equivalent head-down load-movement curve can be produced by combining the effective stress analysis fit to the resistance distribution shown in Figure 13 by a t-z and q-z analysis, calculating the movements leading up to the evaluated load distribution. The q-z function is first fitted to the measured pile toe load-movement. For the subject case, the toe fit was made by the "ratio" approach (Fellenius 2012a; 2012b), wherein the ratio between the measured (Q_1) and assumed loads (Q_2) is equal to the ratio of the measured and the ratio between measured (δ_1) and assumed movement (δ_2) is raised to an exponent (a) per equation $Q_1/Q_2 = (\delta_1/\delta_2)^a$. The measured maximum toe movements were chosen as the "calibrating" movement for Pile TBP-1 and TBP-2, 100 mm and 80 mm, respectively, and the exponent was fitted to the value 0.5. The shaft response was fitted to a t-z curve similar to measured. The so-simulated head-down load-movement curves are shown in Figure 13. Of course, the curves in the figure are extrapolations of the test results; nevertheless, the curves beyond the "Maximum loads in O-cell tests" are quite credible.

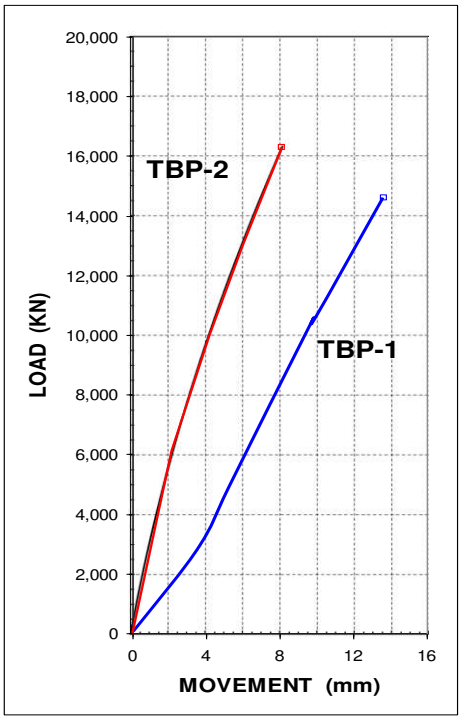


Figure 12. The equivalent head-down load-movement curves of Piles TBP-1 and TBP-2 (data from Fugro Loadtest 2010).

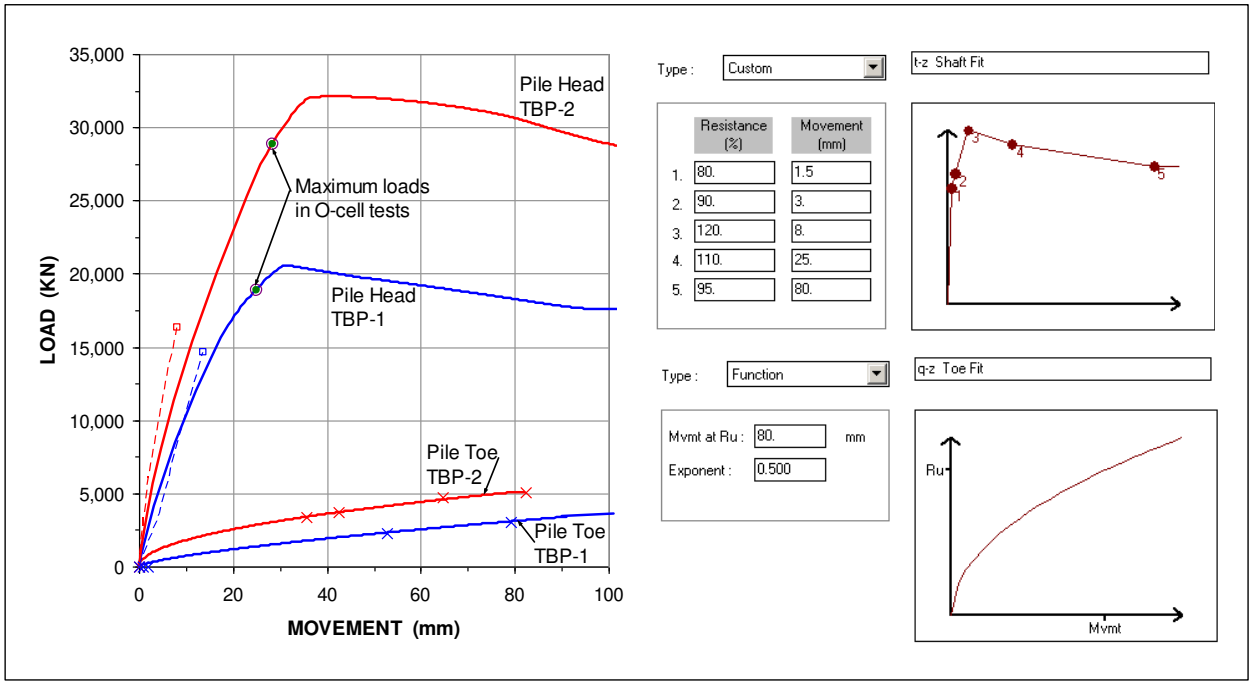


Figure 13. The equivalent head-down load-movement curves calculated from t-z and q-z functions fitted to measured toe load-movements and shaft capacities.

The intended unfactored working load is 8,000 kN/pile. The test piles were intended to represent the most likely piles for

the project; however the pile foundation design is as yet not finalized. It is useful to apply the intended working loads (long-term conditions) to the distributions found for the test piles. If assuming that the cleaning up of the pile toe before concreting is at least as successful as for Pile TBP-1, then, at a pile toe movement of about 30 mm, the long-term pile toe load will amount to about 4,000 kN for both piles. Moreover, the long-term load distributions for the piles will range as shown in Figure 14, taking the two test piles as the pile length and size boundaries.

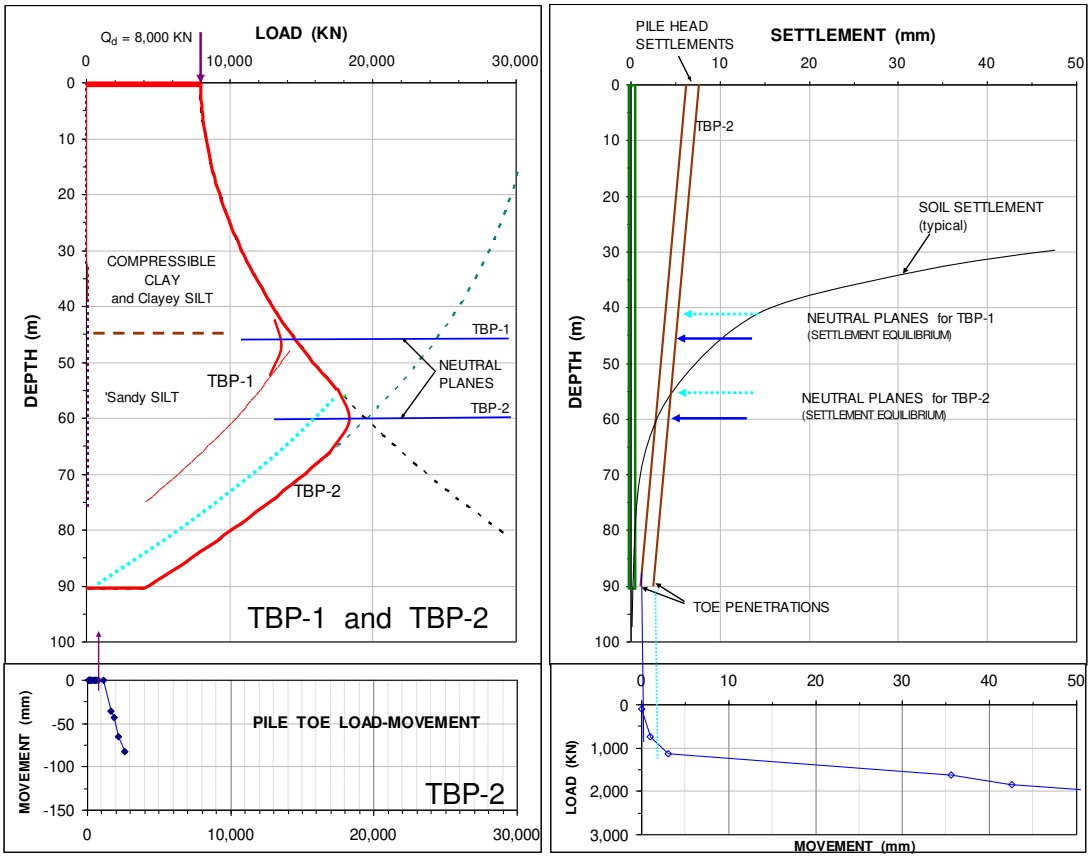


Figure 14. Long-term load distribution and settlement profile for working piles with same diameter and length as the two test piles.

The capacity of the longer and larger diameter pile to 90 m depth is more than adequate even if the toe stiffness would be smaller than the 4,000 kN value due to the toe resistance being debris affected. (As suggested to be the case in the bottom left diagram showing the actual O-cell pile-toe load-movement). In contrast, acceptance of the shorter and smaller diameter pile would be conditional on the success in ensuring that its construction achieves reasonable toe stiffness.

For both piles, the drag load is more or less equal to the dead load. The maximum load, i.e., the sum of dead load and drag load appearing at the neutral plane is well below the axial strength of the piles. (The neutral plane is the depth of force and settlement equilibria. The force equilibrium is located at the depth where dead load plus drag load equal positive shaft resistance plus mobilized toe resistance. The settlement equilibrium is located where the pile and the soil settle equally). A more important result of the analysis is that the neutral plane for both piles lies in the low compressibility soil (the clayey sandy silt). This is where long-term settlements due to the loads from the towers as well as the regional settlement in the delta will be small and not significantly impact the foundations of the towers. The right side graph of the figure shows a typical soil settlement distribution. For the 90 m long pile, a reduction in pile toe stiffness to that measured in the test would have little effect on the location of the neutral plane, i.e., the expected long-term pile head settlement. In contrast, were the pile toe stiffness be reduced for shorter pile, the subsequent rise of the neutral plane location would result in a significant increase of settlement. In fact, the settlement aspect of the piles is more important for success or no success of the design than whether or not the capacity is 2.5 times the working load or "merely" 2 times.

SUMMARY AND CONCLUSIONS

The O-cell load-movement result showed that the pile shaft resistance above the O-cell was fully mobilized below about 30 m to 40 m depth, leaving the shaft resistance along the upper length of the piles only partially mobilized.

The remaining amount of resistance to mobilize is estimated to be a mere 2,000 to 3,000 kN. It would have been a simple and low-cost effort to perform a head-down test on the piles with the O-cells open to determine the shaft resistance in the upper portions of the piles. No more than about 3,000 KN of reaction would have been necessary. A concluding head-down test will always add value to an O-cell test (Fellenius and Tan 2010). No such test was undertaken, unfortunately.

The incremental stiffness evaluation of the strain-gage records showed that the pile average stiffnesses (EA) were 44 GN and 63 GN, respectively, corresponding to a nominal E-modulus of the piles of about 25 GPa.

The loads evaluated for the various strain-gage levels showed the loads at the pile toe were small and that the stiffness response of the pile toes was soft despite the fact that the pile toes were in dense sand. This is an indication of slurry and drilling debris being left in the shaft when concrete was poured. Subsequent coring of an adjacent test pile confirmed this to be the case.

The shaft resistance showed an approximately linear response to a movement of about 3 to 4 mm, where a sudden change to plastic response occurred. The response is probably a post-peak softening behavior, but the peak occurred too suddenly for the data acquisition system to pick up.

The load distributions evaluated from the strain-gage records were fitted to an effective stress analysis and showed that the effective stress proportionality coefficient, β, was for the post-peak shaft resistance equal to 0.12 to 0.13 above the O-cell (where the shaft resistance had been fully mobilized). Below the O-cell, for Piles TBP-1 and TBP-2, the evaluated β-coefficients were 0.14 and 0.13, respectively—practically equal. From a depth of about 50 m, the soil is composed of dense to very dense sandy silt. The β coefficients are very low for being from shaft resistance in dense soil. It is probable that the shaft resistance was affected by the slurry filter cake.

The pile movements relative to the soil at the peak values were the same for the two piles and the records showed no correlation to pile diameter in this regard.

If the necessary cleaning out the pile toe from debris would not be successful for the production piles, then, the neutral plane would develop about 4 m and 10 m higher up for the pile depth range. This might make the 75-m depth less suitable than the 90-m depth.

The neutral plane lies below the compressible soil layers and neither the settlement due to the load imposed by the towers nor the regional settlement will be large enough to negatively impact the tower foundations. The thickness of the compressible layers may be larger at other locations of the site, and the neutral plane may there show to be located up in the compressible soil. If so, the piles will be subjected to downdrag and may have to be taken deeper so that the neutral plane will be in the low compressibility layers or be subjected to positive efforts to ensure large toe resistance.

The long-term maximum load in the piles, made up of dead load plus drag load, will be well below the pile axial structural strength.

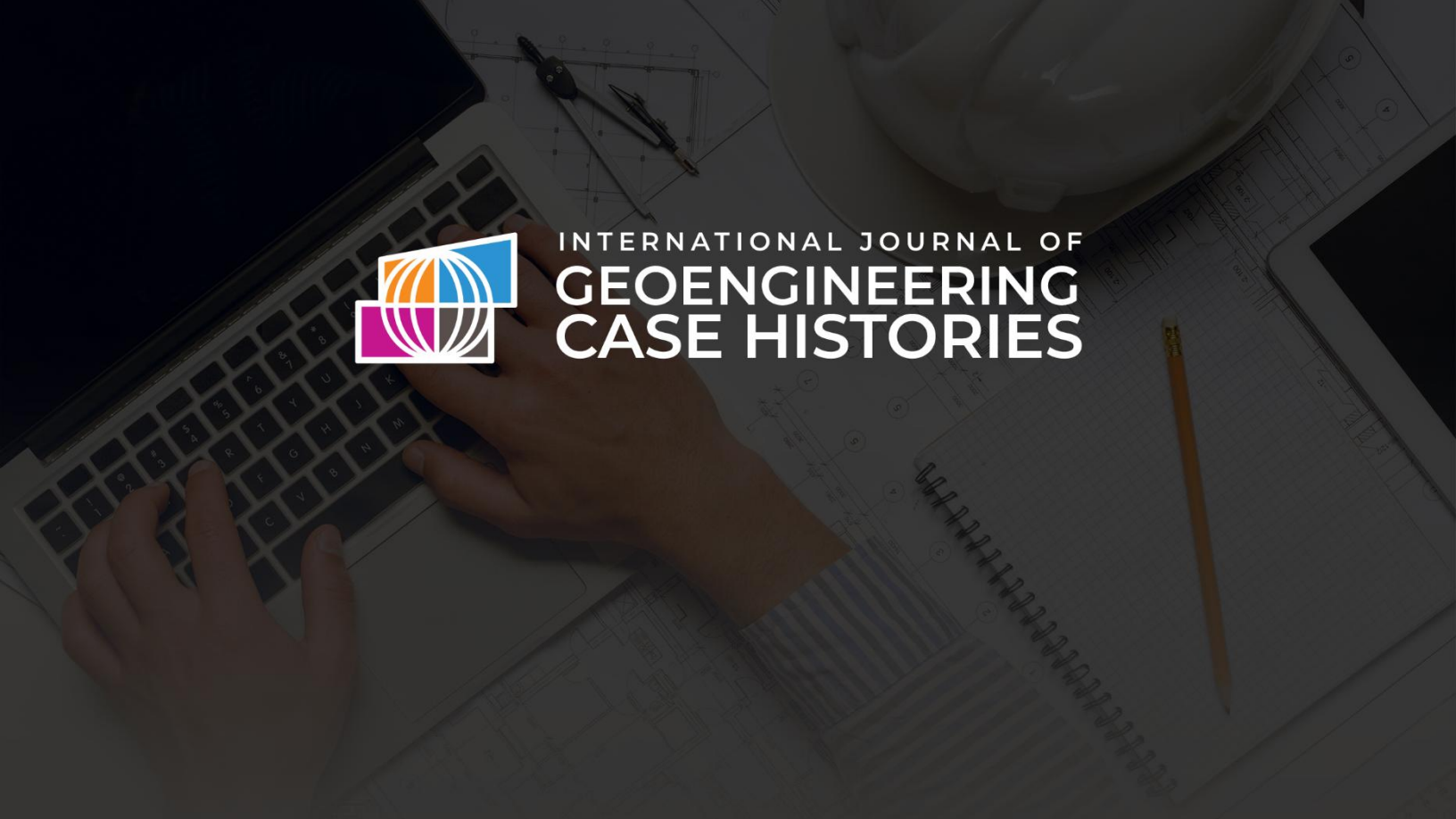
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