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331. Fellenius, B.H., 2014. An instrumented screwpile load test and connected pile-group load-settlement behavior. Discussion. Journal of Geo-Engineering Sciences, IOS Press, 1(2) 101-108.

Journal of Geo-Engineering Sciences 1 (2013/2014) 101–108
DOI 10.3233/JGS-130014
IOS Press

Discussion Paper

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Bengt H. Fellenius*
Rothesay Ave, Sidney, BC, Canada

The authors report the results of the static loading test on an extensometer-instrumented, 460 mm diameter, bored pile installed to 21.56 m depth (an Omega screw pile) through a 15 m thick old fill consisting of sand with clay deposited on about 4 m of silt and clay and 5 m of sand, followed by clay at 24 m depth. The authors applied a CPT-sounding to predict the shaft and toe resistances of the pile. The case history is interesting and worth additional attention.

Figure 1 presents the CPT cone-stress diagram together with the authors' CPT-calculated shaft resistance distribution and the shaft resistance distributions calculated using four additional methods. The figure is supplemented with the distribution of shaft resistance measured in the static loading test at the maximum applied load and the interpreted soil profile. The CPT methods suggest that the ultimate shaft resistance of the test pile should be about 1,500 kN through about 2,000 kN.

The four additional methods used for the calculation of the pile resistance are the "Dutch" [2], the "LCPC" [1], the "Schmertmann" [12], and the "E-F" [3], Eslami and Fellenius 1995; 1996], summarized in Fellenius [6]. The calculations are by means of the UniPile program (Goudreault and Fellenius 2012).

Figure 2 shows the toe resistances calculated using the same methods as determined at the maximum load applied to the pile head in the static loading test. Obviously, the CPT methods have all vastly overestimated the maximum toe resistance developed in the static loading test. The pile toe is 2.5 m, about 5 pile diameters, above the clay layer, which ordinarily should be enough to prevent a softening of the toe response due to the presence of the clay. Therefore, a question is if the depth to the clay layer varies appreciable across the site, the distance to the clay layer might be smaller at the test pile location. However, the authors' Fig. 14 indicates that the variation of the depth to

*Corresponding author: Bengt H. Fellenius, Dr.Tech., P.Eng., 2475 Rothesay Ave, Sidney, BC V8L 2B9, Canada. E-mail: Bengt@Fellenius.net.

ISSN 2213-2880/13/14/$27.50

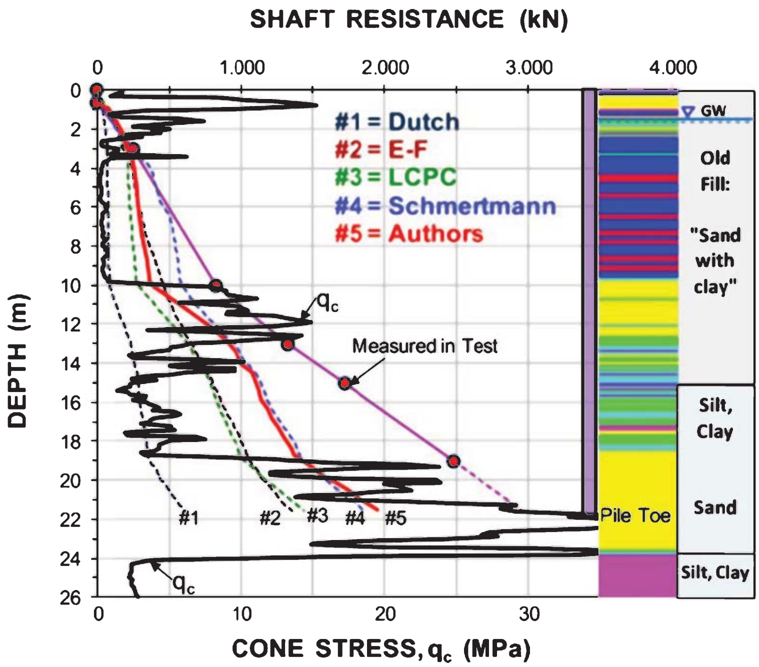

Fig. 1. Pile ultimate shaft resistance — Measured and as calculated by 5 CPT methods. And soil profile with CPR qc-diagram.

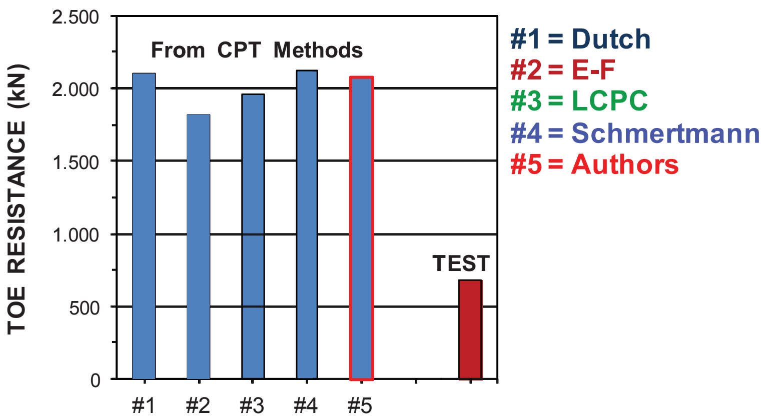

Fig. 2. Pile toe resistance — Maximum in test and "ultimate" as calculated by five CPT methods.

the clay is minimal at the site. They also point out that the imposition of the pile reinforcing and the instrumentation cage might have resulted in a less than perfect concreting of the pile toe.

Figure 3 shows the load-movement curves measured in the static loading test for the pile head, the pile shaft, the pile toe, and the pile compression. The pile toe curve is plotted using the toe load extrapolated from the extensometer measurements. (The authors instead plotted the pile-head load versus the pile-toe movement.)

The measured shaft resistance load-movement curve does not indicate a clear value of ultimate shaft resistance. It can be estimated to range between 2,750 kN at a pile head movement of 40 mm through 2,930 kN at about 70 mm movement. Thus, the evaluation of the extensometer records indicates that the ultimate shaft resistance determined from the static test is about one-and-a-half times larger than that calculated by the CPT methods.

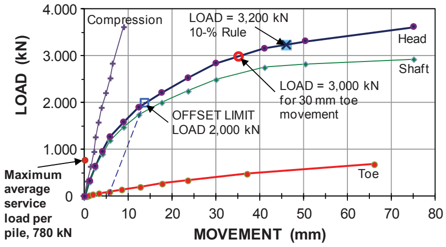

Fig. 3. Load movement curves for head, shaft, toe, and compression.

It is important to notice that the pile toe load-movement response is by a gently rising convex curve that does not have the slightest tendency toward a value one could reasonably define as the ultimate toe resistance. This is no surprise, it is what all properly instrumented and analyzed tests will show. Simply, ultimate toe resistance does not exist [4]. It is bewildering that the concept of ultimate toe resistance and a plastic response of the pile toe is still almost universally believed to exist—and applied to piled foundation design.

The movement of the pile head for the applied load is a sum of the axial shortening of the pile, which is a linear response to the load, the movement due to the mobilization of the shaft resistance (which often does trends to a recognizable ultimate resistance, albeit far from always a plastic response), and the pile toe response, which is a simple non-linear stiffness-governed response. No wonder then that pile capacity, thought to be the "ultimate load" or "failure load" rarely appears. The profession, therefore, has developed many different definitions in lieu of a clear indication. Figure 3 includes the Davison Offset Limit, which is in widespread use in North-America, and the load for a movement equal to 10-% of the pile diameter, mentioned by the authors. Personally, I find the pile head load that generated a 30 mm pile toe movement useful in the context. The authors applied "the 10-% definition", which has its origin in a mistaken quotation of a now 70 years old statement by Terzaghi (1942). Terzaghi wrote: "*the failure load is not reached unless the penetration of the pile is at least equal to 10% of the diameter at the tip* (toe) *of the pile*". For full quotation and context, see Likins et al. [11]. Note, Terzaghi did not define the capacity as the load generating a movement equal to 10 percent of the pile head diameter, he emphatically stated that whatever definition of capacity or ultimate resistance used, it must not be applied until the pile toe has moved at least a distance corresponding to 10 percent of the pile toe diameter. (The pile head will then have moved an additional distance equal to the pile compression.)

The authors suggest that the disagreement between shaft resistance calculated using the CPT-method and that found in the evaluation of the extensometer records of the test could be due to the presence of locked-in loads in the test pile, so-called residual load. They provide a load distribution corrected for residual load, which distribution is approximately equal to that produced by their CPT-method. The distribution, labeled AUTHORS' "True" Distribution, is shown in Fig. 4 together with a more direct construction of the residual load per the principles published by Fellenius [5, 6]. The figure also includes the CPT cone stress superimposed on a column listing ß-coefficients. The latter are the beta effective stress beta-coefficients back-calculated from, first, the measured load distribution (called "False"), and, then, the distribution corrected for residual load (called "True"). The three curves—the "Residual", the "False" and the "True"—are linked, that is, any two of them determine the third. The three distributions indicate that the residual load resulted in fully mobilized negative skin friction down to about 17 m depth. Hereunder, a transition occurred toward partially mobilized positive shaft resistance.

It is not possible to tell from the records that residual load was actually present in the test pile at the start of the static loading test. It could be present and it could be as large as indicated by the authors' "True" distribution. There is no definite indication either way. However, the ß-coefficients back-calculated from the test records (the "False" distribution) are somewhat high overall and it is not logical that a ß-coefficient of 0.80 would represent the shaft shear in the soft zone between the depths of 3 and 10 m. Neither is it logical that a ß of 0.65 would represent the shaft shear in the very dense sand between the 10 through 22 m depth range. The coefficients back-calculated from the "True" distribution are, in contrast, logical in relation to each other and reasonable for the soil types.

The real "true" distribution probably lies somewhere in between the suggested "True" distribution shown in Fig. 4 and the distribution evaluated from the extensometer records. It is certainly disappointing that after all the effort

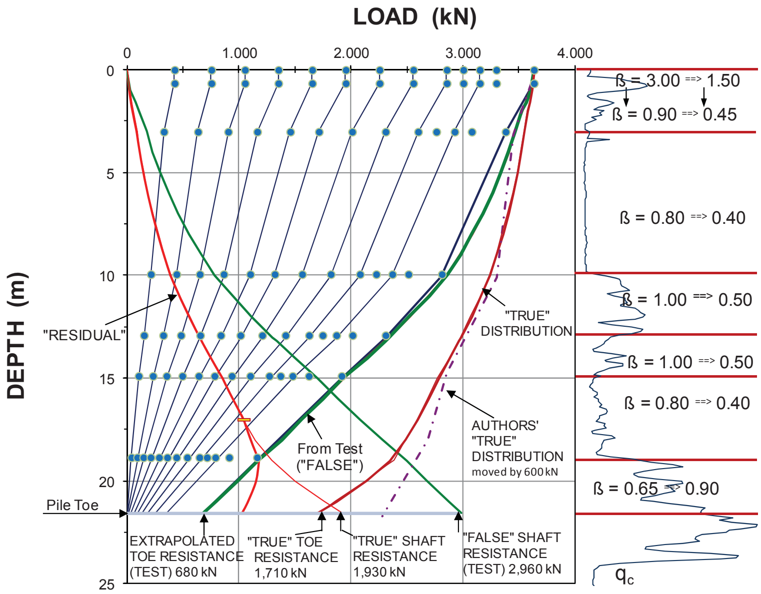

Fig. 4. Load and resistance distributions as measured and as adjusted for residual load with distribution of back-calculated beta-coefficients.

invested in the instrumentation, to find that the test results are unable to be more definite about the results. For example, the possibility of using shorter piles cannot be determined. Still, by simply performing an uplift test after the compression test, which would have been a relatively easy and low-cost addition to the testing programme, the issue would have been resolved. Not only would combining push and pull tests have provided the true ultimate shaft resistance (because the toe resistance would not be involved in the test), the residual load (which would have acted also on the uplift test) would have "worked the other way" and the analysis of the records from the two instrumented tests together would have determined the amount of residual load and its distribution.

The hydrotesting of Tank 1 during six days of building up to a load 18.0 m height of water in the tank (full service load, 780 kN/pile) showed linear increase of deformation to a mean maximum of 20.5 mm and a range from lowest to highest of ±1 mm. After one day at full load, the tank was emptied over a span of about 3 days. The net mean deformation measured along the tank perimeter was then 4 mm. Judging from the observed pile shortening and transfer movements during the static loading test, I estimate that the mean gross settlement below the pile toe depth was about 17 mm. As there was hardly any time for consolidation to develop, this settlement consisted of "immediate" settlement in the 2.5 m thick layer of sand below the pile toe and in the clay below 24 m depth. The unloading rebound would also have been affected, albeit marginally, by the simultaneous loading of Tank 2 (the water in Tank 1 appears to have been pumped over to Tank 2). Because the sand and clay layers are overconsolidated well beyond the applied stress, the unloading moduli will be about the same as the loading moduli. The piles prevented a full rebound in the soil above pile toe depth.

The observations allow for an approximate back-calculation of the immediate compressibility parameters of the sand and the clay below the pile toe depth. The calculation was made for a circular, 48-m diameter, equivalent raft placed at the pile toe. The stress on the raft (180 kPa) was distributed by Boussinesq to the full 100 m depth. The software used was UniSettle (Goudreault and Fellenius 2011). The so-estimated moduli in the sand and clay were taken to be about equal and the back-calculation gave a value of 200 MPa for the immediate compression modulus, which is approximately similar to the values indicated in the authors' paper (Table 1 and Fig. 3b) for the zone nearest the pile toe level.

The authors mention that the interaction of unloading Tank 1 when loading Tank 2 affected Tank 2 resulted in a "tilt" of Tank 2 away from Tank 1. The term "tilt" for the differences in the observed deformation for Tank 2 is misleading, however. The tanks did not tilt. The reason for the difference in observed records is that no observations were taken of the Tank 2 perimeter benchmarks during the hydrotesting of Tank 1 (for reasons of

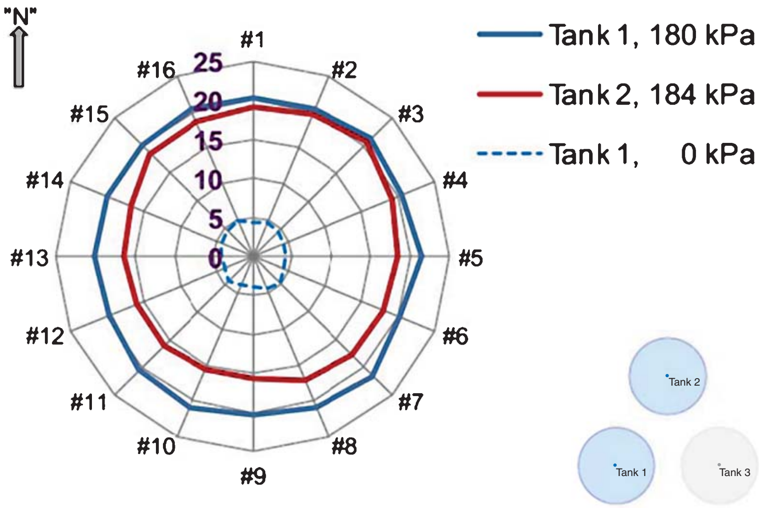

Fig. 5. Observed deformations for Tanks 1 and 2 at maximum load and Tank 1 after unloading. The North direction is assumed vertical.

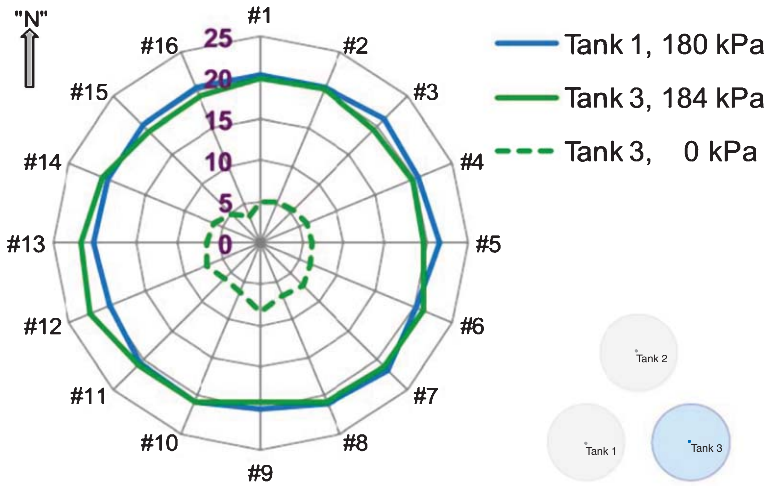

Fig. 6. Observed deformations for Tanks 1 and 3 at maximum load and Tank 3 after unloading. The North direction is assumed vertical.

ongoing construction, I understand). This is clear in Fig. 5, showing a radial diagram for the maximum measured deformations along the Tanks 1 and 2 perimeters.

The measurements show that the deformations along the north-east portion of the tanks are very similar. However, in the south-west portion, the deformations on Tank 2 are affected from the outset by the fact that the stress induced from Tank 1 underneath Tank 2 was being reduced due to the fact that Tank 1 was simultaneously emptied of water. Similarly, the 0 kPa stress indicated for Tank 1 does not mean that the soil below the pile toe level is unstressed by tank load; it is stressed by the load applied to Tank 2 spreading over to the area below Tank 1. The dashed curve in the center of the diagram shows the net measured deformation for Tank 1 indicating a slight "tilt" toward Tank 2 because of the stress spreading effect. After the unloading of Tank 2 with now both tanks at zero load, had the true zero conditions then been recorded, the measurements would have shown no such "tilt".

Tank 2 had been emptied before Tank 3 was hydrotested. Therefore, any influence of the stress from the hydrotesting of the adjacent tanks was gone before Tank 3 was hydrotested. Consequently, as indicated in Fig. 6, the

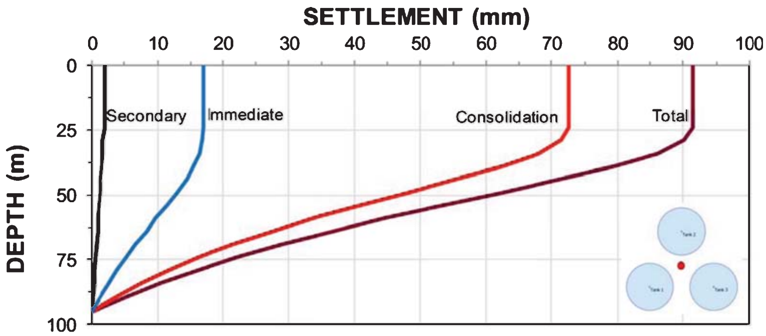

Fig. 7. Long-term settlement: immediate compression, residual compression, consolidation, settlement, and total settlement for a point at the center of the site.

observed deformations for Tank 3 do not show any tendency of mutual interaction similar to the at shown in Fig. 5.

The authors estimated the consolidation settlement to be about 100 mm in the clay layer between 24 and 100 m depths for the fully loaded tanks in service. I assume they consider this to be as an average value. They did not provide the compressibility parameters for the soil that resulted in the estimated value. Of course, in the absence of proper soil parameters for the clay, one cannot do much more than produce an estimate applying best judgment. However, the best judgment estimate should be directed toward the soil parameters, not the final settlement. Once a best judgment estimate is made for the soil compressibility, the rest is simple calculations applying known soil principles and usually acceptable stress distributions. The clay is overconsolidated and I believe a reasonably conservative estimate of the re-loading compressibility is a Janbu modulus number of 80 and a stress exponent, j, of 0. (This correlates to a conventional re-loading C_{cr}-value of 0.053 for the clay, which the authors indicate to be 1,900 kg/m^3 and, therefore, the void ratio, e_0, can be determined to 0.85). As the loading lies well within the preconsolidation range, the consolidation can be expected to be rapid. I have assumed that a 90-% degree of consolidation will be attained in 3 years.

As indicated by Fellenius and Ochoa [8], a piled foundation raft is flexible (valid even for rafts much thicker than the 600 mm slab of the subject case). Consequently, the settlement distribution due to a uniform stress applied to the raft is best calculated using Boussinesq stress distribution. The stress and settlement calculations were performed using UniSettle for an equivalent raft placed at the pile toe level with due consideration of the location of the neutral plane. As the c/c spacing for the 422 piles is about 5 pile diameters and assuming that the load distribution shown by the analysis for residual load acting on the test pile shows the correct load distribution for the pile, the neutral plane for the subject tank pile groups lies close to the pile toe level. The applied stress (180 kPa, 780 kN/pile) acts uniformly over the 48 m circular equivalent raft assumed placed at the same depth as the test pile, 21.5 m.

Figure 7 shows the long-term settlements versus depth below the center of the site: the immediate compression, residual compression, consolidation, settlement, and total settlement, as based on the mentioned soil compressibility (modulus number) and site specifics.

Figure 8 shows the development over time of the settlements for two points: the Site Center and Point #5 on the perimeter of Tank 1. Based on the assumed time for attaining 90-% degree of consolidation, only a marginal amount of settlement will develop after about 5 years. Of course, filling and emptying the tanks will result in 'elastic" compression and rebound, more or less immediate, of the piles and the soil.

The calculations provide two important results: first, in the long-term the tanks will experience a small tilt due to the interaction of the loading, and second, the center of the tanks will settle considerably more than the perimeter. Figure 9 shows the calculated settlements for a diameter west to east of Tank 1 indicating that the tilt will be about 30 mm across the 65 m distance, i.e., about 1/20,000, which is close to the authors estimated value of 1/12,000. The settlement difference between the center of the tank and the perimeter will be about 100 mm, which might be of some concern for the tank base, should the predicted deformation actually develop.

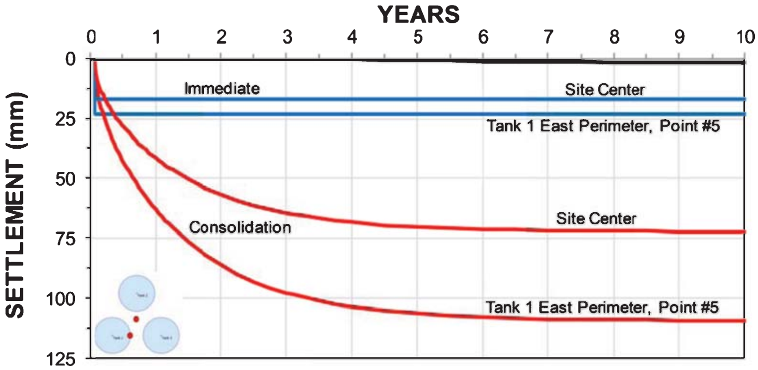

Fig. 8. Settlement versus time (years) as calculated for the Site Center and Tank 1 Point #5.

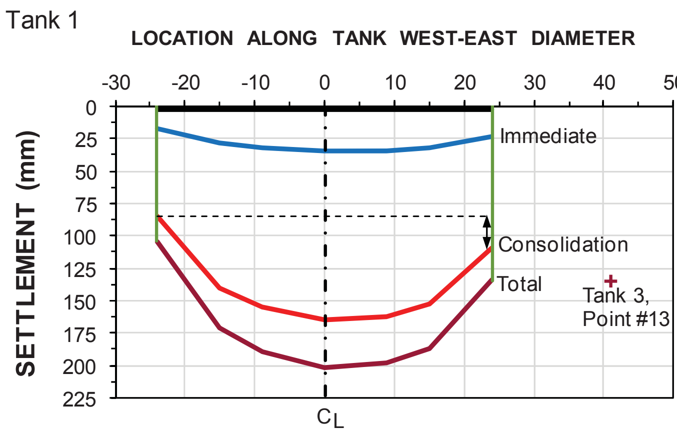

Fig. 9. Settlement distributions below Tank 1 center line.

The estimated—predicted—long-term deformations depend primarily on the relevance of the assumed compressibility of the clay, the modulus number of 80. I intended for this to be a realistic value, but one on the cautious side. The assumption of flexible foundation is conservative for the settlement difference between the tank center and perimeter; but not overly so. Regardless of the actual magnitude of the settlements, I expect that the center of the tanks will settle about twice as much as the perimeter.

It would be of considerable interest to the profession if the long-term settlement of the tanks would be monitored for the next several years along with the use of the tanks.

I note the authors' mention that a drag force due to long-term settlement of the soils in the upper 15 m of the soil profile of 180 kN was expected and that this value was added to the 780 kN service load for the piles. First, the drag force is probably going to be close 1,000 kN, not 180 kN, and, second, the drag force is only of concern for the pile structural strength, which is more than adequate for the total axial load of close to 2,000 kN. The Eurocode advocating that the drag force be considered a load similar to the service load is quite in error and very wasteful. For example, several codes and standards recommend the drag force not be lumped in with the service load, notable the Canadian, Australian, and several US codes and standards [7].

Moreover, I realize that the design was probably finalized before the pile loading test was performed. However, the test results—capacity was about 3,000 kN—suggest that a much larger service load than 780 kN could have been assigned to the foundation piles. This would have resulted in fewer piles for the project. If the pile lengths had been the same, having fewer piles would not have changed the amount of foundation settlement. Thus, in conclusion, significant costs and time savings could have been realized for the project had the results of the static loading test been available before the design had been finalized.

Acknowledgments

After my contacting the authors for clarifying information, I received an immediate reply and copies of all test data and original files that the authors used in preparing their paper. I very much appreciate this show of professionalism and support.

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