Design, Installation and Verification of Driven Piles in Soft Ground

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ABSTRACT: Soft compressible marine clay is a common shallow deposit along the western coastal shorelines of Malaysia, which is within the belt of active economy development. As such, there are many structures and infrastructures constructed over this deposit to sustain the economy growth in view of convenience. In this paper, the processes of design, installation and verification of the floating piles of varying lengths as the foundation system for a factory structure will be presented. There are total four test piles with one fully instrumented to reveal the performance of the foundation pile design. High strain dynamic pile tests (HSDPT) and conventional static load tests (SLT) on these test piles have been planned and implemented to establish good correlation for quality control of foundation construction. The test pile results have been generally shown a very satisfactory performance. In this case study, comparisons have been carried out between the HSDPT, SLT and load transfer behaviour from the instrumentation results. Conventional theory on pile capacity and pile deflection have also been compared and verified with the test results.

1 Introduction

1.1 General

This paper presents an alternative foundation design using fixed length floating pile system for a plastic drum factory constructed on the reclaimed land underlain by soft marine clay as part of the value engineering exercise. The factory primarily consists of the four major structures, namely Production Factory, Raw Material Store, Finished Goods Store with 3-storey Office Block, Sprinkler Water Tank and Pump House. Figure 1 shows the layout plan of the proposed development with the locations of boreholes and piezocones.

1.2 Site geology and conditions

Based on the Geological Map of Port Klang & Klang (sheet 93 Selangor 1976) published by the Geological Survey Malaysia, the site is underlain by alluvium generally consisting of quaternary deposits of marine clay, silty sand and clayey sand. The general geological condition of the site is shown in Figure 2. The bedrock at the site is very deep, which is likely to be the weathered metasedimentary formation locally known as Kenny Hill formation. As to be discussed in Section 1.3, three boreholes carried out at the site have not encountered rock even though the depths of the boreholes have reached 45m to 60m below the ground level. The site is located on a levelled reclaimed land of approximate 3m to 5m thick hydraulic sand fill.

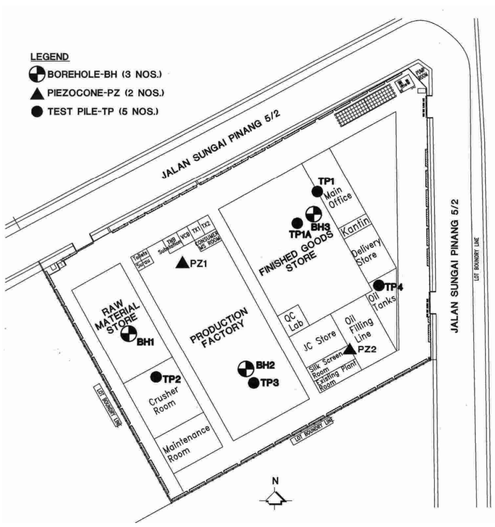

Figure 1. Project site layout plan.

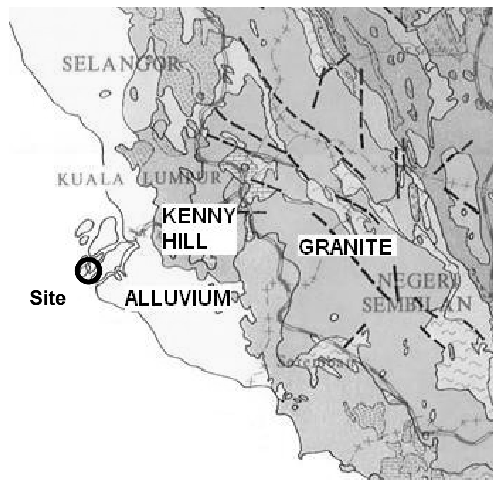

Figure 2. Project site geology.

1.3 Subsurface investigation

Subsurface investigation (SI) works of three boreholes and two piezocones have been planned and implemented for the foundation design. All boreholes have been carried out using rotary wash boring method. The borelog profile showing SPT 'N' values and major/minor components of soil classification are presented in Figure 3.

Two piezocones have revealed the continuous subsurface stratigraphy and obtained relevant engineering parameters of the subsoil. The interpreted subsoil profile from the piezocones as shown in Figure 4 has indicated that the upper marine clay unit is primarily interbedded layers of thin sand layers and 2m to 3m thick clay layers, whereas the lower marine clay unit is predominantly clay. These findings are clearly evidenced in the pore pressure measurement. The piezocones have also detected the sandy material between the upper and lower marine clay units with relatively high cone resistance and hydro-static pore pressure profile. It can be shown in the later section that the interbedded sand layers in the upper clay unit provide efficient drainage to dissipate the pore pressure induced as a result of reclamation and therefore accelerate the rate of strength gain as reflected in the test pile performance.

From the piezocone pore pressure profile, the hydrostatic water profile indicates the groundwater level is at 1.5m below ground. Vane shear tests have also been carried out at specified depths in the boreholes to establish the peak and remoulded undrained shear strength profile of the subsoils. The interpreted undrained strength profile from piezocone results matches very well with undrained shear strength profile from the vane shear tests results if the cone factor, N_{kt} of 14 is used. The sensitivity of the upper marine clay unit ranges from 2 to 3, whereas that of the lower marine clay unit ranges from 1.8 to 2.

Together with the laboratory test results, the geotechnical model established for the design is shown in Figure 5.

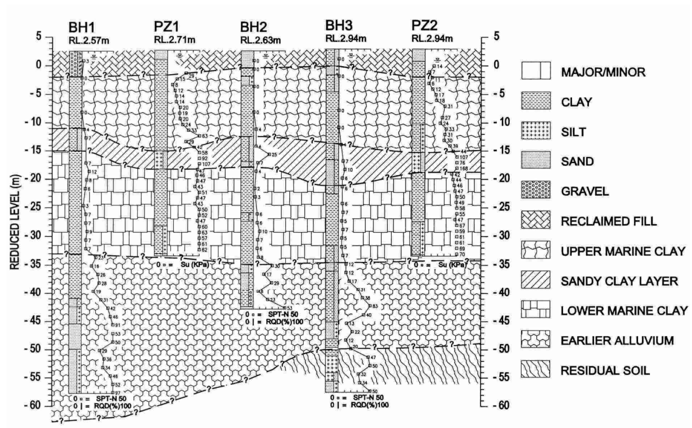

Figure 3. Subsurface profile of project site.

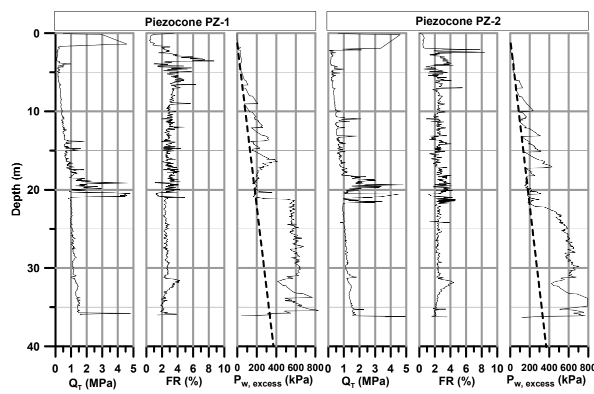

Figure 4. Piezocone results.

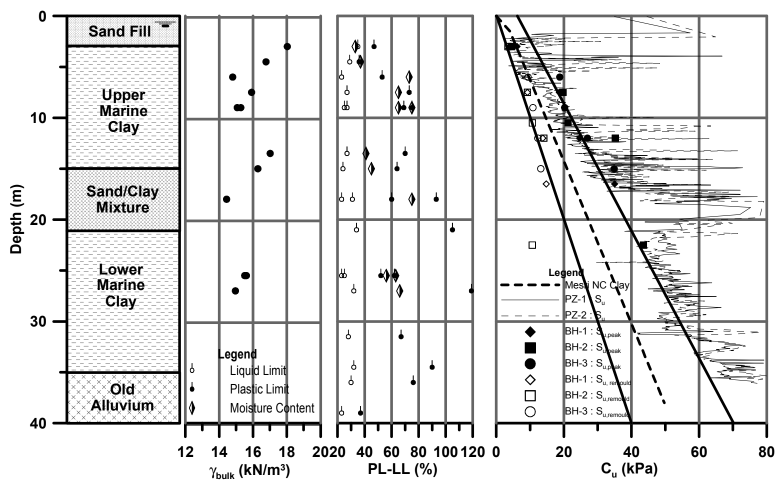

Figure 5. Interpreted subsurface condition.

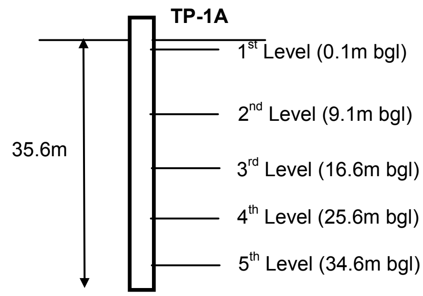

Figure 6. Location of strain gauges at TP-1A.

2 Pile Construction

2.1 Pile installation

The circular prestressed concrete piles of 250mm diameter and 55mm wall thickness with 80MPa concrete strength have been adopted as the foundation pile system for the entire project. Varying pile lengths have been designed to provide different working loads with no pile head hacking as all the piles have been installed with pile head reaching within certain tolerance of the designated pile cut off level.

2.2 Pile testing

2.2.1 Static maintained load tests

The four static maintained load tests, namely, TP-1A, TP-2, TP-3 and TP-4 have been subjected to test load to failure, except TP-3. The tabulated load test results is summarised in Table 1 and the graphical plotting of the individual test pile result has been shown in Figures 7 to 10.

From test pile, TP-3, it is interesting to note that the rebounce of pile head after first cycle unloading shows pile heave, whereas test pile, TP-2, shows the rebounce after unloading from second load cycle. This could well be a good indication of residual compressive stress residing in the pile body. This phenomenon can be explained by the following situations and usually is more prominent in long pile due to more temporary pile compression and larger soil disturbance in pile penetration:

a. lock-in temporary pile compression during pile driving installation or temporary loading on pile
b. dissipation of excess pore pressure surrounding the pile body as result of penetration of the test pile and also the adjacent piles

Such stress in the pile before any imposed load is applied on the pile head is commonly known as residual stress. This phenomenon is further illustrated in the interpretation of instrumented test pile results.

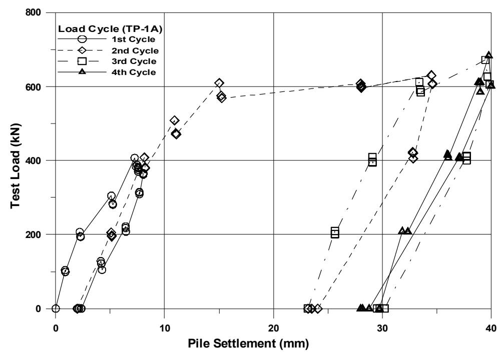

Figure 7 Instrumented test pile (TP-1A)

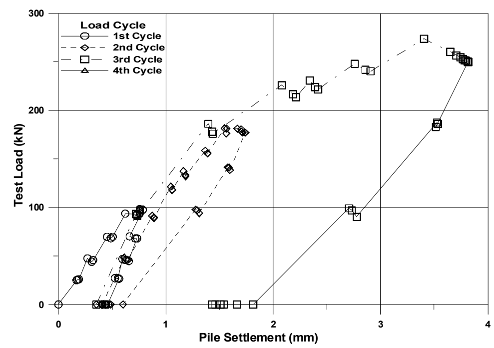

Figure 8 Working test pile (TP-2)

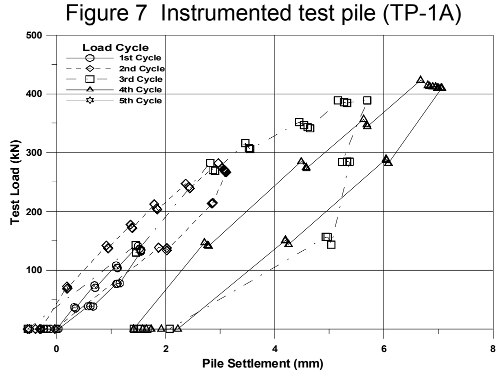

Figure 9 Working test pile (TP-3)

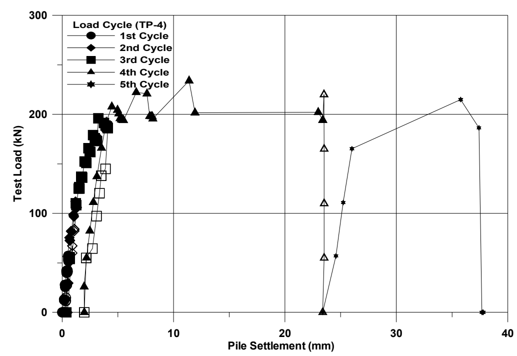

Figure 10 Working test pile (TP-4)

Table 1 summarises the details, pile deflection and maximum mobilised pile capacity in the pile testing. Most test piles have reached ultimate resistance with excessive pile head deflection except test pile TP-3, in which the test load is believed to have reached plastic failure of the soil. Comparisons between the predicted ultimate pile capacity and the mobilised pile capacity are carried out. The general trend of the test results suggests that the mobilised skin resistance along the pile shaft seems to approach a constant value of 22kPa. This could be explained by the possibilities:

a. Existence of residual stress in the pile, which was previously misunderstood as limiting shaft resistance below the “critical depth” (Fellenius & Altaee, 1995)

b. The sand layers in the upper clay unit encourage faster dissipation of excess pore pressure and therefore leading to better and faster gain in strength when comparing to the lower clay unit. This can be further verified in the instrumented test pile results.

Table 1. Summary of test pile results

TP	Pile Length (m)	Design Load (kN)	Date of Installation	Date of Testing	Day	$P_{ult, est}$ (kN)	$P_{ult, test}$ (kN)	Type of Test	δ_c (mm)
1	33.0	405	26-Mar-2004	26-Mar-2004	0	440*	137	EOD	
				2-Apr-2004	8	817#	910	RS	14.2
1A	35.5	405	2-Apr-2004	27-Apr-2004	26	510*	**600**	SLT	15
				2-Apr-2004	0	900#	138	EOD	5.7
				30-Apr-2004	29		956	RS	15
2	14.5	90	26-Mar-2004	17-Apr-2004	23	145*	**250**	SLT	2.5
				20-Apr-2004	26	304#	340	RS	4.5
3	23.5	140	26-Mar-2004	13-Apr-2004	19	270*	**400**	SLT	5
				16-Apr-2004	22	584#	593	RS	8
4	11.5	55	20-Apr-2004	7-May-2004	18	127*	**200**	SLT	3
				30-Apr-2004	11	230#	320	RS	6

Note: * LCPC method; # Eslami & Fellenius method (1997)

2.2.2 Instrumentation results

The instrumented test pile TP-1A was equipped with five levels of strain gauges, as shown in Figure 6. Two strain gauges were provided at each level. These instruments were installed after pile installation. The third level strain gauges were found to be faulty after installation, and therefore only the results of four levels were available for interpretation.

The strain readings did not change in all levels of strain gauges when the pile was loaded up to 100kN. This is likely due to relieve of prestress in the pile while the compressive load was transferred throughout the pile, and therefore there was no significant net change in strain within the pile until sufficient relaxation of prestress.

Residual load was believed to exist in this test pile as a result of “lock-in stress” developed between the pile-soil interface during driving and dissipation of excess pore water pressure. This can be evidenced in the unusual portions of the instrumentation results, as plotted in Figure 11. For instance, the shaft resistance at upper 9m seems to be insignificant based on the measured load distribution. But, the mobilised shaft resistance should be larger than the measured value. Moreover, the load distribution is steeper at depth below 26m, which contradicts with the increasing soil strength profile with increasing depth.

As the load distributions interpreted from the first and second load cycles were not consistent with insignificant strain change during initial loading stage and negative shaft resistance, the instrumentation results for the third load cycle were used for determining the residual load in this paper.

The residual load in the instrumented test pile was determined using method recommended by Fellenius (2002). Due to limited levels of strain gauges in the test pile, matching of theoretical distribution in effective stress analysis with “half of the measured load reduction” curves was approximated with intersection between these two curves, which is at depth of about 10.6m below ground level. The effective friction angle (ϕ’) of 20°, which is typical range for marine clay in Klang, was assumed in the effective stress analysis, as effective stress strength test was not carried out at this site.

The interpreted residual and true load distributions are shown in Figure 12. The load in the pile increased to depth of about 9m. This phenomenon is similar to “dragload”, in which negative shaft resistance was induced at the upper part of the pile. Lock-in load due to pile rebounce after unloading at the previous load cycle contributed to negative shaft resistance. Damage of the third level strain gauges affected determination of true load distribution profile, as there was no strain gauge data for depth between 9m and 26m. Consequently, depth of interface between negative and positive shaft resistances could not be determined accurately, and it is reasonably assumed to be at depth of 9m where strain gauge readings were available at this depth. The average positive shaft resistance is about 32kPa based on true load distribution, and it is higher than the shaft resistance of 22kPa as computed based on measured load distribution.

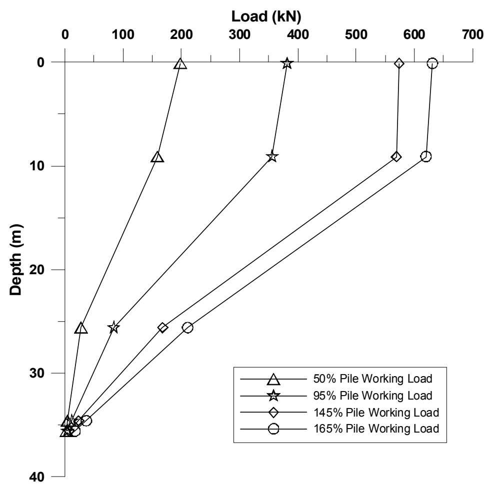

Figure 11. Measured load distribution (TP-1A).

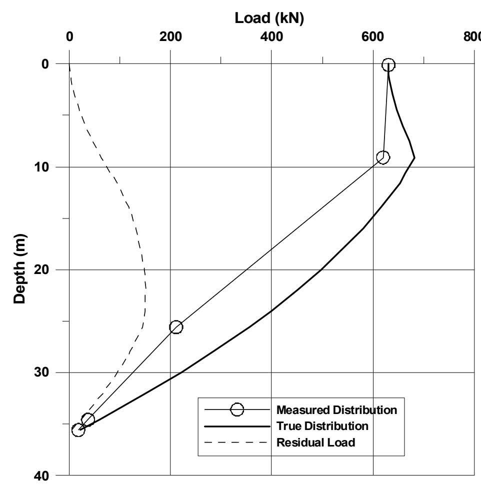

Figure 12. Residual and true load distributions.

2.2.3 High strain dynamic pile tests

Figures 13 and 14 show the results of driving monitoring for test piles TP-1 and TP-1A.

As shown in Table 1, the high strain dynamic pile tests (HSDPT) during the end of drive (EOD) produced much lower ultimate pile capacity for test piles TP-1 and TP-1A, which indicated significantly reduced shear strength of highly sensitive clay due to disturbance during pile driving.

Restrike tests (RS) using (HSDPT) overestimate the ultimate pile capacity by 36% to 60%. Therefore, the results of restrike tests should be used in the pile design with caution.

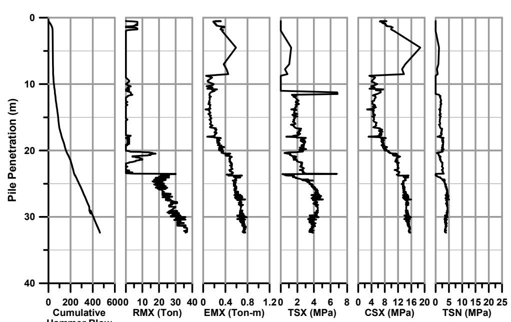

Figure 13. Driving monitoring results (TP-1).

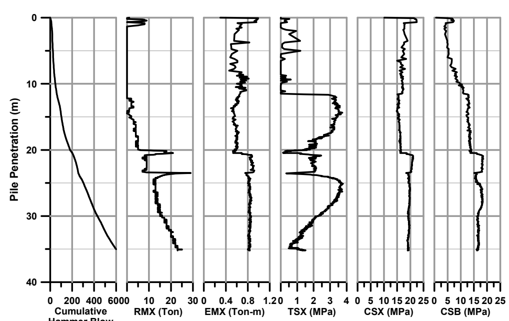

Figure 14. Driving monitoring results (TP-1A).

3 Comparison with Conventional Theory

3.1 Pile capacity

The estimated ultimate pile capacity is also computed in Table 1 using two direct methods based on piezocone results, which are LCPC method and method recommended by Eslami & Fellenius (1997). The estimated ultimate pile capacity ($P_{ult,\ est}$) as shown in Table 1 is taken as the average value computed from piezocones PZ1 and PZ2.

The estimated ultimate pile capacity interpreted from LCPC method is generally much lower than the value computed using Eslami & Fellenius method. The estimated ultimate pile capacity computed using Eslami & Fellenius method is closer to the pile capacity established from RS test, but is higher than the ultimate pile capacity. LCPC method produces value closer to the ultimate pile capacity even though this method is conservative. Hence, LCPC method may be more suitable to be used in the pile design at soft marine clay deposit.

As shown in Figure 15, development of driven pile capacity with time is scattered. Scattered data could be due to influence by adjacent pile driving operation at the site during pile tests. Generally significant increase in pile capacity can be observed one week after pile installation.

3.2 Pile deflection

The pile deflection is estimated based on conventional theory of elasticity. E_{50}/s_u ratio is about 700 to 800 to match the results of the third load cycle (Figure 16). This ratio is higher than E_{50}/s_u of 400 for clay with average plasticity index of 40 (Duncan & Buchignani, 1976) at this site. This can be explained by existence of residual load in the pile that caused stiffer pile response. Further research with more case studies can be carried out to establish the influence of residual load to the pile deflection.

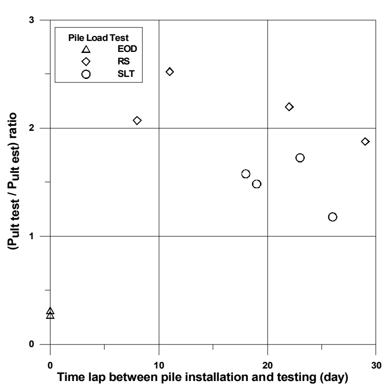

Figure 15. Development of pile capacity.

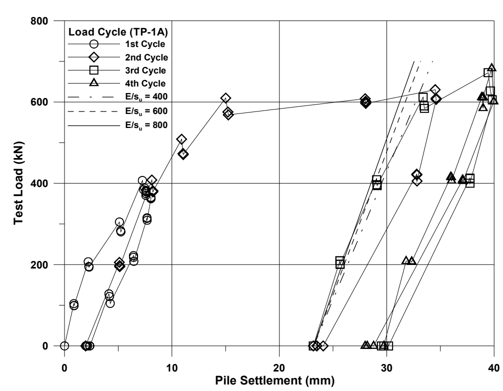

Figure 16. Correlation with elasticity theory.

4 Conclusions

Case study on the design and construction of driven piles in soft ground has been presented. From the pile test results, it is found that the shaft resistance at the upper portion of the test piles is better than expected while low shaft resistance at the lower portion of the piles. Shorter pile also performed better than longer pile. These observations did not tally with increasing soil strength profile with depth. This phenomenon is likely due to fast dissipation of excess pore water pressure

at the upper marine clay layer with sand lenses at every 2 to 3m. Installation of large displacement piles also improves the soil effective stresses to three to five times the initial undrained shear strength after consolidation around the pile (Randolph & Wroth, 1982).
Existence of residual load due to pile driving, pile loading and dissipation of excess pore water pressure is also demonstrated in the interpretation of the instrumented test pile results. Residual load significantly affects interpretation of the pile load test results. With consideration of residual load, interpretation of the pile load test results reveals true load transfer behaviour, and unusual instrumentation test results can be explained. However, sufficient quantity of instrumentation data is required to produce more accurate residual and true load distributions in the instrumented test pile.
The results of restrike test and end of drive test using HSDPT should be used with correlation with static maintained load test, as it is possible that the ultimate pile capacity interpreted from the restrike test or end of drive is different significantly from the actual pile capacity.
Prediction of ultimate pile capacity using piezocone direct methods does not necessarily produce safe design value. In this case, LCPC method is more appropriate to be used, as the computed pile capacity is close to or lower than the ultimate pile capacity established from static maintained load test. The load-settlement behaviour of the test pile shows stiffer pile response compared to typical range recommended in literatures using conventional elasticity theory. Hence, further study with more case studies is required to establish appropriate correlations between the pile tests and conventional theory on pile capacity and deflection in marine clay in Malaysia.

5 References

Bustamante M., Gianeselli L. 1982. *Pile bearing capacity predictions by means of static penetrometer CPT.* Proc. of the 2nd European Symposium on Penetration Testing. Amsterdam, Vol. 2, 493 – 500.

Duncan J. M., Buchignani A. L. 1976. *An engineering manual for settlement studies.* Geotech. Engrg. Rep., Dept. of Civ. Engrg., Univ. of California, Berkeley.

Eslami A., Fellenius B. H. 1997. *Pile capacity by direct CPT and CPTu methods applied to 102 case histories.* Canadian Geotechnical Journal, 34, 886 – 904.

Fellenius B. H., 2002. *Determining the resistance distribution in piles. Part 1: Notes on shift of no-load reading and residual load.* Geotechnical News Magazine, Vol. 20, No. 2, 35 - 38.

Fellenius B. H., 2002. *Determining the resistance distribution in piles. Part 2: Method for determining the residual load.* Geotechnical News Magazine, Vol. 20, No. 3, 25 - 29.

Fellenius B. H., 2002. *Determining the true distribution of load in piles.* American Society of Civil Engineers, ASCE, International Deep Foundation Congress, An International Perspective on Theory, Design, Construction, and Performance, Geotechnical Special Publication No. 116, Edited by M.W. O'Neill, and F.C. Townsend, Orlando, Florida, February 14 - 16, 2002, Vol. 2, 1455 – 1470.

Fellenius B. H., Altaee A. A. 1995. *Critical depth: how it came into being and why it does not exist.* Proc. Instn Civ. Engrs, Geotech. Engng, 111, Apr 1995, 107-111.

Randolph M. F., Wroth C. P. 1982. *Recent development in understanding the axial capacity of piles in clay.* Ground Engineering, October 1982, 17 – 32.