Discussion of "Observed Performance of Long Steel H-Piles Jacked into Sandy Soils" by J. Yang, L. G. Tham, P. K. K. Lee, and F. Yu

January 2006, Vol. 132, No. 1, pp. 24–35.
DOI: 10.1061/(ASCE)1090-0241(2006)132:1(24)

Bengt H. Fellenius[1]

[1]11905 Alexander St. SE, Calgary, Alberta, Canada T2G 4J3. E-mail: Bengt@Fellenius.net

The authors have presented a factual compilation of a very interesting project and are commended for having taken on this effort. As for all good case histories, the authors' paper will undoubtedly serve several researchers well when searching data of specific interest for researches' different perspectives. However, the paper lacks some of the necessary background information and it would be helpful if the authors could clarify the following points.

1. The main soil body at the Hong Kong site is a saprolite formed from weathering of granite. Such soils are usually not saturated. However, the authors indicate groundwater tables located at depths of 2.8 and 3.7 m at the sites of Piles J1 and J2, respectively. Do the tables represent perched water tables in the nonweathered surficial soil with the deep soils being nonsaturated or is the entire soil profile saturated below this water table? If so, are the pore water pressures hydrostatically distributed?
2. The description of the saprolite is very brief; however, Saprolite is a rather unusual soil, outside Hong Kong, that is. Yet, the authors make comparison references to papers reporting results from pile test in other soil types of similar grain size but having different genesis and mineralogy. Would the authors be able to expand on the particulars of the saprolite? Perhaps add the results of a CPTU sounding from the vicinity of the site?
3. In Fig. 14, the authors present the shaft resistance along three lengths of piles as a function of shaft movement. Were telltales used to measure movement attached to the pile or are these movements determined from integration of the strain measurements?
4. Figs. 7, 12, and 15–17 show the distributions of stress determined from the strain measurements. But, while the pile size and weight are presented, the added areas from the steel angles and, potentially, telltale guide pipes are not presented, which makes the accuracy of conversion from the reported stress to load somewhat imprecise.
5. While the distance between strain-guage pairs and the pile toe depths are mentioned, the total length of the pile and the pile length above ground (the "stick-up") are not. It would be good if the authors could provide this information.
6. The static loading tests on Piles J1 and J2 were performed 4 days and 2 days, respectively, after the piles were installed. This would seem to be early and before full setup would have occurred. However, the 34-day repeat test on Pile J2 implies very little change from the early test. It would be interesting if the authors could add the load-movement curve of the repeat test to their Fig. 11.
7. The most needed clarification is the strain measurements taken immediately before and after the completion of the static loading tests. Do the strain values behind the pile stresses and pile loads include the strains that obviously have been locked into the pile both from installation jacking and from each loading cycle during the static loading tests, or were the guages "zeroed" before each test? It would be a very valuable addition to show the distributions of the locked-in stress (or load) in the piles for each of these events. Moreover, Fig. 21 appears to show the change of load in Pile J2 due to the jacking of the adjacent Pile J5. It would be exceptionally interesting and useful to see the load distribution in Pile J1 immediately before the jacking of Pile J5 started.
8. The measured changes of stress in Pile J2 due to the jacking of the adjacent Pile J15 (Fig. 21) are one of the singularly noteworthy observations reported by the authors. Were similar measurements in Pile J2 also taken when Piles J3 and J4 were jacked near Pile J2? If so, it would be valuable if the authors could also present these measurements.
9. The authors do not state how the reaction force for the jacking frame was arranged. Is it possible that some of that tension forces induced into the soil from the jacking of Pile J5 shown in Fig. 21 could have affected the measurements in Pile J2?

The authors determined the distributions of unit shaft resistance shown in Fig. 13 by differentiation of the load from one strain guage to the next. Such differentiations will invariably enlarge data imprecisions. For example, if the load difference between two guage locations is 8% of the load value and the imprecision (error) in each load value is about ±4% of the load, then the potential imprecision in the evaluated unit shaft resistance determined by differentiation between the two guages will range from zero unit shaft resistance through a unit shaft resistance of twice the correct shaft resistance value for a case where the loads are infinitely precise. The discusser believes the scattered distributions of unit shaft resistance illustrated in Fig. 13 is not due to variation of the actual shaft resistance, but to imprecisions of the strain measurements (and their conversion to load) in the piles. A more realistic distribution of the unit shaft resistance can be obtained by approximating the load distributions of Fig. 12 in an effective stress analysis and determining the unit shaft resistance from that approximation. The discusser's assumptions are hydrostatic pore pressure distribution, shear forces developing on the surface of the "H" rather than on the "square," and that the authors' distributions do account for all locked-in loads—the pile toe depth is as scaled from the authors' figure. The discusser's Fig. 1 shows the so-approximated load distributions and the data of Fig. 12 (after conversion from pile stress to pile load using the nominal steel area of the H-piles). The distributions shown are those for the maximum loads applied in the static loading tests on the two piles, 7,788 and 5,900 KN, respectively.

The load distribution approximations correspond to a distribution of unit shaft resistance plotted in Fig. 2 together with the

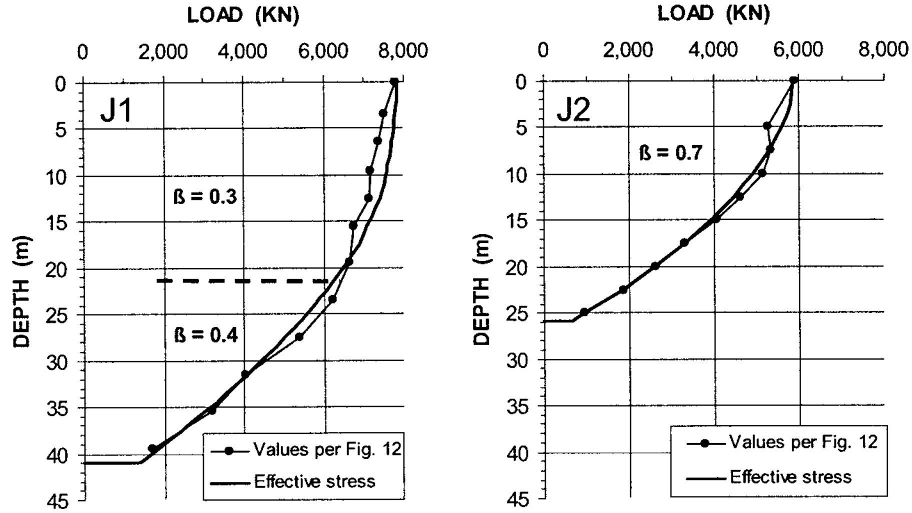

Fig. 1. Authors' load distribution showed in Fig. 12 converted from pile stress to pile load and approximated (solid lines) in an effective stress analysis employing the Beta-coefficients shown

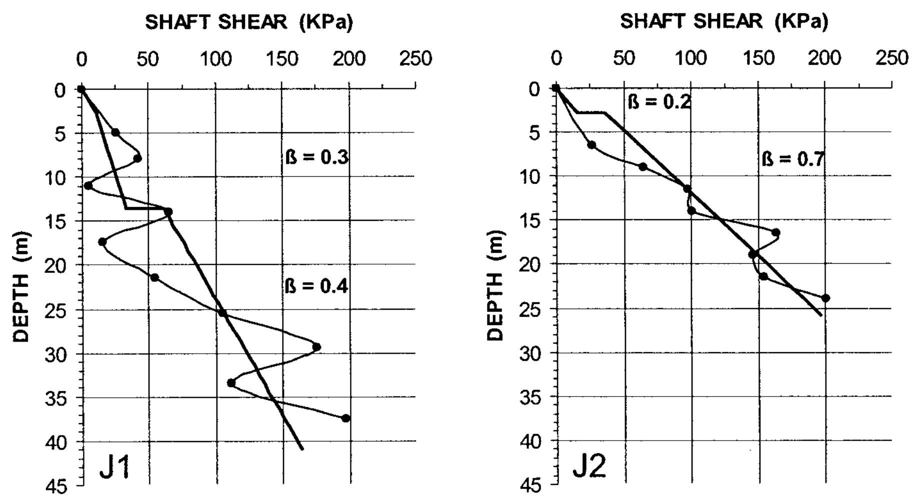

Fig. 2. Distribution of unit shaft resistance from the effective stress analysis (solid lines) and the distributions presented in the authors' Fig. 13

distributions of Fig. 13. The unit shaft resistance values determined from the approximated measured load distributions imply that the scatter shown in Fig. 13 is not representative for the site conditions. Note that the measured pile toe loads (stress) during the jacking of the piles shown in Fig. 7 indicate almost linear increase with depth, which supports the contention that effective stress governs the load and resistance distributions at the two sites. It also supports that the varying distributions of unit shaft resistance shown in Fig. 13 are not representative for the actual conditions at the two sites.

Moreover, the analysis demonstrates that the two sites show distinctly different magnitudes of shaft resistance (as do also the original figures, albeit this is disguised by the authors' use of different depth scales in Fig. 13). It would be of interest if the authors could expand on the potential cause of the difference between the two sites.

The toe resistances determined by fitting the data to the effective stress analysis correspond to a toe bearing coefficient, N_t, of 100 when calculated on the actual steel cross section area, and to N_t, equal to 30 if calculated over the area of the circumferential square. Neither value conflicts with the reported SPT N-indices shown in Fig. 1.

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William M. Isenhower, M.ASCE[1]; and
Lymon C. Reese, Hon.M.ASCE[2]
[1]Project Manager, Ensoft. Inc., 3003 Howard Lane, Austin, TX 78728. E-mail: ensoft@ensoftinc.com
[2]Principal, Ensoft. Inc., 3003 Howard Lane, Austin, TX 78728.

The authors have made a useful contribution to the technical literature. Increased knowledge about the behavior of deep foundations under loading, both axial and lateral, is strongly dependent on the results of carefully performed field experiments. The results are especially valuable if the foundations are instrumented for the measurement of load as a function of depth. Such data can be analyzed, taking the axial deformation of the pile into account, to develop *load-transfer* curves, such as shown in the authors' Fig. 14. The prediction of load-transfer curves, for side resistance and for end bearing, based on soil properties and method of installation and time after installation, allow for the analysis of deep foundations under a wide range of conditions (Reese 2005).

The discussers note the importance of the method of installation of a deep foundation on the properties of soil as exists around a deep foundation after installation. If a pile is driven into normally consolidated clay, the clay is not only remolded but excess pore-water pressure is developed, and the capacity of the pile under axial load can increase significantly with time, along with the dissipation of the excess pore-water pressure.

If a pile is driven into loose, cohesionless soil, the pile driving in most instances will cause a densification of the soil with a resulting depression in the ground surface near the pile. The assumption might be made that the driving of a pile will push back the soil at the wall of the pile and the earth pressure at the wall of the pile may be assumed to be passive. Careful analysis of field experiments show that, in fact, the pressure at the wall is much less and may be fairly close to earth pressure at rest.

If a pile is constructed by drilling, the soil around a drilled shaft after installation will depend on whether the construction was done with the dry method, the wet method, or the casing method. The response of a drilled shaft to loading will be affected not only by the method of installation but by the details employed in each of the methods, for example, by the specific nature of the concrete and its method of placement (Isenhower, unpublished course notes, 2006).

Research such as reported by the authors, extended to a determination of the specific nature of the soil at the wall of the pile after installation, can lead to improved methods of design of deep foundations. The paper is valuable particularly for the methods employed, and the results are useful for piles pushed into place into soil as at the site. The results are not applicable for piles installed by a different method into different soils. The authors have written, not necessarily in jest, that a student in a class later in this century could ask, "Professor, did engineers in the last century really design piles on the basis of soil properties determined before the pile was constructed"? (Reese and Isenhower 2000). The importance of the effects of pile installation on soil properties is being recognized by the profession (U.S. Army Corps of Engineers 1993) and there is hope that definitive research will be accomplished in time to lead to the prediction of numerical values of the modified soil.

The authors are to be congratulated on presenting an interesting and informative paper and one that should be of considerable use to engineers who plan to install deep foundations in similar soil and in a similar manner. The details of the instrumentation for measurement of axial load and porewater pressure are particularly valuable.

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Closure to "Observed Performance of Long Steel H-piles Jacked into Sandy Soils" by J. Yang, L. G. Tham, P. K. K. Lee, and F. Yu

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J. Yang, M.ASCE[1]; L. G. Tham, M.ASCE[2]; P. K. K. Lee[3]; and F. Yu[4]
[1]Assistant Professor, Dept. of Civil Engineering, The Univ. of Hong Kong, Pokfulam, Hong Kong, China. E-mail: junyang@hku.hk
[2]Professor, Dept. of Civil Engineering, The Univ. of Hong Kong, Pokfulam, Hong Kong, China.
[3]Head, Dept. of Civil Engineering, The Univ. of Hong Kong, Pokfulam, Hong Kong, China.
[4]Research Student, Dept. of Civil Engineering, The Univ. of Hong Kong, Pokfulam, Hong Kong, China.

The interest of the discussers in the paper is highly appreciated. The writers totally agree with the comments of Isenhower and Reese that carefully designed field tests with highly instrumented piles play an important role in understanding the mechanisms involved in pile behavior. With regard to the points raised by Fellenius, the writers would like to provide the following clarifications.

1. Both sites are located on reclaimed land whose water-table conditions are generally different from those of the natural sloping terrain. It is considered acceptable to assume that the soil profile below the water table at the sites is saturated and the pore-water pressure distributes hydrostatically.
2. The decomposed granite is a residual soil formed by weathering of the parent rock. This type of soil exists widely in Hong Kong and other areas of the world, such as Japan and Malaysia. Typical particle-grading curves of the decomposed granite soil in Hong Kong are shown in Fig. 1, which were established by Lumb (1962) using 72 samples. The soil is essentially sandy and relatively permeable. Details on various properties of the soil can be found in the works of Lumb

(1962; 1965). Because of the nature of the soil, cone penetration tests (CPT) are rarely used in ground investigation. The standard penetration tests (SPT) are dominant in local practice.

3. The shaft movement in Fig. 14 was not directly measured but derived from the pile head settlement and strain measurements.
4. The cross-sectional area of the 40×40 steel angle is $308\ mm^2$. The small additional area from the steel angles was ignored in data interpretations in the paper.
5. The pile heads were about 40 cm above ground.
6. Figs. 9 and 10 of the paper show clearly that the pore-water pressures generated during pile installation were completely dissipated in about 2 h. This observation suggests that the load tests carried out 2–4 days after pile installation would not be affected by the set-up effect. Moreover, the results from the repeat test conducted 34 days after the first test (Figs. 9 and 10 of the paper) do not show a strong set-up effect that may arise from soil creep or aging. Given limited time, no measurements were made of the load-settlement curve during the repeat test.
7. The existence of locked-in stress in a pile after the pile installation has been known for a long time. However, very few evaluations of residual stress, as pointed out by Van Impe (1994), have been presented in the literature. This is mainly because the conditions for a shift in a guage reading before the start of a load test are influenced by many details of the pile installation procedure and pile group effects. It has thus been common practice to zero the guages before the start of a load test. This common practice was followed in the present study.
8. The changes of axial stress in Pile PJ2 due to jacking of Piles PJ3 and PJ4 were measured. Generally, they exhibit patterns similar to those shown in Fig. 21. It should be noted that the installation of adjacent piles induced significant tensile stress in PJ2, which could substantially reduce the locked-in stress (mainly in compression) due to installation of PJ2 itself. Detailed data interpretations with regard to this issue will be reported in future papers.
9. The reaction force for the jacking frame was provided by kentledge instead of tension piles or soil anchors, which would not induce tension force in the soil.
10. The unit shaft resistance for any section between two guage levels was determined as the difference of the pile loads at the two levels divided by the surface area of the pile section. The shaft resistance should thus be regarded as an average value for the section, and the plotted shaft resistance distribution should be treated as an approximate rather than an exact representation. This is a widely used practice in Hong Kong. The writers consider the method described by Fellenius to be an alternative for shaft resistance interpretation, which may help view the test results in a different way. The writers disagree, however, with the discusser's opinion that the method is superior over the common practice in that it is able to provide "a more realistic" distribution of shaft resistance. The discusser's method assumes that the distribution of shaft resistance is perfectly linear with a constant β value, which should be an idealized rather than a real case, as natural deposits are never perfectly uniform. Fitting of the pile-load curve using a smooth curve will simply remove the clue for the recorded real-life variations, which might be due to other reasons such as the existence of thin soft layers that were not discovered by soil borings at the site or due to the existence of a gap between the pile and surrounding soil caused by pile installation.
11. Possible reasons for the observed difference in shaft resistance between Piles PJ1 and PJ2 have been mentioned in the original paper. One of the reasons is the differences of ground conditions at the two sites. The second is probably related to the different treatments of the gap between the pile and surrounding soil generated during pile installation. The third reason may come from the difference of the termination criteria adopted for jacking of the two piles. A more detailed discussion of the effect of termination criteria on the behavior of jacked piles has been given by Yang et al. (2006).

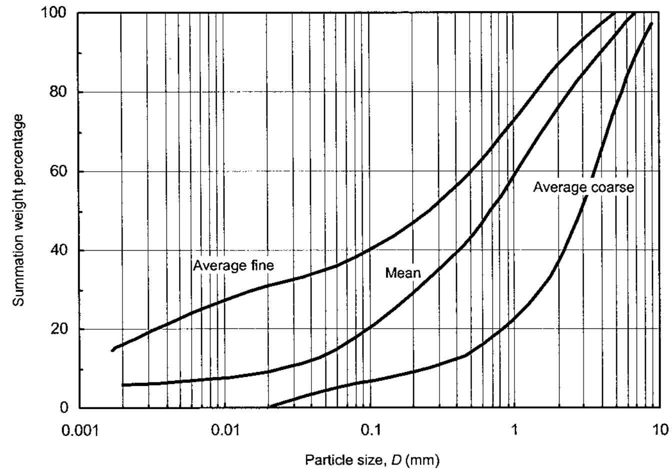

Fig. 1. Grading curves of decomposed granite soil in Hong Kong (after Lumb 1962)

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