Discussion on two parallel papers presenting results of full-scale loading tests on H-piles in Hong Kong

Fellenius, B.H., 2007. Behavior of jacked and driven piles in sandy soil. Geotechnique 57(5) 245-259.

and

Fellenius, B.H., 2007. Observed performance of long steel H-piles jacked into sandy soils. ASCE Journal of Geotechnical and Environmental Engineering, (133)7 898-899.

Discussed papers (not included with the discussions)

J. YANG, L.G. THAM, P.K.K. LEE, S.T. CHAN, AND F. YU (2006). Behavior of jacked and driven piles in sandy soil, Geotechnique 56(4) 245-259.

and

J. Yang, L.G. Tham, P.K.K. Lee, and F. Yu (2006). Observed performance of long steel H-piles jacked into sandy soils. ASCE Journal of Geotechnical and Environmental Engineering, (132)1 24-35.

Discussion

Behavior of jacked and driven piles in sandy soil

J. YANG, L.G. THAM, P.K.K. LEE, S.T. CHAN, AND F. YU (2006). *Geotechnique* 56(4) 245-259

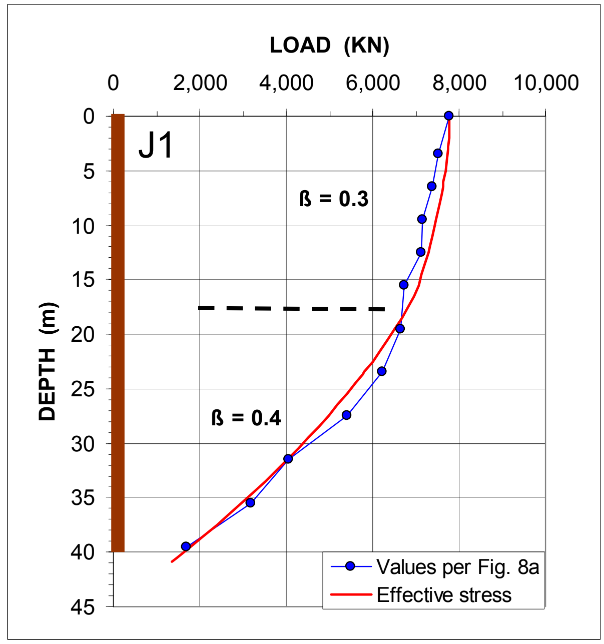
B. H. Fellenius, *Calgary, Canada*

The discusser questions the authors' method of determining the distribution of unit shaft resistance from the distribution of measured loads in the piles. The authors calculated the unit shaft resistance from the change of load between the strain-gage levels divided by the lengths between the gage level and circumferential area of the pile. Such differentiation will invariably amplify data imprecision. For example, the load in the pile diminishes on average about 8 % from one gage location to the next. Assuming, realistically, that the measurement imprecision (error) in each load value is about ±4 % of the value, then, as opposed to a case where the loads are infinitely precise, the potential error in the evaluated unit shaft resistance determined by differentiation between the two gage levels will range from zero to twice the correct value. Thus, the discusser believes the erratic distributions of unit shaft resistance illustrated in Figs. 10 and 11, are not due to the implied variation of the unit shaft resistance, but to imprecision of the strain measurements (and their conversion to stress or load) in the piles. A more realistic distribution of the unit shaft resistance can be obtained by first fitting the measured load distributions to an effective stress analysis, which results in a smoothened distribution curve; a filtering of the errors. The unit shaft resistance can then be determined from the arithmetic relations of the so filtered distribution of load in the pile.

The discusser has fitted the data points in Figs. 8 and 9 to an effective stress analysis assuming hydrostatic pore pressure distribution, shear forces developing on the surface of the "H" rather than on the square box around the "H", and, for this analysis, employing the authors' zeroing of the gages before the test. Figs. 23 and 24 show the so fitted load distributions together with the data of Figs. 8 and 9 (after conversion from pile stress to pile load). The distributions shown are those for the maximum loads applied in the static loading tests on the piles. The fit is good for Piles J1, J8, and D2, but less so for Pile D8. However, the authors load distribution for Pile D8 (Fig. 9b), indicating no shaft resistance between the depths of 4 m through 30 m, is not believable. It is probable that the gages have malfunctioned, supplying data with larger scatter and errors than shown by the gages in the other three piles. Therefore, the discusser has chosen to use the same effective stress distribution for Pile D8 as that for Pile D2. Each graph lists the effective stress proportionality coefficients (the ß-coefficients) used for the particular fitting, taken as having approximately constant values within the two soil layers identified by the borings.

The unit shaft resistance values determined from the effective stress calculation fitted to the measured load distributions are plotted in Figs. 25 and 26 together with the distributions of Figs. 10 and 11, respectively. In the discusser's opinion, the calculated straight line distributions are more representative for the unit shaft resistance distributions than the authors' differentiation approach. With regard to Pile D8, the authors' four values of negative shaft resistance per the differentiation approach further emphasizes that the gage readings from Pile D8 cannot be correct.

The ß-coefficients determined from the jacked piles are larger than those of the driven piles, seemingly agreeing with the authors' conclusion that larger shaft resistance was obtained for the jacked piles as opposed to the driven piles. However, in the discusser's opinion, this conclusion is not necessarily correct.



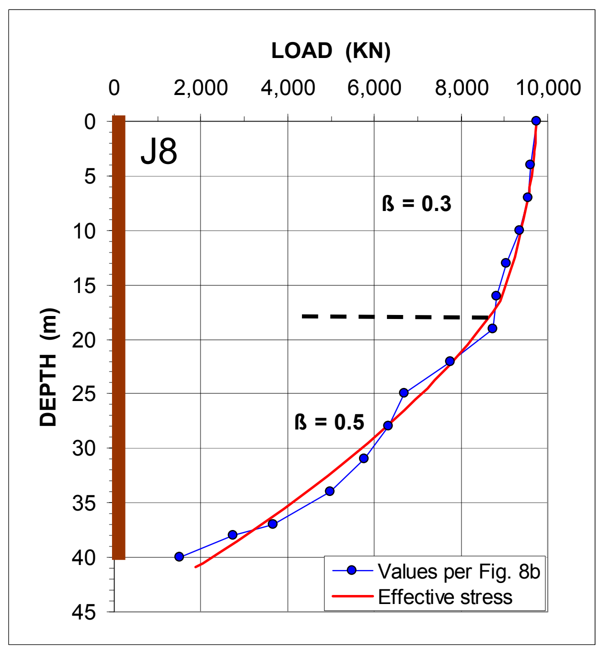


Fig. 23 Load distributions for Piles J1 and J8 per data points showed in Figs. 8a and 8b, converted from axial stress to load and approximated in an effective stress analysis employing the ß-coefficients shown.

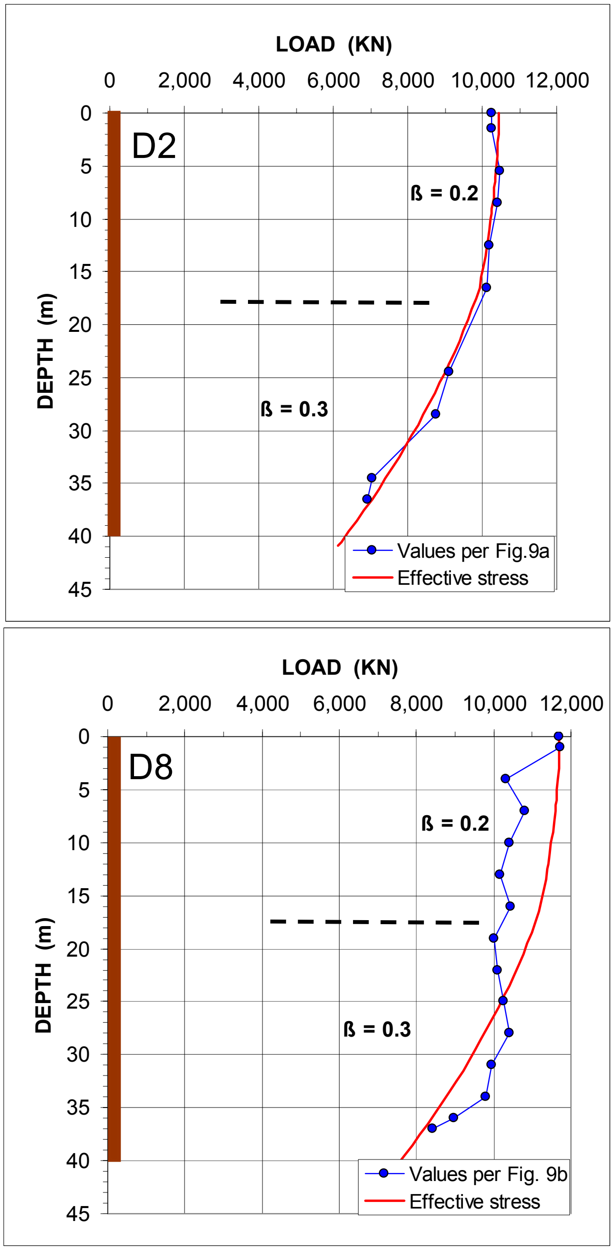


Fig. 24 Load distributions for Piles D1 and D8 per data points showed in Figs. 9a and 9b, converted from axial stress to load and approximated in an effective stress analysis employing the ß-coefficients shown.

The authors have — stating this to be intentional! — disregarded the effect of residual load (locked-in load) in the piles at the start of the static loading tests. This is surprising in the light of that the gages were present in the pile during the installation of the piles and available for taking readings before the start of the static loading tests. Specifically for the jacked piles, the load remaining in the pile after unloading surely must have indicated that were locked-in as a result of the jacking. Gage readings before the start of the tests would have established the residual load distribution in the pile. Moreover, the fact that the testing of five of the nine driven piles showed a "negative settlement" after unloading is a direct indication of presence of residual load. (When in the loading test, the pile toe is not moved appreciably, and the shaft resistance is degraded somewhat by the test, a portion of the residual load is unloaded, which results in a corresponding elongation of the pile and a higher elevation of the pile head after the test in relation to the before-the-test elevation).

When omitted from the analysis, presence of residual load, will cause the evaluation of the gage data to show the shaft resistance to be larger than the true value. Where the residual load is fully developed, the so evaluated shaft resistance will be twice the true shaft resistance. Moreover, the evaluation will result in an underestimated magnitude of the pile toe resistance. Finally, the pile-head load-movement curve will appear stiffer for a pile with larger residual load as opposed to a pile with smaller.

It is possible, indeed probable, that the residual loads in the driven piles will be smaller than in the jacked piles. If so, the omission of the presence of residual load will result in the evaluated shaft resistances being larger for the jacked piles as opposed to the driven piles, even if the ultimate unit shaft resistance would be the same whether the piles are jacked or driven. Indeed, as the authors excluded the residual load in their analysis, the difference shown by the authors is not likely true.

The toe resistance is of course a function of how hard the piles were driven. The case indicates that the driven piles were probably installed to a somewhat larger toe resistance as opposed to the jacked piles. This not-withstanding that the authors' analysis is likely to have shown a toe resistance smaller than the true value because of the omission of the locked-in toe loads.

In the discusser's opinion, the conclusion that the jacked pile develop more shaft resistance than the driven piles is not supported by the authors' data. Whether or not there is a difference is unknown because the residual loads are omitted from the authors' analysis not known. It is also the discusser's opinion that all other conclusions of the authors are invalid, inasmuch they are based on the apparent differences of shaft resistance and pile stiffness originating in the omission of gage errors and residual loads.

Authors' reply

The discusser raised two points in his discussion. The first one is concerning the method of determining the distribution of unit shaft resistance. As indicated in the paper, the unit shaft resistance for any a section between two gauge levels was determined as the difference of the pile loads at the two levels divided by the surface area of the pile section. The unit 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 and also is, to our best knowledge, common practice in many other regions outside Hong Kong. We consider the discusser's method one of the alternatives for shaft resistance interpretation that may help view the test results in a different way. We do not agree, 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 beta value, which should be an idealised rather than a real case, as natural deposits are never perfectly uniform. Lehane et al. (1993) conducted a load test on an instrumented model pile at a sand deposit that was considered highly uniform. The local shear stress (i.e. shaft resistance) was measured directly from stress

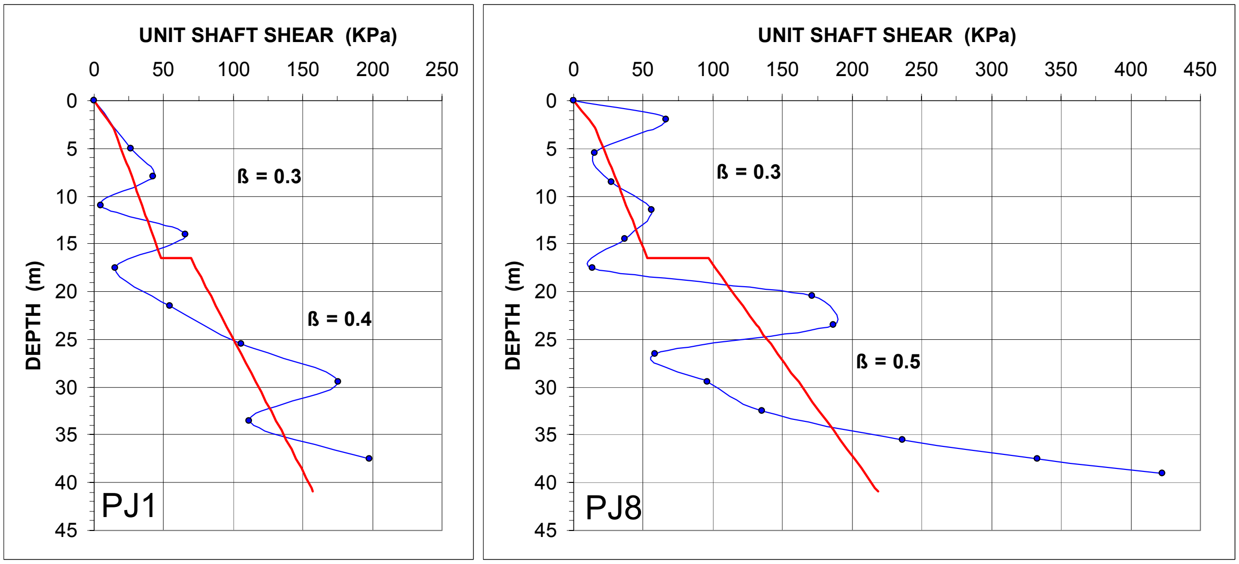


Fig. 25 Distribution of unit shaft resistance from the effective stress analysis and the distribution s presented in the authors' Fig. 10

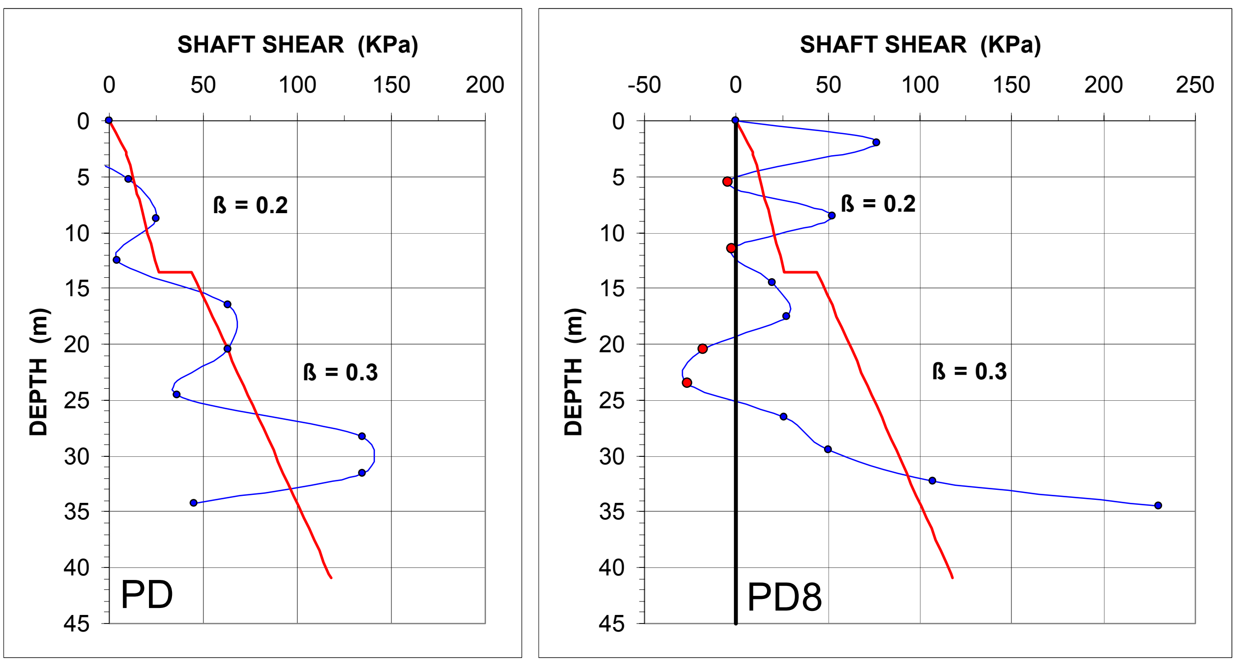


Fig. 26 Distribution of unit shaft resistance from the effective stress analysis and the distribution s presented in the authors' Fig. 11

transducers at three different distances (h) from the tip of the pile as it was jacked into the ground. Shown in Fig. 27 here are profiles of the local shear stress (where R is pile radius), which clearly indicate the phenomenon of friction degradation at a given depth. They state that the reduction in stress with increasing pile penetration provides a rational explanation for the concept of critical depth, a much debatable issue in the area of pile foundations over the past decades. The friction degradation is considered to be connected with the cyclic loading action of the surrounding soil during pile installation. This fatigue mechanism also provides a good explanation for the difference in shaft friction between jacked and driven piles observed in Fig. 18 in our original paper.

The above example highlights the complexity of the problem we are dealing with. In this regard, as stated in the original paper, while we presumably consider that some anomalous values of shaft resistance (for PD8) may be due to rogue gauge measurements, we also recognize that there exist other potential reasons. These potential reasons may include, for example, the existence of soft layers that were not discovered by the limited number of boreholes at the site or the existence of a gap between the pile and surrounding soil due to pile installation. It has been frequently observed that, when an H-pile penetrates into the ground, whether by driving or jacking, the overlying soil may be dragged down by the pile to lower levels, leaving a gap between the pile and soil at upper levels. This phenomenon was also reported by, for example, Tomlinson (1977) and Poulos and Davies (1980). Fitting of the pile-load curve using a smooth curve will simply remove the clue for these real-life variations.

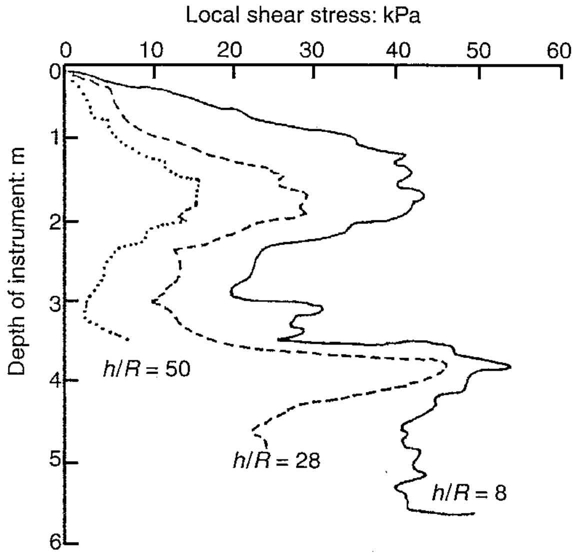


Fig. 27. Measured profiles of shaft friction (Lehane et al., 1993)

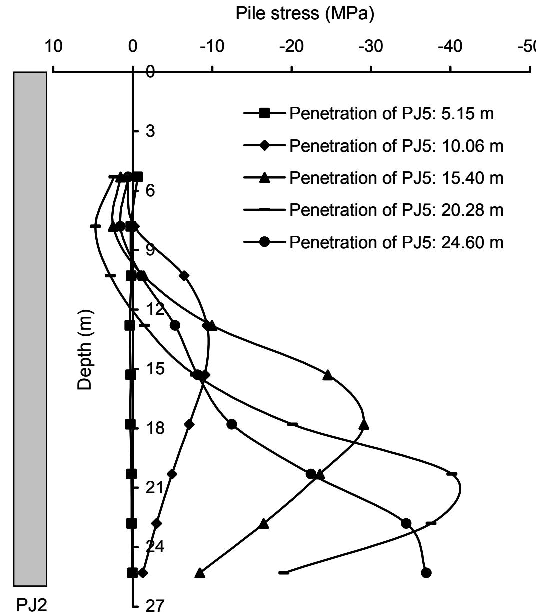


Fig. 28. Measured profiles of axial stress in PJ2 induced by jacking of PJ5 (Yang et al., 2006)

The second point raised by the discusser is about the residual stress effect on pile load tests. The existence of locked-in load in a pile after the pile installation has been known for a long time (Nordlund, 1963; Gregersen et al., 1973). However, it is not easy to demonstrate and even more difficult to quantify the effect on test data, because the conditions for shift of the gauge reading before the start of load test are complicated and influenced by many details of pile installation (e.g., placing the pile in the lead before installation and splicing pile segments during installation). It has thus been common practice to zero the gauges before the start of pile load test. Very few evaluations of residual stress, as pointed out by Van Impe (1994), have been presented in the literature. In a prediction symposium that was held by the ASCE Geotechnical Engineering Division in conjunction with the 1989 Foundation Engineering Congress, results of load tests on four instrumented piles were used as the basis for prediction (Finno, 1989). Of the 23 people who made predictions, only three chose to predict the residual loads induced by pile driving. This small number of predictions for residual loads emphasizes the difficulties associated with residual load interpretations.

One particular difficulty or uncertainty in the evaluation of the residual load effect, in our opinion, comes from the impact of adjacent pile installation. Yang et al. (2006) show that both the magnitude and distribution of the stress in an existing pile are largely influenced by nearby pile driving, as shown in Fig. 28, where the profiles of axial stress in pile PJ2 induced by jacking of a nearby pile (PJ5) are given. Note that significant tensile stresses dominating in the major portion of PJ2 were measured, which may substantially reduce the locked-in stresses (mainly in compression) due to installation of PJ2 itself. Therefore, in order to have meaningful interpretations, both the testing programme and piling programme need to be well managed, which, given the time and economics in most practical projects, would not be easy to achieve. In our field study, we do have made effort to investigate the residual load effect for pile PJ2 by using strain measurements in different phases. A preliminary analysis (Yang and Sze 2005) indicates that the existing methods do not give a satisfactory prediction of the residual loads. Detailed data interpretations are in progress and will be reported in future papers.

In summary, the residual load effect remains a very tricky and difficult issue and current practice is considered acceptable. In this regard, we agree with Van Impe (1994) that we should not "overestimate the possibilities of predicting residual loads, since they are too sensitive to the pile installation procedure and pile group effects, usually rendering its prediction errors above all acceptable levels."

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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

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Bengt H. Fellenius[1]

[1]1905 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 gages "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 gage 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 gages 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

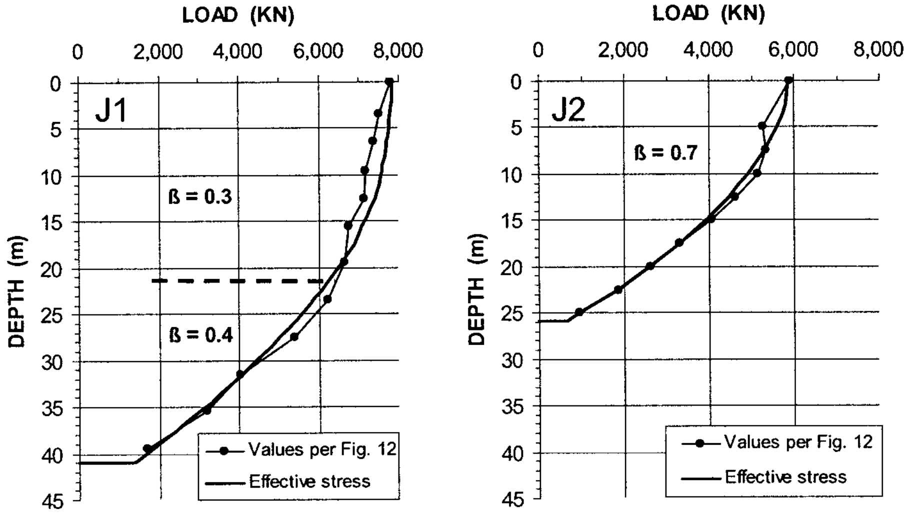

Fig. 1. Authors' load distribution showed in Fig. 12 converted from pile stress to pile load and approximated (solid lines) in an effective stress analysis employing the Beta-coefficients shown

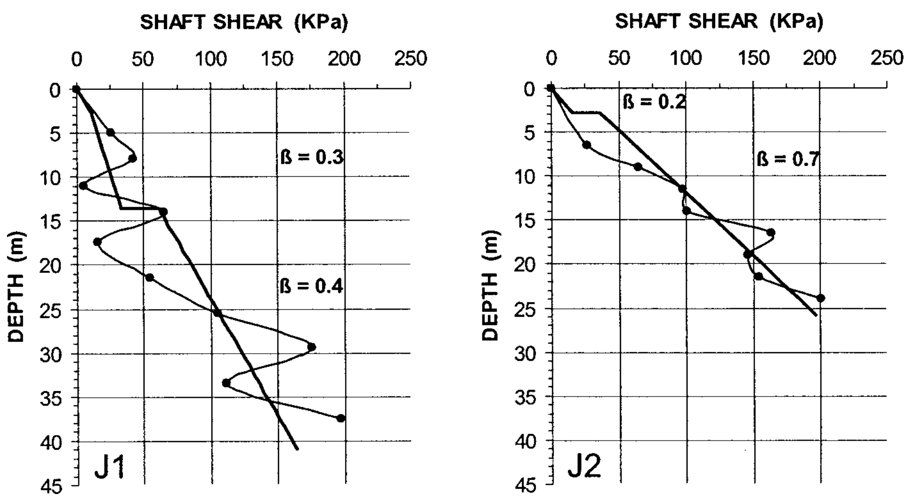

Fig. 2. Distribution of unit shaft resistance from the effective stress analysis (solid lines) and the distributions presented in the authors' Fig. 13

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.

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 gage 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 gages 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 gage 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).

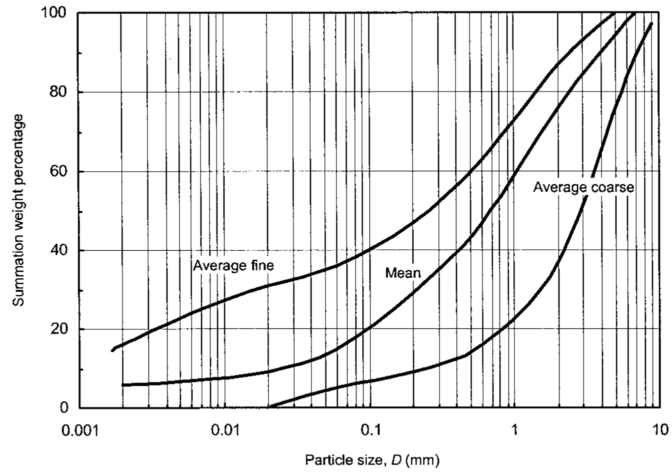

Fig. 1. Grading curves of decomposed granite soil in Hong Kong (after Lumb 1962)

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Yang, J., Tham, L. G., Lee, P. K. K., Chan, S. T., and Yu, F. (2006). "Behaviour of jacked and driven piles in sandy soil." *Geotechnique*, 56(4), 245–259.

COMMENTS ON THE AUTHORS' REPLY

On the reply to the Discussion in Geotechnique

To support the scattered distribution of unit shaft resistance, the authors state that such scatter could be due to "friction degradation" and refer to similarities presented in a paper by Lehane et al. (1993). However, the soil in that paper is not "highly uniform", as suggested by the authors, the CPT q_c distribution from the site presented in that paper shows a definite variation with depth. Also, the distribution of unit shaft and toe resistances reported by Lehane et al. (1993) agrees closely with the CPT q_c distribution. Moreover, the shaft resistance distribution in Fig. 27, which the authors copied from the paper by Lehane et al. (1993), refers to the installation of the test pile as it is being pushed in, and it does not show measurements obtained in a static loading test some time after the installation, as implied by the authors' figure caption. Finally, contrary to the authors' statement, no cyclic action occurred during the installation of that test pile. Note also that the test pile reported by Lehane et al. (1993) is very short, 6 m as opposed to the authors' pile, which is long, 40 m. In the discusser's opinion, the referenced figure is therefore not supportive of the Authors' approach.

The authors rationalization that their indicated variation of unit shaft resistance could be due also to "*existence of soft layers that were not discovered by the limited number of boreholes at the site*" is an unsupported conjecture. A as the Discusser points out, a more plausible cause of the shown erratic variation in unit shaft resistance values (determined from differentiation between load values) is that it is due to the resulting magnification of the error in the load values — the small difference in load values between gage levels being about equal to the error in the load values.

Maybe only three contributors made direct reference to residual load at the Evanston prediction event. Whether or not that proves the prediction or assessment of residual load from soil data is difficult, this is irrelevant. The authors did not need to predict or interpret, as they had access to direct measurements of the residual loads; measurements at end of jacking and at the start of static loading tests. It would have been illuminating if the authors had presented these measurements in their reply.

Indeed, the discusser's evaluated beta-coefficients are not true for the pile response at the site. Not because of any inaccuracy in the method of evaluation, but because they only represent the shaft resistance generated during the test and do not include the shaft resistance present in the pile before the start of the test.

In referring to residual loads, the paper by Lehane et al. (1993) states that "*If neglected, these can cause the compressive shaft capacity to be overestimated significantly and result in large errors in the evaluated shear stress distribution with depth*", which neglect is very much what the authors have displayed.

The authors' Fig. 28 is from their "parallel" report of the tests, which was sent to the ASCE journal. While that graph is interesting, note that — again — the residual load present in the pile at the start of the measurements is omitted from the graph and, therefore, the graph only presents the loads imposed by the jacking of the adjacent pile, that is, the loads in the pile before the jacking are unknown. The omission makes it impossible to draw any general conclusion from the test.

The shaft resistance distribution shown by the discusser is not the results of a "perfectly constant beta-coefficient", but of a best fit of the reduction of load in the pile with depth as imposed during the test for the maximum load applied to that of the distribution of accumulated effective stress. It is noted that the authors "have made effort to investigate the residual load effect for pile PJ2". The statement that "a preliminary analysis (Yang and Sze 2005) indicates that the existing methods do not give a satisfactory prediction of the residual loads" is bothersome. The gages would provide direct measurements of the load distribution, i.e., the residual load, in the pile at the start of the static loading test. A difficulty in evaluating these measurements would imply that also the gage values recorded during the authors' loading test are questionable. In the absence of a discussion of why the change in gage readings during the static loading test would show correct values of the loads imposed during the test, if the readings obtained at the end of the installation and before starting the test are not reliable, would have a definite bearing on the accuracy of the data reported in the authors' paper.

On the reply to the Discussion in ASCE Journal

The reply clarifies the issues raised in Questions 1-9 of the Discussion. Reply 10 in effect states that the Discusser's analysis of the strain-gage loads is an alternative to that of the authors'. The authors' approach is erroneous and, as such, it cannot be accepted as an alternative. The similarity of the two papers means that much of the comments on the reply to the discussion of the Geotechnique version of the report on the test results also apply to the reply to the to the discussion of the ASCE version. It is very clear that the Authors' load values are influenced by significant residual loads. It is baffling that the authors decided to ignore these loads despite having the data available. The reference to that this is in conformity with Hongkong's "*widely used practice*" is no acceptable reason. If true, that practice is wrong and should be rectified. The authors' information (Geotechnique reply) that detailed data interpretations are in progress and will be reported in future papers is disappointing. It would have been preferable that the authors had completed the analyses before publishing the tests results—now in three papers and then in the as indicated forthcoming fourth paper.

The word "gage" is often also spelled "gauge". ASCE's spelling "guage" is rather rare, though.