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Settlement of building foundations based on field pile load tests

Tassement de fondation de bâtiment base sur des essai de pile charger

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ABSTRACT
Serviceability limit requirements are an important concern in pile foundation design. In places such as Hong Kong, where depths of competent founding soils/bedrock vary greatly, the pile lengths can vary from a few meters to over 100 m. Following the current practice of pile design and acceptance, the settlement of pile foundations can be considerably different from one site to another. The objective of this paper is to evaluate ranges of the settlement of buildings supported by driven pile foundations. Settlement measurements from full-scale static load tests on a large number of piles are reported. From the load-displacement curves of these load tests, the distributions of the settlement of single piles under the intended design load, twice the design load, and the failure load are determined. The settlements of single piles at the intended design load vary from less than 10 mm for relatively short piles to about 30 mm for long piles and the settlements at twice the design load vary from 18 mm to 85 mm. The single pile settlement follows approximately a lognormal distribution. The settlement of a pile group is then obtained by multiplying the single pile settlement by a group settlement ratio that is defined as the ratio of the pile group settlement to the single pile settlement under the same load per pile. The distribution of the settlement ratio for freestanding pile groups in sand is established based on data from some field and model pile group tests. The results show that considerable pile group settlement can occur even if all individual piles satisfy specified acceptance criteria. Settlement problems may be particularly serious for buildings supported by very long piles or piles founded on undulating bedrocks.

RÉSUMÉ
Les exigences des limites de serviabilité représentent une inquiétude dans le dessin de piles. Dans de tels lieux que Hong Kong, où la profondeur de la terre/roche en place compétente varie énormément, la profondeur de piles varie, de quelques mètres à plus que cent mètres. En poursuivant l'exercice courant en ce qui concerne le dessin et acceptation de piles, le tassement de fondations peut varier considérablement, selon le chantier. Cet article a, comme objectif, l'évaluation de tassement des bâtiments qui se servent de fondations dont les piles sont enfoncées'. Des mesures de déposition résultant d'épreuves statiques de pleine échelle de nombreuses piles seront rapportées. Selon les courbes indiquant le déplacement de chargements tirés de ces épreuves, les distributions de tassement de piles individuelles soumises au chargement prévu, deux fois prévu, et le chargement de défaillance, sont calculées. Le tassement de piles individuelles au chargement de dessin prévu varient de moins que 10mm en en cas de piles relativement courte, vers 30mm en cas de piles relativement longues, et les dépositions à deux fois le chargement prévu varient de 18mm à 85mm. Le tassement de piles individuelles poursuive une distribution approximativement lognormal. Par conséquent, le tassement d'un groupe de piles est obtenu en multipliant le tassement d'une pile individuelle par le ratio du tassement d'un groupe défini comme le ratio du tassement d'un groupe de piles contre le tassement individuel sous le même chargement par pile. La distribution du ratio de tassement, en ce qui concerne les groupes de piles indépendantes positionnées en sable, est établie selon des données provenant des essais pratiques et modèles. Les résultas montrent bien qu'un tassement considérable se produise, même si toutes les piles individuelles remplissent des critères spécifiques d'acceptation. Des bâtiments qui se servent de très longues piles ou de piles basées sur un soubassement onduleux risquent de graves problèmes de tassement.

1 INTRODUCTION

In the construction of driven piles, a required pile capacity is often prescribed and the piles are driven to depths that meet the prescribed capacity. Some completed piles are then selected for static loading testing. The foundation piles will be considered acceptable if specified acceptance criteria, such as Davisson's criterion and the residual settlement criterion (Buildings Department, 2002) are satisfied during the load tests. In places such as Hong Kong, where depths of competent founding soils/bedrock vary greatly, the pile lengths also vary from a few meters to over 100 m. Following the current practice of pile design and acceptance, the settlements of pile foundations can be highly different from one site to another.

Although both the ultimate limit and serviceability limit requirements must be considered in designing a pile foundation (Becker, 1996; Honjo & Kusakabe, 2002; Zhang et al., 2002), the current practice appears to have paid less attention to the serviceability requirements. One of the reasons may be that it is more difficult to estimate the foundation settlement than the foundation capacity. Indeed, in order to reasonably estimate the foundation settlement under a serviceability load combination, both the shear strength properties and the nonlinear stiffness of the foundation soil need to be characterized, and the cap-soil-pile interactions must be understood. These are not routinely carried out in the current practice.

The objective of this paper is to evaluate the range of the settlement of buildings supported by driven piles. To circumvent uncertainties in theoretical models for settlement analysis, the settlement evaluation will be based entirely on field evidence, i.e., results of full-scale static load tests on a large number of driven piles. First, a database of static load tests of driven piles is compiled. From the load-displacement curves of these static loading tests, the distributions of the settlement of single piles under the intended design load, twice the design load, and the projected failure load are determined. The settlement of a pile group is then obtained by multiplying the single pile displacement by a group settlement ratio. Finally, the distributions of the settlements of pile groups at the intended design load, twice the design load and at the projected failure are determined.

2 THE DRIVEN PILE DATABASE

A database of static loading tests on driven piles in Hong Kong was developed at the Hong Kong University of Science and Technology supported by the Hong Kong Housing Authority. This database contains over 250 cases of field static load tests on steel-H piles. The piles are either 305x305x180 (55C) piles or 305x305x223 (55C) piles. The sectional widths of the two types of pile are 319.7 mm and 325.7 mm, respectively and their cross-section areas are 22930 and 28480 mm^2, respectively. Depending on the founding level, the penetration depths of the test piles range from 9.8 to 86.0 m with an average length of 41.8 m.

Most of the test piles were driven into founding layers with the blow count from the standard penetration tests around 200. Three ground conditions at the founding level, i.e., decomposed granite, decomposed tuff, and marble formations, were encountered. Above the founding layer, superficial fill and marine deposits were commonly encountered.

The static loading tests followed two procedures outlined by the Hong Kong Housing Authority (1998) and the Buildings Department (2002). In this paper, only recent load tests following the Buildings Department procedure are reported. Two types of tests are specified in the procedure. The first type of tests was conducted on working piles. In each test, two load cycles were applied. The peak loads of the two cycles corresponded to one times and twice the design compression capacity of the pile (intended design load), respectively, which were 3548 kN for 305x305x223 (55C) piles and 2950 KN for 305x305x180 (55C) piles unless adjusted for various considerations. The second type of tests was conducted on preliminary piles. The test piles were not used as working piles and were therefore loaded to failure in terms of either the geotechnical pile capacity or the structural capacity. The structural capacity is defined as the load at which the stress in the pile section reaches the yield stress of the pile material, which is 3.3 times the design load for steel–H piles. The ultimate geotechnical capacity of the pile is the load which produces a settlement of the pile head exceeding

$$S = \frac{PL}{AE} + \frac{D}{120} + 4\,mm \qquad (1)$$

where S=settlement at failure in mm; D=least lateral dimension of the pile; P=applied load at failure; L=pile length; A=cross sectional area of pile; and E = Young's modulus for pile material.

From the load-displacement curves of the static load tests, the settlements under the intended design load, twice the design load, and the failure load are determined. In the next section, the settlements will be analysed and used as the basis for the distribution of the settlement of single piles.

3 ANALYSIS OF DISTRIBUTIONS OF SETTLEMENT OF SINGLE PILES

The settlement measurements of 41 preliminary piles, including settlements at the intended design load, two times the design load, and at failure, are presented in Fig. 1. The sequence of the load tests is arranged in a descending order in terms of the settlement at the intended design load. In Fig. 1, the settlement of a pile is not linear. At a given load, the pile settlement is a function of the pile length, soil conditions, the method of pile installation and time. Figure 2 shows the relationship between the settlement at the intended design load and the gross pile length.

Not surprisingly, under the same loading, longer piles will settle more due to larger elastic compressions of the pile shaft. When the pile length increases from 30 m to 65 m, the pile settlement nearly doubles from 15 mm to 30 mm. If acceptance of piles is entirely based on the bearing capacity requirements, the resulted settlements of the pile foundations can be considerably different from one site to another. Particularly, large differential settlement can occur in buildings supported by piles that are founded

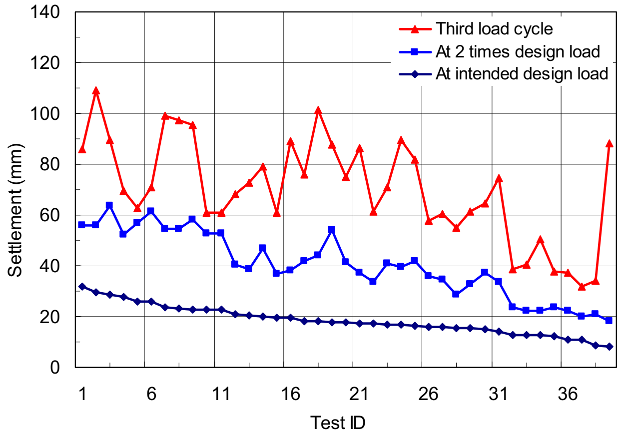

Figure 1. Settlement of 41 preliminary piles at the intended design load, twice the design load and at failure.

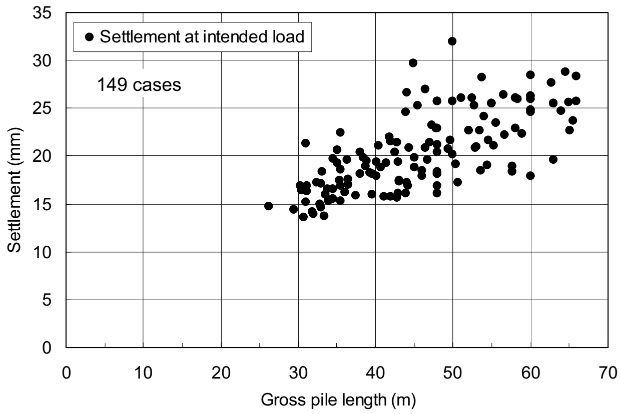

Figure 2. Variation of settlement at the intended design load with the gross pile length.

on undulating bedrocks therefore have uneven lengths.

Ideally, the distribution of the pile settlement should be investigated by dividing the test piles into several subsets according to the influence factors. In this preliminary study, the dada are not further classified, with the understanding that, in practice, the estimated pile foundation settlement is to be compared with an allowable settlement and that the allowable settlement is usually taken to be independent of such factors as pile length and soil types.

Figures 3(a) and (b) show the frequency diagrams of the settlements of 149 test piles at the intended design load and twice the design load. At the intended design load, the average settlement and maximum settlement are 19.9 and 31.9 mm, respectively, and 66 out of the 149 test piles settled more than 20 mm. At twice the design load, the average settlement and maximum settlement are 49.5 and 85.3 mm, respectively, and 147 out of the 149 test piles settled more than 20 mm. Fig. 3(c) shows the frequency diagram of the settlement of the 41 preliminary piles that were tested for third load cycles. The average settlement at failure reaches 70.0 mm.

The frequency diagrams in Fig. 3 appear to follow a lognormal distribution. The lognormal distribution is selected for convenience of reliability analysis in the future. The validity of this distribution can be checked using the χ–square test. At the significance level α=5%, the lognormal distribution can indeed be substantiated.

Based on the values of the average settlement and the corresponding standard deviation, the lognormal probability density functions (PDF) are superimposed in Fig. 3. Values of the mean and standard deviation of the logarithm of the settlement, λ and ξ, are also given in Fig. 3.

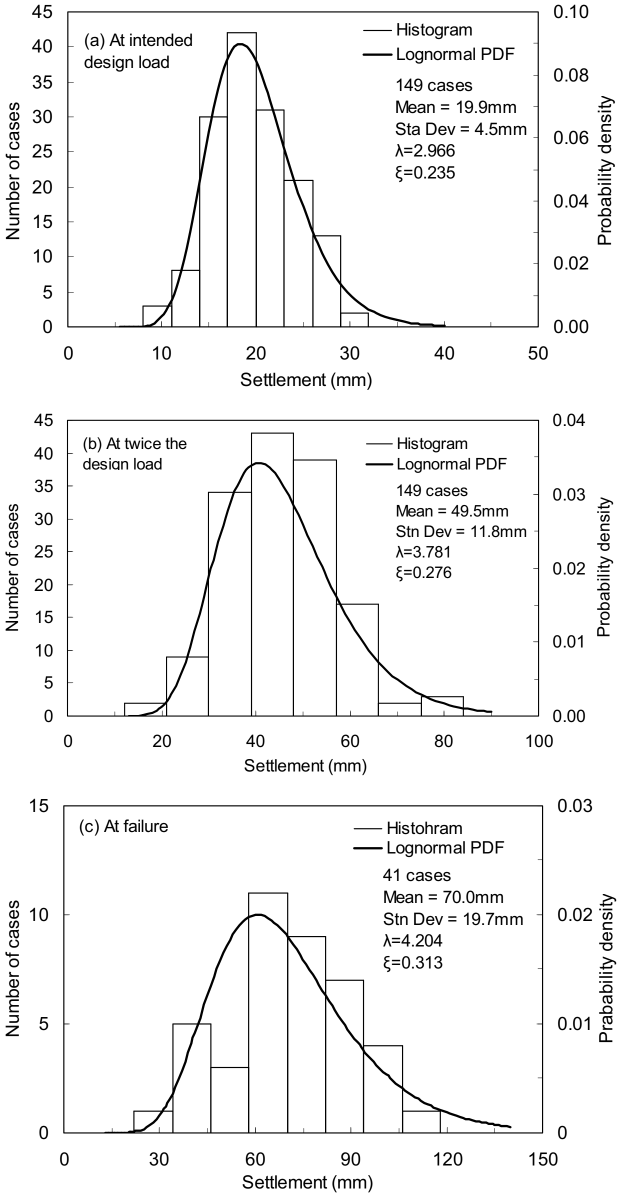

Figure 3. Frequency diagram of settlements of single piles at the intended design load, twice the design load and at failure.

4 STATISTICAL ESTIMATION OF SETTLEMENT OF PILE GROUPS

A pile foundation system often consists of several groups of piles and the group settlement differs from the single pile settlement due to pile group interactions. The settlement of a pile group can be obtained by multiplying the single pile settlement by a group settlement ratio R_S that is defined as (Poulos and Davies, 1980),

$$R_S = \frac{\textit{Average Group Settlement}}{\textit{Settlement of Single Pile at Same Average Load per Pile}} \qquad (2)$$

The settlement ratio for a particular site can be determined through field tests on single piles and pile groups at that site. The settlement ratio is a random variable that depends on the soil type, the pile spacing, the size of the pile group, and cap-soil interactions. It can be smaller than 1.0 for pile groups in loose sand but can be larger than 5.0 for pile groups in clay (O'Neill, 1983).

The majority of the test piles reported in this paper were constructed in completely decomposed granite and tuff, which are essentially cohesionless. Thus, only the settlement of pile groups in sand is considered in this section. Figure 4 presents the distribution of the settlement ratio of freestanding, driven pile groups in sands. The data in the figure were based on field and reduced-scale tests reported by Kezdi (1957), Woodward-Clyde (1979), O'Neill (1983), Briaud et al. (1989), and other researchers. The mean and COV of the settlement ratio are 0.77 and 0.93, respectively. The mean is smaller than 1.0, which is consistent with the result of a study on group interactions in driven piles (Zhang et al., 2001). In that study, the average group efficiency factor for the capacity of 3-diameter spaced freestanding pile groups in sand was 1.41. Accordingly, the average settlement ratio should be smaller than 1.0. Similar to the single pile settlement, the probability distribution of the settlement ratio can also be approximated by a lognormal distribution, and the distribution can be confirmed using a goodness-of-fit test.

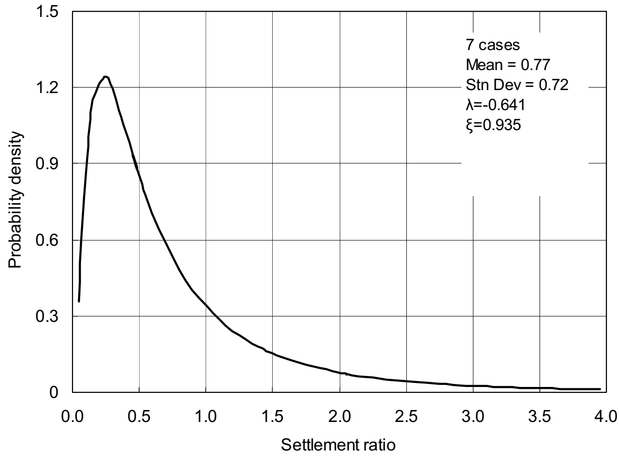

Figure 4. Frequency diagram of settlement ratio of freestanding, three-diameter spaced pile groups.

The group settlement ratio distribution shown in Fig. 4 cannot be taken as a general distribution of the settlement ratio of pile foundations. For pile groups in clay, the settlement ratio is usually larger than 1.0. In addition, although the settlement ratio measured from load tests on a single pile and a pile group reflects the pile group interactions, it usually does not reflect the long-term settlement of the soils beneath the pile group since a pile group test is completed in a relatively short period of time. The latter settlement can consist of a large fraction of the total settlement of the foundation system.

Using the settlement ratio, the settlement of a pile group, ρ_G, can be expressed as

$$\rho_G = R_S\, \rho_S \qquad (3)$$

If ρ_S and R_S are considered as two statistically independent lognormal variables, then the distribution of ρ_G is also lognormal and the mean and standard deviation of ρ_G are (Tang, 1989)

$$\overline{\rho}_G = \overline{R}_S\, \overline{\rho}_S \qquad (4)$$

$$\sigma_{\rho_G} = \overline{\rho}_G \sqrt{COV_{R_S}^2 + COV_{\rho_S}^2} \qquad (5)$$

where $\overline{\rho}_G$, $\overline{R}_S$, and $\overline{\rho}_S$ are the means of ρ_G, R_S and ρ_S, respectively; $\sigma_{\rho G}$ is the standard deviation of ρ_G; COV_{Rs} and $COV_{\rho s}$ are the coefficients of variation of R_S and ρ_S, respectively.

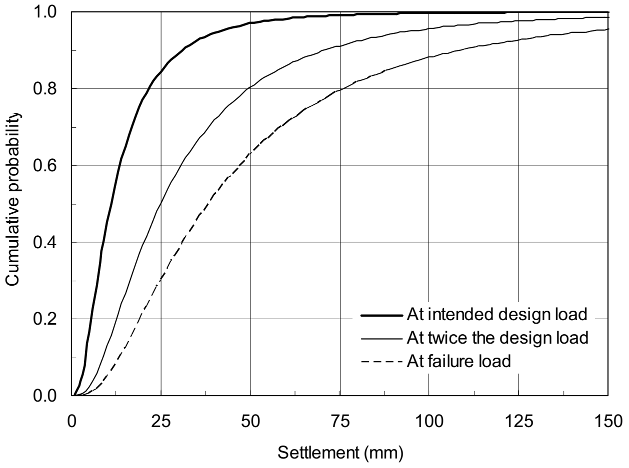

Figure 5. Estimated distributions of settlements of pile groups at the intended design load, twice the design load, and at failure.

The calculated distributions of the pile group settlement at the intended design load, twice the design load, and the failure load are shown in Fig. 5. Fig. 5 reveals that, adopting the current design practice and ignoring the cap-soil interactions, considerable pile group settlement can occur even if all individual piles satisfy specified acceptance criteria. For instance, if the selected allowable settlement is 25 mm, then the probability that the allowable settlement is exceeded will be 15% at the intended design load and will be 50% at twice the design load.

The exceedance of the selected allowable settlement, however, does not mean that the foundation cannot satisfy serviceability requirements. In fact, the tolerable settlement of foundations is also a random variable that depends on the type of structure, intended uses, ground conditions, and other factors. As there are uncertainties in soil properties, construction effects, interaction effects in large pile groups, as well as loads and load combinations, serviceability issues should be addressed in a more systematic way. A reliability-based approach that addresses uncertainties in both estimated foundation settlement and tolerable settlement is preferred.

5 SUMMARY AND CONCLUSIONS

Settlement information from full-scale static load testing of a large number of steel-H piles is presented to illustrate the possible settlement of pile foundations. The average length of the test piles was 41.8 m. At the intended design load, the average and maximum settlements of the test piles were 19.9 and 31.9 mm, respectively. At twice the design load, the average and maximum settlements were 49.5 and 85.3 mm, respectively, and 147 out of the 149 test piles settled more than 20 mm. When either the structural capacity of the pile section or the geotechnical capacity is reached, the average settlement reached 70.0 mm. The probability distributions of the pile settlements at these load levels can be considered lognormal.

The settlement of a pile group is obtained by multiplying the single pile settlement by a group settlement ratio that is defined as the ratio of the pile group settlement to the single pile settlement under the same load per pile. The settlement ratio itself is also a lognormal random variable.

The distributions of the settlement of freestanding pile groups in sand are estimated assuming the single pile settlement and the group settlement ratio being two statistically independent variables. Based on the estimated distributions, large total settlements can occur following the prevalent practice even if all individual piles satisfy specified acceptance criteria. For instance, if the selected allowable settlement is 25 mm, then the probability that the allowable settlement is exceeded will be 15% at the intended design load and will be 50% at twice the design load. To address the serviceability requirements and the ultimate limit requirements equally, a more unified reliability-based design approach is needed.

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