GEOTECHNICAL CRITERION FOR SERVICEABILITY LIMIT STATE OF HORIZONTALLY-LOADED DEEP FOUNDATIONS

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Abstract

This paper presents a first approximation of the elastic limit displacement of soil resistance to horizontally loaded piles. 37 field test data sets were consistently chosen from a vast database. Both mathematical and graphical approaches were adopted to interpret the elastic limit of soil resistance from the measured load-displacement curve. The data sets indicate that the mean value of the elastic limit displacement is 5% to 6% of the pile diameter, and its coefficient of variation is approximately 40% to 60%. A design horizontal threshold displacement can be also proposed as 2% to 4% of the pile diameter, considering the variation in elastic limit displacement.

Introduction

The advent of performance-based design specifications for bridges occurred in 2002 with the Japanese Specifications for Highway Bridges (Japan Road Association, 2002). The required bridge performances are clearly described for Normal Situation, Frequent Earthquake Situation, Rare Earthquake Situation, and so forth. The limit states of structural components are also called for, so that integrating the components that are verified for their limit state criteria can be considered to achieve the required performance of total bridge systems.

The required bridge performance for the Normal Situation and Frequent Earthquake Situation is such that highway bridges have to be fully functional without changing the way they respond to loads, and the corresponding limit state for foundations is the elastic limit. Verification is still implemented using a traditional allowable stress design format. Sectional stress in shafts must not go beyond allowable stresses from the structural viewpoint, and a load effect at the pile top must not exceed the allowable bearing capacity. In the current allowable stress design, safety factors are set considering both serviceability and safety. Now the Specifications are being revised toward implementation of the load and resistance factor design (LRFD) format with a reliability design concept. Therefore, this study considers that the elastic limit belongs to the serviceability limit state, and deals with the horizontal soil resistance to piles. The ultimate limit state design, to keep a safety margin against ultimate resistance capacity, will be studied elsewhere.

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Horizontal soil resistance is nonlinear, in reality, even from a very small displacement level. However, if the pile displacement remains within a certain level and no notable residual displacement appears, it is expected that a steady horizontal soil resistance to piles is maintained and nothing is going to happen in service. Traditionally, an allowable horizontal pile bearing capacity is defined with a displacement at the design ground level. Before 1990, empirical threshold horizontal displacements were 10 mm in the Normal Situation and 15 mm in the Frequent Earthquake Situation in Japan. From 1990, the Specifications have described that the allowable horizontal displacement is the larger value of 1% of the pile diameter or 15 mm in both Situations. This is based on Okahara et al. (1991a, 1991b) in which they collected in-situ test data of piles subjected to horizontal loads, and examined the elastic limit displacements on the observed load-displacement curves.

Also, 1% of the pile diameter may be considered reasonable from the practical viewpoint, because the typical design calculation method is conceived to apply only to small displacement. However, we cannot understand whether it was soil resistance or shaft resistance that attributed to the elastic limit points on load-displacement curves derived in the past study. Structural verification is separately implemented in design and the threshold displacement should be a function of the soil resistance, not the shaft resistance. In addition, larger diameter shafts and stronger materials are now available. A new nonlinear design calculation model has been recently introduced along with the advent of a new seismic design of foundations in the Rare Earthquake Situation. Technically they can calculate the foundation response of larger displacement levels to take advantage of shaft strength, while traditional values have been seen as a barrier against newer technologies.

This study attempts to re-examine the elastic limit of horizontal soil resistance using a PWRI database for field tests of piles subjected to horizontal loads. First, we propose an interpretation to extract the elastic limit of soil resistance from the load-displacement curve measured in testing. Then, a number of field test data sets are consistently analyzed to illustrate the statistical characteristics of the elastic limit displacement in terms of soil resistance, so that we can furnish model statistics for reliability calibrations. Finally, this paper attempts to suggest a revised design threshold horizontal displacement of piles under the reliability design concept.

Behavior of Horizontally-Loaded Piles

A typical nonlinear behavior of horizontally loaded piles is shown in Figure 1. Nonlinearities in total pile resistance are attributed to the nonlinearities in the soil and shaft resistances as indicated with the labels of A and X in Figure 1. Accordingly, it is necessary for shafts not to become plastic, so that the elastic limit in soil resistance marked with A in Figure 1 can be identified from a total load-displacement curve. Finally, the load test data should accord with the following requirements to identify the elastic limit point of soil resistance:

A. The load-displacement curve has clear nonlinearity.
B. The shaft can be assumed not to become plastic up to a large displacement level.

As shown in Figure 2(a), in which R = total pile resistance (or measured load) and d = displacement, it is assumed that the total pile resistance is described by superposing the soil resistance, R_{soil}, and the shaft resistance, R_{shaft}.

$$R = R_{\text{soil}} + R_{\text{shaft}} \tag{1}$$

Dividing both sides of Eq. (1) by a displacement of d leads to the corresponding instantaneous secant stiffnesses, K_{total}, K_{soil}, and K_{shaft}, of the total resistance, soil resistance, and shaft resistance, respectively.

$$K_{\text{total}} = K_{\text{soil}} + K_{\text{shaft}} \tag{2}$$

This study estimates the resistance components of soil and shaft using a beam-on-Winkler foundation model. When an instantaneous subgrade reaction coefficient is assumed to be constant over the whole depth and renewed with a step-by-step load increment, the governing equation can be described as

$$EIy'''' + k_i By = 0 \tag{3}$$

for a cross-section underground, in which E = Young's modulus of shaft, I = second sectional moment of inertia of shaft, k_i = instantaneous subgrade reaction coefficient at an i-th loading step, and y = deflection of shaft. For horizontal load H applied at a height of ground level to a free-head pile that is long enough, solutions to Eq. (3) in closed form are shown in many design specifications and text books like the Japanese Specifications for Highway Bridges (2002) and Poulos and Davis (1980). For example, the secant rigidity, K_{total}, for the total load-displacement curve at a load of R may be derived as

$$K_{\text{total}} = 2EI\eta^3 \tag{4}$$

and a depth of L_0 of the first zero-deflection point from the ground level, O, that is indicated in Figure 2(b) may be estimated as

$$L_0 = 0.5\pi/\eta \tag{5}$$

in which $1 / \eta$ = the characteristic length of pile, and $\eta = [(k_iB) / (4EI)]^{1/4}$. Then, when the shaft resistance is assumed equal to the horizontal resistance of a fixed-ended cantilever beam with a length of L_0 and a bending rigidity of EI, as delineated in Figure 2(b), the corresponding instantaneous stiffness of the shaft resistance is obtained as

$$K_{\text{shart}} = 3EI / L_0^3 \tag{6}$$

Using Eqs. (4), (5), and (6), the ratio of K_{total} to K_{shaft} is obtained by

$$K_{\text{total}} / K_{\text{shart}} = \pi^3 / 12 = 2.58 \tag{7}$$

Therefore, from Eqs. (1), (2), and (7), the following relationships may hold for any loading step.

$$R_{\text{soil}} = 1.58R / 2.58 \qquad \text{and} \qquad R_{\text{shaft}} = R / 2.58 \tag{8}$$

In conclusion, determining the elastic limit displacement on the total load-displacement curve is equivalent to interpreting the elastic limit displacement on the soil resistance-pile displacement curve. Based on Eq. (8), the behavior of the total, soil, and shaft resistances can be illustrated as Figure 2(c). Even though the shaft does not become plastic and has a linear bending rigidity, *EI*, the resistance carried by the shaft seems nonlinear. This is because the first zero-deflection point, O, shifts deeper as an increase in the applied horizontal load because of soil-pile interactions. Even if the point of loading is above ground level, similar solutions are available in the same theoretical background.

Interpretation of Elastic Limit Point

There is no specific definition to identify the elastic limit point of soil resistance from a load-displacement curve, and both mathematical and graphical approaches are available to determine the point at which the transition occurs from the initial linear portion to the second linear portion of a load-displacement curve (e.g. Hirany and Kulhawy, 1989). A mathematical model is employed herein, because it is considered more robust than graphical approaches. This study employs the Weibull curve fitting in reference to a study by Uto et al. (1985) and a study by Okahara et al. (1991a, 1991b).

$$\frac{R}{R_{uw}} = 1 - \exp\left[-\frac{d}{d_0}\right]^m \quad \text{or} \quad \frac{R}{R_{uw}} = 1 - \exp\left[-\frac{d/B}{d_0/B}\right]^m \tag{9}$$

in which R_{uw} = ultimate total pile resistance estimated via the Weibull fitting, d = displacement, d_0 = elastic limit displacement estimated in the Weibull fitting, B = pile diameter, and m = a constant that defines the shape of curve. Typical curves are shown in Figure 3. A load level corresponding to the elastic limit displacement is always $R_0 = R_{uw} \times (1 - e^{-1}) = 0.63R_{uw}$ for any m. If the ultimate horizontal soil resistance to the pile, $R_{u\text{soil}}$, could be given, a generalized soil resistance and pile displacement curve would be described as follows:

$$\frac{R}{R_{u\mathrm{soil}}} = 1 - \exp\left[-\frac{d/B}{d_0/B}\right] \tag{10}$$

with an assumption of $m = 1$. For the sake of simplicity, this study assumes $m = 1$ except for the cases of drilled shafts, because the assumption of $m = 1$ for drilled shafts seems to err based on a trial and error analysis. When it also comes to drilled shafts, the data points that exist under a displacement level of $0.01B$ are omitted in the Weibull fitting analysis. This is because a pseudo elastic limit could appear on load-displacement curves because of the development of cracks in cover concrete prior to the yield of longitudinal reinforcement.

<u>Selection of Single Pile Load Test Data</u>

For single piles, only in-situ tests are dealt with herein. The data adopted herein agrees to the following conditions:

1) The pile is as slender as it can, is assumed to have a semi-infinite length, and it is free-head and straight-side.
2) The height of point of loading from the ground level is lower than the pile diameter, B.
3) The maximum displacement level is larger than 5% of the pile diameter, B.
4) The observed maximum load in the load test is larger than 1.2 times the calculated elastic limit load, R_0, with the Weibull fitting.
5) The shaft is not considered to become plastic up to a load level of 1.2 times the calculated elastic limit load, R_0.

The items 3), 4), and 5) are associated with the requirements A and B mentioned above. A very simple method is employed to check about item 5) above. With the same theory as Eqs. (3) through (8), the maximum bending moment in the shaft at a load level of $1.2R_0$ is estimated using sort of a representative secant total stiffness, R_0 / d_0, of the total load-displacement curve.

Eventually, 37 data sets are available. The observed and fitted load-displacement curves are shown in Figure 4. The displacement at ground level is adopted if available, while the displacement at the closest point to the ground level is taken if that at ground level is not available. If both are not available, the displacement at the loading point is adopted in this study. When it comes to cyclic load tests, only the backbone curves are shown in Figure 4. A summary of the data sets used herein is shown in Table 1 and Figure 5 in terms of construction types and pile geometries. Approximately 81% of the employed data sets are associated with the category of non-composite piles (or typical piles) and the rest are associated with the category of composite piles. Driven piles, large-diameter screw piles, and inner-augured compressively-installed piles are classified as non-composite piles. Pre-boring piles, and steel pipe-soil cement piles are to be classified as composite. In the data selection process, all drilled shaft tests were excluded because of the violation of item 5) above.

Inner-augured compressively-installed piles are constructed as follows: (1) A pile is set, and an augur is put through the pile, making a hole with a diameter somewhat smaller than the pile diameter. (2) As the auger is moved down, the pile is simultaneously pushed into the ground. (3) The steps (1) and (2) are repeated again and again until the pile reaches a predefined depth.

Pre-boring piles are constructed as follows: (1) Soil is augured with a diameter of *B* and a predefined pile length. (2) Runny cement is mixed with residual soil in the augured hole. (3) A factory made high-strength prestressed concrete pile with a diameter somewhat smaller than *B* is placed into the augured hole. The soil cement made in step (2) rises up, and the space between the soil and pile is supposed to be filled so that a composite pile can be made.

Steel pipe-soil cement piles are constructed as follows: (1) A soil cement column with a diameter of *B* is constructed. (2) A pile with a diameter somewhat smaller than *B* is pushed into the soil cement column, so that it becomes a composite pile with the soil cement column part. The steps (1) and (2) are sometimes simultaneously conducted and sometimes separately conducted.

An augured diameter, *B*, is somewhat larger than the inserted factory-made pile diameter in pre-boring piles and steel pipe-soil cement piles. This study deals with the diameter of *B* as the representative diameter of these piles, because the soil cement part should respond together with the factory-made pile and it should be directly subjected to the soil resistance. The bending rigidity of shaft is put equal to that of the inserted factory-made pile.

Statistical Characteristics of Elastic Limit of Horizontal Soil Resistance

The elastic limit displacements estimated with the Weibull fitting, d_0, are plotted in the left panel of Figure 6 versus the pile diameter, *B*. A displacement level of 20 mm safely covers almost all data. However, there is a tendency for the value of d_0 to increase with an increase in the pile diameter, *B*. Therefore, a normalized displacement level, d / B, that is sort of a representative strain in soil in front of the pile is used to characterize the elastic limit. The relationship between the elastic limit displacement level, d_0 / B, and the pile diameter is shown in the right panel of Figure 6. The data distributes around the mean value of $0.565B$ with the coefficient of variation, COV, of 0.39, in which the coefficient of variation is obtained by a square root of $(\sigma^2 / (n - 1))$, in which σ = standard deviation and n = the number of load tests. The calculated normalized Weibull curve with the mean value of d_0 / B, 0.565, and $m = 1$ is also shown in the right panel of Figure 4.

The empirical distribution of elastic limit displacement level, d_0 / B, is shown in Figures 7 and 8. It should be noted that there are small variations in the number of tests cited herein because some test results in the database do not have adequate boring log data.

Based on Figure 7, it seems that the values of d_0 / B of composite piles tend to be somewhat smaller than those of typical non-composite piles. Composite piles had soil cement layer around the factory-made shaft. Therefore, the development of crack in the soil cement part during the loading may be considered to cause nonlinearity in the load-displacement curve even at smaller displacement levels, and the load-displacement curve may have a false elastic limit point. For example, all driven piles in the adopted data sets are steel pipe pile. As shown in Table 1, while the mean value of d_0 / B for the driven piles is 0.599, the mean value for steel pipe-soil cement piles is 0.356.

The relationships among the estimated elastic limit displacement level, d_0 / B, soil types, and soil stiffness or strength, i.e. SPT-N values are shown in Figure 8. For each pile load test, the predominant soil type depends on the subsoil layers within the characteristic length of pile, 1 / λ. The total depths of sandy subsoil layers and clayey subsoil layers are calculated, and the predominant soil type is based on the larger amount of the total depth either the sandy subsoil layers or the clayey subsoil layers. The subgrade reaction factor for calculating the characteristic length of pile is obtained with an empirical formula in the Specifications. The SPT-N value is the mean values of the measured values within the characteristic length of pile. The difference in the estimated elastic displacement level, d_0 / B, resulting from the difference in soil type or SPT-N value is little.

Graphical Method to Determine the Elastic Limit

Sometimes, a graphical interpretation is conceived more susceptible than a mathematical interpretation from the viewpoint of physical meaning, and accordingly, an alternative graphical method proposed by Okahara et al. (1991a, 1991b) is applied to confirm the above results. Figure 9(a) shows a cyclic load test result. From the test result, the sets of the residual displacement at an unloaded point of $R = 0$ and the preceding peak point displacement are extracted as [1], [2], [3], [4], and [5] as shown in Figure 9(b), and they are plotted in Figure 9(c). Figure 9(c) shows that a sudden change in the increment of residual displacement occurs at a peak displacement level of d_1. Therefore, this displacement of d_1 can be regarded as the elastic limit displacement, because, if the pile did not go beyond this threshold displacement, a notable residual displacement would not remain, and the initial soil resistance property still would be able to be mobilized.

The cyclic load test data are chosen from the load test data sets employed above, and the number of employed load tests, n, is eventually twelve. The elastic limit displacement level identified with the graphical analysis, d_1 / B, is compared with that estimated through the Weibull fitting, d_0 / B. For example, a Weibull curve fitted for the backbone curve is also shown in Figure 9(a), and it gives an elastic limit point as marked on the fitted curve. As shown in Figure 10, they agree with each other very well, and this confirms the fact that both methods are capable of accounting for the elastic limit of soil. The bias factor, $\lambda = d_1 / d_0$, has the mean value of 1.01 and the coefficient of variation of

0.45. In addition, the mean and COV of d_1 / B thereof interpreted via the graphical method are 0.048 and 0.39.

Design Threshold Displacement and Its Reliability

A general design equation can be expressed as

$$\text{Resistance capacity} \geq \text{Effect of loads} \tag{11}$$

As long as the resistance capacity is larger than the effect of loads, there is a margin of safety for the limit state under consideration. However, because both the resistance capacity and the load effect are considered to be random variables, factored resistance and loads are usually used in design to achieve a predefined safety margin. In terms of the soil resistance to horizontally loaded piles, Eq. (11) can be translated into

$$G = \phi\, d_L / B - d(\gamma\, Q) / B \geq 0 \tag{12}$$

in the Load and Resistance Factor Design (LRFD) format, in which ϕ = resistance factor, d_L = elastic limit displacement, γ = load factors, Q = loads, and d = calculated pile displacement in design with factored loads $\gamma\, Q$.

A crude estimation of the threshold value considering uncertainty in the elastic limit displacement is still better than that given in a deterministic way. Accordingly, this study shows a conditional reliability analysis, disregarding the statistic issues in the effect of loads, d, and conceiving the loads in the current Specifications as the deterministic factored values. Eventually, Eq. (12) can be rewritten as

$$G = \phi\, d_L / B - d(Q) / B = d_d / B - d / B \geq 0 \tag{13}$$

in which d_d = threshold displacement. The elastic limit displacement level, d_L / B, is assumed to follow a log-normal distribution. Hence, the First Order Second Moment (FOSM) method gives a relationship for the design threshold displacement level, $d_d / B = \phi\, d_L / B$, and reliability index, β, as follows:

$$\beta = \frac{\ln\left(\dfrac{1}{\phi}\sqrt{\dfrac{1}{1+\mathrm{COV}_R^2}}\right)}{\sqrt{\ln(1+\mathrm{COV}_R^2)}} = \frac{\ln\left(\dfrac{d_L / B}{d_d / B}\sqrt{\dfrac{1}{1+\mathrm{COV}_R^2}}\right)}{\sqrt{\ln(1+\mathrm{COV}_R^2)}} \tag{14}$$

in which COV_R = the coefficient of variation of the resistance capacity, d_L / B.

The mean and COV for the resistance capacity, d_L / B, can be estimated based on the results of the Weibull fitting analysis and graphical analysis. Although it appeared in

Figure 7 that there may be a difference in the uncertainties because of the difference in construction types, this study does not take this difference into account, assuming that the elastic limit in the horizontal soil resistance is not a function of the behavior of the soil cement part in composite piles. If the graphical interpretation may be considered more reasonable, the transformation error in elastic limit displacement between the Weibull fitting analysis and the graphical analysis should be incorporated into the mean and COV of d_L / B. Thus the mean and coefficient of variation of elastic limit displacement, d_L / B, are $d_L = \lambda \times 0.565 = 0.057$ and $\mathrm{COV}_R = (0.39^2 + 0.45^2)^{0.5} = 0.60$. Accordingly, based on all the results obtained above, the values of the mean and COV of d_L / B are likely to place somewhere in the range 0.05 to 0.06 and 40% to 60%. Eventually, this study sets the values of the mean and COV of d_L / B to be 0.055 and 50%.

An estimation of a target reliability index, β_T, is attempted hereafter. As mentioned above, the allowable horizontal displacement in the current Specifications is based on Okahara et al. (1991a, 1991b). They have adopted an equivalent theory to Eq. (14) and considered a reliability level of approximately 1 with the load test data compiled by them. Accordingly, the target reliability level should be set around that value. In addition, a typical safety margin associated with design of shallow foundations and piles subjected to vertical loads can assist in the calculation. Safety factors of $\mu = 3$ and 2 have been applied to the ultimate bearing capacity to estimate the allowable bearing capacities in the Normal Situation and Frequent Earthquake Situation, respectively, and the allowable bearing capacities are considered to be involved with both serviceability and safety in the current Specifications. The idea herein is that equivalent safety factors should have been applied to horizontal soil resistance. A generalized load-displacement curve can be expressed as

$$R / R_{u\mathrm{soil}} = 1 - \exp[-(d / B)/(d_L / B)] \quad \text{and} \quad R_L = 0.63 R_{u\mathrm{soil}} \tag{15}$$

in which R_L = soil resistance at the elastic limit load corresponding to the elastic displacement capacity, d_L. Therefore the safety factors of $\mu = 3$ and 2 to the ultimate horizontal soil resistance are translated into the safety factors of $\mu' = 1.89$ and 1.2 to the elastic limit soil resistance, R_L, respectively. In addition, because the resistance factor ϕ is equal to $1 / \mu'$, the safety factors of $\mu' = 1.89$ and 1.2 correspond to the resistance factors of $\phi = 1 / 1.89$ (= 0.53) and $1 / 1.2$ (= 0.89). Therefore, Eq. (14) leads to the design threshold values and reliability levels. The design threshold displacement level, d_d / B, at a resistance factor $\phi = 0.53$ is estimated as 0.022 with a reliability level $\beta = 1.67$ and that at a resistance factor of $\phi = 0.89$ is estimated as 0.038 with a reliability level $\beta = 0.54$. Based on these results, the target reliability indices can be eventually rounded to be $\beta_T = 1.5$ and 0.5 in the Normal Situation and Frequent Earthquake Situation. The corresponding values of design threshold displacement level, d_d / B, are obtained as 0.024 and 0.039, respectively.

As it turns out, we can expect at least $0.02B$ in the Normal situation and $0.035B$ in the Frequent Earthquake Situation as the rounded design threshold displacements for the serviceability limit state.

Horizontally-Loaded Pile Groups

While the results shown above are associated with single piles, pile foundations are usually comprised of several piles with a closed spacing between neighboring piles equal to 2.5 to 3.5 times the pile diameter. Accordingly, a rough analysis for the elastic limit displacement of pile groups is conducted for reference using the Weibull fitting. Eventually, a drilled shaft group and nine steel-pipe-pile groups are adopted. To make the amount of data as large as possible, laboratory test data are also used as well as in-situ test data. The adopted data sets accord with the following conditions:

1) the pile group was comprised of several pile rows in both parallel and perpendicular directions to the loading direction,
2) the observed maximum displacement level is larger than 5% of the pile diameter,
3) the observed maximum load in the load test is larger than 1.2 times the calculated yield load, R_0, with the Weibull fitting.

The relationship between the elastic limit displacement level, d_0 / B, estimated with the Weibull fitting and the pile diameter, B, is shown in Figure 11. The solid and dash lines indicate the threshold displacements in the Normal Situation and Frequent Earthquake Situation that are proposed above, and those threshold displacements cover most data on the safety side. This result supports the application of the proposed threshold displacements to general design for pile groups having a closed pile-to-pile spacing.

Concluding Remarks

In accordance with the 2002 Japanese Specifications for Highway Bridges, when the soil resistance to piles behaves within the elastic limit, the piles have a reversible restoring soil resistance force, and are expected to be firmly supported in years against service loads and frequent earthquakes. Accordingly, this study examined the statistical issues for the elastic limit displacement. The following results are clarified in this study.

1) This study has proposed a simple method to identify the elastic limit of soil resistance from the load-displacement curve in the horizontal load test of a pile.
2) When the elastic limit displacement is interpreted consistently from a number of field load test results, it appears that the mean value is 4% to 6% of the pile diameter with a coefficient of variation of 40 to 55%.
3) We have found that design threshold displacements at the ground level can be set to approximately 2% to 4% of the pile diameter for the serviceability limit state in the Normal Situation and the Frequent Earthquake Situation. These values are generally larger than the past values of $0.01B$ and 15 mm for a typical range of pile diameters in highway bridge foundations.
4) The proposed threshold displacements also can be considered relevant to the design of pile groups.

As a remaining issue, we further need to examine the ultimate limit state criteria for horizontally loaded piles to complete the new design specifications. In addition, we need to study the behavior of the soil cement portion of steel pipe-soil cement piles and pre-boring piles to elaborate the serviceability limit criteria.

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Table 1 Numbers of load test results in terms of pile construction methods for single piles

Construction methods		# of data sets	Mean of d_0 / B*	COV of d_0 / B*
Non-composite (Typical)	Driven pile	21	0.0599	0.39
	Large-diameter screw pile	7	0.0693	0.22
	Inner-augured compressively-installed pile	2	0.0515	0.23
Composite	Steel pipe-soil cement pile	6	0.0356	0.28
	Pre-boring pile	1	0.0329	–

* d_0 = Elastic limit displacement obtained with the Weibull fitting analysis

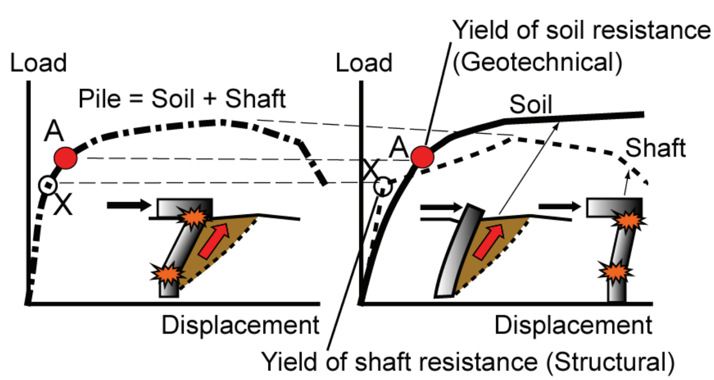


Figure 1 Typical nonlinear behavior of horizontally-loaded piles

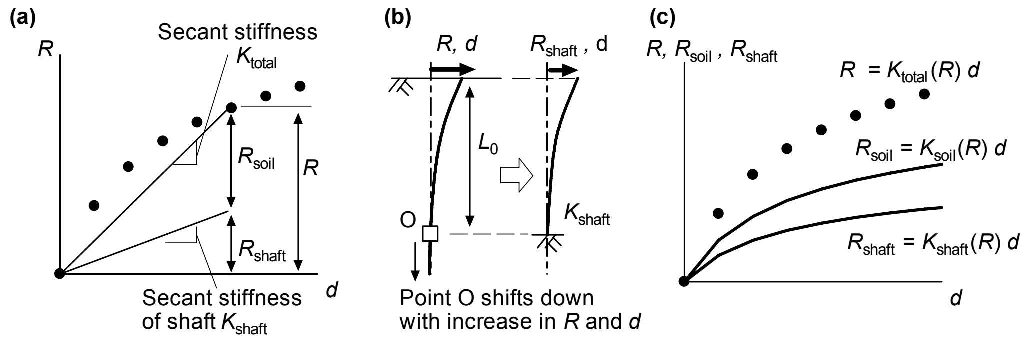


Figure 2 Decomposition of total pile resistance into soil resistance and shaft resistance

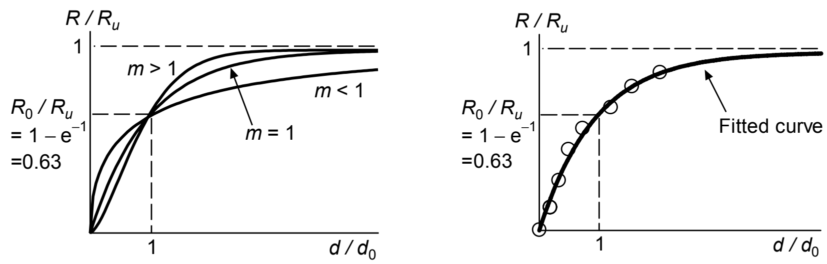


Figure 3 Weibull curve

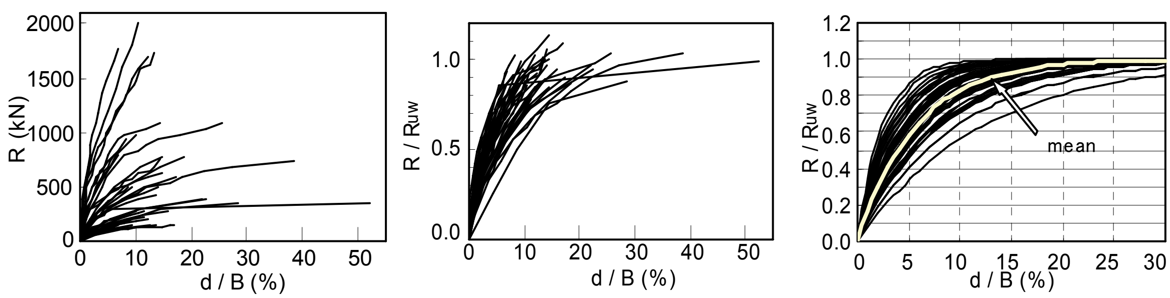


Figure 4 Observed, normalized observed, and fitted load-displacement curves

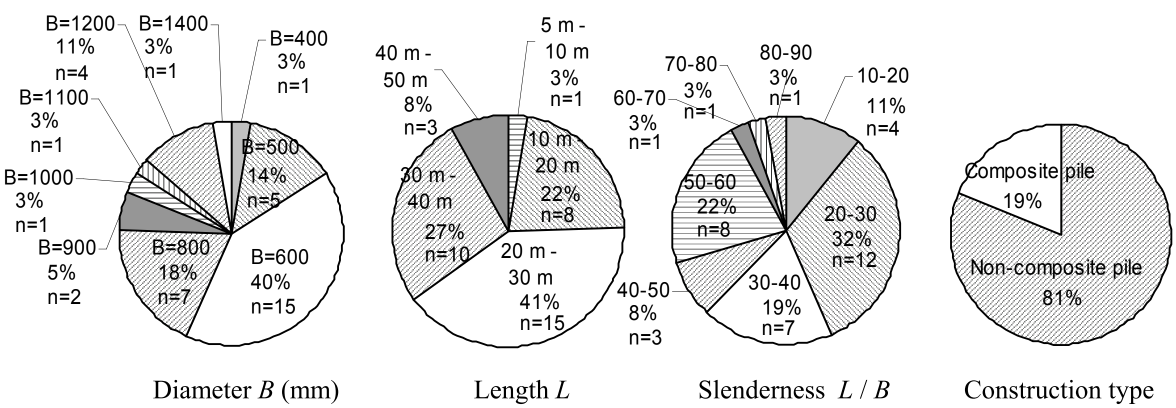


Figure 5 Frequencies in pile geometry and types of construction

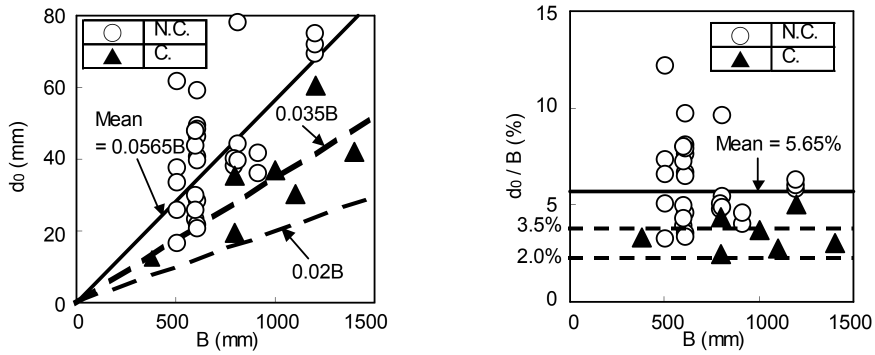
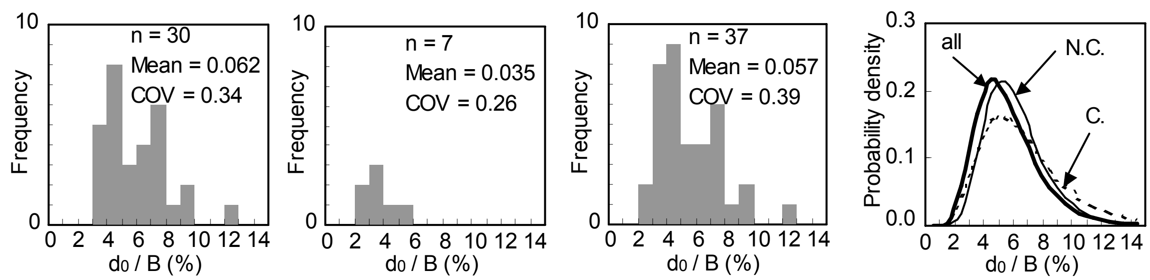
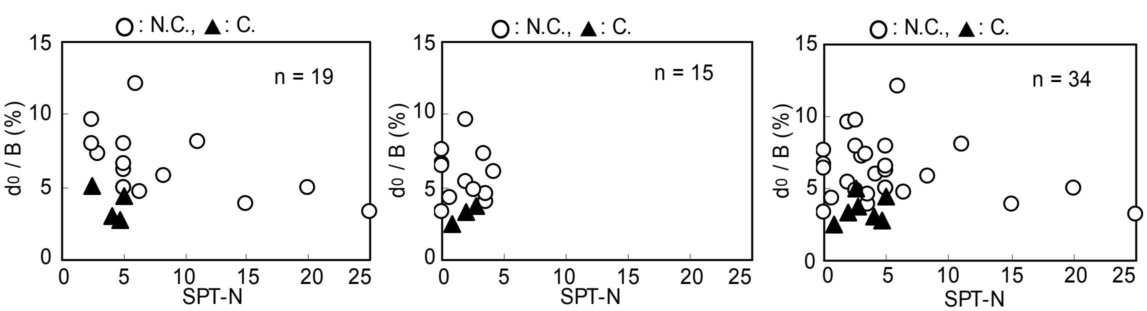


Figure 6 Relationship between estimated elastic displacement level and pile diameter (N.C. = non-composite piles and C. = composite piles)



(a) Non-composite pile (b) Composite pile (c) All pile types (d) Probability density

Figure 7 Variation in elastic limit displacement level, d_0 / B, of horizontally loaded piles for each piling type (N.C. = non-composite piles and C. = composite piles)



(a) sand (b) clay (c) all

Figure 8 Relationship of elastic limit displacement level, d_0 / B, with soil type and SPT-N value (N.C. = non-composite piles and C. = composite piles)

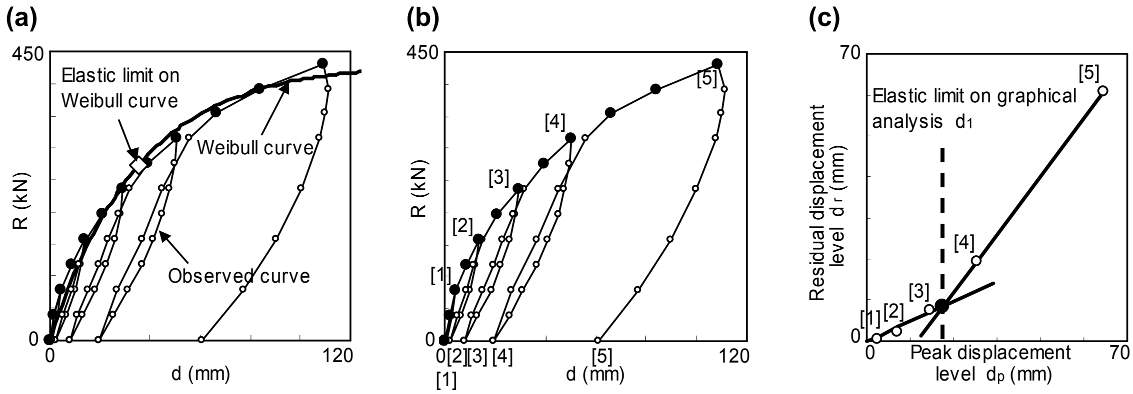


Figure 9 Graphical analysis for estimating threshold displacement in terms of rapid evolution of residual displacement in cyclic load test

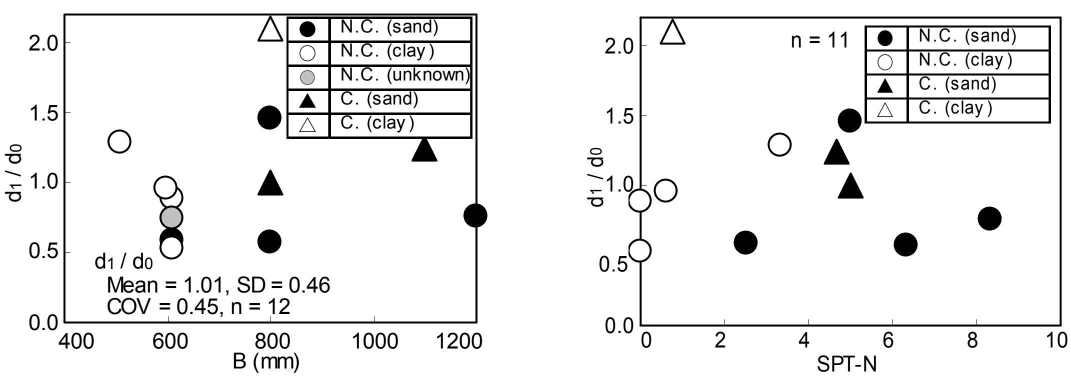


Figure 10 Comparison of elastic limit displacements, d_1 and d_0, obtained with the Weibull fitting and graphical analysis (N.C. = non-composite piles and C. = composite piles)

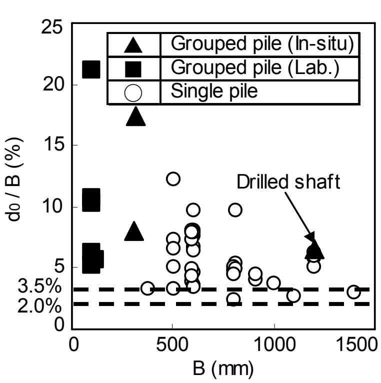


Figure 11 Elastic limit displacement obtained with the Weibull fitting analysis for horizontally-loaded pile groups