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SYNOPSIS The authers have proposed a method for evaluation of vertical bearing capacity of bored friction pile which might be capable to refrect the uncertainty of soil properties on the evaluation. The rationality of the method is examined from the application to the bridge design in this paper. A vertical bearing capacity of pile foundation is practically estimated by expressions with N-value, in which there are two kinds of uncertainties which depend on the N-values at the estimation points and the coefficient of the bearing capacity expressions. It is, therefore, necessary to improve the accuracy in estimaing the bearing capacity that spatial distribution of N-values in the ground are predicted with a high accuracy and in-situ loading test results are refrected in the bearing capacity expressions.

INTRODUCTION

Where the vertical bearing capacity of a pile foundation is to be considered, it needs to be noted that the bearing capacity expression based on the N-values by the standard penetration test has two uncertainties, i.e; one deriving from the bearing capacity factor and the other deriving from the soil properties(represented by N-valves in this case). The uncertainty related with the bearing capacity factor can be reduced by conducting in-situ vertical loading tests of piles and by reflecting such test results on the evaluation of the bearing capacity factor. Also, uncertainty concerning the evaluation of soil properties can be reduced by conducting soil investigations at narrow spacing; however, it is not always possible to conduct such investigations at all the proposed locations of foundation. In such a case, prediction of the spatial distribution of soil properties by the probability theory provides an effective solution. The anthors have proposed a method of evaluating performance factors used in the factored resistance. This method takes into account uncertainties related with those soil properties (i.e; bearing capacity factors based on the loading test results and spatical distribution of soil properties) which are used in the assessment of factored bearing capacities of foundations supported by friction piles.

In this paper, the authors describe the case where the performance factor proposed by them was used for evaluation of the vertical bearing capacity of the foundation of a certain expressway bridge, and further discuss the factored resistance determined by the proposed performance factor and the allowable bearing capacity obtained by a conventional safety factor on a comparative basis in order to verify that the former capacity is more rational than the latter.

VERTICAL LOADING TESTS OF PILES

The bridge now under consideration is a 190m long hollow slab bridge composed of three prestressed concrete spans and six reinforced concrete spans. The foundations are supported on bored piles having a pile diameter of 1.2m. Fig.1 shows the soil profile and the embeded depth of the

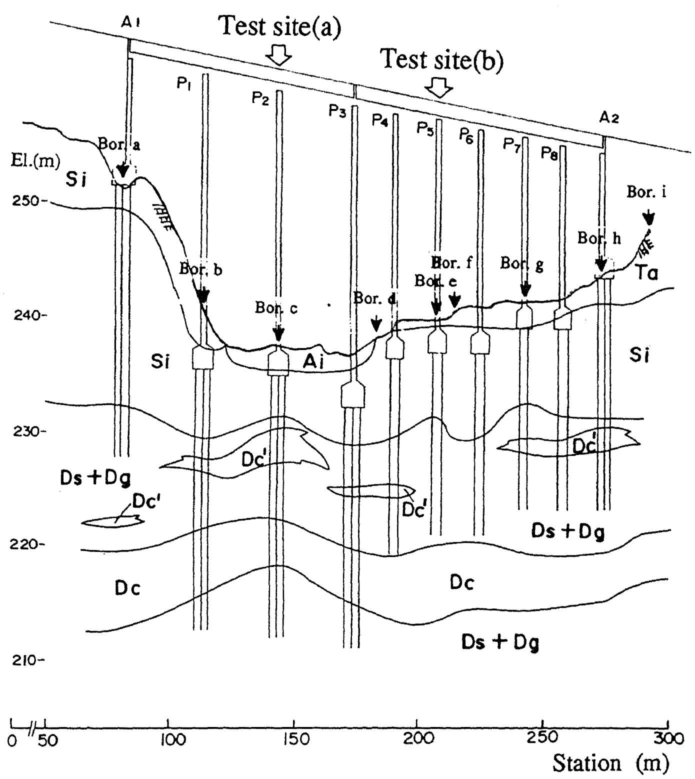

Fig. 1. Soil Profile and Embeded Depth of Piles

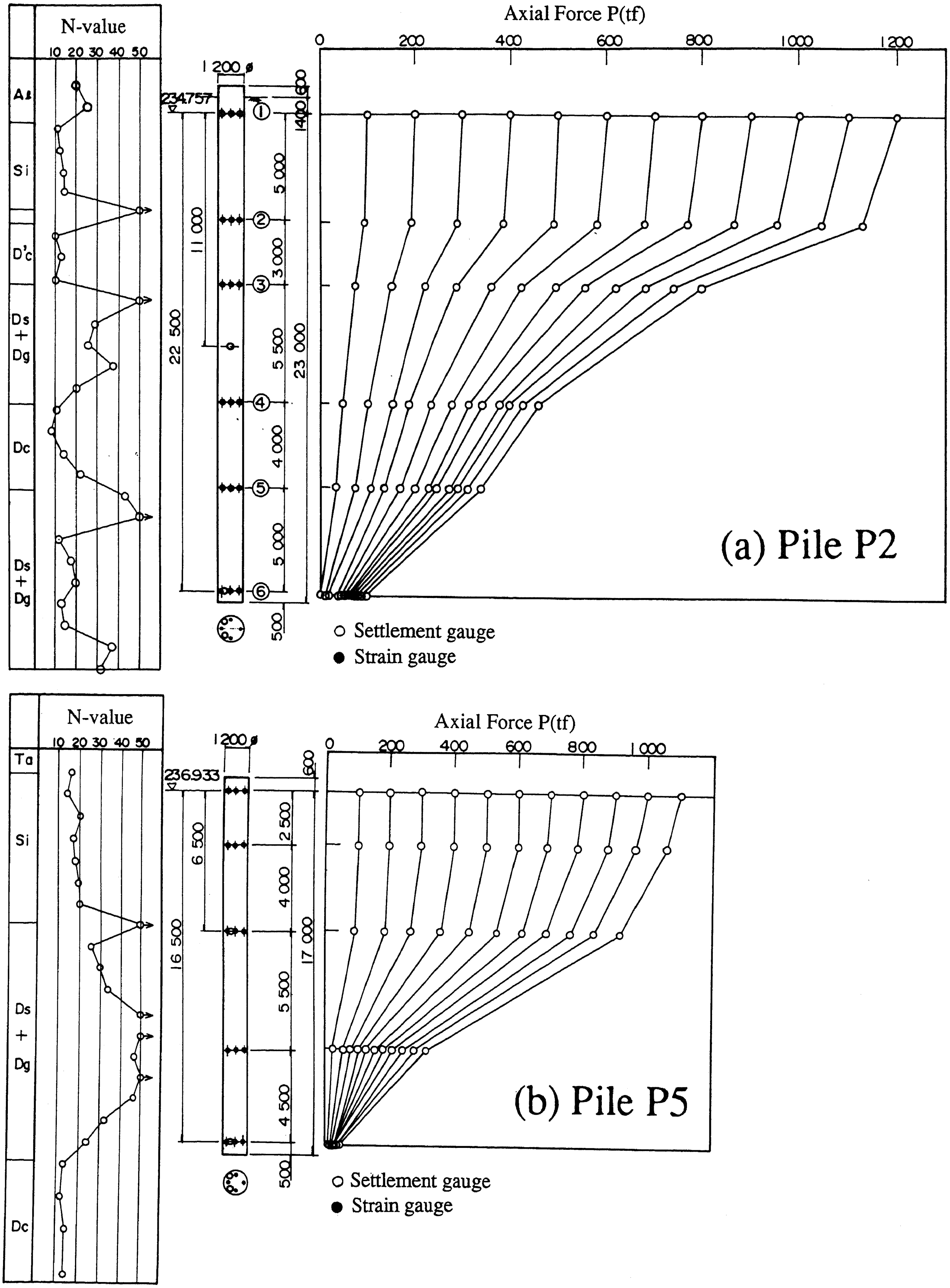

Fig. 2. Boring Logs and Axial Force Distributions of Test Piles

foundation piles below the suface of ground at the bridge site. The stratum Si(a tuffaceous sand layer of the quaternary deposit), which exists near the pile heads, is a submerged sedimentary stratum of the "Shirasu" soil which overlies a diluvial deposit. This Si stratum has N-values which are generally in a range of 11 to 26. The aforesaid diluvial deposit consists of two layers: one being a layer composed of alternating beds of sandy soil and gravelly soil (Ds+Dg) and the other being a clayey layer (Dc). This Dc layer consists of hard clay which generally has an N-value of 14. The layer is nearly horizontal and has a thickness of 4 to 5m with the upper limit of the layer located at E1.+220m. This specific layer is discriminated from a thin clayey layers (Dc') which is often intercalated in Ds+Dg. No reliable bearing stratum which has a substantial thickness and has also an N-value of 30 or over is found at the site. Hence, friction piles were used for the foundations of this bridge and the pile tip resistance was disregarded in view of the loading test results.

The piles differ in length depending on the loads imposed on them by the superstructures, and are divided into Group P1 to P3 and Group P4 to P8 by the length. While the piles in the former group rest on Layers Ds + Dg which are at lower levels, those in the latter group have their bottoms at levels above Layer Dc. In order to ascertain how the bearing capacities of friction piles are influenced by such difference in pile length and subsurface soil composition, two piles, P2 and P5, having different lengths were subjected to the in-situ vertical loading tests (with the maximum loads(Pmax) of 1,200 tf for P2 and 1,100 tf for P5). Fig. 2 shows the boring logs and the distributions of axial force in direction of the depth. These results apparently indicate that Layer Dc' gives large pile shaft resistance and that the friction piles have very small tip resistance.

Fig. 3 shows the relation between the head load (Po) and the normalized settlement (So/D, D: pile diameter) of test piles. The bearing capacities of the piles turned out to be much larger than were expected before hand and thus the pile settlement under the maximum loading ,Pmax, did not exceed about 2% of the pile diameter in any of the piles. As clarified by Okahara et al. (1990), the ultimate bearing capacity of a bored pile is exhibited generally when the pile settlement equal to about 10% of the pile diameter has been caused. If the pile bearing capacity at that time is estimated by the Weibull curve as proposed by Uto et al. (1985), it is presumed that the ultimate bearing capacity will be Ru=1700 to 1900tf approximately for the both piles.

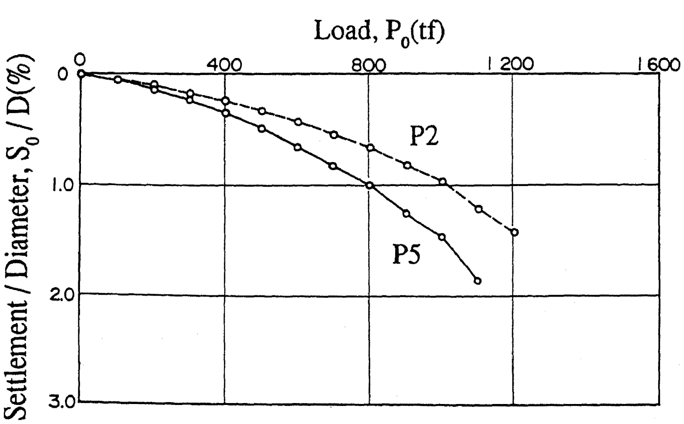

Fig. 3. Load–Normalized Settlement Curves of Test Piles

Fig.4 shows the relation between the normalized shear resistance ($\tau/\bar{N}$) and relative settlement ($\bar{S}$) along the pile shaft for each type of strata. In this relation, $\tau/\bar{N}$ is a shear resistance (τ) divided by an average N-value ($\bar{N}$) for the soil layer under consideration and $\bar{S}$ is a relative settlement of the pile and the ground in that soil layer. Si and Ds+Dg showed the similar $\tau/\bar{N}$ — $\bar{S}$ curves for both piles P2 and P5 . The strata Dc and Dc' were encountered by pile P2 only. The bearing capacity factor for Dc , which is a stratum that spreads continuously and extensively at a level below El.+220m is conspicuously different from that for Dc', intercalated in Ds+Dg.

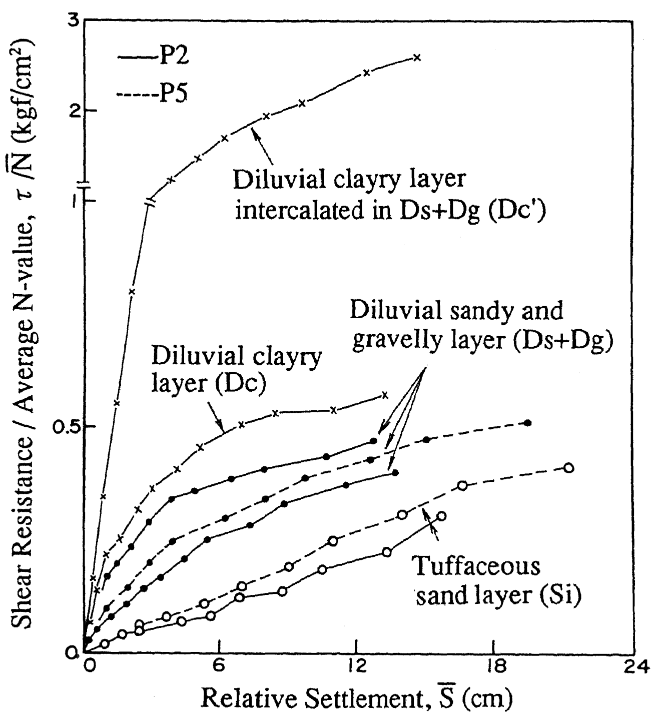

Fig. 4. Normalized Shear Resistance — Relative Settlement Curves for Each Type of Strata

Table 1. Bearing Capacity Factors

Soil	Pile	Average $\bar{N}$	Unit shaft resistance f (kgf/cm²)	Bearing capacity factor α
Si	P2	13.0	3.97	
	P5	17.9	7.39	.413
Ds+Dg	P2	41.0	16.3	
	P2	27.8	13.0	
	P5	45.5	23.2	.511
Dc'	P2	11.0	29.0	2.63
Dc	P2	13.8	7.91	.573

Table 1 indicates the bearing capacity factors for various strata as arranged for presentation in a tabulated form. Since it is known from Fig. 4 that τ mearly reached the peak, τ at the time of the maximum loading may be taken as an unit shaft resistance of pile, f. A plurality of bearing capacity factors were obtained for Si and Ds+Dg; however, since all of them are characterized by similar $\tau/\overline{N}-\overline{S}$ relation, the value for pile P5 which gave a larger $\overline{S}$ value was adopted for the purpose of design. While the test results for the sandy strata (Si and Ds+Dg) nearly conform to the values, 0.5, specified in Specifications for Substructures (1990),those for the clayry strata (Dc and Dc') depart conspicuously from the values, 1.0, given in the said Specifications.

ESTIMATION OF SPATIAL DISTRIBUTION OF N-VALUES

The spatial distribution of N-values by the concept of the sample field is expressed by the mean value and the variance of N-values and also the auto-correlation coefficient indicating the correlation between two points. What is known as Kriging technique is one of such estimating method. Journel and Huijbregts (1978) proposed a method in which the distribution was estimated by multiplying the sample values for the sample point (i.e; a known point of soil investigation) by the weights obtained from the distance between the sample point and the estimation point.

Among the statistical properties of a sample field, a mean value and a variance can be readily obtained, but it is often difficult to estimate an auto-correlation coefficient,$\rho(\Delta x)$ expressed by Equation (1) because of a limited number of samples.

$$\rho(\Delta x) = \exp\left[-(\Delta x/A)\right] \qquad (1)$$

where, Δx:horizontal distance between the two points, A :correlation parameter in horizontal direction (m)

Based on their investigations on the spatial distribution of N-values in five types of soil strata in Japan, Matsui et al.(1991) clarified the relation between horizontal direction correlation parameter (A) of N-values and average distance of sample points ($\overline{L}$) shown in Fig. 5. It is known from this figure that A varies apparently depending on the spacing of soil investigation points, that A becomes smaller as the soil vestigation points are spaced out closer and that the value of A is independent of the types of soil strata. For the parameter A to be ture, it is reasonable to take the value obtained when $\overline{L}$ is 0 as the value of the parameter, and about 15m may be regarded as a commonly acceptable value.

Further, Matsui and Ochiai (1992a) verified the effectiveness of the Kriging technique in estimating the spatial distibution of

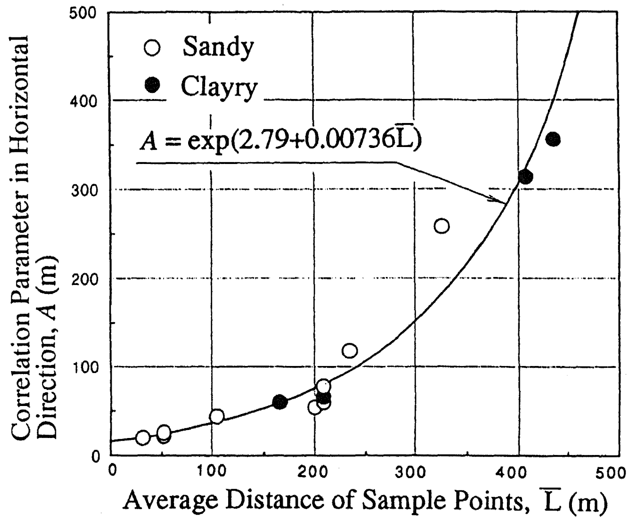

Fig. 5. Relation between Correlation Parameter of N-values and Average Distance of Sample Points

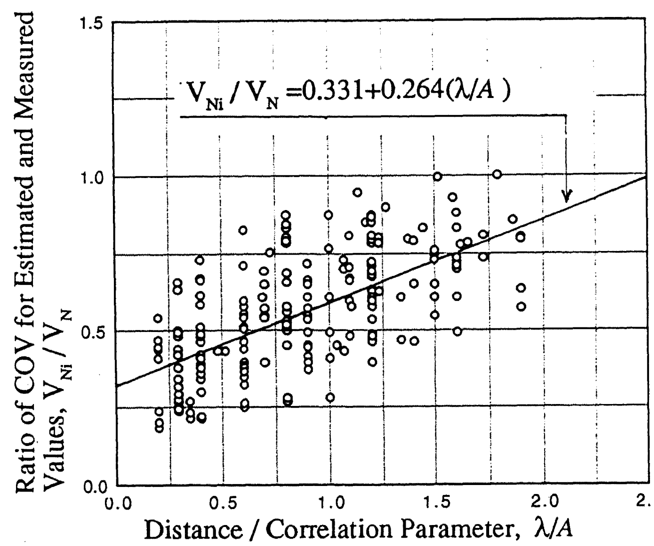

Fig. 6. Relation between COV Ratio and Normalized Distance

N-values and at the same time obtained a relation of $V_{Ni}/V_N-\lambda/A$ shown in Fig. 6. In this equation, V_{Ni} is a COV (coefficient of variation) of estimation for the N-values of the stratum at the point of estimation (i), V_N is a COV for N-values of the stratum under consideration and λ is a minimum distance between a sample point and an estimation point. There exists a relation shown in Equation (2) between V_{Ni}/V_N and λ/A , and where λ/A is 2.5, V_{Ni}/V_N is approximately equal to l. This means that if A is assumed as 15m, V_{Ni} agrees with V_N (i.e; a COV for the N-value of the stratum in question) when λ is about 40m. Futher, even if λ/A is zero, V_{Ni} becomes $0.331V_N$, which means that the N-value in this case is not free of uncertainty.

Thus, $V_{Ni}/V_N = 0.331 + 0.264(\lambda / A)$ (2)

Because a correlation between two points in horizontal direction prevails over that between widely spaced out sample points, the mean value of the N-values($\overline{N}_i$) at the point of estimation can be obtained analogically by connecting the sample values with a straight line. By combining this mean value of estimation with the COV of estimation by Equation (2), the spatial distribution of N-values can be brifly estimated.

Table 2 indicates the first and second order statistics of the N-values in each layer of the ground. The COV of each layer is 36 to 45% approximately and this is with in a normal range of N-value variability. Since it was not possible to obtain the

Table 2. First and Second Order Statistics of N-values

Soil	Number of Sample,n	Average $\overline{N}$	COV V_N
Si	85	18.0	.364
Ds+Dg	84	41.8	.452
Dc	39	11.2	.413

value of A for this ground because of the limited number of the samples, a value of 15m mentioned above was adopted. Fig. 7 shows the estimated spatial distibution of N-values of Si and Dc as obtained by the aforesaid statistical characteristics. In this connection, estimation errors were obtained by multiplying the COV of estimation (V_{Ni}) by the mean value of estimation ($\overline{N}_i$). The figure clearly shows that the estimation errors were small at the sample points and were large at the midpoint between the two sample points. Where no soil investigation was conducted between P2 and P5 as in the case of the Dc layer, estimation errors for any point in between inevitably become large.

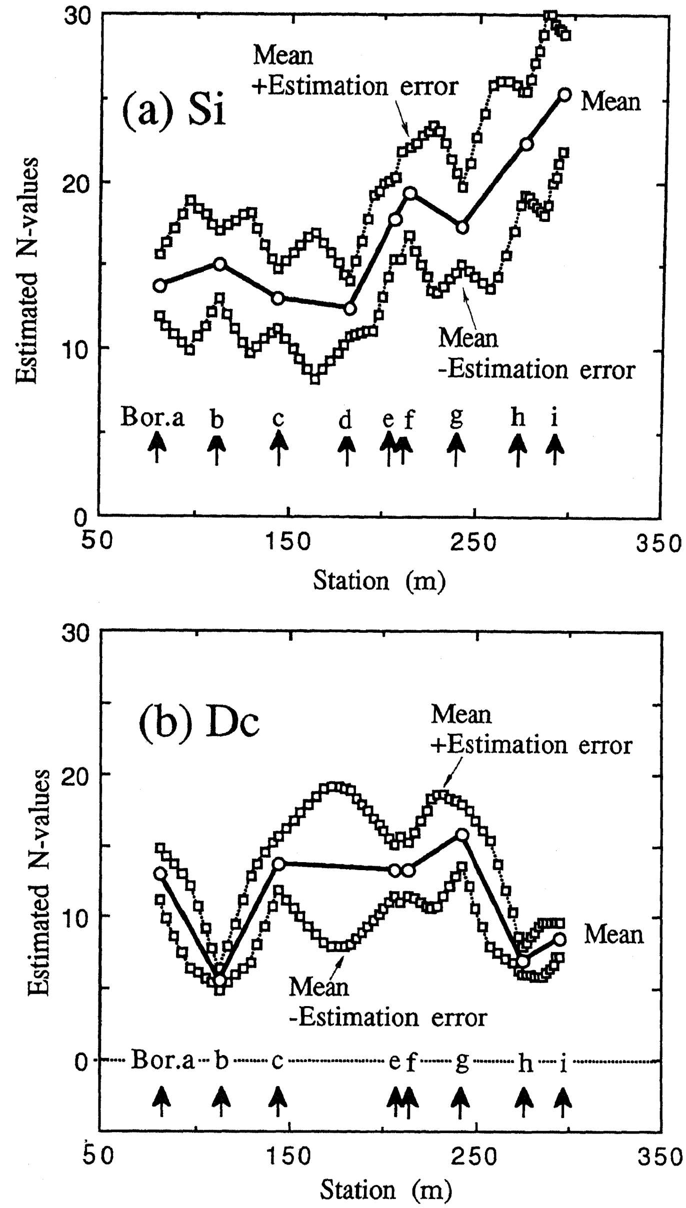

Fig. 7. Estimated Spatial Distribution of N-values

EVALUATION OF BEARING CAPACITY OF FRICTION PILES

where the pile tip resistance is disregarded, the ultimate bearing capacity in vertical direction of a friction pile (Ru) may be expressed by Equation (3).

$$Ru = U\Sigma \ell_i f_i = U\Sigma \ell_i \alpha_i N_i \quad (3)$$

Where, U: perimeter length of a pile (m), i:the identification number of the stratam along pile shaft, ℓ:thickness of stratum (m), f:unit shaft resistance of pile (tf/m^2), α :bearing capacity factor,and N:mean value of estimation

If in the right term of Equation (3), α and N are treated as random variables, then, V_R, which is the COV of Ru, can be expressed by Equation (4).

$$V_R = \sqrt{V_{R1}^2 + V_{R2}^2} \quad (4)$$

Where, V_{R1}: COV of a bearing capacity factor, and V_{R2}: COV of an estimated N-value

Let it be assumed that design criterion of an ultimate limit state at the pile head of a single pile may be checked by the following expression:

$$Ru / F_R \geqq F_S Pd \quad (5)$$

Where F_R and F_S are the resistance factor and the load factor,respectively, Ru and Pd are the ultimate bearing capacity of the pile and the nominal value (mean value in this case) of the load applied to the pile head, respectively. Now, if the performance function Z is equal to ln Ru-ln Pd, then, the resistance factor (F_R) and the load factor (F_S) can be related to the safety index β in the first-order and second-moment method by the following equation:

$$F_R=\exp\ (\alpha'\beta V_R) \quad \text{and} \quad F_S=\exp(\alpha'\beta V_S) \qquad (6)$$

Where, α': separation coefficient and V_S: coefficient of variation of the pile head load

If, in this case, the load factor can be considered constant, the load factor and the resistance factor may be arranged into one performance factor F_R' as given by Equation (7) below.

$$F_R'=F_RF_S=\exp(\beta\sqrt{V_R^2+V_S^2}) \qquad (7)$$

Therefore, a factored bearing capacity (Rf) in the limit states design method and an allowable bearing capacity (Ra) in the conventional allowable stress design method may be expressed by Equation (8) below by using Ru, a performance factor (F_R') and a safety factor (n).

$$Rf=Ru\ /\ F_R' \quad \text{and} \quad Ra=Ru\ /\ n \qquad (8)$$

With respect to the above equations, it should be remembered that whereas a safety factor is a conventional constant derived from the experience in the past, a performance factor is a function of the uncertainties related with loads and resistances and the safety index. Therefore one is basically different from the other. By reflecting loading test results and spatial distribution of N-values on V_R in Equation (7) by the probability theory, a performance factor in which uncertainty concerning the soil properties is taken into account can be obtained.

If a performance factor is to be established, a target of safety index β needs to be established. Based on the reseach results by Hoshiya and Ishii (1986) and by Yamada et al. (1983), β is taken here as 3 for the Ordinary state and 1.5 for the During earthquake. V_R can be expressed as a square root of V_{R1}^2 plus V_{R2}^2. According to Matsui and Ochiai (1992b), a COV of a bearing capacity factor (V_{R1}) and a COV of the estimated N-alues (V_{R2}) can be expressed by Equations (9) and (10), espectively.

$$_{1}=0.3\overline{\alpha} \qquad (9)$$

$$=(0.331+0.264\ \lambda/A)\ V_N \qquad (10)$$

ere, $\overline{\alpha}$:average value of bearing capacity factor

nce a COV of the ultimate bearing capacity of a pile) may be expressed by Equation (11). By introducing this o Equation (6), a performance factor (F_R') which takes into count the loading test values and the spatial distribution of -values can be obtained by Equation (12).

$$V_R=\sqrt{V_{R1}^2+V_{R2}^2}=\sqrt{(0.3\overline{\alpha})^2+(0.331+0.264\lambda/A)^2V_N^2} \qquad (11)$$

$$F_R'=\exp[\beta\sqrt{(0.3\overline{\alpha})^2+(0.331+0.264\lambda/A)^2V_N^2+V_S^2}] \qquad (12)$$

If F_R' in Equation (12) is calibrated into conventional safety factor (n) used in the conventional design method, the values 0.1 and 0.3 will be obtained for Ordinary state and During earthquake, respectivly, provided that λ/A is 0 and V_N is 0.4. Fig. 8 shows the relation between a performance factor and λ/A for a case where $\overline{\alpha}=0.5$. V_N in the figure is a COV of an N-value of each stratum. This figure indicates that a performance factor increates in proportion to

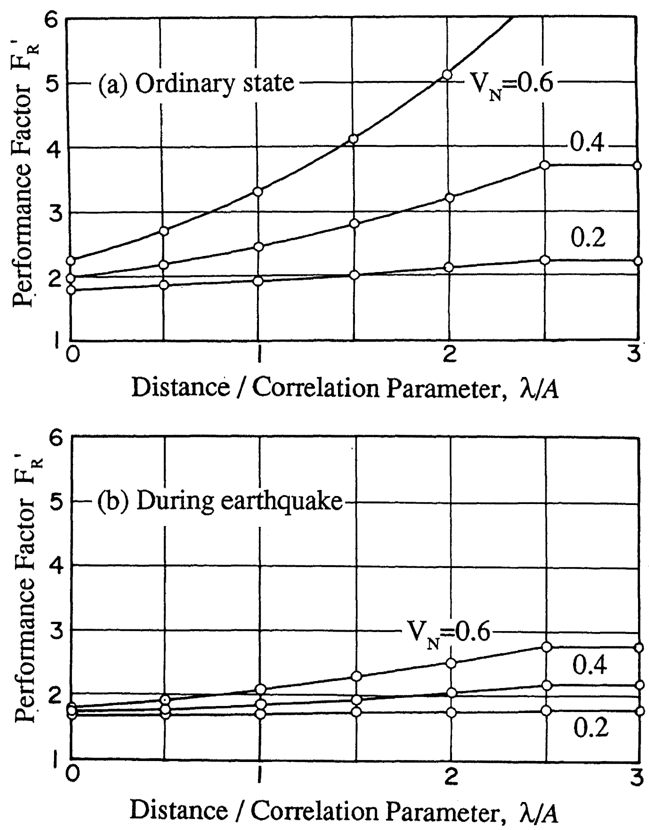

Fig. 8. Relation between Performance Factor and Normalized Distance (in case of $\overline{\alpha}=0.5$)

λ/A, and a rate of such increase is larger when the variability of N-values is larger.

The ultimate bearing capacity (Ru), the factored bearing capacity (Rf) and the allowable bearing capacity (Ra) for each foundation as obtained by Equations (3) and (8) are shown in Fig. 9. The safety factors used in computing allowable bearing capacities are taken as 3 for Ordinary state and 2 for During earthquake as suggested by Spcifications for Substructures (1990).

Since the allowable bearing capacity at Ordinary state is equal to the ultimate bearing capacity divided by 3 (i.e; Ru/3), the bearing capacity of each foundation is determined almost

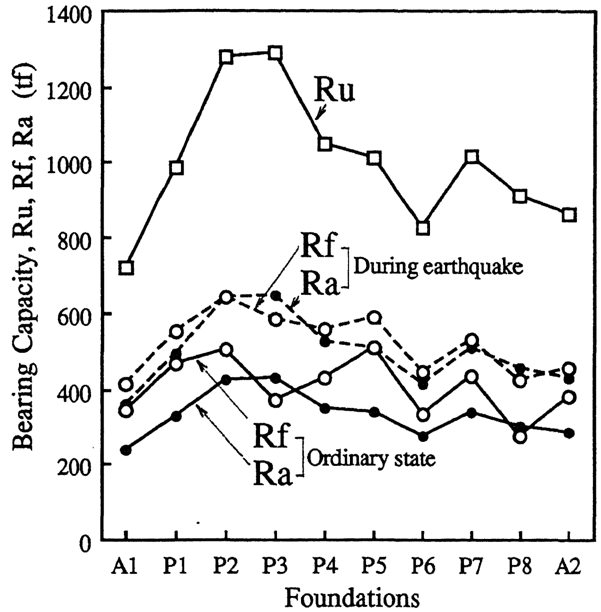

Fig. 9. Ultimate, Factored and Allowable Values of Bearing Capacity for Each Foundation

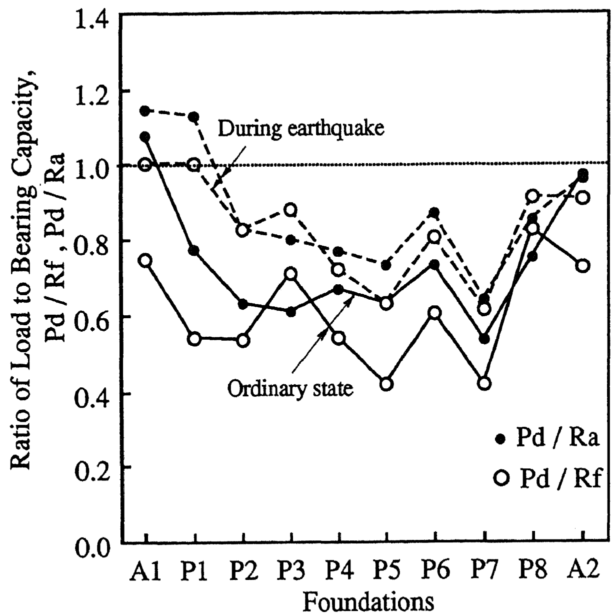

Fig.10. Ratio of Load and Bearing Capacity for Each Foundation

solely by the magnitude of its ultimate bearing capacity and is irrelevant to the uncertainty of the soil properties.

On the other hand, the evaluation of Rf is dependent on the uncertainty of the N-values of each foundation. For example, in the case of pile P3, Rf is given lower evaluation than Ra. This is because the N-values of Dc and Ds+Dc below it are estimated on the basis of the sample values at the positions of piles P2 and P5 and consequently these N-values are liable to large estimation errors which cause the performance factors of these strata to have high values. In the same way, pile P8 is assessed to have a factored bearing capacity that is smaller than the allowable bearing capacity because no soil investigation was conducted at the bridge foundation location in question. The same comment can also be made about the bearing capacity at During earthquake.

Fig. 10 shows the ratio of the load (Pd) to the bearing capacity (Rf or Ra) by the aforesaid two design methods. In this figure, Pd/Rf by the performance factor or Pd/Ra by the safety factor is taken on the ordinate. Where a performance factor which takes into account the uncertainties relative to the soil properties is used. Pd/Rf is smaller than one for all the foundations both at Ordinary state and During earthquake. Thus, it is believed that the use of the performance factors as proposed by the authors enables the safety of bearing capacity to be assessed in a more rational manner and makes it possible to achieve more economical design.

CONCLUSIONS

From what has been described so far, the conclusions may be summarized as follows:

(a) The bearing capacity factors for the ground under consideration were obtained by the in-situ vertical loading tests and compared with those by Spcifications for Substructures' expressions. As a result, it was found that one differed from the other pronouncedly in case of clayry soils.

(b) The spatial distribution of the N-values of the aforesaid ground was estimated by the statistical characteritics of N-valves and the estimation method proposed by the authors. Through this process, it was clarified that the uncertainties of the estimated N-values were apparently affected by distance of the soil investigations.

(c) A method which took into account uncertainties of the soil properties was proposed for evaluation of performance factor, and this method was applied to a certain bridge foundation. In consequence, it was ascertained that the factored bearing capacity by the performance factor proposed by the authors was more rational than the bearing capacity by the conventional method; hence, the effectiveness of the proposed method was confirmed.

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