PERFORMNCE BASED DESIGN OF BRIDGE FOUNDATIONS

Makoto Suzuki[1] and Masahiro Shirato[2]

ABSTRACT: The aim of this paper is to discuss issues based on foundation design in conjunction with partial factor format for the future development of foundation design codes. That the resistance factor approach is superior to the material factor approach in practical terms for foundation design is concluded. Then, a common procedure from geotechnical investigation/ testing to determine the design geotechnical parameters is proposed, paying particular attention to the evaluation of characteristic values. Finally, inherent problems in code calibration of foundation structures based on our experience are pointed out, and some improvements to the calibration procedure are suggested.

1. INTRODUCTION

Recent design codes are based on the concept of performance based design for structures. For example, ISO2394[1)] which was developed based on this concept, specifies general requirements to be verified for ultimate and serviceability limit states and proposes verification formats for partial factors. While comprehensive guidelines for design code development based on reliability are certainly given in ISO2394, pertinent design codes based on our own background of design practice must be created reviewing them carefully.

Design codes for foundations have also moved toward the introduction of reliability-based design, and the revision work for the next version of the Japanese Specifications for Highway Bridges[2)] has been implemented in order to introduce reliability-based design. Introducing reliability-based design is one of the most important themes in Japanese codes, as well as accelerating performance-based design[3)].

In this paper, the problems related to the following three issues based on our revision work are discussed:

(1) Determination of the characteristic value of a geotechnical parameter,
(2) Pros and cons of the material factor approach and the resistance factor approach,
(3) Implementation of code calibration of partial factors for the design of pile foundations in Japan including seismic design.

Through discussion, one of the realistic ways to accelerate the introduction of reliability-based design to foundation design codes is demonstrated, which would be welcomed by ownership authorities and practical designers for civil engineering structures.

2. PILE DESIGN METHOD ADOPTED IN THIS PAPER

Regarding reliability in the current Specifications, Shirato et al.[3)] stated that most of the Specifications are not based on a probabilistic background; rather, they are based on the safety factor method in principle. For example, the allowable ductility factors of members including foundation systems consisting of ground and structural members are derived by dividing the values of the ultimate displacement by safety factors depending on the seismic performance levels, while the values of the specific return period for the design

[1] Institute of Technology, Shimizu Corporation, Tokyo, Japan
[2] Foundation Engineering Research Team, Public Works Research Institute, Tsukuba, Japan

earthquake motions are not elucidated. On the other hand, the capacity design concept is clearly employed in order to control the failure mode of bridges. An increasing factor for design loads is provided in the design of foundations, as will be mentioned later. In addition, the ratio of bending and shear strengths is based on capacity design.

Code development is highly dependent on background at the region and country levels. Accordingly, first we clarify the design verification that is adopted throughout this paper to make it easy to understand the discussions.

2.1 Estimation of geotechnical parameters

We deal with the design of foundations, especially grouped-pile foundations, in the present discussion. The determination of the values of geotechnical parameters differs considerably from that of the values of structural material parameters, as follows:

(1) We cannot know the values of mechanical parameters until ground exploration is performed on the construction site.

(2) Various geotechnical investigation methods are available for estimating the value of a geotechnical parameter of a subsoil layer, such as through laboratory tests and in-situ tests with empirical equations based on past data.

(3) Many geotechnical investigation methods and empirical equations are being developed even now.

(4) The reliability of estimated geotechnical parameter values changes depending on the soil test method used.

2.2 Ductility design of grouped-pile foundations

This paper adopts the design of grouped-pile foundations of highway bridges in Japan, especially for severe earthquakes. Design situations that must be checked are roughly composed of: normal situations, extreme wind situations, and dual-stage seismic situations of Level 1 and Level 2. Level 1 earthquakes are of small to medium magnitude, while Level 2 earthquakes are extremely strong, but are very unlikely to strike a structure during its service period. The Japanese Specifications for Highway Bridges recommend a ductility design method to verify seismic performance and to control damage to structures for Level 2 earthquake situations, in which both the capacity and ductility of the foundation are taken into account. Foundation design results are usually dominated by Level 2 earthquake situations and consequently, design code calibration for Level 2 can be seen as playing a key role in the introduction of reliability-based design.

Figure 1 shows a schematic diagram of the behavior of foundations subjected to seismic loads. The behavior of a foundation is checked by performing pushover analysis. As for pile foundations, nonlinear properties in terms of the bending of foundation members are considered by evaluating their moment-curvature relations. The vertical bearing resistance of a pile is modeled as an elastic-perfectly plastic bilinear spring having its yield points at the ultimate bearing capacity for compressive forces and the ultimate pullout capacity for tensile forces, respectively. The horizontal subgrade reaction is modeled with the use of a Winkler-type distributed spring having bilinear properties.

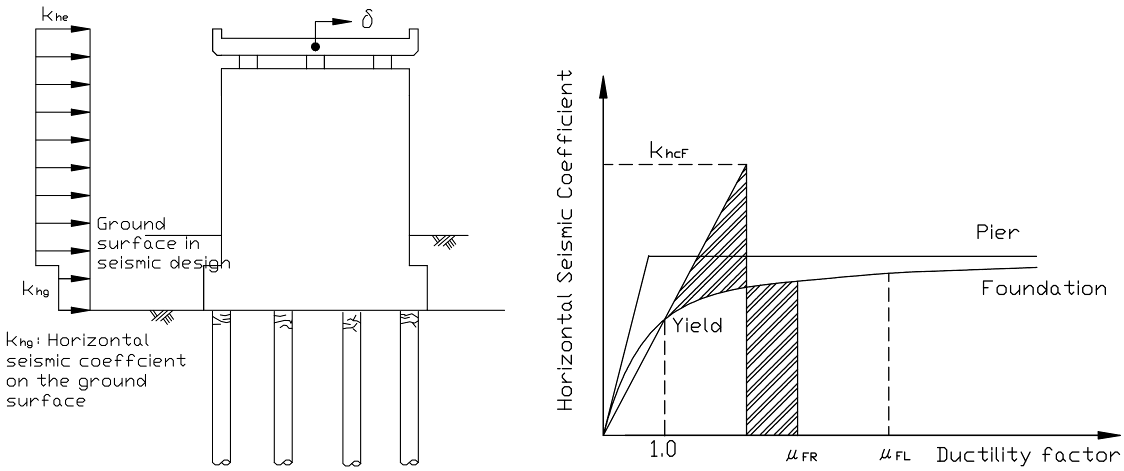

Fig. 1 Ductility design of pier foundations for verification in cases where foundations have sufficient ductility capacity

The yield point of the system behavior of a foundation can be derived from the relationship between the seismic coefficient, k_h, and the horizontal displacement, δ, at the point of seismic lateral load in the superstructure. Generally, the yield point in the system behavior of a highway bridge pile foundation with normal dimensions is characterized as follows:

- when all piles yield, or
- when a compressive reaction force at a pile top reaches the bearing capacity of the pile.

The Specifications regarding the ductility design for Level 2 earthquake situations stipulate that clear inelastic behavior of foundations as systems is not expected in principle, and that the main seismic energy dissipation should be expected at other structural sections and devices, such as the bottoms of piers.

However, it is necessary to consider the dominant inelastic behavior of a foundation for rare-scale earthquakes, especially when a pier ends up possessing a large capacity due to factors outside the seismic design process or when liquefaction of subsoil layers occurs. In this case, it has been verified that the foundation does not reach the limit state point on the ductile response at which damage to the foundation cannot be repaired, and large residual displacement of the foundation does not result in difficulty when reopening the bridge. The nonlinear response is estimated by the energy conservation method, and verification is implemented by use of the response and allowable ductility factors, μ_{FR} and μ_{FL}, respectively.

$$\mu_{FR} \leq \mu_{FL} \tag{1}$$

The ductility factor, μ, is defined by the following equation by dividing the displacement, δ, by the yield displacement, δ_y, at the point of seismic lateral load in the superstructure.

(2)

The allowable ductility factor, μ_{FL}, is recommended based on previous large-scale experimental studies such as experiments on grouped piles subjected to lateral and overturning moment cyclic loads, as shown in Fig. 1, and the allowable ductility factor of piers. Regarding the grouped-pile foundations, the allowable ductility factor, μ_{FL}, itself is deterministically specified to be four, because conventional numerical analyses cannot predict the onset of the post-peak behavior of grouped-pile foundations, i.e. the ultimate limit state. Note that this value of the allowable ductility factor, four, is also applied in practice when the foundation reaches the yield point due to mobilizing the ultimate compression bearing capacity of the leading piles. Regarding column-type foundations such as caisson foundations, a safety factor α is considered in determining the value of the allowable ductility factor, μ_{FL}, in order not to reach the ultimate displacement, δ_u, when a foundation sustains an ultimate curvature for maintaining bending strength, which is regarded as the onset of spalling of cover concrete in practice, as shown by the following equation:

$$\mu_{FL} = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y}, \tag{3}$$

From the viewpoint of implementing reliability-based design, the distinguishing features that should be carefully considered in this paper are summarized as follows:

(1) Consideration of the nonlinear behavior of foundations is not rare, but is a routine procedure.

(2) We deal with the behavior of a grouped-pile foundation as a system in the framework of displacement-based design. The performance of a foundation depends on both structural resistance elements and soil resistance elements, each having nonlinear properties. This means that both safety during an earthquake and serviceability after an earthquake are checked together in the design calculation, which differs from the conventional design involving two calculations through limit analysis and deformation analysis. On the other hand, most past studies on reliability design and design codes addressed only force-based design, in which the bearing capacities of soil resistances and strength capacities of piles are checked based only on comparisons of forces.

Advances in seismic design technologies of foundations have accelerated at a pace far beyond the presumptions of general code calibration tools after the disasters of the 1995 Hyogo-ken Nanbu earth-quake in Japan, and the 1989 Loma Prieta earth-quake and 1994 Northridge earthquake in the USA.

3. CHARACTERISTIC VALUES OF GEOTECHNICAL PARAMETERS

The problem of how to decide the value of a geotechnical parameter used for the design calculation is one of the most important issues in the design of foundations. As pointed out in Section 2.1, many options are available, such as laboratory tests and in-situ soil tests, for estimating the value of a geotechnical parameter, and the reliability of the estimated value changes depending on the use of the soil test method used. For the purpose of simplification, we limit the area of discussion to shear strength parameters of internal friction angle f and cohesion c for typical sand and clay as an example.

We can generally assume that the use of the values of ϕ and c corresponding to triaxial compression tests, instead of plane strain tests or direct shear tests, are adopted in standard design calculation methods and theoretical formulas in design books are based on these values. On the other hand, in practice, the values are usually estimated indirectly based on in-situ measurements of standard penetration tests, cone penetration tests and so on. Accordingly, while the engineer can freely choose the method, the reliability of each soil test method should be reflected on the design result, because some of the test options can directly assess a required geotechnical parameter, and some require a transformation model to obtain a required geotechnical parameter from the test results. Furthermore, others are suitable to a particular soil type, but are barely applicable to another soil type.

However, it is not reasonable to prepare the same number of sets of partial factors as the number of types of soil tests. This is costly and time consuming. It is also not a good idea to perform the code calibration only for a representative test method such as the standard penetration test, because there would be no incentive for the designer to employ other superior soil test methods.

A possible alternative method for incorporating the reliability of a soil test into the design result is as follows. First, we consider characteristic values to be the values input into the design calculation and considered as the representative average values on a subsoil layer. The reasons for using the average value as the characteristic value are as follows. Design codes should not force practical designers to predict the tail shapes of probabilistic distributions of soil test results. That is not an easy task even for researchers. In addition, designers can regard the calculated foundation response as shown in Fig. 1 as the most likely nonlinear behavior. We cannot account for the influence on the calculated load-displacement curve of Fig. 1 and judge the physical adequacy of the load-displacement curve, when we use an extremely underestimated value in the nonlinear calculation process.

Secondly, when the characteristic values are estimated based on a soil test without triaxial compression tests, the average value on the subsoil layer is obtained, but it is estimated in consideration of the estimation accuracy of the average value depending on the transformation model. As for the shear strength parameters of f and c, while the values of f and c based on triaxial compression tests can be directly used in the estimation of the characteristic value, the values obtained from other soil test methods should be adjusted beforehand for the degree of uncertainty relative to the values that could be obtained in the case of using triaxial compression tests.

Thirdly, the partial factors are calibrated only for cases where triaxial compression tests are used. Those calibrated partial factors are used independently of the kind of soil tests, since the difference in the reliability level of the soil tests has already been adjusted in the determination of characteristic values.

Figure 2 is a diagram showing the procedures up to the point of determining the characteristic soil resistance from geotechnical investigation results[4)].

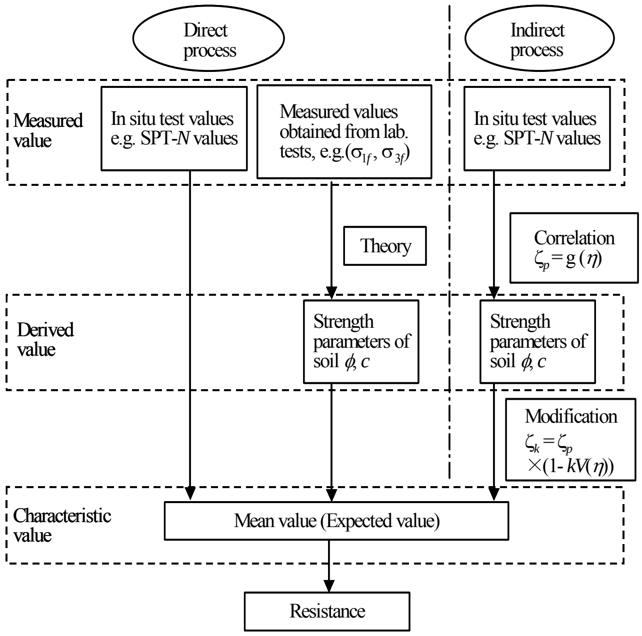

Fig. 2 Process for obtaining characteristic values of geotechnical parameters from geotechnical investigation results[4)]

3.1 Direct process

The method of estimating a characteristic value directly based on the measured values of soil tests is referred to as a direct process. For example, the values of f and c are derived based on a theory (such as a failure criterion with Mohr's circle) with principal stresses at failure in laboratory tests or in-situ soil tests for objective resistances. Engineers determine the representative values of f and c of a subsoil layer as the characteristic values based on the distributed derived values.

3.2 Indirect process

When in-situ soil test results are used for the design work, a transformation model is needed to relate a measurement result to a required geotechnical parameter. This is referred to as an indirect process herein. At this point it is necessary to have a basic agreement on the procedure from soil testing to de-termination of the characteristic value of a geotechnical parameter in the indirect process for achieving consistent reliability despite the difference in soil test methods.

There are many transformation equations for various in-situ tests, and they are obtained by empirical data fitting. In general, many transformation equations in design books are consciously given smaller than the simple regression relations, and the data scattered around the average relation is empirically taken into account. Statistically, this kind of reduction can be accounted for as follows:

(1) While regression analysis is used to estimate the average (or expected) relationship between the laboratory test values of f and c and in-situ measurement values,
(2) The accuracy of estimating the expected value is included in the transformation equation.

Based on this observation, a procedure can be proposed for obtaining the characteristic value of the soil resistance from the geotechnical investigation results in consideration of the accuracy of estimating the expected value.

When we derive a value of parameter *z* from a measurement *h*, we use the following relationship:

$$\varsigma_p = g(\eta) \tag{4}$$

$$\varsigma_k = \varsigma_p \times (1 - kV(\eta)) \tag{5}$$

Equation (4) is a simple regression equation for *z*-*h*, based on past data, and z_p is referred to as the derived value of *z*. Equation (5) yields the characteristic value of *z*, z_k, from the derived value zp. V(*h*) is the COV of the distribution of the data for Eq. (4), and k is an adjustment coefficient to incorporate the accuracy of the derived value relative to the derived value obtained in the direct process. The value of *k* can be derived by comparing current transformation equations to available measurement data.

For example, as far as foundation design practice of the Japanese Specifications for Highway Bridges is concerned, *k* = 1 seems relevant for ϕ and *c*[4].

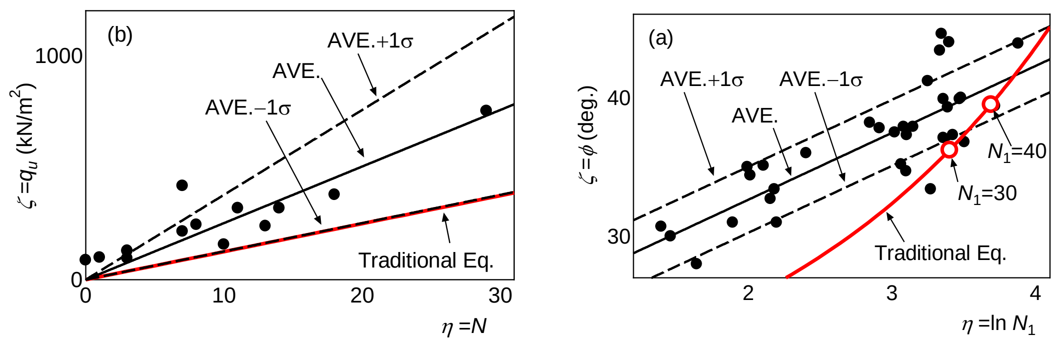

Fig. 3 Examples of data distribution between SPT-N value and ϕ[5] and between SPT-N value and q_u[4]

3.3 Comments on the proposed method

Although the proposed method is not theoretically correct in the strictest sense, it is impossible to pre-pare sets of partial factors to cover all geotechnical investigation situations, and in-situ soil test methods are being improved and newly developed even now. The proposed method can be used to prepare the partial factors that widely cover many kinds of soil tests with the least time and cost, and incorporate future technologies of geotechnical investigation, since it does not need code calibration as many times as the number of geotechnical investigation methods.

Furthermore, under the concept of performance-based design code, design codes must not hinder the creativity of code users. The proposed method fits the engineers' demand in selecting geotechnical investigation methods. According to the results of an international joint questionnaire survey conducted by the Foundation Engineering Research Team of the Public Works Research Institute and TC23 of the International Society for Soil Mechanics and Geo-technical Engineering, while the SPT-N blow count is commonly used to estimate the values of many geotechnical parameters, engineers want to

use other geotechnical investigation methods in addition to the conventional one especially for cohesive soil[5), 6)]. The proposed procedure enables engineers to actively introduce better-quality geotechnical investigation methods having less degree of variation in data distributions for the average relationships, even though the geo-technical investigation cost increases. The increase in the geotechnical investigation cost would be paid, since we could end up using larger characteristic values than the values derived from conventional soil tests and achieving a more economical design achievement.
A different example of the design formula for the compressive bearing capacity of a pile has also been covered by Shirato et al. (2003c).

4. PARTIAL FACTOR APPROACH

There are basically two partial factor approaches to evaluating the design value of the soil resistance in partial factor design from geotechnical investigation results. One is a material factor approach (MFA) and the other a resistance factor approach (RFA), simply represented by the following equations:

$$R_d = f\,(x_k \,/\, \gamma) \qquad \text{(MFA)} \tag{6}$$

$$R_d = f\,(x_k)/\gamma \qquad \text{(RFA)} \tag{7}$$

where R_d is the design value of the soil resistance, x_k is the characteristic value of a geotechnical parameter, and g is the partial factor. The ground resistance is associated with passive resistance intensities such as the bearing capacities of shallow foundations and the horizontal resistances of deep foundations. On the other hand, the geotechnical parameters indicate, for example, the internal friction angle and cohesion of soil, etc., which are used to evaluate the soil resistance R_d.

In the MFA, a partial factor is applied to the characteristic value of each geotechnical parameter to set the design value of the geotechnical parameter. The design value of the soil resistance is calculated by using a set of design values of the geotechnical parameters, i.e. the factored values. In the RFA, on the other hand, the characteristic soil resistance is first calculated by using the characteristic values of the geotechnical parameters. Then, the resistance factor is applied to the characteristic soil resistance. In the case of soil resistance comprising the sum of several resistance elements, a resistance factor is applied to each resistance element. For example, the compression bearing capacity of a pile is evaluated as the sum of the side resistance and the base resistance. Then, the design value of the pile bearing capacity is obtained as the sum of the factored side resistance and the factored base resistance. When partial factors are applied to two or more resistance elements, it is sometimes called the multiple resistance factor design (MRFD) format[8)].

The pros and cons of MFA and RFA have been described in reports[e.g., 9), 10), 11)]. We conclude from the practical viewpoint that the RFA, especially the MRFD, is more suitable than the MFA for the code of practice. The reasons are similar to Phoon et al.[10)] and Becker[11)], and we pay particular attention to the seamless transition of design results from the current design approach to the reliability-based design approach.

Generally, the resistances to foundations are in the category of passive resistance problems of soil. Accordingly, let us consider the estimation of a bearing capacity coefficient for the design of shallow foundations, N_γ, as an example. Because the value of the bearing capacity coefficient exponentially increases with increase in the internal friction angle ϕ, the rate of change of the bearing capacity coefficient also increases exponentially with increase in the value of ϕ.

Conventional global safety factor approaches are analogous to the RFA in that we may think of a global safety factor to be applied to the foundation resistance. In this case, the design bearing capacity obtained with N_γ, is reduced at a certain ratio from the characteristic value for any values of ϕ. On the other hand, in the MFA, the ratio of reduction in the design bearing capacity varies considerably depending on the characteristic value of ϕ, since the partial factor is applied to the characteristic value of f prior to the calculation of N_γ. This means that the introduction of the MFA will cause significant changes in the dimensions of the design results compared to the traditional design results. A large change in the dimensions would not be welcomed by practical engineers, since they would be unable to find the errors in the design calculations based on their past design experience. Furthermore, it becomes more difficult for code writers to scrutinize the effect of a partial factor on the behavior of foundation systems in the code calibration, when the incremental relationship between a partial factor and soil resistance is highly nonlinear, more so than the conventional force-based design, especially such as ductility design widely used in seismic design in high seismicity regions around the Pacific Ocean. A more foreseeable de-sign approach regarding the effects of partial factors on design results has a greater advantage in code calibration. The RFA can cover these disadvantages of the MFA, and the MRFD is likely to give a clearer perspective to code writers and users on how a traditional global safety factor is separated into partial factors.

5. ISSUES OF CONVENTIONAL CODE CALIBRATION PROCEDURES AND POSSIBLE ALTERNATIVES

5.1 Issues in code calibration

Calibration of partial factors is described in several books[e.g.12)]. It is assumed that the limit state in consideration can be specified by a calculation model in terms of one function $g(...)$ of a set of variables X_1, X_2, ... X_n, comprising actions, material properties, etc., so that a condition for the safety of the structure of the form , can be associated with the limit state. The design requirement may be written as: $g(x_{1d}, x_{2d}, \ldots x_{nd}) \geq 0$, where x_{1d}, x_{2d}, ... x_{nd} are design values defined in the following.

For an arbitrary distribution $F(x_i)$ the design values are given by

$$F(x_{id}) = \Phi(-\alpha_i \beta_t) \tag{8}$$

If X_i is assumed to be normally distributed, then

$$x_{id} = \mu_i(1 - \alpha_i \beta_t V_i) \tag{9}$$

If the random variables are independent, the factors α_i should be found from a number of FORM calculations. In principle, this would require many iterative calculations, which are very inconvenient. However, based on experience, a set of standardized α_i values has been developed, which is presented in ISO2394.

In the procedure outlined above, the partial factor approach is introduced as an elaboration of the design value method. An alternative approach is to start with some arbitrary partial factor format and to require that the partial factors are chosen in such a way that the reliability of the existing structures is as close as possible to some selected target value.

We have not yet acquired partial factor values that seem reasonable, nor found reasonable results that are likely to be widely accepted from the viewpoint of the issues considered in this paper. However, we believe that we have nearly determined the precise causes preventing the code calibration from working well. Apart from the lack of some statistical information, we can summarize the main causes as follows:

(1) Difficulty in reliability analysis of an integrated structural system: A grouped-pile foundation comprises several piles and soil resistances. In the system response of a grouped-pile foundation in a severe earthquake situation, it often happens that some piles deform in the plasticity region even though the pile foundation has not reached the yield point of a system as listed in Section 2.2. Accordingly, reliability in the sys-tem performance of a grouped-pile foundation is attributed to the variations in structural strength of the piles, ductility of the piles, axial resistance of the piles, and lateral soil resistance to the piles, and it is impossible to assess the contribution of each variability factor of resistance to the system performance in the code calibration process.

(2) Existence of several failure modes, Part 1: As mentioned in Section 2.2, the limit states of systems of grouped-pile foundation are caused roughly by either the structural damage to piles or the upper limit of the mobilized compressive resistances of piles. In addition, the intensities of the mobilized structural resistance and pile compressive resistance interact with each other in the response as a system. Therefore, due to Items (1) and (2), it is difficult to set the performance functions.

(3) Existence of several failure modes, Part 2: Based on our code calibration experience, we recognize that the estimated reliability indices of existing pile foundations seem to depend on the failure modes. This makes it difficult to as-sign a partial factor γ_i to each fundamental variable X_i, which is available for any failure mode.

(4) Design factors outside the equilibrium of forces: Pile diameters are determined not only by mobilized stress and deformation of piles but also by the specifications of drilling machines, pile installation machines, and standardized values. Consequently, pile diameters are not minimized from the viewpoint of equilibrium of forces. Sometimes, the number of piles is not minimized either, because piles are aligned in a foundation. As a result, grouped-pile foundations usually possess surplus resistance, and this result in large variation in the system reliability index of existing grouped-pile foundations.

(5) Variety of design alternatives: There are various design alternatives once a prototype design of the foundation does not satisfy the required performance. For example, in the case of insufficient compression bearing capacity of a pile, we can choose an alternative from the following three: 1. Extend the pile length, 2. Enlarge the pile diameter, 3. Increase the number of piles.

Therefore, we must seek a code calibration strategy to avoid these difficulties.

5.2 Alternatives

Honjo et al.[13)] have shown a new strategy of using a kind of substructure method to perform reliability analyses and derive multi-resistance factors for the design of axial loaded piles in grouped-pile foundations of highway bridges. The system reliability problem of grouped-pile foundations is converted into a reliability problem of single piles subjected to only compressive loads. This suggests important tactics in order to avoid the difficulties of code calibration.

Here, we describe the essence of the tactics extracted from the methodology of Honjo et al. Although it is equivalent to the conventional way to start with collecting design results of existing pile foundations, the following stages are completely different. First, as for compressive outermost piles in grouped-pile foundations, the ratios of vertical loads acting on the pile top in seismic situations to the dead loads acting on the pile top in normal situations are determined.

Next, several representative soil profiles and corresponding pile lengths are chosen as the deterministic parameters in the prototype design from the collected design results, and prototype single piles are designed for the design situations using original global factors of safety. Two important points in the design of prototype single piles are as follows. First, some dead load conditions acting on the pile top due to the supported structures are assumed referring to past design results, and then the seismic loads are set based on the correlation derived from the above compilation of dead load and seismic compressive load in the existing design results. This manipulation reduces the integrated performance of pile foundations as a system into a pile bearing capacity problem of single piles, while the overall behavior of pile foundations is still physically incorporated in the load combination. In addition, since only one performance function is handled at a time, the reliability analysis becomes much easier. Secondly, the pile diameters of the prototype single piles are minimized so as to just support the vertical loads, ignoring the factors outside the bearing capacity such as the specifications of drilling machines. In other words, the prototype single piles are unlikely to have typical diameters in the realistic sense. This achieves a greater degree of uniformity in the reliability levels of the prototype single piles, and makes it easier to determine the target reliability. In the end, code calibration based on the design value method is implemented.

We consider it possible to perform code calibrations for other failure modes such as the bending strength of piles, deformation capacity of piles, and displacement of piles by applying this kind of substructure method. This method seems effective, since it deals with substructures converted from integrated space depending on each failure mode, instead of directly computing the system reliability.

On the other hand, there remain theoretical and practical problems. One theoretical problem is that the values of target reliability will differ depending on the failure modes even though we attempt to consider the reliability of a system. This is unavoidable, because well-designed pile foundations are highly robust and ductile. Foundation behavior is not sensitive to exceeding the critical state criterion of each resistant element in the ductile behavior region. In addition, the critical states of grouped-pile foundations still cannot be adequately predicted numerically, and thus we do not strictly define the critical states of foundations. One of the practical problems is that the cost of design would increase, since the same number of sets of partial resistance factors are prepared as the number of failure modes, and de-signers must repeat the computations of foundation response (i.e. pushover analyses) the same number of times as the number of prepared sets of partial resistance factors.

We should also retain one easy choice that could be widely accepted by engineers, namely to temporarily assign the value of 1 to all partial factors except for the factor applying to the critical verification index such as the limit value of ductility factor in seismic design for severe earthquakes. Note that, even in this case, we can make the performance-based design progress since we are able to reward efforts and innovations by practical engineers by getting the model uncertainties of transformation and design equations to factor in the estimation of corresponding characteristic values as shown in Section 3.

6. CONCLUDING REMARKS

Reliability-based design is a tool for evolving the performance-based design codes of foundations, as reliability theory can quantify the progress of technology based on the difference in uncertainty. However, the practical application of code calibration theories has not been thoroughly studied nor advanced to the same degree as recent seismic design of foundations in high seismicity regions along the Pacific Rim. This paper clarified the issues on code calibration for seismic design of grouped-pile foundations, and showed some alternatives to facilitate resolving such issues.

(1) It is effective to use an average value considering the confidence level of estimation of mean value as the characteristic value of a geotechnical parameter. For code writers, this significantly reduces the time and cost for code calibrations, while for practical engineers, it encourages designers to use more reasonable and diverse geotechnical investigation methods and design formulas.
(2) MRFD seems to be more practical than the MFA.
(3) The main causes for why code calibration of the pile foundation design has not yet succeeded were shown.
(4) One of the solutions to deriving reasonable partial resistance factors is to subdivide a foundation system having integrated performance into simplified substructures having unique performances depending on the failure modes.

This research was conducted in order to revise the Japanese Specifications for Highway Bridges in Japan. Comments and support from the Working Group for Partial Factor Design of Foundations (Chairman Dr. Kenji Matsui) established in the Committee for Substructures in the Japan Road Association are gratefully acknowledged. Note that this report reflects the authors' views based on our work in the Working Group to date; the revision work of the Working Group is still in progress.

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