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A design method concerning horizontal resistance of piles constructed in improved ground

Méthode de calcul relative à la résistance horizontale d'un pieu planté dans un sol amélioré

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ABSTRACT
The purpose of this study is to consider the application of strength generated following ground improvement as horizontal resistance of piles constructed in improved ground and to systematize it as a rational design method. In the proposed design method, the range of influence of horizontal resistance of piles in improved ground, or the necessary range of ground improvement, was established based on engineering assessment and horizontal subgrade reaction of piles was determined from shear strength increased by ground improvement. The validity of the design method was empirically verified by on-site horizontal loading tests of piles and finite element analysis that was conducted to supplement its parameters. Also, because the improved ground would become stronger than the unimproved original ground, the effect of the hardness of surrounding ground on piles during earthquakes was verified using a centrifugal excitation experiment and the dynamic finite element method.

RÉSUMÉ
La présente recherche a pour objectif de tenir systématiquement en compte la rigidité du sol amélioré dans le calcul de la résistance horizontale d'un pieu planté dans ledit sol amélioré et d'établir ainsi une méthode de calcul rationnelle. La méthode de calcul proposée détermine la portée d'influence sur la résistance horizontale d'un pieu dans un sol amélioré, c'est-à-dire la région dans les limites desquelles l'amélioration du sol est nécessaire, selon l'appréciation technologique ainsi que la réaction horizontale du sol sur le pieu selon la résistance au cisaillement du sol renforcée par l'amélioration. La pertinence de cette méthode a été démontrée par l'essai de chargement latéral du pieu sur place et l'analyse selon la méthode d'élément fini effectuée pour compléter les paramètres dudit essai. En outre, comme le sol amélioré a une résistance plus élevée que le terrain naturel non amélioré, l'influence de la rigidité du sol environnant sur le pieu en cas de séisme a été vérifiée par l'essai de tremblement à l'aide d'un équipement centrifuge.

1 INTRODUCTION

The former design methods for piles in improved grounds are not reasonable, because the design is based on ground properties that are different from the actual properties, and subgrade reaction can be underestimated depending on the specific site conditions. This study presents a rational design method (Tomisawa *et al* 2002) in which ground improvement is conducted around piles constructed in soft ground and ground subject to liquefaction, where the ground strength after improvement is reflected as horizontal resistance. Although similar concepts have been proposed in the past (Maeda *et al* 2001,Kitazume *et al* 2003), design methods vary greatly and are far from being systematized. Thus this paper established the range of influence of the horizontal resistance of piles in improved ground, or the necessary range of ground improvement based on an engineering assessment, and determined the horizontal subgrade reaction of piles from the amount of the shear strength that increased from ground improvement. Validity of the design method was confirmed empirically through several on-site horizontal loading tests performed on piles from actual bridges. Because improved ground is much stronger than the original unimproved ground, the effects of the improved ground strength and boundary conditions on the dynamic behavior of piles depending on ground vibration levels were also verified by a centrifuge excitation experiment.

2 EVALUATION METHOD FOR HORIZONTAL RESISTANCE OF PILES IN IMPROVED GROUND

2.1 *Range of influence of horizontal resistance of piles on the surrounding ground*

In the case of horizontal resistance of piles, the limit equilibrium state will be maintained because the front ground will be compressed in the horizontal direction, so horizontal earth pressure will increase, if the maximum value of horizontal subgrade reaction is treated as the limit resistance proportional to the force of action in the same way as in the limit subgrade reaction design of rigid foundations. Thus, the range of horizontal resistance of the front ground when horizontal force is applied to the piles can be set as a passive earth pressure area (Akai *et al* 1997), which can be represented as the failure angle of the soil. It is known that the horizontal behavior in the linear range of semiinfinite long piles in soft ground, where the horizontal force is applied to pile heads, is controlled by ground properties within the range of the passive earth pressure in the front, considering the characteristics length $1/\beta$(Japan Road Association *et a*l 1997). In this study, therefore, the range of influence of the front ground horizontal resistance of piles in improved ground was set as the area raised to the working gradient θ $=(45° + \phi/2)$ （ϕ: angle of shear resistance of soil）of passive earth pressure from the depth of $1/\beta$. Also, the two-dimensional range of influence was assumed to be a fan-shaped area $\theta=(45° + \phi/2)$ spreading from the direction of force, in the same way as the horizontal subgrade reaction in the front. As a result, the horizontal subgrade reaction was established as a three-dimensional cuneiform area centered on the piles. Considering the workability of ground improvements, however, a pile design method reflecting the improved strength was proposed. In this method, a three-dimensional square shape is formed by combining the necessary area of soil improvement in two directions, parallel and perpendicular to the pile axis against the horizontal force shown in Figure 1 In this case, piles

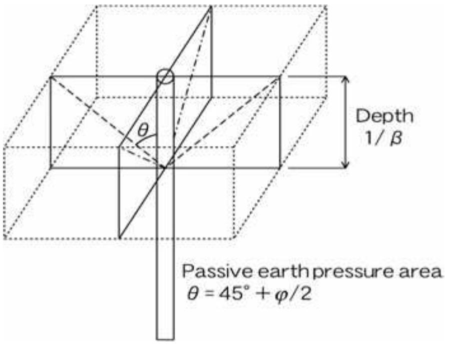

Figure 1. Three-dimensional range of influence of the horizontal resistance of composite ground piles.

and improved ground, including composite ground formed using the deep mixing method, are not an integral structure, and the horizontal behavior of piles is in conformity with the design of the subgrade reaction method.

2.2 *Calculation of the horizontal subgrade reaction of piles in improved ground*

The horizontal resistance of piles is determined by the coefficient of horizontal subgrade reaction *k*, which is calculated from the modulus of deformation of ground *E* using Eq. (1). It is thus necessary to evaluate the effect of the increased shear strength *C* as the degree of increase in modulus of deformation of ground *E* in the design of piles to be constructed in improved ground.

$$k=(\alpha \cdot E/0.3) \cdot \left[(D/\beta)^{1/4} \cdot (1/0.3)\right]^{-3/4} \quad (1)$$

Where, *k* is the coefficient of horizontal subgrade reaction of piles (kN/m^3), *E* is the modulus of deformation of improved ground (kN/m^2), α is the estimated coefficient of horizontal subgrade reaction, *D* is the pile diameter (*m*), β is the characteristic value $((kD)/4EyI)^{1/4}$ (m^{-1}), *Ey* is Young's modulus of piles (kN/m^2) and *I* is the geometrical moment of inertia (m^4).

The shear strength *C* of composite ground, in which improved columns are installed using the deep mixing method, is determined by Eq. (2) (Public Works Research Center *et al* 1999), combining the original ground strength and improved column strength based on the improvement rate.

$$C = Cp \cdot ap + \alpha s \cdot Co\,(1 - ap) \quad (2)$$
$$Cp = qup/2,\ Co = quo/2,\ ap = Ap/A$$

Where, *C* is the shear strength of composite ground (kN/m^2), *Cp* is the shear strength of improved columns (kN/m^2), *Co* is the shear strength of original ground (kN/m^2), αs is the fracture distortion reduction rate, *ap* is the ground improvement rate, *qup* is the unconfined compressive strength of improved columns (kN/m^2), *quo* is the unconfined compressive strength of original ground (kN/m^2), *Ap* is the cross-section area of improved columns (m^2), *A* is the distribution area per improved column (m^2) and αs is the strength reduction rate of *quo* for fracture distortion of *qup* (1/2～1/3).

The relationship between the shear strength *Cp* and unconfined compressive strength *qup* is *Cp = qup/*2, as shown in Eq. (2). It is also known that the unconfined compressive strength *qup* and modulus of deformation *Ep* are in a relative relationship depending on the properties of the original ground before improvement. For example, *Ep*=100*qup* in the case of improving cohesive ground using a cement-based soil stabilizer. It is therefore considered that the modulus of deformation of composite ground *E* is similar to the rate of the shear strength *C*. In the proposed design method, the modulus of deformation of composite ground *E* is treated similarly to the shear strength *C*

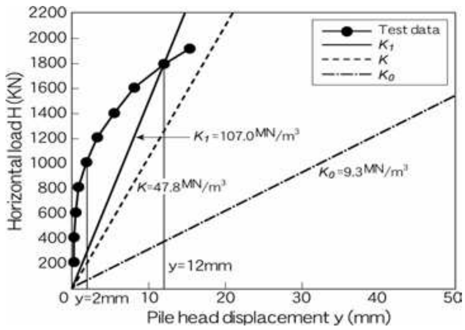

Figure 2. Relationship between horizontal load *H* and pile head displacement *y*.

of composite ground based on the improvement rate *ap* as shown in Eq. (3), from the shear strength *Cp* and unconfined compressive strength *qup* of improved columns and the relationship between the unconfined compressive strength *qup* and modulus of deformation *Ep*. As a result, the modulus of deformation of composite ground *E* is determined as the total of the modulus of deformation *Ep* of improved columns combined with the improvement rate *ap* and the modulus of deformation of the original ground *Eo*.

$$E = Ep \cdot ap + \alpha s \cdot Eo\,(1 - ap) \quad (3)$$

Where, *E* is the modulus of deformation of composite ground (kN/m^2), *Ep* is the modulus of deformation of improved column (kN/m^2) and *Eo* is the modulus of deformation of the original ground (kN/m^2).

By reflecting the increased modulus of deformation of ground *E* found by each ground improvement method in the above manner, it becomes possible to determine the horizontal subgrade reaction of piles in large improved ground that is closer to the actual condition than using the modulus of deformation of the original unimproved ground.

3 A CASE EXAMPLE OF VERIFICATION OF THE PROPOSED DESIGN METHOD BY THE ON-SITE HORIZONTAL PILE LOADING TEST

This section presents a case example of confirming the validity of the proposed design method by examining an onsite horizotal loading test on piles. The site was a one-span box-type abutment constructed on soft ground for high-standard highway in Hokkaido, Japan. To increase the horizontal resistance of the piles, composite ground was formed around them within the range of the working gradient of passive earth pressure $\theta = (45° + \phi/2)$ from the depth of $1/\beta$=3.65 m using the deep mixing CDM method (Public Works Research Center *et al* 1999) (slurry-based mechanical mixing method) with design unconfined compressive strength of *qup*=200kN/m^2 and the improvement rate of *ap*=78.5%. As a result, it was possible to reduce the number of piles to n =3 × 4=12 (pile diameter: 1,200*mm*, pile length: 17.0*m*), as well as the size of the abutment body, whereas an unrealistic number of piles, n =14×5 = 70, is necessary to ensure adequate horizontal
resistance using a conventional design without ground improvement. Construction costs could be reduced by up to 45%.

Using proposed Eqs. (1) and (2), the design value *k* of composite ground for the pile foundation of the abutment was set as *k*=47.8MN/m^3, approximately 5 times as large as the design value k_O=9.30MN/m^3 for original ground where N=5 or lower before ground improvement. The unconfined compressive strength of the CDM at the material age of 28 days was *qup'*=408kN/m^2averaged over four points at the depth of $1/\beta$. This value was twice as large as the design unconfined compressive strength *qup*=200kN/m^2. The measured value of composite ground k_1 found from the unconfined compressive strength of improved columns qup was therefore set as $k_1 = 107.0 MN/m^3$, while the design value of composite ground was $k=47.8 MN/m^3$.

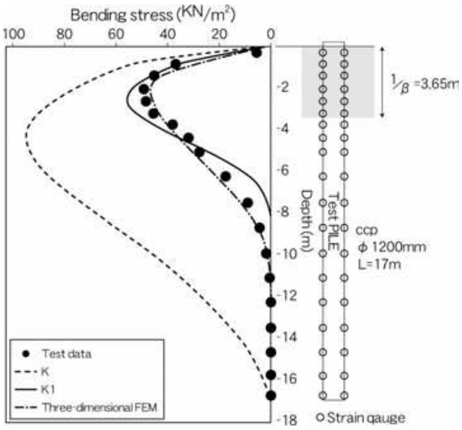

Figure 3. Bending stress distribution of test piles.

On-site horizontal loading test of piles was conducted by themulti-cycle load-control method in accordance with criteria from the "Method of horizontal loading test of piles and instruction manual" established by the Japan Geotechnical Society (Japanese Geotechnical Society *et al* 1993) The test piles were equipped with reinforcing bar stress transducers and strain gauges in the depth direction to measure the bending stress. Figure 2 shows the relationship between the horizontal load H and pile head displacement y, obtained as a result of the on-site horizontal loading test of piles. The figure also shows the relationship between H and y calculated by the linear elastic ground reaction method using the measured value of composite ground k_1 found from the design value of original ground before improvement ko, the design value of composite ground k and the actual unconfined compressive strength of improved columns. The H-y relationship obtained in the horizontal loading test was greater than the measured value of the original ground $ko = 9.30\ MN/m^3$ and the design value of composite ground $k=47.8 MN/m^3$. The coefficient of horizontal subgrade reaction k_2, which is equivalent to the standard displacement of 1% of the pile diameter (12 mm), corresponded with the measured value of composite ground $k_1=107.0 MN/m^3$.

Figure 3 illustrates the pile bending stress of the horizontal load $Hmax=1800 kN$, measured when the calculated value k_2 corresponded with the measured value k_1 in the on-site horizontal loading test, together with the bending stress calculated by elastic analysis of the two-dimensional FEM using the design value of the original ground k_O and the measured value of composite ground k_1. The bending stress calculated by the two-dimensional FEM using measured subgrade reaction of piles in composite ground was considered to be allowable in practice, although there was a slight variation in the maximum value compared with the measured value. The bending stress distribution closely corresponded with that of the measured value, unlike the value calculated from the design value of the original ground ko. Figure 3 also illustrates the bending stress distribution of piles determined using the three-dimensional FEM. The analysis model of the three-dimensional FEM featured the reproduction of an improved cubic sphere that underwent ground improvement based on the proposed method, and the sphere centered on single piles above the $1/\beta$ position was set as the range of passive earth pressure. Physical properties of the on-site ground were input for the modulus of deformation E and Poisson's ratio ν. Pile rigidity was set taking non-linearity into account depending on the strain level. As a result, the bending stress distribution of piles obtained using the three-dimensional FEM closely corresponded with the measured bending stress in almost the same way as the analytic value of the two-dimensional FEM. Similar results have been obtained in verification tests of other actual bridges. As a result, the range of influence of the horizontal resistance of piles was set as the area raised to the working gradient of passive earth pressure $\theta = (45° + \phi/2)$ from $1/\beta$, and the validity of the proposed design method, in which the coefficient of horizontal subgrade reaction k was determined from the shear strength of improved ground C, was empirically verified.

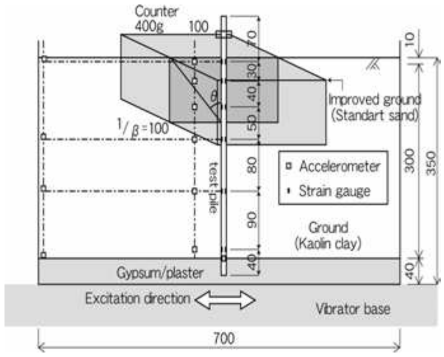

Figure 4. Composite ground model for centrifuge excitation experiment.

4 SEISMIC PERFORMANCE OF PILES IN IMPROVED GROUND

The greatest concern about behavior of piles constructed in improved ground during an earthquake is thought to be the seismic performance of piles at the boundary between the improved section and the original ground, where strength differs. A centrifuge excitation experiment was thus conducted to verify the seismic performance of piles in improved ground.

4.1 *Centrifuge excitation experiment model*

For the centrifuge excitation experiment, model ground and piles at a scale of 1:50 were prepared using a steel model container with inner dimensions of $L=700mm \times B=200mm \times H=50mm$. The centrifuge force field was 50G to satisfy the law of similarity for the stress level. Model piles with outer diameter $\phi = 10mm$, thickness $t = 0.2\ mm$ and pile length $L = 400\ mm$ were prepared by stretching and specially processing steel. The pile head was equipped with a weight ($W=400g$) to simulate the substructure, and was in the embedded fixed-head state. The model piles were equipped with strain gauges to measure the stress generated on the piles, and acceleration sensors were installed on the pile heads and in the ground for measurement at the time of excitation. Dynamic excitation was unidirectional using sine waves of a uniform level, $10 m/s^2$ (=1,000gal). A single layer of kaolin clay was prepared to simulate completely soft, unimproved ground, and a kaolin clay soft-ground section with an improved section around the pile heads, which was simplified by substituting standard sand, were used as the model ground for the experiment. Figure 4 shows the model for this experiment. The area of improved ground using standard sand was set as a cubic area ($L20cm \times H10cm \times B20cm$) in front of the piles to simulate the area from $1/\beta$ to the passive earth pressure gradient $\theta = (45° + \phi/2)$ based on the proposed design method. The intensity ratio of the cone index qu between kaolin clay and standard sand of the model ground was calculated, assuming that the loading method was used, and was set as 1:3 in a 1-G field to confirm the qualitative tendency.

4.2 *Dynamic horizontal behavior of piles in improved ground*

Figure 5 presents a comparison of frequency curves of piles in the single-layer ground of kaolin clay and composite ground of standard soil, which were obtained as a result of the sine wave excitation experiment. The vertical axis of the figure represents the amplitude ratio of transmissibility found by dividing the Fourier amplitude of the piles by the foundation input value, and curve fitting of the frequency curve was performed by the three-dimensional function. As a result, the natural frequency of

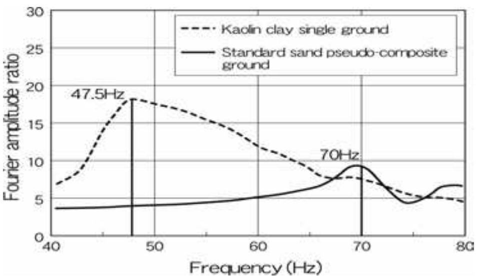

Figure 5. Pile frequency for kaolin clay single ground and standard sand improved ground.

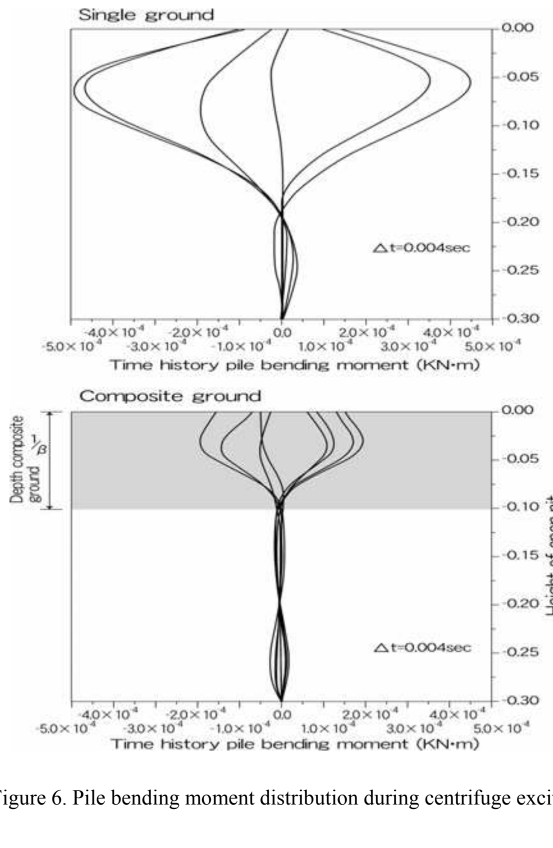

Figure 6. Pile bending moment distribution during centrifuge excitation.

piles in the $50G$ field was found to be $47.5Hz$ in the single layer ground of kaolin clay and $70Hz$ in the improved ground of standard sand. Since the improved ground is solidified, the period of the piles becomes short during an earthquake. Because the damping effect of the entire bridge system changes with the shortening of the period of piles, attention should be paid to radiational damping from the foundation to the ground when considering the performance of piles in improved ground. The natural frequency during excitation was confirmed to be approximately 300 *Hz* in standard sand and 200 *Hz* in kaolin clay, while it was 70 *Hz* for piles in improved ground. When converted to the ground property value TG (Japan Road Association *et al* 2002), standard sand is equivalent to Type I ground (TG<0.2), and kaolin clay is equivalent to Type II ground ($0.2 \leqq TG<0.6$). As a result, it was presumed that the improved ground and piles did not resonate as an integral structure, and that the piles, standard sand and kaolin clay showed different dynamic behavior depending on their respective deformability and response characteristics.

Figure 6 shows the bending moment distribution of the piles in the single-layer ground of kaolin clay and improved ground of standard soil in the time history at intervals of Δt=0.004*sec*, which was obtained during response of the piles to sine wave excitation. The maximum bending moment of piles occurred at $1/\beta$, decreased to 1/2.5 of the single-layer ground (maximum value in composite ground / maximum value in single-layer ground $\fallingdotseq 2.0\times10^{-4}/4.5\times10^{-4}$ $kN \cdot m$), and converged in composite ground. This was thought to be due to the restrictions on the pile heads in improved ground and the decrease in pile frequency during excitation. While there was a concern about strain of piles at the bottom of the boundary between the improved ground of standard sand and original ground of kaolin clay, both the displacement and bending moment of piles were smaller than the maximum values, and there was no problem concerning stress. In the case of this experiment, however, the strength of the improved ground was three times as high as that of the original ground, and the pile stress tended to be slightly dominant in some boundary conditions between improved and original ground as a result of analysis using dynamic FEM with a varied intensity ratio. It was therefore considered necessary to verify pile stress at the boundary, depending on site conditions, when applying this method. As a verification method, it is considered effective to employ the dynamic response analysis method, taking the deformation of ground during an earthquake into account, and creating a pile-ground spring model[8)], in addition to the verification of the horizontal load capacity method of the superstructure's inertial force during earthquakes.

5 CONCLUSIONS

The knowledge obtained through the case study for empirical verification can be summarized as follows:

(1)The pile design method used to reflect the increase in ground strength resulting from the construction of piles in improved ground as horizontal resistance aids in efficient use of construction costs under certain site conditions.

(2) Horizontal subgrade reaction of piles in improved ground was set from increased shear strength *C* after improvement by using the results of several on-site static horizontal loading tests of piles from an actual bridge and two and three dimensional finite element analysis. The validity of the proposed design method, in which the range of influence of horizontal resistance was set from $1/\beta$ to an area raised to the working gradient of passive earth pressure θ=(45° + ϕ/2), was empirically verified.

(3) The centrifuge excitation experiment and dynamic finite element analysis revealed the tendency for the period of piles to be shortened in improved ground. The piles, improved columns and original ground showed different dynamic behavior depending on their respective deformability and response characteristics. The tendency for a relatively large amount of strain to occur on pile bodies in certain boundary conditions was also observed, and it is considered necessary to verify pile stress depending on site conditions.

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