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EXPERIMENTAL AND NUMERICAL VERIFICATION OF SEISMIC PERFORMANCE OF PILE FOUNDATION IN COMPOSITE GROUND

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ABSTRACT

This paper deal with a new construction method of pile foundation in composite ground, in which, prior to installing piles, the ground is improved around the heads of the piles in soft ground or ground subject to liquefaction. This construction method uses a combination of pile foundation construction together with common ground improvement methods, including deep mixing, preloading, and sand compaction piling, and is referred to as the composite ground pile method. Since an artificial ground with relatively high rigidity comparing with that of the original ground was formed around the pile in this method, and the seismic performance has not been made clear, thus the seismic performance of piles in composite ground was systematically analyzed through a series of centrifuge model tests and numerical analyses by using dynamic nonlinear finite element method, and a verification method for the seismic performance of piles in composite ground was proposed on the basis of the experimental and analytical results.

Keywords: pile foundation, composite ground, centrifuge model test, dynamic nonlinear FEM

INTRODUCTION

In order to improve not only cost performance, but also structural reliability during strong earthquake, different types of seismic strengthening methods for pile foundations were proposed and put into practical use recently in Japan. Ground improvement around piles is a typical seismic strengthening method for pile foundations. Although this method is being used for practical construction of pile foundations (Akiyoshi et al., 2001, Nanjo et al. 2000), its design method has not been systematically established yet. There are, in particular, still many unclear points concerning the seismic performance of piles in improved ground. The composite ground pile method, in which ground improvement is carried out around piles constructed in soft ground or ground subject to liquefaction, was studied for the purpose of saving construction costs, and a design method reflecting the ground strength increased by improvement mainly on the horizontal resistance of piles was proposed and put into practical use (Tomisawa & Nishikawa, 2005a, 2005b). This construction method uses a combination of pile foundations with commonly used ground improvement methods, such as deep mixing, preloading and sand compaction pile. In this method, the horizontal subgrade reaction of piles is determined from the shear strength of the improved ground and the necessary range of ground improvement is established as a range of the horizontal resistance of piles, based on an engineering assessment. The validity of this method has already been verified from *in-situ* static horizontal loading tests of piles and static finite element analyses. Earthquake resistance at the boundary between the improved and original ground has also been confirmed by the seismic intensity method and the dynamic linear finite element method (equivalent linear method). There are, however, still some unclear points concerning the seismic performance of piles in improved ground such as the dynamic responses of both pile and improved ground at different earthquake levels and ground conditions, resistance of the improved

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ground to the pile and seismic behavior at the boundary between improved and original ground of pile foundations and so on. Although several studies have been conducted on composite foundations combining piles and improved columns (Maeda et al., 2001, Maenaka et al., 2001), they are limited to the case studies, and the general design procedure has not been discussed. To properly estimate the seismic performance of pile foundation in composite ground, it is necessary to establish a reasonable design and construction procedure for such foundations. In this study, therefore, the earthquake resistance of pile foundations in composite ground under Level 1 and Level 2 earthquake motions was verified through a series of dynamic centrifuge model tests and numerical analyses by using dynamic nonlinear finite element method. In the centrifuge model test, static horizontal subgrade reactions to the pile and seismic responses of the pile before and after shaking with Level 1 and Level 2 earthquake motions were confirmed respectively. In the numerical analysis, the target site was a composite ground pile foundation by using deep mixing method, which is a ground improvement method with the highest strength and rigidity, and the deformation of the pile under Level 1 and Level 2 earthquake motions was mainly focused. On the basis of the experimental and numerical results, the seismic performance of the composite ground pile method was discussed.

DESIGN PROCEDURE OF THE COMPOSITE GROUND PILE METHOD

Setting of the range of ground improvement

The range of influence of horizontal resistance in the ground when horizontal force is applied to a pile spreads gradually as load increases. As a result, when the failure limit state of the ground is reached following the horizontal displacement of the pile, a state of equilibrium is considered to be maintained between the maximum value of the horizontal subgrade reaction and the passive earth pressure. In the composite ground pile method, therefore, the necessary range of ground improvement, i.e., the range of horizontal subgrade reaction to the pile, is proposed to be a three-dimensional domain formed with the gradient of the surface of passive failure $\theta = (45^\circ + \phi/2)$ (ϕ : angle of shear resistance of soil) from the depth of the characteristic length of piles, $1/\beta$ ($\beta = (kD/4E_vI)^{1/4}$), which is the depth of influence of the horizontal resistance of piles on the basis of the limit equilibrium and the Mohr-Coulomb failure criterion. Therefore, the necessary range of improvement is set as a three-dimensional inverted cone shape centered on the pile. However, since it is difficult to conduct ground improvement in a cone shape due to construction limitations, a cubic body covering the range of the invert cone shape shown in Figure 1 was proposed for the range of ground improvement. The method for setting the range of ground improvement for groups of piles is the same as that for a single pile.

Determining horizontal subgrade reaction of pile

When using the deep mixing as the ground improvement method, the ground is formed by placing improved columns with required strength around the piles. The shear strength of composite ground S is calculated from Equations (1) & (2), combining the shear strength of the improved column S_p and that of original ground S_0 according to the improvement rate;

$$S = S_p \cdot \alpha_p + S_0(1 - \alpha_p) \quad (1)$$

$$S_p = q_{up} / 2, S_0 = q_{u0} / 2, \alpha_p = A_p / A \quad (2)$$

where S_p = shear strength of the improved columns (kN/m^2); S_0 = shear strength of the original ground; α_p = ground improvement rate; q_{up} = unconfined compression strength of improved columns (kN/m^2); q_{u0} = unconfined compression strength of the original ground; A_p = area of the cross-section of the improved columns (m^2); and A = distribution area per improved column (m^2). Greater strength can be expected for the composite ground when deep mixing method is adopted for ground improvement. In this case, improved columns are installed one by one in a rectangular arrangement with an improvement rate of $\alpha_p = 78.5\%$ or higher to ensure a certain level of the horizontal subgrade reaction of the piles.

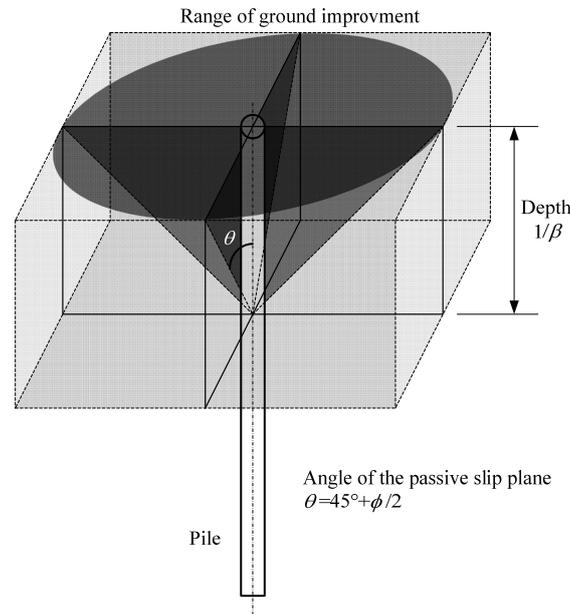


Figure 1. 3-D image of the lateral resistance of pile foundation and the range of ground improvement

To properly select the horizontal subgrade reaction of piles in improved ground in a design for the composite ground pile, it is necessary to evaluate the increases in shear strength S and modulus of deformation E of the ground due to ground improvement. In deep mixing method, the shear strength of improved columns S_p can be obtained from the unconfined compression strength of improved columns q_{up} ($S_p = q_{up}/2$), as shown in Equation (2). It is also well established that the unconfined compression strength q_{up} is proportional to the modulus of deformation E_p of improved columns. Therefore, the modulus of deformation of composite ground E can be determined as the total of the modulus of deformation of improved columns E_p combined with the improvement rate α_p and the modulus of deformation of the original ground E_0 as follows;

$$E = E_p \cdot \alpha_p + \alpha_s \cdot E_0(1 - \alpha_p) \quad (3)$$

where, α_s = strength reduction rate of original ground related to the fracture strain of improved columns (1/3 ~ 1/2). The modulus of deformation of improved columns E_p in clay soil ground can be found from the relationship of $E_p = 100q_{up}$, based on the unconfined compressive strength of improved columns q_{up} . The design strength of improved columns is usually $q_{up} = 200$ to 500 kN/m². The coefficient of horizontal subgrade reaction of piles in composite ground k_h can be calculated by using following equation from the modulus of deformation of composite ground E .

$$k_h = 1/0.3 \cdot \alpha \cdot E \cdot (\sqrt{D/\beta}/0.3)^{-3/4} \quad (4)$$

where, α = estimated coefficient of horizontal subgrade reaction which depends on the estimation methods of E_p and E_0 (Japan Road Association 2002a); D = pile diameter; and β = characteristic value of the pile. By setting the coefficient of horizontal subgrade reaction k_h using the above method, it becomes possible to design pile foundations under a static load in composite ground.

EVALUATION OF THE SEISMIC RESISTANCE OF COMPOSITE GROUND PILES
THROUGH CENTRIFUGE MODEL TEST

The main purposes of the centrifuge model tests in this study are to analyze the effects of strong earthquake motion on both pile and surrounding improved ground, and to confirm the seismic response of pile foundation in composite ground under different level of earthquake motion. In composite ground pile method, as an important issue of design when deep mixing was adopted as the ground improvement method, it is the soundness of improved column during high level earthquake motion. Thus, a series of dynamic centrifuge model tests was conducted to evaluate the seismic resistance of the pile foundation in composite ground first.

Outline of the centrifuge model test

For the centrifuge model test, model ground and pile with a scaling ratio of 1:50 were prepared and installed in a pair of model containers with inner dimensions of $L = 0.7 \text{ m} \times B = 0.2 \text{ m} \times H = 0.5 \text{ m}$. The strong steel container was used for static horizontal loading tests, and the laminar container was used for dynamic shaking tests. A 50 G (G: gravitational acceleration) centrifugal acceleration field was adopted for both static horizontal loading test and shaking test to satisfy the similarity law on the stress level. Figure 2 shows the configuration and instrumentation of the model for the tests. Model pile with outer diameter $D = 0.01 \text{ m}$; wall thickness $t = 0.002 \text{ m}$; and length $L = 0.4 \text{ m}$ was made and specially finished from steel pipe, and lower end of the model pile was fixed and filled with gypsum mortar as the model base ground. A proto-type scale steel pile with outer diameter $D = 0.5 \text{ m}$; wall thickness $t = 0.1 \text{ m}$; and length $L = 20 \text{ m}$ was simulated in the centrifugal acceleration field by using this model pile. Strain gauges were installed in the inner wall of the model pile to measure both axial and bending stress generated along the axis of the pile, and miniature acceleration sensors were installed on the pile head and in the ground to measure the responses of both pile and ground during shaking. A steel mass with the weight of $W = 3.92 \text{ N}$ (equivalent to 490kN in prototype scale) was fixed to the pile head to simulate the substructure.

Peat ground was supposed to be the model original ground. It was prepared by mixing peat moss and kaolin clay at a dry weight ratio of 1:1 and saturating it with an initial moisture content of 300%. Model improved columns around the pile with the diameter of 0.02 m and the length of 0.1 m, were prepared by mixing the peat ground material with cement and curing them in acrylic tubes and then installed in a rectangular arrangement with an improvement ratio of $\alpha_p = 78.5 \%$. The required unconfined compressive strength of improved column was set as $q_{up} = 200 \text{ kN/m}^2$.

In the test, sine waves with a maximum amplitude equivalent to approximately 150 gal and 800 gal in prototype scale were used as the input waveforms to simulate Level 1 and Level 2 earthquake motions respectively. Displacement controlled static horizontal loading tests of pile were conducted for both the model grounds with and without improved columns. For the improved ground condition, i.e. the composite ground model, static horizontal loading tests were also carried out before and after shaking with both Level 1 and Level 2 earthquake motions. To confirm the seismic response behavior of pile and ground in the elastic deformation range, shaking tests were also carried out before and after shaking with Level 1 and Level 2 earthquake motions by using sine waves with low level acceleration amplitude of 10 m/s^2 (equivalent to 20 gal in prototype scale) and different frequency as input motions. After the shaking test with Level 1 Level 2 earthquake motions, the situation of the improved column around the pile was carefully observed.

Test results and discussion

The average coefficients of horizontal subgrade reaction of piles for both original and composite grounds before and after shaking with Level 1 and Level 2 earthquake motions predicted from the static horizontal loading tests are summarized as shown in Table 1. Comparing with the case of pile in original ground, the average coefficients of horizontal subgrade reaction of piles in composite grounds were approximately 12

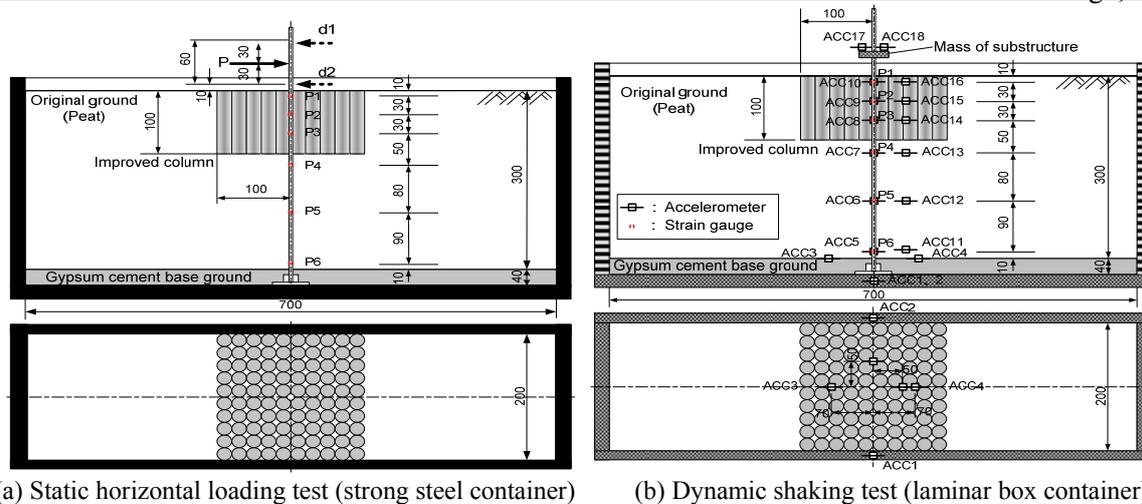


Figure 2. Configuration and instrumentation of centrifuge model test

times larger, and the shaking almost does not affect the average coefficients of horizontal subgrade reaction of the piles in composite grounds. It also means that even under high level earthquake motion the shaking does not change the rigidity of improved ground around the pile. The effectiveness of horizontal subgrade reaction of the composite ground on piles was therefore validated from the centrifuge model tests. Figure 3 shows the loading curves obtained from the static horizontal loading tests before and after shaking tests by using Level 1 and Level 2 earthquake motions. Such loading curves are almost the same indicating that the improved columns around the pile did not change too much due to the shaking with strong earthquake motions. Sufficient seismic resistance of pile in composite ground was recognized from the results of static horizontal loading tests. Figure 4 shows a comparison of the frequency curves for piles before and after shaking tests with Level 1 and Level 2 earthquake motions. These data were obtained from low level shaking tests (acceleration amplitude in prototype scale is 20 gal) using sine waves with various frequencies. The vertical axis in the figure represents the ratio of Fourier amplitude of the acceleration response at the pile head to that of the input acceleration at the base ground, i.e., the ratio of transfer functions and curve fitting for the frequency curves was performed to remove noise by a high-order function. As a result, the natural frequencies of the pile before and after shaking with Level 1 earthquake motion were not changed, and those before and after shaking with Level 2 earthquake motion were found to be slightly lowered (from 0.85 Hz to 0.75 Hz). This indicates that even under strong earthquake motion there were no major changes in the seismic response of the pile, and that the piles in composite ground have sufficient seismic resistance.

Table 1. Average coefficients of horizontal subgrade reaction of piles predicted from static horizontal loading tests

Case	Condition	k_h (kN/m ²)
1	Original ground	675
2	Composite ground, before shaking with Level 1 earthquake motion	8127
3	Composite ground, after shaking with Level 1 earthquake motion	8010
4	Composite ground, before shaking with Level 2 earthquake motion	8460
5	Composite ground, after shaking with Level 2 earthquake motion	7887

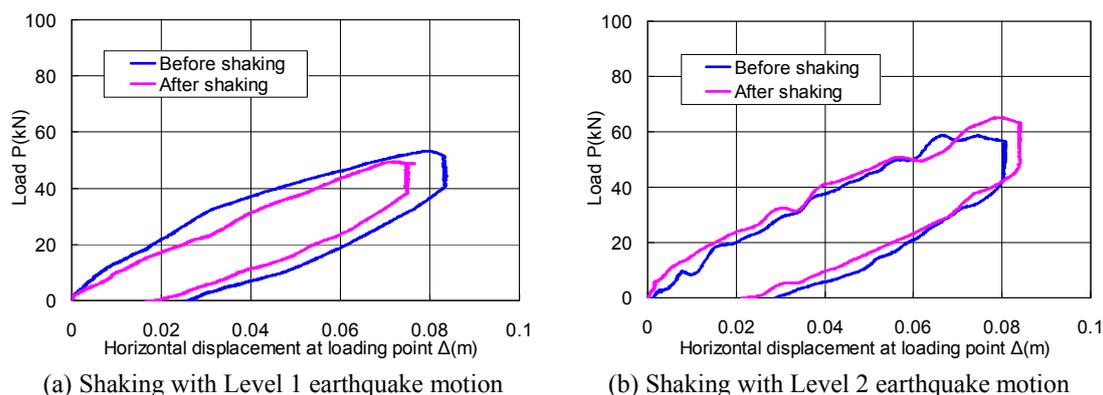


Figure 3. Loading curves obtained from static horizontal loading tests before and after shaking with different level of earthquake motion (in prototype scale)

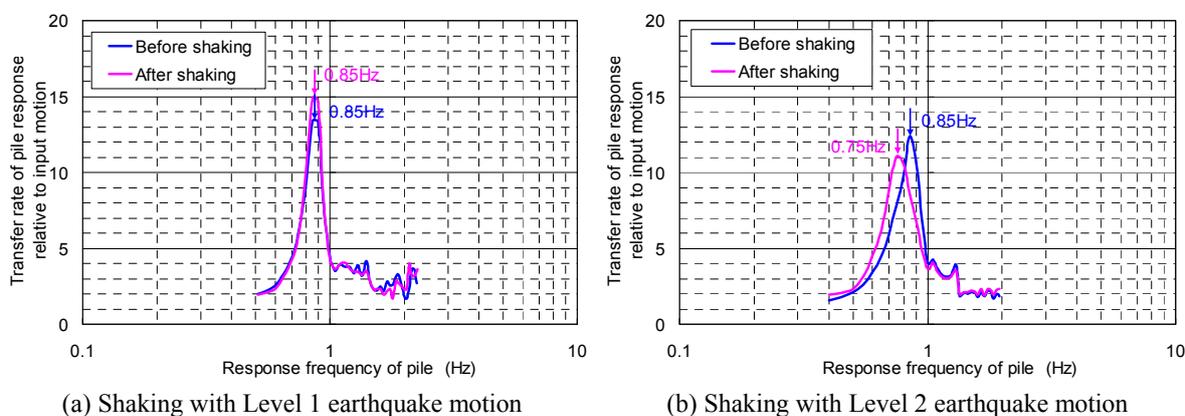


Figure 4. Change in frequency spectrum of piles before and after shaking with different level of earthquake motion (in prototype scale)

VERIFICATION OF THE SEISMIC PERFORMANCE OF COMPOSITE GROUND PILES THROUGH NUMERICAL ANALYSIS

Target site for analysis

The model used for analysis was an actual bridge abutment foundation, for which the composite ground pile method was adopted. The abutment was constructed on ground consisting of a sand layer subject to liquefaction at the top and soft silt at the lower layers. Figure 5 shows the structure of the abutment foundation. Cast-in-place piles (diameter: $D = 1.2$ m, length: $L = 13$ m, pile arrangement: $n = 3 \times 5 = 15$) were constructed on a bearing layer of shale bedrock. The range of ground improvement was set in accordance with the proposed basic design method. The coefficient of horizontal subgrade reaction k_0 was calculated based on the modulus of deformation E_0 of each layer of original ground, and the improvement depth was $1/\beta = 7.0$ m. An improvement width equivalent to the range of passive failure was set as 7.0 m from the piles of both ends, on the assumption that the angle of shear resistance of the original ground was $\phi = 0$. As specifications for ground improvement, the improvement rate of $\alpha_p = 78.5\%$ and the unconfined compressive strength $q_{up} = 400 \text{ kN/m}^2$ of improved columns were adopted. The earthquake resistance of piles used for this abutment was verified by the seismic intensity method under Level 1 earthquake motion and the horizontal load-carrying capacity method under Level 2 earthquake motion in accordance with the Specifications for Highway Bridges (Japan Road Association, 2002a and 2002b).

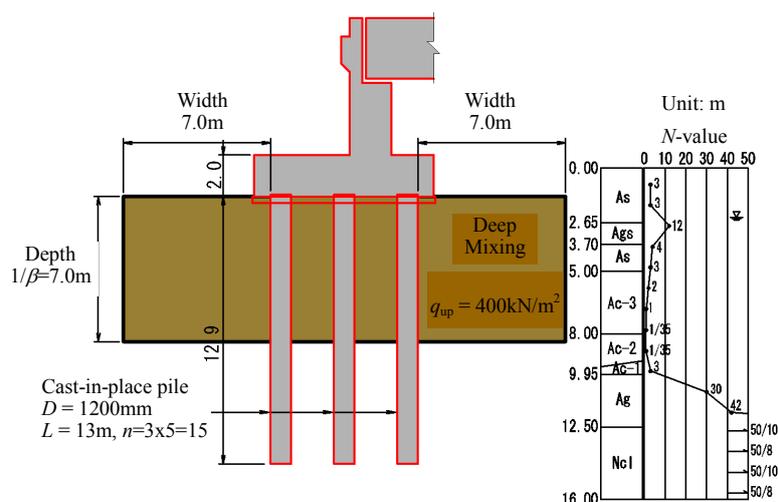


Figure 5. Outline of the target site for analysis

Analysis model and input earthquake motion

A plate element was used as a two-dimensional analysis model (Figure 6). The footing width was used as the depth of the analysis model taking the correlation between the results of three- and two-dimensional pile foundation analysis into account, based on the results of previous studies (Ishihara et al., 1994, Kurosawa et al., 1994). A nonlinear constitutive law of materials was applied to the piles and ground, and the footing and abutment were treated as linear elastic elements (Ashif & Maekawa, 1996). Pile components with circular cross sections were replaced by those with rectangular cross sections, with which the second moment of area of the piles would be equivalent. Joint elements were inserted at all the boundaries between the structure and ground. The width of analysis model was set as approximately 10 times of the total ground thickness (width: 157,300 mm), and viscous boundary elements were applied at vertical boundaries.

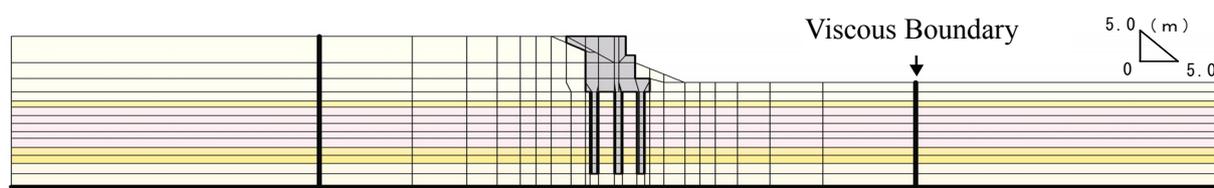


Figure 6. 2-D dynamic nonlinear finite element model

In the model, eight-node plane stress elements were used for the abutment and piles and eight-node plane strain elements were used for the ground. As viscous boundary elements, six-node joint elements, which were obtained by reducing the degree of freedom from the eight-node plane elements, were applied. The contact and detachment between the structural element and the ground were also taken into account by inserting similar joint elements. When using joint elements in the analysis model, attention was paid to the connection, in which ground elements positioned at the back of the structure were placed in an overlapping position. It means that joint elements were connected between the contact points of the piles and abutment with the ground in this model, to maintain the continuity of the ground at the back of the structure. For joint elements between the abutment/piles and the ground, the tensile and shear rigidity was assumed to be zero (i.e. equivalent to disregard for surface friction), and high compressive stiffness was applied in the contacting direction to avoid the overlapping of the ground elements and RC structure elements.

For the RC elements of piles, the history-dependent nonlinear constitutive law of reinforced concrete presented by Okamura (1991) and Maekawa et al. (2003) was applied. Applicability of this constitutive law to the non-orthogonal multidirectional crack model, the buckling model of reinforced concrete and other strongly nonlinear ranges was considered. In this constitutive law, confining pressure from the surrounding ground is also taken into account automatically. As ground elements, the Osaki's model (Osaki, 1980) was applied to the relationship between the deviator stress and strain, and linear elasticity was used as the hydrostatic element. As the properties of ground materials, the unit volume weight γ_0 and γ_c , the modulus of deformation E_0 and E_c , Poisson's ratio ν , the shear modulus of rigidity G_0 and G_c , shear strength S_u and C and the shear elastic wave velocity V_s were set respectively for the original and improved ground (Tables 2 and 3). G_0 , E_0 and S_u of the original ground were calculated by using following equations (Ashif & Maekawa, 1996).

$$G_0 = 11760N^{0.8} \quad (5)$$

$$E_0 = 2(1 + \nu) \cdot G_0 \quad (6)$$

$$S_u = (1000 \cdot G_0) / 600 \quad (\text{cohesive soil}) \quad (7)$$

$$S_u = (1000 \cdot G_0) / 1100 \quad (\text{sandy soil}) \quad (8)$$

Where, N is the N -value of original ground. The shear elastic wave velocity V_s of original and composite ground was calculated as follows.

$$V_s = \sqrt{gG / \gamma} \quad (9)$$

Where, g is the gravitational acceleration ($= 9.8\text{m/s}^2$) and γ is the unit volume weight of original or composite ground.

Table 2. Input parameters for soil ground

Symbol	Soil type	N -value	γ_0 (kN/m ³)	E_0 (kN/m ²)	ν	G_0 (kN/m ²)	S_u (kN/m ²)	V_s (m/s)
Bd	Sandy soil	3	19.0	74,000	0.3	28,000	33	118
As	Sand	1	17.0	31,000	0.3	12,000	11	76
Ac1	Clayey silt	2	16.5	53,000	0.3	20,000	24	100
Ag	Gravel	36	20.0	536,000	0.3	206,000	242	317
Ns1	shale	50	20.0	699,000	0.3	269,000	244	363

Table 3. Input parameters for the improved ground

q_{up} (kN/m ²)	γ_c (kN/m ³)	E_c (kN/m ²)	ν	G_c (kN/m ²)	C (kN/m ²)	V_s (m/s)
400	17.0	124,000	0.17	53,000	157	175

Material characteristics input for RC structure elements were the design compressive strength $f'_c = 24\text{N/mm}^2$ and tensile strength $f_t = 1.914\text{N/mm}^2$ of concrete and the design yield strength $f_y = 345\text{N/mm}^2$ of reinforcing bars (JSCE, 2002). As the input wave motions, the earthquake wave motions specified in the Specifications for Highway Bridges (Japan Road Association, 2002b) was adopted as shown in Figure 7. Level 2 earthquake motion was assumed to be that of a Type I inland strong earthquake.

Earthquake motion waves are the acceleration time-history waveform of phase characteristics, which is set by converting the acceleration response spectrum of past observation records into the spectrum immediately above a fault using a distance decay formula, while taking the fracture process of the fault into account. In dynamic analysis, the principal earthquake motion (12 seconds) of the waveform was extracted and direct integration was performed by Newmark's β method ($\beta = 0.36$). The time interval was counted as 0.01 seconds.

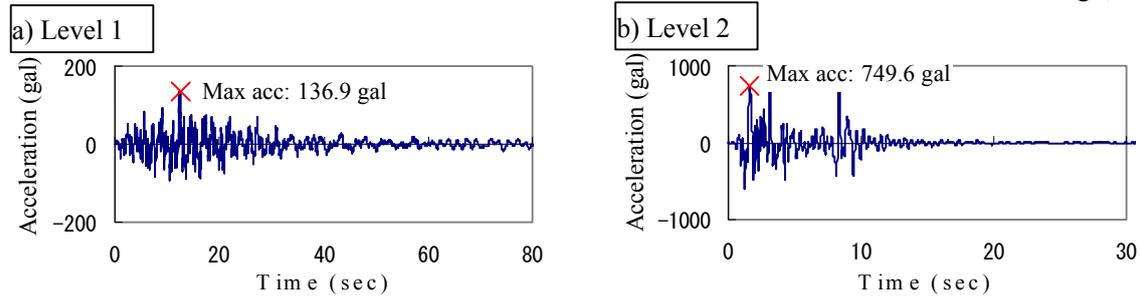


Figure 7. Input earthquake motion

Analysis results and discussion

As the analysis results, the deformation of pile under Level 1 and Level 2 earthquake motions was mainly focused. The data were analyzed by comparing those under the conditions with and without ground improvement.

Pile displacement

Figure 8 shows the time-history analysis results of the horizontal displacement of the footing under Level 1 and Level 2 earthquake motions in the cases with and without ground improvement. The displacement is the relative displacement at the bottom centre of the footing against the lower ends of the piles. The positive and negative values represent the displacement to the front and back sides, respectively. As a result, the maximum displacement of 12.7 mm on the front side under Level 1 earthquake in the case without ground improvement decreased by almost 50% to 11.1 mm in the case with ground improvement. The maximum displacement of 172.9 mm on the front side of the abutment under Level 2 earthquake in the case without ground improvement also decreased approximately 70% to 127.6 mm in the case with ground improvement. It means that pile displacement during earthquakes was controlled and seismic performance improved by ground improvement. On the back side, however, no significant difference in horizontal displacement was observed due to the influence of backfill at the back side of the abutment.

Pile strain

Figure 9 shows the time history of compressive and tensile strain in the axis direction of the pile heads under Level 1 earthquake in the cases with and without ground improvement. The maximum tensile strain, $\varepsilon_{tmax} = 0.42 \times 10^{-3}$, and compressive strain, $\varepsilon_{cmax} = -0.37 \times 10^{-3}$, at the pile heads in the case without ground improvement decreased slightly to $\varepsilon_{tmax} = 0.36 \times 10^{-3}$ and $\varepsilon_{cmax} = -0.28 \times 10^{-3}$ due to ground improvement. Similarly, Figure 10 shows the time history of compressive and tensile strain in the axis direction of the pile heads under Level 2 earthquake. The maximum tensile strain, $\varepsilon_{tmax} = 4.80 \times 10^{-3}$, and compressive strain, $\varepsilon_{cmax} = -2.15 \times 10^{-3}$, at the pile heads in the case without ground improvement decreased by half to $\varepsilon_{tmax} = 2.53 \times 10^{-3}$ and $\varepsilon_{cmax} = -0.89 \times 10^{-3}$ due to ground improvement. It means that improvement in earthquake resistance by ground improvement was more significant under Level 2 earthquake than under Level 1 earthquake.

Verification of seismic performance

The tensile strain ε_t under the yield stress of reinforcing bars was set as the limit value of Seismic Performance 1 (performance with which the soundness of the bridge will not be damaged by an earthquake) in the Specifications for Highway Bridges (Japan Road Association, 2002b). If tensile strain generated on reinforcing bars is smaller than the yield stress, it means that RC structure members are within the elastic range and the soundness can be maintained. The compressive strain ε_c at the maximum strength of concrete was also set as the limit value of Seismic Performance 2 (performance with which

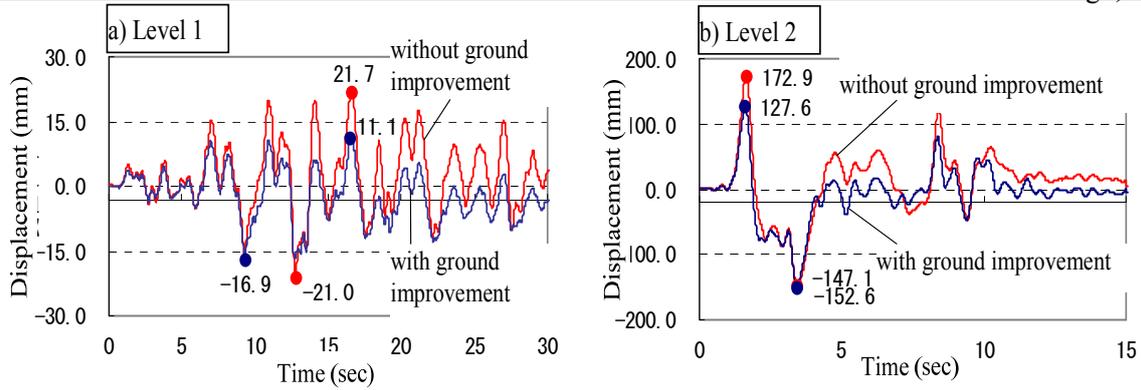


Figure 8. Time history of horizontal displacement of the footing

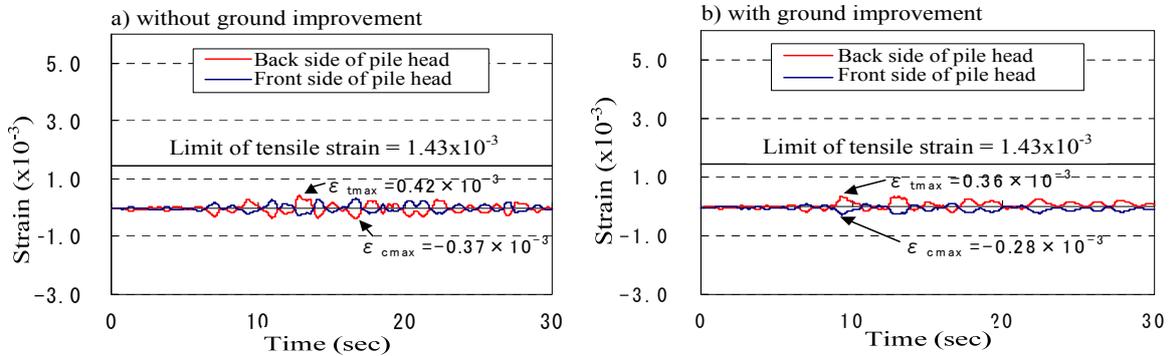


Figure 9. Time history of strain at pile head under Level 1 earthquake motion

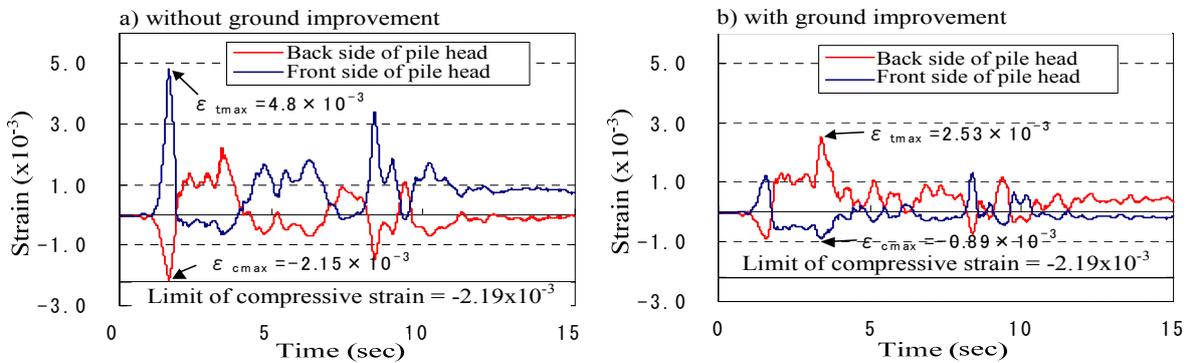


Figure 10. Time history of strain at pile head under Level 2 earthquake motion

damage by an earthquake can be limited and the bridge functions can be recovered immediately). If the compressive strain of concrete is smaller than the strain at the maximum strength, it means that damage to concrete is slight and limited, the immediate recovery of functions is possible and the seismic performance can be maintained (JSCE, 2002). In the case of a highway bridge, it is necessary to maintain Seismic Performance 1 and 2 under Level 1 and Level 2 earthquake motions, respectively.

The average tensile strain ε_t of reinforcing bars under the yield stress and the compressive strain of concrete ε_c at the maximum strength were calculated by Ashraf and Maekawa (1996). As a result, the limit value was set as $\varepsilon_t = 1.43 \times 10^{-3}$ under Level 1 earthquake and $\varepsilon_c = -2.19 \times 10^{-3}$ under Level 2 earthquake, as shown in Figures 9 and 10. The limit value of the tensile strain generated on piles was $\varepsilon_t = 1.43 \times 10^{-3}$ or lower in the cases with or without ground improvement, satisfying the required Seismic Performance 1.

The compressive strain under Level 2 earthquake was almost the same as the limit value of $\varepsilon_c = -2.19 \times 10^{-3}$ in the case without ground improvement, and the value was near the limit value of the required Seismic Performance 2. In the case with improvement, however, the value was much smaller than the limit value and Seismic Performance 2 was satisfied.

From the above study, it was made clear that it is possible to reduce pile displacement and strain under Level 1 and Level 2 earthquake motions and improve the seismic performance of piles by forming composite ground in the $1/\beta$ range of pile foundations for the bridge abutment. It is confirmed that the composite ground pile method provides sufficient seismic resistance of pile foundation to strong earthquake motion. On the basis of the results drawn from the numerical analysis together with the centrifuge model test, a verification flow chart for the seismic performance of the pile foundation in composite ground was proposed as shown Figure 11. The main consideration of the verification method is that the seismic performance of piles in composite ground should not only be verified by static analyses such as seismic intensity method and verification method of ultimate earthquake resistance, but also the detail checking through dynamic analysis using finite element method, although the dimension setting of the composite ground is mainly based on the characteristic length of pile $1/\beta$. Through the verification from numerical analysis and centrifuge model test, the flow chart shown in Figure 12 is considered as a reasonable design method for checking seismic performance of pile foundation in composite ground.

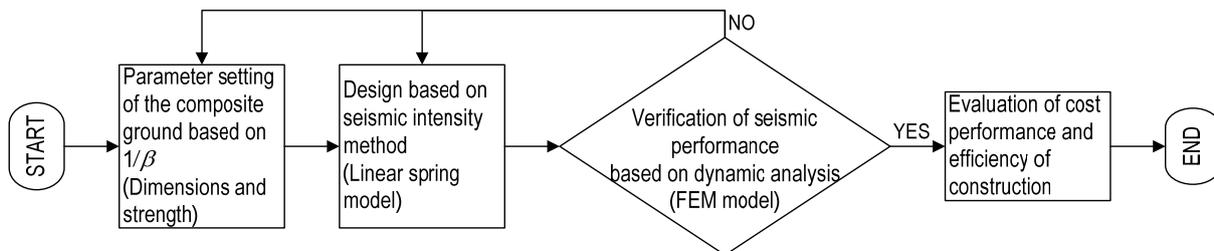


Figure 11. Flow chart for verification of seismic performance of pile foundation in composite ground

CONCLUSIONS

In this study, the earthquake resistance of piles in composite ground was verified through a series of centrifuge model test and dynamic nonlinear analysis using finite element method. The conclusions drawn from this study are summarized as follows:

- 1) Centrifuge model tests confirmed the effectiveness of horizontal subgrade reaction of the composite ground on piles comparing with the original ground. As a result of examining the response characteristics of pile through low level input motion shaking (acceleration amplitude in prototype scale: 20 gal) before and after shaking with strong earthquake motion, no significant changes were observed in the rigidity of the composite ground around the pile, and it was concluded that sufficient seismic performance was ensured in the composite ground pile method.
- 2) Two-dimensional nonlinear finite element analysis revealed that the horizontal displacement of the footing and compressive/tensile strain of concrete pile decreased by forming composite ground around piles. As a result, earthquake resistance improved under both Level 1 and Level 2 earthquake motions.
- 3) Pile foundations with composite ground designed by the seismic intensity method, where the improvement depth was set at the characteristic length of $1/\beta$, satisfied the required seismic performance under Level 1 and level 2 earthquake motions, as a result of verification by setting limit levels in accordance with the seismic performance guidelines provided in the Specifications for Highway Bridges.

- 4) On the basis of the results drawn from the centrifuge model test and numerical analysis, a reasonable design method for checking the seismic performance of pile foundation in composite ground was proposed for practical application.

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