EARTHQUAKE RESISTANCE OF PILE FOUNDATIONS IN COMPOSITE GROUND THROUGH DYNAMIC NONLINEAR NUMERICAL ANALYSIS

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ABSTRACT :

Earthquake resistance of pile foundations, established in the composite ground which was formed using the deep mixing method for the purposes of improving shear strength in soft ground was verified by a two-dimensional nonlinear dynamic finite element analysis. As a result, it was revealed that the displacement of pile foundations and the strain of pile bodies were restrained by composite ground around piles, and that the earthquake resistance of pile foundations was improved. It was also found that the earthquake resistance of pile foundations depends on the improved strength, improved width and improved depth of composite ground. The composite ground pile method is applicable for both Level 1 and Level 2 earthquake loadings.

KEYWORDS: piles, ground improvement, seismic design, dynamic finite elemennt method

1. INTRODUCTION

Although methods of ground improvement around piles [Akiyoshi et al., 2001, Nanjo et al. 2000] are being used for seismic strengthening of pile foundations, design methods have not been systematically established yet. There are, in particular, still many unclear points concerning the seismic performance of piles in improved ground. A 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 reducing 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 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 using *in-situ* static horizontal loading tests of piles and static finite element analysis. 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 pile foundations depending on earthquake levels and ground conditions. While several studies have been conducted on composite foundations combining piles and improved columns [Maeda et al., 2001, Maenaka et al., 2001], it is necessary to establish analytical and application methods for such foundations.

In this study, therefore, the earthquake resistance of pile foundations in composite ground under Level 1 and 2 earthquake motions was verified through a series of two-dimensional dynamic nonlinear finite element analyses. 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. On the basis of the analytical results, the seismic performance of the composite ground pile method was discussed.

2. DESIGN PROCEDURE OF THE COMPOSITE GROUND PILE METHOD

2.1 Consideration 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, $1/\beta$ ($\beta = (kD/4E_yI)^{1/4}$), 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. Broms [1964] and Reese et al. [1974] indicated the similar failure patterns of ground around a pile horizontally loaded. 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.

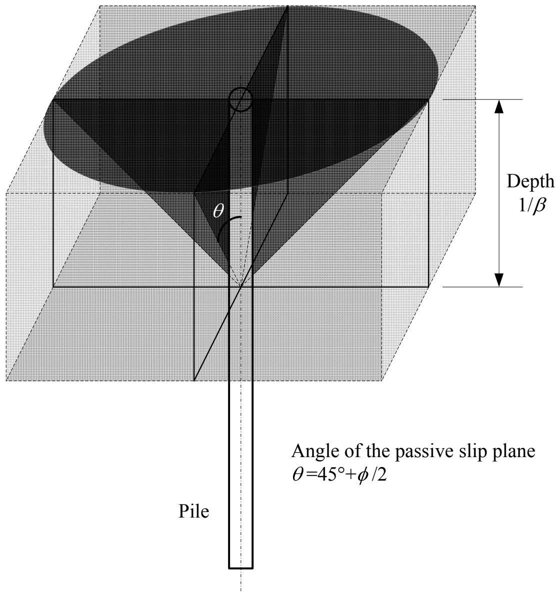

Figure 1 3-D image of the lateral resistance of pile foundation and the range of ground improvement

2.2 Method for determining horizontal subgrade reaction

When using the deep mixing method as the ground improvement method, the modulus of deformation of composite ground E_c is 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.

$$E_c = E_p \cdot \alpha_p + \alpha_s \cdot E_0(1-\alpha_p) \tag{1}$$

Where, α_s is the reduction rate of fracture strain. 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/m^2.

The coefficient of the horizontal subgrade reaction of piles in composite ground k_c can be calculated by using following equation from the modulus of deformation of composite ground E_c.

$$k_c = 1/0.3 \cdot \alpha \cdot E_c \cdot (\sqrt{D/\beta}/0.3)^{-3/4} \tag{2}$$

Where, α is the estimated coefficient of horizontal subgrade reaction, D is the pile diameter, β is the characteristic value of the pile. By setting the coefficient of horizontal subgrade reaction k_c using the above method, it becomes possible to design pile foundations under a static load in composite ground.

3. VERIFICATION OF THE SEISMIC PERFORAMCE OF COMPOSITE GROUND PILES

Dynamic analysis using the two-dimensional nonlinear finite element method was conducted to verify the validity of the range of ground improvement in the composite ground pile method and the difference in seismic performance in cases with or without improvement.

3.1 Target site for analysis

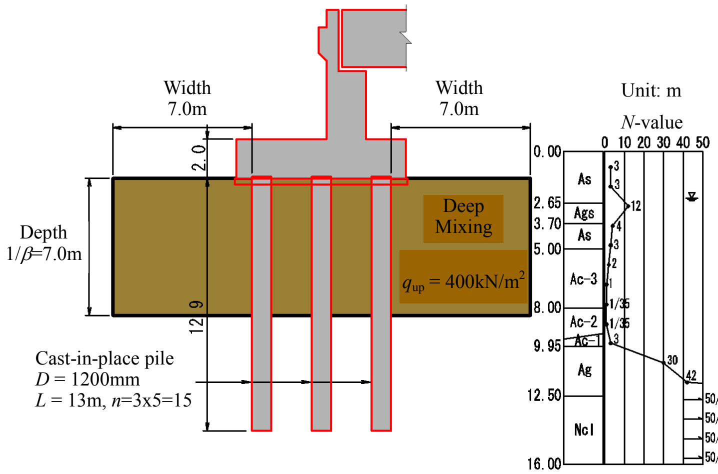

Figure 2 Abutment and the ground profile

The model used for analysis was an actual bridge abutment foundation, for which the composite ground pile method was adopted, taking the versatility and commonality of study results into account. 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 2 illustrates the structure of the abutment foundation. At this site, a static horizontal loading test was conducted after the construction of piles to check the static coefficient of the horizontal subgrade reaction k_c of composite ground [Tomisawa and Nishikawa, 2005a, 2005b]. Cast-in-place piles (diameter: D = 1,200 mm, length: L = 13 m, pile arrangement: n = 3×5 = 15) were constructed on a bearing layer of shale bedrock. The range of ground improvement is as shown in Figure 2. This range 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 ϕ = 0. As specifications for ground improvement, the improvement rate of α_p = 78.5% and the unconfined compressive strength q_{up} = 400kN/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, 2002b].

3.2 Analysis model and input earthquake motion

A plate element was used as a two-dimensional analysis model (Figure 3). 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

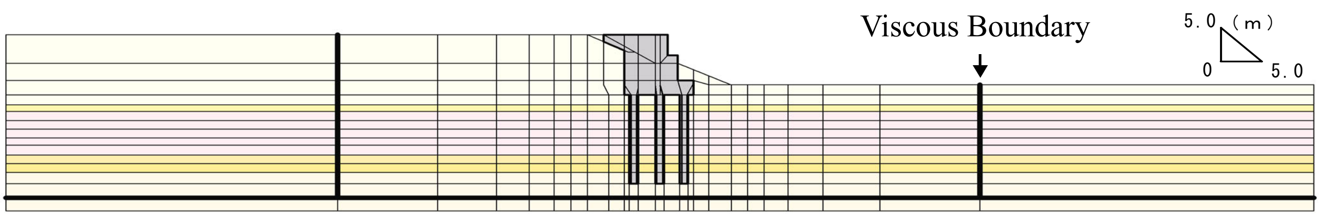

Figure 3 2-D dynamic nonlinear finite element model

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 I 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) as shown in Figure 3, and viscous boundary elements were applied at vertical boundaries.

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 1 and 2). G_0, E_0 and S_u of the original ground were calculated by using following equations [Ashif & Maekawa, 1996].

$$G_0 = 11760N^{0.8} \tag{3}$$

$$E_0 = 2(1+\nu)\, G_0 \tag{4}$$

$$S_u = (1000\, G_0)/600 \quad \text{(cohesive soil)} \tag{5}$$

$$S_u = (1000\, G_0)/1100 \quad \text{(sandy soil)} \tag{6}$$

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.

$$V_s = (gG/\gamma)^{1/2} \tag{7}$$

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

Material characteristics input for RC structure elements were the design compressive strength f'_c = 24N/mm^2 and tensile strength f_t = 1.914N/mm^2 of concrete and the design yield strength f_y =345N/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 4. 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 (β=0.36). The time interval was counted as 0.01 seconds.

Table 1 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 2 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

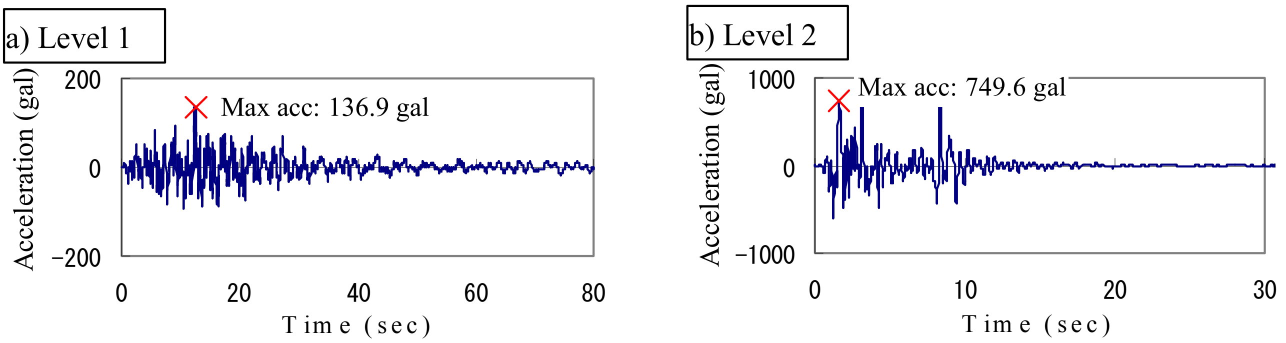

Figure 4 Input earthquake motion

3.3 Analysis results and discussion

3.3.1 Pile displacement

Figure 5 illustrates 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 center 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.

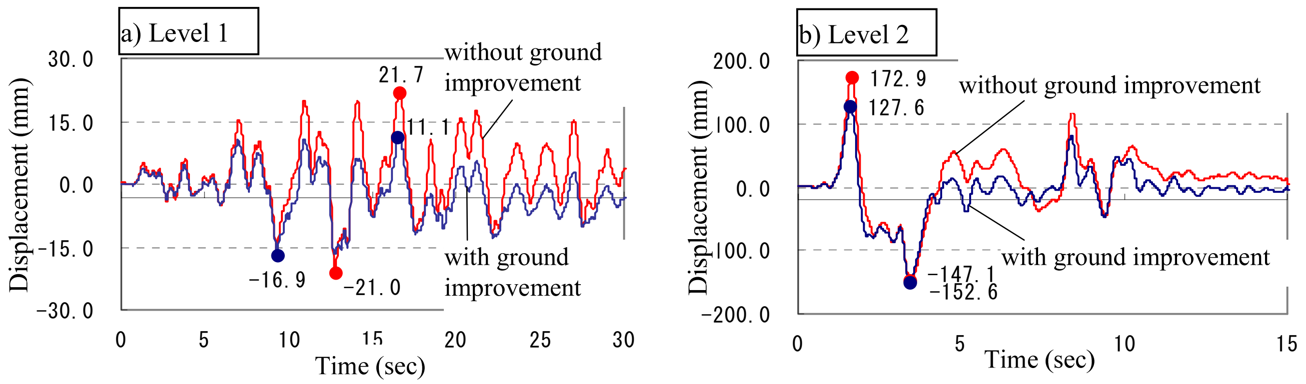

Figure 5 Time history of horizontal displacement of the footing

3.3.2 Sectional force of piles

Figure 6 shows the distribution of the maximum bending moment M and maximum shear force S of the piles under Level 2 earthquake, which were obtained from analysis conducted for the cases with and without ground improvement. The target piles were front side piles. The bending moment of piles M at the pile heads, which was M = 0.18 kN-m in the case without ground improvement, decreased less than 30% to M = 0.05 kN-m due to ground improvement. Similarly, shear strength S, which was S = 450 kN at the pile heads in the case without ground improvement, decreased almost 1/3 to S = 120 kN in the case with ground improvement. Although the shear strength S increased at the improvement boundary in the case with ground improvement, it was not a problem since it was the same as the value at the pile head of 450 kN in the case without ground improvement. Thus the sectional force of piles also tended to decrease similarly to the displacement due to the ground improvement. The composite ground pile method leads to the improvement in seismic performance of the pile foundation.

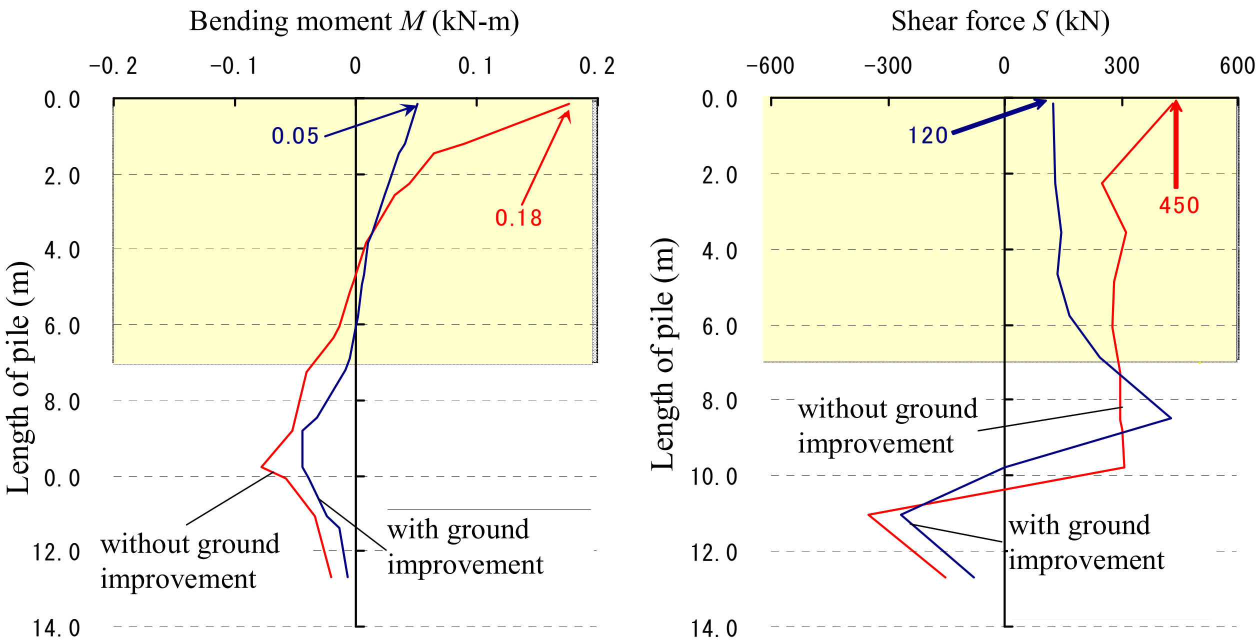

Figure 6 Distribution of sectional force of pile under Level 2 earthquake motion

3.3.3 Pile strain

Figure 7 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, ε_{tmax} = 0.42×10^{-3}, and compressive strain, ε_{cmax} = -0.37×10^{-3}, at the pile heads in the case without ground improvement decreased slightly to ε_{tmax} = 0.36×10^{-3} and ε_{cmax} = -0.28×10^{-3} due to ground improvement. Similarly, Figure 8 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, ε_{tmax} = 4.80×10^{-3}, and compressive strain, ε_{cmax} = -2.15×10^{-3}, at the pile heads in the case without ground improvement decreased by half to ε_{tmax} = 2.53×10^{-3} and ε_{cmax} = -0.89×10^{-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.

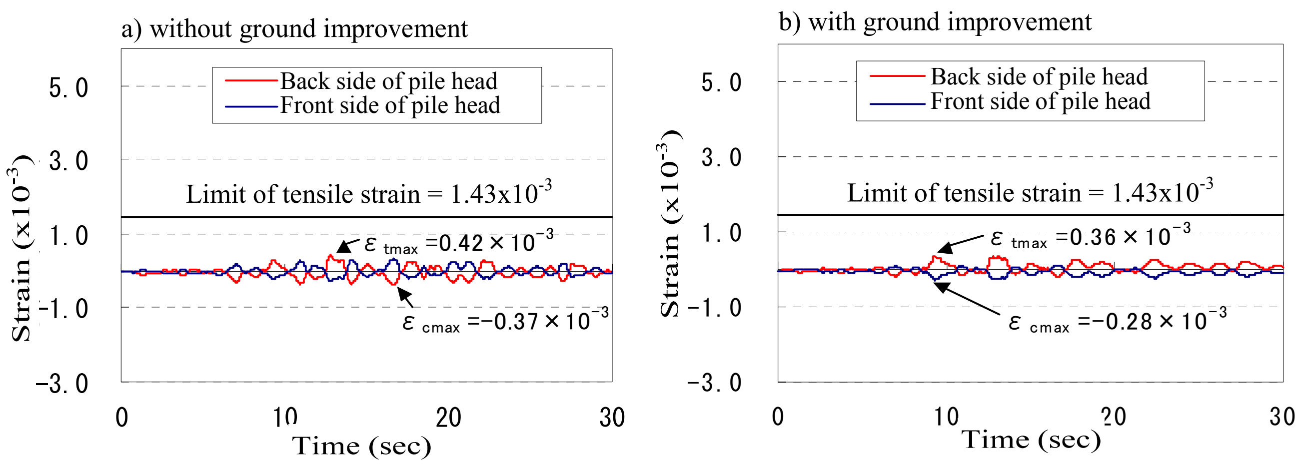

Figure 7 Time history of strain at pile head under Level 1 earthquake motion

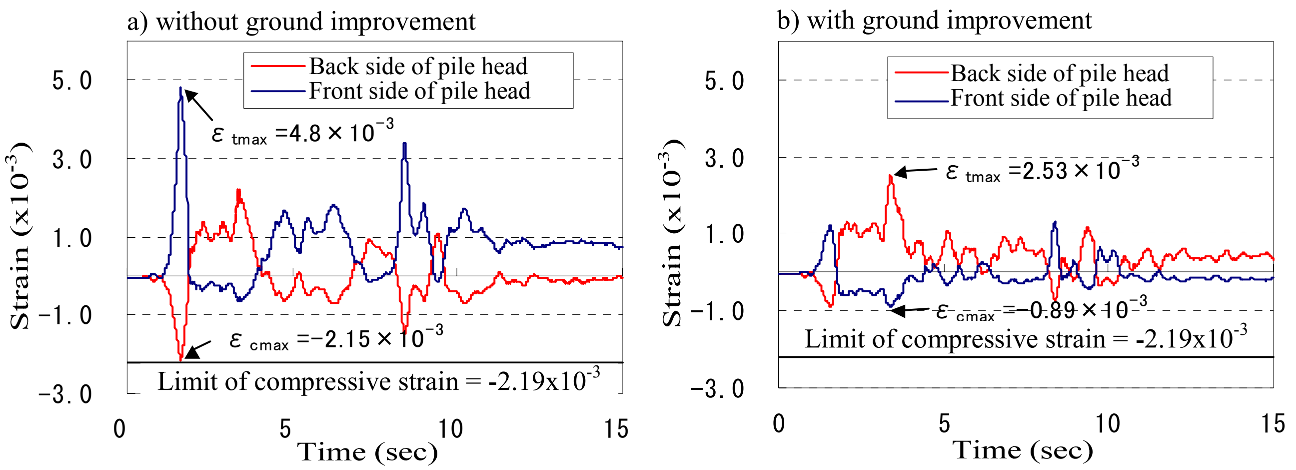

Figure 8 Time history of strain at pile head under Level 2 earthquake motion

3.3.4 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 [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 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 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 $\varepsilon_t = 1.43\times10^{-3}$ under Level 1 earthquake and $\varepsilon_c = -2.19\times10^{-3}$ under Level 2 earthquake, as shown in Figures 7 and 8. The limit value of the tensile strain generated on piles was $\varepsilon_t = 1.43\times10^{-3}$ 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 $\varepsilon_c = -2.19\times10^{-3}$ 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 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.

4. CONCLUSIONS

In this study, the earthquake resistance of piles in composite ground was verified by the dynamic nonlinear finite element analysis method. The results can be summarized as follows:

1) Two-dimensional nonlinear finite element analysis revealed that the horizontal displacement of the footing, sectional force generated on piles 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 2 earthquake motions.

2) Although the shear strength of piles increased due to the difference in ground rigidity in the area near the boundary between the composite and original ground, it did not exceed the shear strength at the pile heads in the case without improvement and did not have significant influence on earthquake resistance.
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 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.

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