Aseismic Reinforcement Method for Existing Pile Foundation with Improvement Soil

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SUMMARY:
In this paper, a new simple earthquake resistance reinforcement method for existing pile foundation with improvement soil was developed in order to reduce the damage of pile foundation structures. During earthquakes, pile foundation of structure is subjected to lateral force. The largest bending moment and shear force of pile are appearing at pile head, and pile may be broken at this position due to severe seismic force. In this new aseismic reinforcement method, improvement soil will be set around pile to reduce the bending moment and shear force at pile head and increase its protection level against seismic force.

Keywords: Reinforcement, Existing pile foundation, Numerical analysis, Bending moment, Shear force

1. INTRODUCTION

In recent years, the great quake happened frequently. A lot of lives were lost. Various structures and lifelines were heavily damaged. Therefore, the earthquake resistant design methods of various structures are developed (Fellenius, 2004). However, a lot of structures designed by old foundation engineering still exist now, including pile foundation on soft ground without lateral resistance. They are very dangerous if earthquake occurs. The aseismic reinforcement method for existing pile foundations is difficult. It is limited by construction conditions. In this study, a new simple aseismic reinforcement method for existing pile foundation with improvement soil was developed in order to reduce the damage by seismic force.

2. CONCEPT AND PRINCIPLE OF THE REINFORCEMENT METHOD FOR EXISTING PILE FOUNDATION WITH IMPROVEMENT SOIL

Fig. 2.1 is view showing a frame format of the reinforcement method for existing pile foundation with improvement soil. This reinforcement method has aimed to decrease bending moment and shear force at pile head. In other words, it is reinforcement method to decrease horizontal seismic force which acts on pile. Improvement soil was set around pile foundation in order to complete this aim. Passive earth pressure in front of improvement soil is acting as bearing force to horizontal seismic force. Therefore, required horizontal bearing force of pile head could be decreased.

Various reinforcement methods were developed for pile foundation (Tomisawa et al, 2005). When this reinforcement method for existing pile foundation could put to practical use, it is not necessary to dig up the ground but only requisite area need to improved by the mixing method or other methods. Therefore, construction is relatively simple, the cost and time for completion could be reduced. On the other hand, the depth and the thickness of the improvement can be examined reasonably and handily by following procedure.

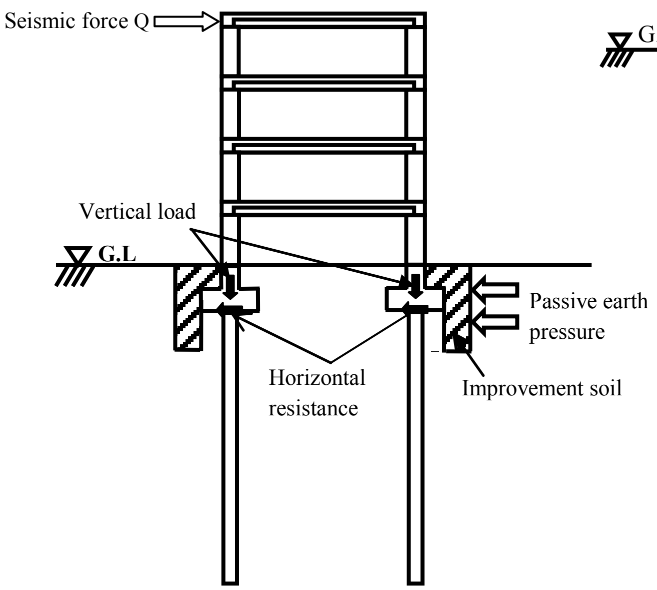


Figure 2.1. Prototype of the reinforcement method for existing pile foundation with improvement soil

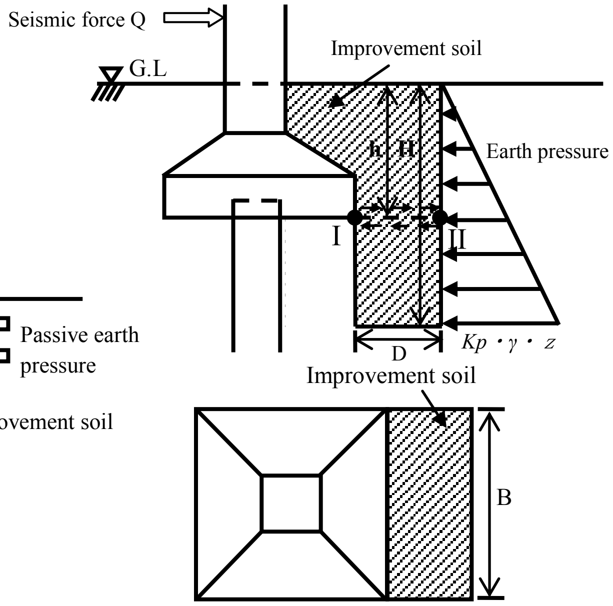


Figure 2.2. Detailed view of footing and pile head part

Fig. 2.2 shows the detailed view of footing and pile head part. Inequality (2.1) could be derived from the equilibrium of horizontal force and inequality (2.2) could be derived from the shear failure condition of section (I-II) as shown in Fig. 2.2.

$$\left(K_p \cdot \gamma \cdot H^2/2\right) \times B > Q \tag{2.1}$$

$$\tau_{max} \cdot B \cdot D > \int_h^H \left(K_p \cdot \gamma \cdot z\right) dz \times B \tag{2.2}$$

where, γ is unit weight of soil, K_p is coefficient of passive earth pressure, H is depth of improvement soil, B is width of improvement soil, D is thickness of improvement soil, Q is horizontal seismic force, z is vertical depth, h is depth from the ground level to the footing bottom, $\tau_{max}=q_u/2$ is maximum shear resistance stress of improvement soil and q_u is unconfined compressive strength of improvement soil. Inequalities (2.3) and (2.4) are obtained by rewritten inequality (2.1) and (2.2).

$$H > \sqrt{2 \cdot Q / \left(K_p \cdot \gamma \cdot B\right)} \tag{2.3}$$

Improvement soil depth H could be decided from inequality (2.3).

$$D > K_p \cdot \gamma \cdot (H^2 - h^2)/q_u \tag{2.4}$$

Improvement soil thickness D could be decided from inequality (2.4).

3. NUMERICAL ANALYSES

An easy model is analyzed by using 3-D finite element method (FEM) in order to examine the improved effect of this improvement method. Decreasing rate of bending moment and shear force of pile head could be checked.

3.1. Analytical model

The assumptive superstructure model is a four-story building, the structural drawing and piling plan of analytical model is shown in Fig. 3.1 and 3.2, respectively. Design load of the floor and roof are assumed as $10kN/m^2$. The area of one span is made an analytical object to simplify the analysis and the structural weight W of one span is 4000kN ($=10kN/m^2 \times 10m \times 8m \times 5$). Therefore, a vertical load of 2000kN (=W/2) is acting on each footing. In addition, the length of the analytical area is designed under considering the angle of failure zone $\theta=\pi/4+\varphi/2$ (φ : internal friction angle of ground). The analysis area is 8m in width ×50m in length ×17m in depth. The finite-element mesh is shown in Fig. 3.3. 222 bar elements are used for beams, columns and piles. 8-node rectangular elements used for ground are 54400 and the number of total nodes is 60225.

Total horizontal seismic force was set as Q=4200kN almost corresponding to the structural weight to examine the improved effect in final plastic state of ground in pre-analysis, and the horizontal seismic force is divided into 21 steps in the incremental analysis.

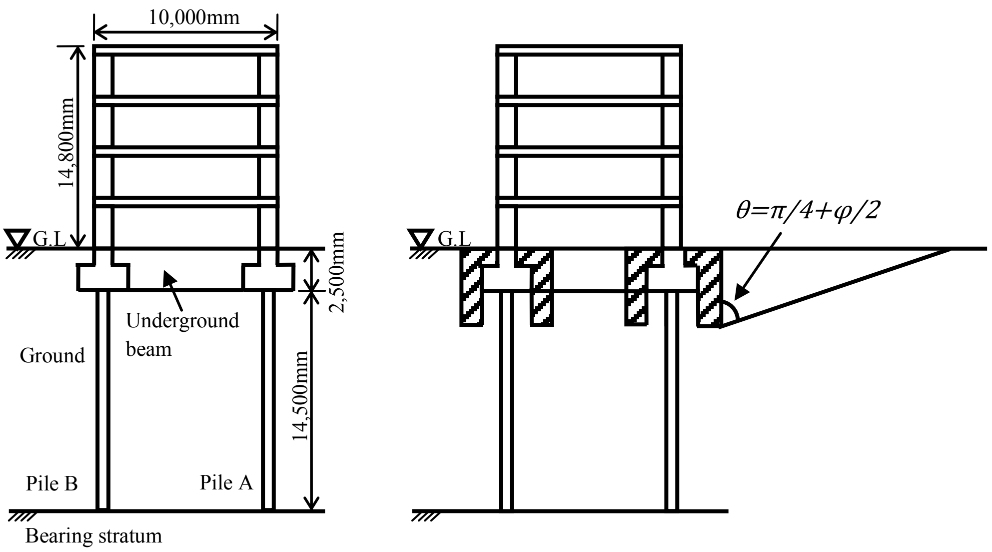
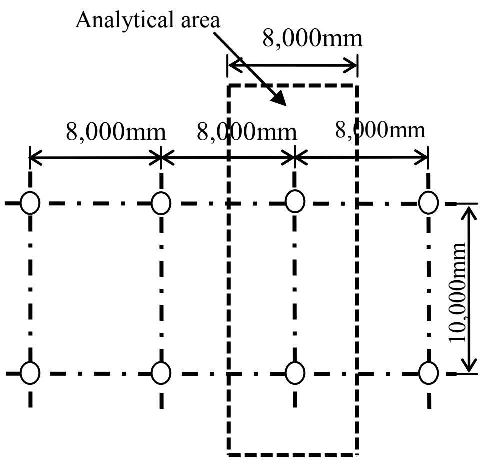


Figure 3.1. Structural drawing



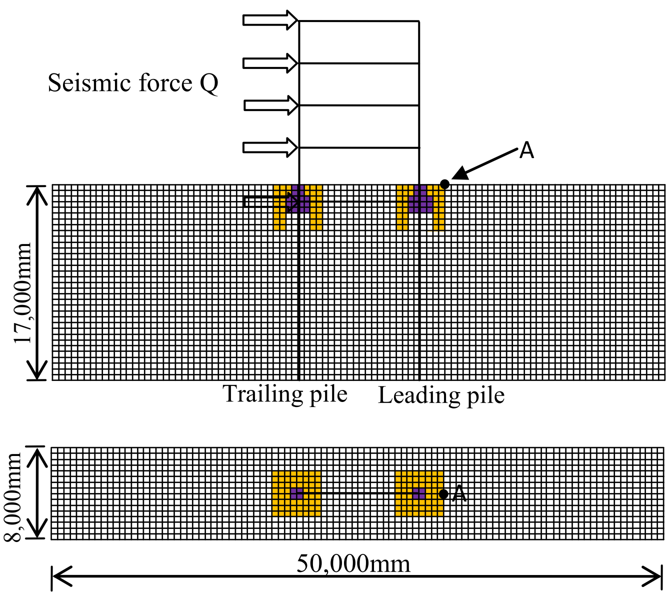


Figure 3.2. Piling plan

Figure 3.3. Finite- element mesh

3.2. Analytical model for ground

On making use of a FEM analysis program AFIMEX(3D), the nonlinear behavior of ground was modeled using the Drucker-Prager model which is the failure criterion on elasto-plastic stress-strain model of ground. It is described in equation (3.1). (Ugai et al. 2003)

$$f = \alpha I_1 + \sqrt{J_2} - \kappa = 0 \tag{3.1}$$

where, I_1 is first stress invariant and J_2 is second deviatoric stress invariant.

α and κ were related with cohesion C and internal friction angle φ in equation (3.2).

$$\alpha = sin\,\varphi/\sqrt{9+3sin^2\varphi},\ \ \kappa = 3C\,cos\,\varphi/\sqrt{9+3sin^2\varphi} \tag{3.2}$$

In this program, associated flow rule is employed as the internal friction angle = dilatancy angle. The initial stiffness coefficient of the ground is obtained by using the concise equation from Duncan-Chang model as equation (3.3). (Duncan and Chang 1970)

$$E_i = K \cdot P_a \cdot (\sigma_3/P_a)^{n_1} \tag{3.3}$$

where, K and n_1 are coefficients concerning the initial stiffness, P_a is atmospheric pressures, and σ_3 is initial minor principal stresses. In this study, K=801 and n_1=0.585 were decided by triaxial compression test on dense sandy soil.

3.3. Numerical parameters

The stiffness coefficient of the improvement soil is determined to be 50 times the stiffness coefficient of the soil using examples from the results of unconfined compressive strength of improvement soil. The stiffness coefficient of pile uses standard stiffness coefficient 2.5×10^7 kN/m^2 of the cast-in-place concrete pile. It is assumed that the beams and the columns are made of concrete, and uses the same elasticity coefficient as pile.

The parameters used in the analysis are shown in Table 3.1. The sandy soil was used as a material with some adhesion in order to make the stability of analysis, though it is usually used cohesion C=0 for sandy soil. C=1000 and φ=5°are set for improvement soil and footing in order to be behave as elastic material with large tensile strength.

Table 3.1. Material parameters

	Stiffness coefficient E(kN/m^2)	Poisson's ratio v	Cohesion C(kN/m^2)	Internal friction angle φ	Unit weight γ(kN/m^3)
Sandy soil	6952	0.3	5	30°	16
Improvement soil	347600	0.2	1000	5°	17
Footing	2.5×10^7	0.2	1000	5°	23.5
	Stiffness coefficient E(kN/m^2)	Geometrical moment of inertia Iz (m^4)	Geometrical moment of inertia Iy (m^4)	Shear modulus G(kN/m^2)	Unit weight γ(kN/m^3)
Pile(ϕ=900mm)	2.5×10^7	0.0322	0.0322	1.04×10^7	24
Beam(1000mm×350mm)	2.5×10^7	0.003573	0.02917	1.04×10^7	24
Column(600mm×600mm)	2.5×10^7	0.0108	0.0108	1.04×10^7	24
Underground beam (2,500mm×600mm)	2.5×10^7	0.7813	0.1528	1.04×10^7	24

3.4. Analytical cases

Ten analytical cases were carried out. The details are listed in table 3.2. The combinations of improvement depth H (0.0m, 2.5m, 4.0m and 5.5m) and thickness D (0.0m, 0.5m, 1.0m and 2.0m) were changed to examine the effect of these two parameters.

Table 3.2 Analytical cases

Thickness of improvement / Depth of improvement	D=0.0m	D=0.5m	D=1.0m	D=2.0m
H=0.0m	Case1			
H=2.5m		Case2	Case5	Case8
H=4.0m		Case3	Case6	Case9
H=5.5m		Case4	Case7	Case10

4. ANALYTICAL RESULTS

4.1. Ultimate load and design criteria load

Fig. 4.1 shows horizontal displacement - horizontal seismic force relationship of point A (in Fig. 3.3) from the analytical results of case1 which both improvement depth H and thickness D are 0.0m (without improvement). Three special steps of analytical results were investigated to check the improved effect of this method. Horizontal seismic force Q=4200kN was set as mentioned above. As shown in this figure, the analytical curve line converge to Q=4200kN, so the horizontal seismic force Q=4200kN of the 21 step is be regarded as ultimate load. One third of the ultimate load (safety factor=3) for the 7 step is regarded as design criteria load, and the 1 step is be regarded as elastic state.

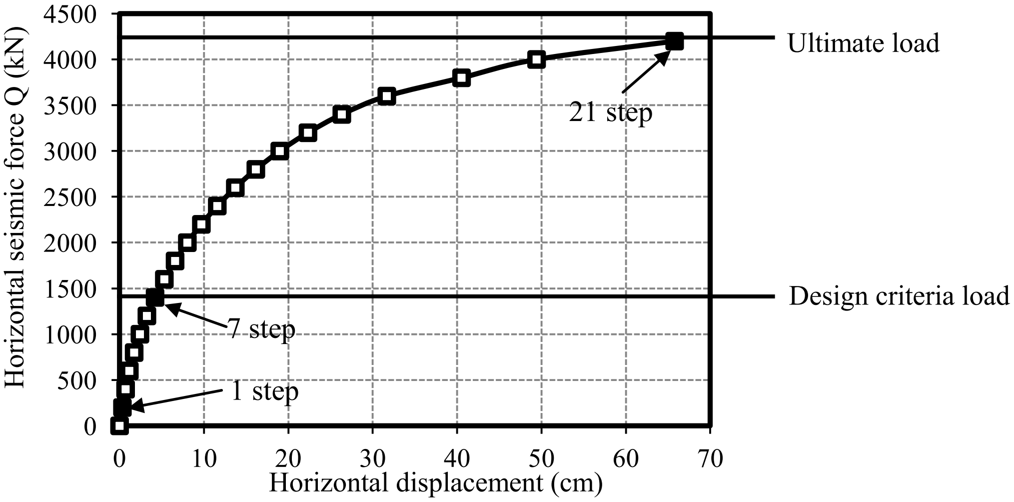


Figure 4.1. Horizontal displacement - horizontal seismic force relationship

4.2. Bending moment

The pile which be located ahead of loading direction is called leading pile and the pile which be located behind to loading direction is called trailing pile as shown in Fig. 3.3. The passive earth pressure in front of improvement soil around the leading pile is larger than that in front of improvement soil around the trailing pile. Therefore, the pile head bending moment and shear force of the trailing pile are lesser than those of the leading pile. However, the difference will becomes smaller when the piles are placed farther away each other. Here, the analytical results of leading pile are presented as an example.

Fig. 4.2 shows the pile bending moment destricution of pile from one set of analytical cases with thickness of improvement soil D=0.0m (case1) or 2.0m (case 8, 9 and 10). There are 3 graphs which show the different steps of analytical results in Fig. 4.2. The bending moment of pile decreases considerably with the increasing depth H of improvement soil, especially at the pile head. For example, the beding moment of pile head for case 8 (D=2.0m, H=2.5m) is nearly 50% of that for case 1 (D=0.0m, =0.0m) in graph (a). However, the beding moment of pile head for case 10 (D=2.0m, H=5.5m) nearly 20% of that for case1.

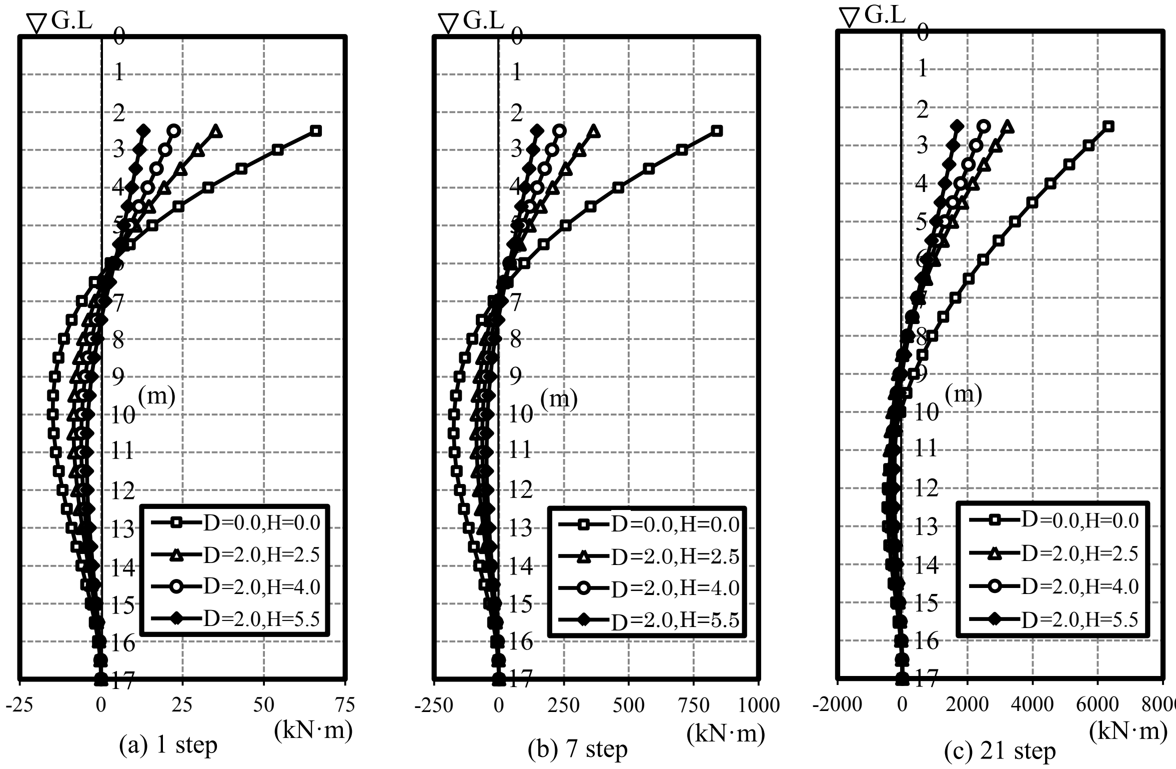


Figure 4.2. Bending moment distribution of pile

In order to clarify the improved effect, the beding moment and shear force of pile head in case 1 are regarded as refernce value. Results of the other cases are divided by this refernce value and the ratio is represented as decresing rate. This is a simple way to find out the decrese of bending moment or shear force of pile head.

Fig. 4.3 shows the decreasing rate for bending moment of pile head. Each curve in graphs (a), (c) and (e) are decreasing rate when the thickness of improvement soil D is constant, and curves in graphs (b), (d) and (f) are decreasing rate when the depth H is constant. It is easily to find out the influence of different parameters by these graphs. Decreasing rate becomes smaller as the thickness or depth of improvement soil is increasing as shown all of these graphs. Moreover, from graphs (a), (c) and (e) of Fig. 4.3, it seems that the tendency of the relationship between the improvement depth and the decreasing rate is almost nearly straight line when improvement thickness is constant. It is to say that the decreasing rate maybe continue reducing if the depth of improvement soil increasing. The decreasing rate changes little when improvement thickness larger than 0.5m as shown graphs (b), (d) and (f). For example, the difference of decrease rate is only 0.2 even if the improvement depth is increase from 0.5m to 2.0m in case of the depth of improvement soil is 5.5m in graph (f). It means that the improved effect could not be expected even if the improvement thickness D is larger than 0.5m in these analytical cases.

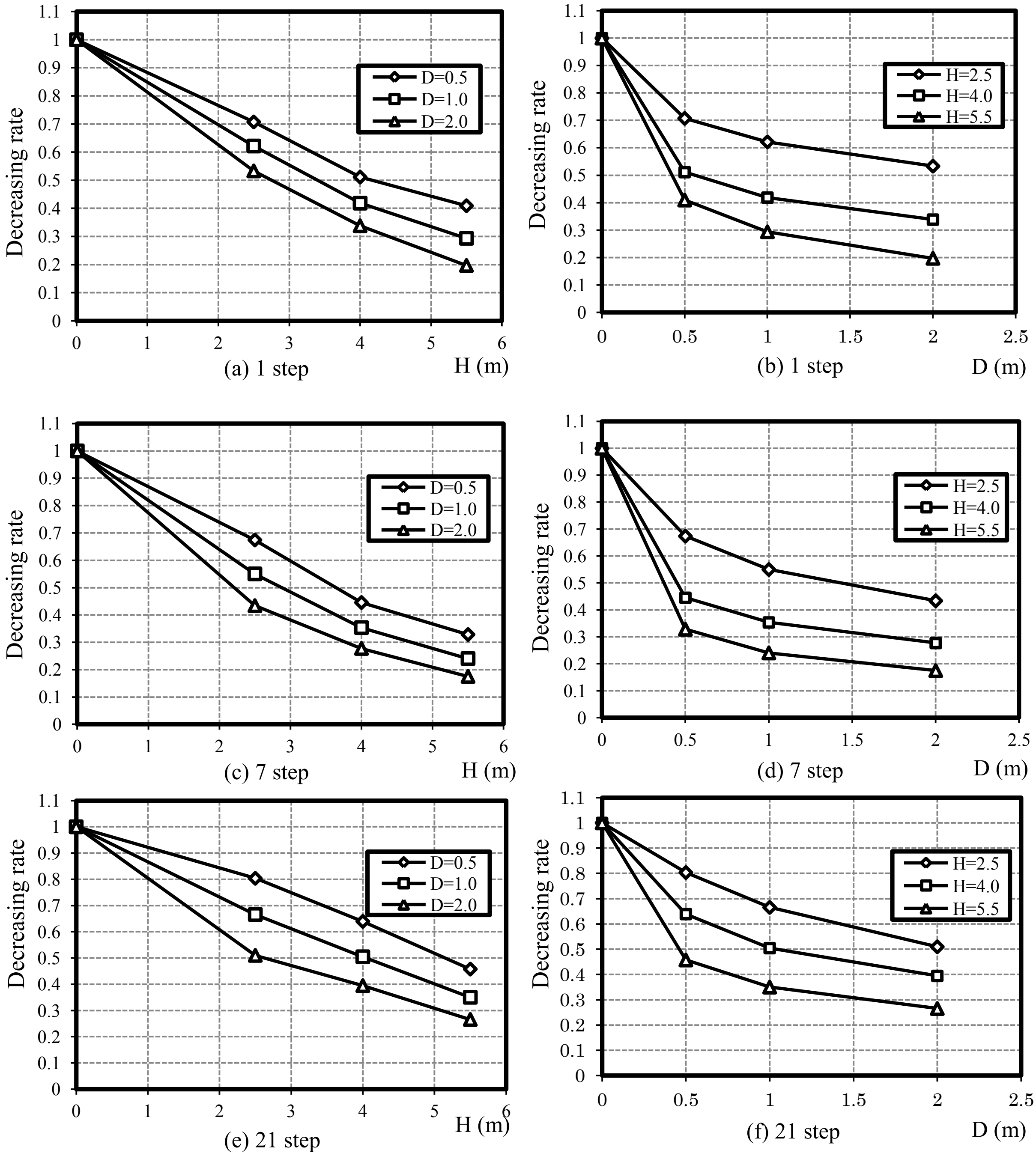


Figure 4.3. Decreasing rate of pile head bending moment

4.3. Shear force

Fig. 4.4 shows the shear force distribution of pile from the same set of analytical cases as shown Fig. 4.2. Three steps (1, 7 and 21) of analytical results are respectively shown in graph (a), (b) and (c). The shear force of pile decreases with the increasing of the depth of improvement soil and this is the same tendency as bending moment. For example, the shear force of pile head for case 8 (D=2.0m, H=2.5m) is nearly 50% of that for case 1 (D=0.0m, =0.0m) in graph (a). Furthermore, the shear force of pile head for case 10 (D=2.0m, H=5.5m) is nearly 10% of that for case1 in graph (a). The maximum shear force is appeared at the bottom of improvement soil when H is 4.0m or 5.5m in graph (c), and these values are much smaller than the maximum shear force of case 1.

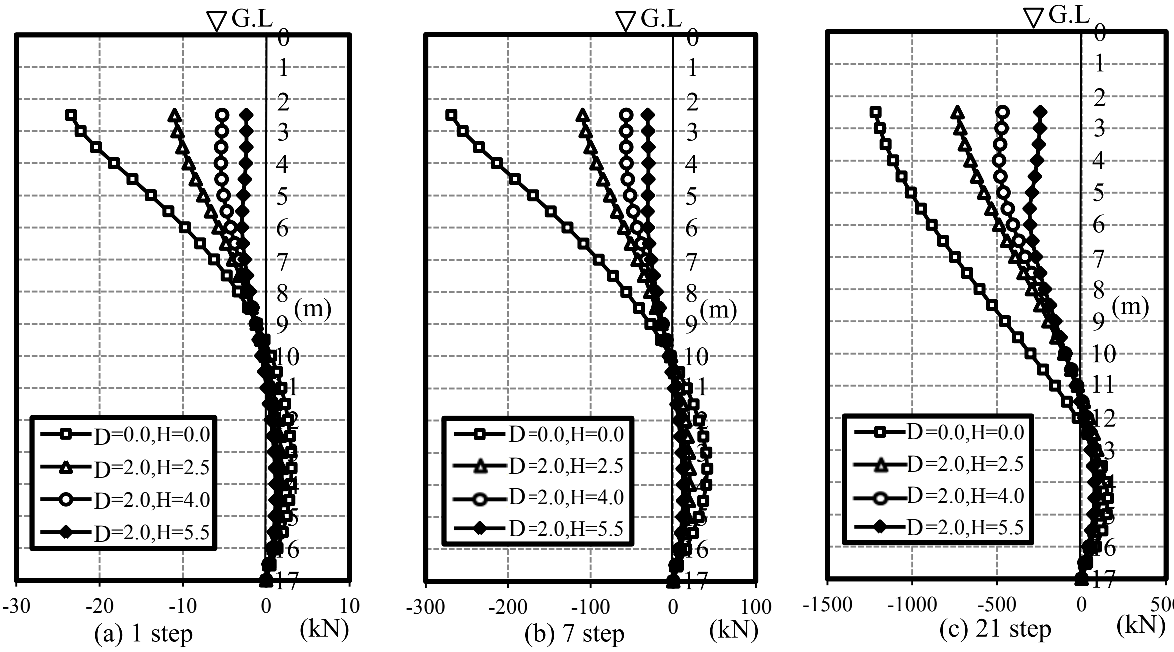


Figure 4.4. Shear force distribution of pile

Fig. 4.5 shows the decreasing rate of shear force of pile head. The curves in graphs (a), (c) and (e) are presented as the relationships between decreasing rate and the improvement depth H with the constant thickness D of improvement soil, and the curves in graphs (b), (d) and (f) are presented as the relationships between decreasing rate and the improvement thickness D with the constant depth H of improvement soil. The smaller decreasing rates are shown with increasing the thickness D or depth H of improvement soil. The shape of the curves in graphs (a) and (c) are very similar, and it seems skew points slightly around H=4.0m. According to the trend of these curves, the decreasing rate will not have a large reduction when improvement depth H is larger than 4.0m. The shape of curves in graphs (a) and (c) are different from the shape of curves in graph (e) which is the analytical results for near ultimate load. There are no skew points which are mentioned above. It seems that the decreasing rate will continue reducing if the improvement depth H increasing when the horizontal force is near ultimate load. The decreasing rate changes little when improvement thickness D is larger than 0.5m, and the curves (improvement depth H=4.0m, 5.5m) are very close as shown graphs (b) and (d). However, the curves (improvement depth H=4.0m, 5.5m) are not close in graph (f). Therefore, it was found that the decreasing rate will continue reducing if the improvement depth H increasing when the horizontal force is near ultimate load as mentioned above.

The analytical results of 1 step, 7 step and 21step when improvement depth D=0.5m are shown in Fig. 4.6. The improved effect of the 21 step (at near ultimate load) is smaller than that of the 1 step and the 7 step (under design criteria load). The improved effect of the 1 step and the 7 step are near. But the improved effect of the 7 step is little better than that of the 1 step. The decreasing rates of pile head bending moment though 21 steps are shown in Fig. 4.7 as an example in order to reveal the reason of inversion.

The decreasing rates of the 1 step and the 2 step are almost the same in Fig. 4.7(a). Theoretically, the decreasing rate is same in case of being in the elastic state. It seems that the decreasing rate is becomes smaller from the 3 step, and the minimum decreasing rate is shown at the 6 step in Fig. 4.7 (a). Since then, the decreasing rate becomes larger step by step with increasing the horizontal seismic force. The ground in front of the improvement soil will failure as the load increasing, and the increment of horizontal resistance will reduce gradually. However, the enclosed ground by the improvement soil has not failure before the 6 step, so the horizontal seismic force was more shared by the ground of this part without failure. This part will failure as horizontal force increasing, and the decreasing rate will increase. Therefore, when the enclosed ground by improvement soil becomes larger, the minimum increasing rate appear in later step as shown in Fig. 4.7(b).

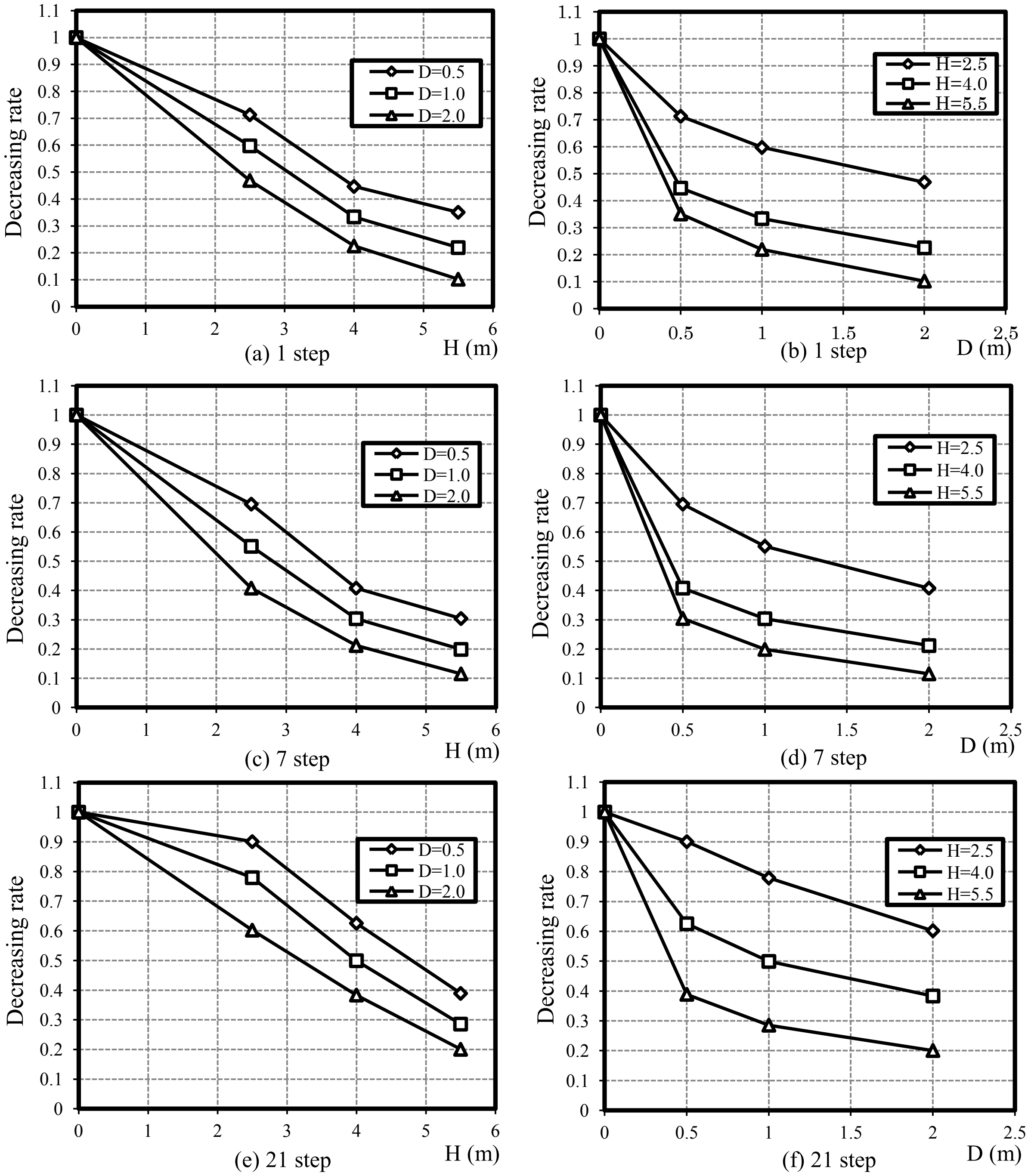


Figure 4.5. Decreasing rate of pile head shear force

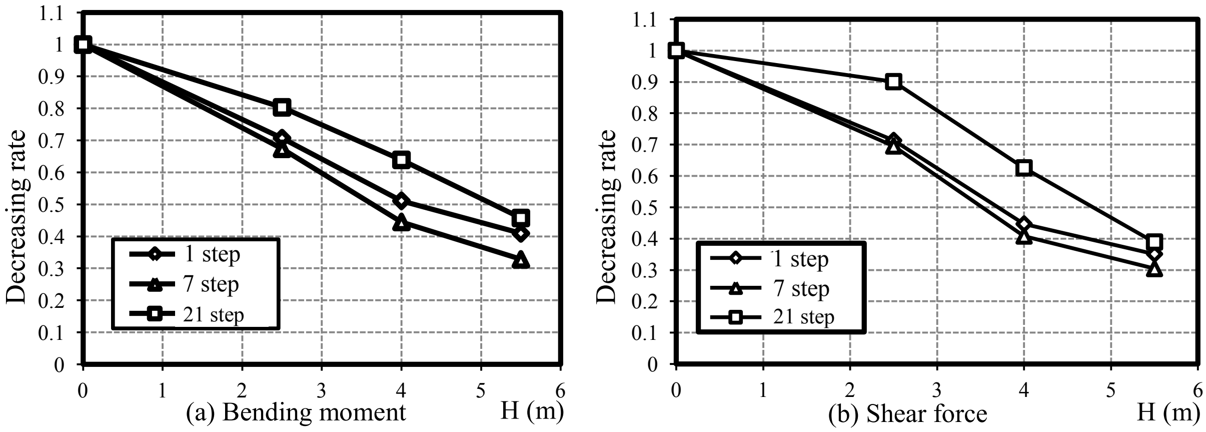
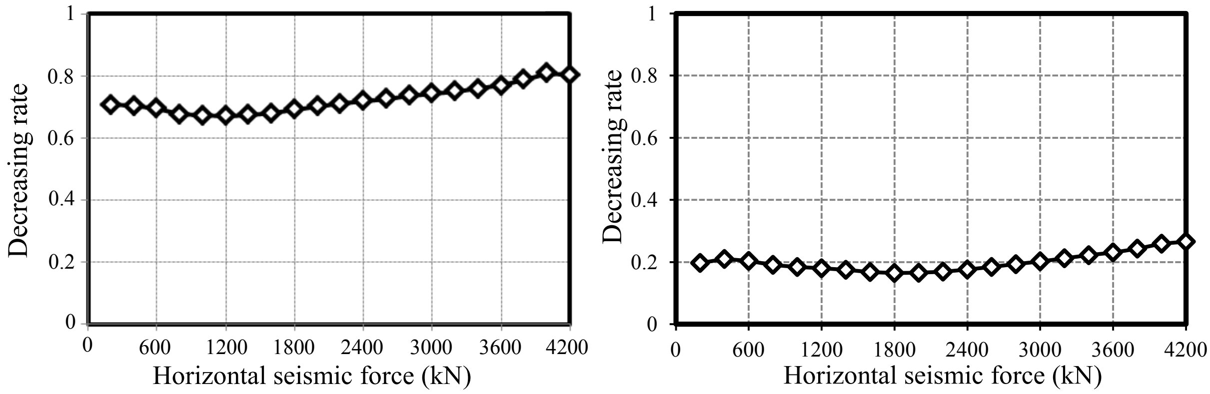


Figure 4.6. Decreasing rate (D=0.5m)



(a) D=0.5m, H=2.5m (b) D=0.5m, H=5.5m

Figure 4.7. Decreasing rate of pile head bending moment

5. CONCLUSIONS

In this study, a new simple aseismic reinforcement method was introduced for existing pile foundation with improvement soil. Improvement soil was set around pile to reduce the bending moment and shear force at pile head. The improved effect of this method was proved by static 3-D FEM analysis. The findings through the case study are as follows:

(1) The bending moment and shear force of pile are reduced significantly when improvement soil was set round the pile.

(2) The decreasing rate change hardly when the thickness of improvement soil larger than a certain value (D=0.5m in these cases).

(3) The decreasing rate is almost constant when the horizontal seismic force is less or equal to design criteria load. Therefore, the results by the elastic analyses could be used when the horizontal seismic force is small.

The depth H and thickness D of improvement soil should be determined by different conditions and experimental study also need to make certain of improved effect in the future work.

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