SEISMIC DESIGN OF REINFORCING PILES FOR AN EXISTING BRIDGE PIER

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ABSTRACT :

Many existing infrastructures constructed prior to 1980 in Japan were designed for a particular seismic load which is smaller than the Level 2 ground motion caused by the maximum considered earthquake (MCE). Those existing structures must be retrofitted to comply with the seismic requirement for the level 2 ground motion to be newly designated after 1995 Hyogoken-Nanbu earthquake. For such pile-supported structures, a new seismic retrofitting approach is proposed by introducing additional reinforing piles. The effectiveness for this approach will be discussed based on the seismic performance-based design method.

KEYWORDS: Reinforcing pile, existing bridge pier, seismic design, retrofit

1. INTRODUCTION

Many existing infrastructures which are deteriorated by corrosive and climate conditions are always threatened by various natural hazards including earthquake loads. Actually, those structures constructed prior to 1980 in Japan were designed for a particular seismic load which is smaller than the Level 2 ground motion caused by the maximum considered earthquake (MCE). After the 1995 Hyogoken-Nanbu Earthquake, revised guidelines which introduce Level 1 and Level 2 ground motions as the design earthquake load were specified making old infrastructure design fall below acceptable limits. This means these structures are vulnerable to strong earthquake in the future.

A typical existing infrastructure is a bridge pier which is supported by steel piles. In this study, this pile-supported structure is adopted as the structural model. The basic concept of seismic design for such structure is based on two purposes; (1) both the structure and pile-supported foundation are in the elastic state corresponding to a Level 1 ground motion, while (2) the structure moves into the inelastic state for a Level 2 ground motion before a plastic hinge is formed at a critical point in a pile.

If the structure is retrofitted with excessive reinforcement for strong seismic effect like Level 2 ground motion, the piles supporting the structure will create plastic hinge or local buckling. On the other hand adding excessive piles to increase the original strength of the foundation will cause an unexpected failure at the weakest portion of the structure.

The above discussions suggest that the adequate selection of reinforcing piles is important when the structure is retrofitted with additional reinforcing piles for Level 2 ground motion. The optimal solution of the additional pile design can be carried out by carefully controlling the failure modes of the structure.

In this study, a simplified design method is proposed for seismic design of reinforcing piles supporting an existing structure. Based on the reliability analysis of an existing bridge structure after possible future earthquakes, the present study discusses the optimal combination of the structural strength and the additional pile reinforcement that are necessary to obtain the effective maintenance strategies of deteriorating structures under seismic risks.

Discussions are devoted on (1) the definition of seismic performance level and its probability of damage states, (2) seismic response of existing structure with pile foundation, especially in stressing the effect of structural characteristics coefficient, and (3) numerical studies based on various parameters.

2. CONCEPT OF REINFORCING PILES FOR SEISMIC RETROFITTING

2.1. Definition of seismic performance

The following three different seismic performance levels are introduced.

Seismic performance 1 is the state that the structure and its supporting foundation can maintain its serviceability function without or with a minor damage after the level 1 ground motion caused by Maximum Operational Earthquake, MOE. The corresponding damage mode is the minor damage which can be defined as

$$D_i^S \equiv \left[R_i^S < S_1^S \right] \tag{2.1}$$

So, the probability of occurrence of minor damage mode, $p_{fi}{}^S$, is given by

$$p_{fi} = P\left[D_i^S \right] = p_{fi}^S \tag{2.2}$$

Seismic performance 2 is the state that the structure and its supporting foundation may obtain subsidiary damages at the critical portions of the system but cannot escape from the collapse with its small occurrence probability after the level 2 ground motion caused by Maximum Considered Earthquake, MCE. The corresponding damage mode is the moderate damage which can be defined as

$$D_o^S \equiv \left[\varepsilon_o^S < \varepsilon_2^S \le \varepsilon_a^S \right] \tag{2.3}$$

So, the probability of occurrence of moderate damage mode, $p_{fo}{}^S$, is given by

$$p_{fo} = P\left[D_o^S \middle| \overline{D}_a^B \right] \cdot P\left[\overline{D}_a^B \right] = p_{fo}^S \cdot \left(1 - p_{fa}^B \right) \tag{2.4}$$

Seismic performance 3 is the state that the structure may obtain a principal damage at the critical portion of the structure with a subsidiary damage at the foundation but cannot escape from the collapse with its occurrence probability after the level 2 ground motion caused by Maximum Considered Earthquake, MCE. The corresponding damage mode is the major damage which can be defined as

$$D_a^S \equiv \left[\varepsilon_a^S < \varepsilon_2^S \right] \tag{2.5}$$

So, the probability of occurrence of moderate damage mode, $p_{fa}{}^S$, is given by

$$p_{fa} = P\left[D_a^S \middle| \overline{D}_a^B \right] \cdot P\left[\overline{D}_a^B \right] + P\left[D_a^B \right] = p_{fa}^S \cdot \left(1 - p_{fa}^B \right) + p_{fa}^B \tag{2.6}$$

The foundation which is supported with a group pile of n members must be kept in the serviceable state with a small probability of collapse failure after the level 2 ground motion caused by Maximum Considered Earthquake, MCE. The corresponding damage mode is the major damage which can be defined as

$$D_a^B \equiv \left[\bigcap_{j=1}^{n} \left\{ u_a^B(x_j) \le u_2^B(x_j) \right\} \right] \bigcup \left[\theta_a^B < \theta_2^B \right] \bigcup \left[\bigcap_{j=1}^{n} \left\{ R_a^B(x_j) < S_{V2}^B(x_j) \right\} \right] \tag{2.7}$$

2.2. Probability of damage states

2.2.1 Seismic assessment of reinforcing system

When the structural system is reinforced with additional piles, the major damage state of the pile foundation can be defined as

$$D_a^{B*} \equiv \left[\bigcap_{j=1}^{n+m} \left\{ u_a^B(x_j) \le u_2^B(x_j) \right\} \right] \bigcup \left[\theta_a^B < \theta_2^B \right] \bigcup \left[\bigcap_{j=1}^{n+m} \left\{ R_a^B(x_j) < S_{V2}^B(x_j) \right\} \right] \tag{2.8}$$

Using the damage state of $D_a{}^{B*}$, the probability of damage mode after the reinforcement can be given as follows.

$$p_{fo}^* = P\left[D_o^S \middle| \overline{D}_a^{B*} \right] \cdot P\left[\overline{D}_a^{B*} \right] = p_{fo}^{S*} \cdot \left(1 - p_{fa}^{B*} \right) \tag{2.9a}$$

$$p_{fa}^* = P\left[D_a^S \middle| \overline{D}_a^{B*} \right] \cdot P\left[\overline{D}_a^{B*} \right] + P\left[D_a^{B*} \right] = p_{fa}^{S*} \cdot \left(1 - p_{fa}^{B*} \right) + p_{fa}^{B*} \tag{2.9b}$$

2.2.2 Probability of failure for structural components

A structural system shall be requested to maintain the daily operation immediately after an earthquake of MOE. Then the structure is assessed to be in a minor damage state, while the foundation is naturally assumed to be in safe. When an earthquake of MCE is applied to the structural system, on the other hand, the foundation failure plays a critical role. Once the foundation is collapsed, the whole structural system is also failed. So the moderate and major damage states must be conditioned on the damage state of the pile foundation. The moderate and major damages can be expressed in terms of the conditional probability as follows.

$$p^S_{fo} = P\left[D^S_o \middle| \overline{D}^B_a\right] = \frac{P\left[\overline{D}^B_a \middle| D^S_o\right]\cdot P\left[D^S_o\right]}{\sum_{k=1}^{3} P\left[\overline{D}^B_a \middle| D^S_k\right]\cdot P\left[D^S_k\right]} \quad \text{and} \quad p^S_{fa} = P\left[D^S_a \middle| \overline{D}^B_a\right] = \frac{P\left[\overline{D}^B_a \middle| D^S_a\right]\cdot P\left[D^S_a\right]}{\sum_{k=1}^{3} P\left[\overline{D}^B_a \middle| D^S_k\right]\cdot P\left[D^S_k\right]} \tag{2.10}$$

It should be noted that the seismic load to be reduced by taking the inelastic response effect into consideration can be only applied when the pile foundation is not in the major damage state.
Accordingly, the major damage of the pile foundation can also be expressed in terms of the conditional probability as follows.

$$p^B_{fa} = P\left[D^B_a \middle| D^S_o\right]P\left[D^S_o\right] + P\left[D^B_a \middle| D^S_a\right]P\left[D^S_a\right] + P\left[D^B_a \middle| \overline{D^S_o \bigcup D^S_a}\right]P\left[\overline{D^S_o \bigcup D^S_a}\right] \tag{2.11}$$

in which, for instance, the probability of major damage for the pile foundation under the condition of moderate damage mode of the structure is developed as follows.

$$\begin{aligned} P\left[D^B_a \middle| D^S_o\right] =\ & P\left[\bigcap_{j=1}^{n}\left\{u^B_a(x_j) \le u^B_2(x_j)\right\} \middle| D^S_o\right] + P\left[\theta^B_a < \theta^B_2 \middle| D^S_o\right] + P\left[\bigcap_{j=1}^{n}\left\{R^B_a(x_j) \le S^B_{V2}(x_j)\right\} \middle| D^S_o\right] \\ & - P\left[\bigcap_{j=1}^{n}\left\{u^B_a(x_j) \le u^B_2(x_j)\right\} \middle| D^S_o\right]\cdot P\left[\theta^B_a < \theta^B_2 \middle| D^S_o\right] - P\left[\bigcap_{j=1}^{n}\left\{R^B_a(x_j) \le S^B_{V2}(x_j)\right\} \middle| D^S_o\right]\cdot P\left[\theta^B_a < \theta^B_2 \middle| D^S_o\right] \\ & - P\left[\bigcap_{j=1}^{n}\left\{u^B_a(x_j) \le u^B_2(x_j)\right\} \middle| D^S_o\right]\cdot P\left[\bigcap_{j=1}^{n}\left\{R^B_a(x_j) \le S^B_{V2}(x_j)\right\} \middle| D^S_o\right] \end{aligned} \tag{2.12}$$

And the probability of moderate damage for the pile foundation can also derived in the same manner.

2.2.3 Discussion on the performance design of reinforcing piles
The effect of reinforcement due to additional piles for the structural-pile-foundation system can be assessed with the following formula;

$$\frac{p^*_{fa}}{p_{fa}} = \frac{p^{S*}_{fa}\left(1 - p^{B*}_{fa}\right) + p^{B*}_{fa}}{p^S_{fa}\left(1 - p^B_{fa}\right) + p^B_{fa}} \tag{2.13}$$

If the probability of major damage for the structure is larger than that of the pile foundation, that effect depends on the improvement of the structure itself.

$$p^{B*}_{fa},\, p^B_{fa} << 1 \ ,\ \frac{p^S_{fa}}{p^B_{fa}} >> 1 \ \text{ and } \ \frac{p^{S*}_{fa}}{p^{B*}_{fa}} >> 1 \Rightarrow \frac{p^*_{fa}}{p_{fa}} \cong \frac{p^{S*}_{fa}}{p^S_{fa}} \tag{2.14}$$

If the probability of major damage for the pile foundation, on the other hand, is larger than that of the structure, the additional piles can express the significant contribution to the whole system in terms of structural reinforcement.

$$p^{B*}_{fa},\, p^B_{fa} << 1 \ ,\ \frac{p^S_{fa}}{p^B_{fa}} << 1 \ \text{ and } \ \frac{p^{S*}_{fa}}{p^{B*}_{fa}} << 1 \Rightarrow \frac{p^*_{fa}}{p_{fa}} \cong \frac{p^{B*}_{fa}}{p^B_{fa}} \tag{2.15}$$

3. SEISMIC RESPONSE OF EXISTING STRUCTURE WITH PILE FOUNDATION

3.1. Modeling

Fig.1 is a schematic illustration of the structure-pile-foundation system. The group piles are allocated in the 4 x 3 system in Fig.2 (1), while the ground response Uh applies to the pile and the surrounding soil can resist for the forced displacement based on the ground shaking in terms of horizontal load q as shown in Fig.2 (2) which is approximately estimated with the maximum soil pressure shown in Fig.3.
The seismic response behavior of a single pile can be estimated based on the response displacement method.
Seismic response forces the pile to be deformed as same as the displacement of soil ground, while the surrounding soil can resist for this forced deformation. This resisting force can be modeled as $q=D\sigma_s$ as shown in Fig.2(2) where σ_s is the maximum soil pressure in Fig.3. A simplified assumption is introduced to estimate the pile displacement in the next section 3.2.

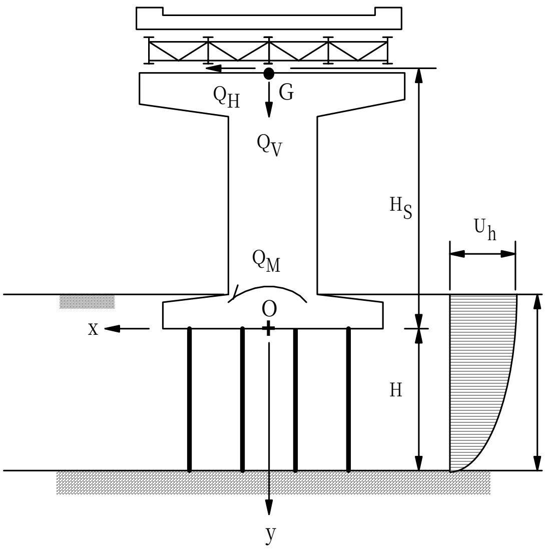
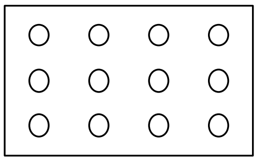
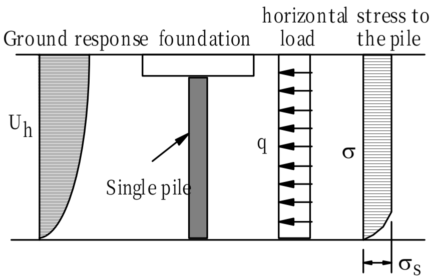


Fig.1 Schematic profile of a bridge pier supported by pile foundation system



(1) Plan view of the foundation and pile allocation



(2) Side view of the pile foundation system forced by ground response

Fig.2 profile of pile foundation system

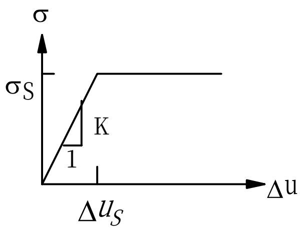


Fig.3 Inelastic relationship of soil and pile interaction

3.2. Seismic behavior of a single pile

Once the ground displacement is given as

$$U_h(y) = \frac{2}{\pi^2} \cdot S_V(T) \cdot T \cos\left(\frac{\pi \cdot y}{2H'}\right) \tag{3.1}$$

, the displacement profile of a single pile in Fig.2 (2) is governed by the equation

$$EI \frac{d^2 u}{dy^2} = -M \tag{3.2}$$

Noting that the boundary conditions at the top of the pile is given as $M=M_p$ and $u'=0$ at $y=0$ with a fully plastic moment of a pile derived from

$$M_p = \frac{4}{\pi} M_y = 4\pi\sigma_y t\left(\frac{D-t}{2}\right)^2 \tag{3.3}$$

, then the critical displacement of the pile having the fully plastic moment at the pile top end is given as

$$u_o^B = \frac{H^2}{3EI}\left(M_p - \frac{qH^2}{8}\right) \tag{3.4}$$

3.3. Seismic behavior of a structure-pile-foundation system

The inelastic response of the structural system due to the level 2 ground motion caused by MCE generate the reduced seismic loads with the structural characteristic coefficient Ds as follows;

$$Q_H(\eta) = D_S(\eta) \cdot S_A(T_S), \qquad Q_V(\eta) = \frac{1}{2} Q_H(\eta), \qquad Q_M(\eta) = Q_H(\eta) \cdot h \tag{3.5}$$

in which

$$D_S = \frac{1}{\sqrt{2\eta - 1}} \quad \text{for the ductility of} \quad \eta = \frac{\varepsilon_2^S}{\varepsilon_o^S} \tag{3.6}$$

By using the probability density function of the ductility factor, the probability of major damage for the structure and its pile foundation is given as

$$p_{fa}^B = \int_0^{\eta_{\max}} \left\{ \begin{aligned} & P\left[D_a^B(\eta) \middle| D_o^S(\eta)\right] P\left[D_o^S(\eta)\right] \\ & + P\left[D_a^B(\eta) \middle| D_a^S(\eta)\right] P\left[D_a^S(\eta)\right] \\ & + P\left[D_a^B(\eta) \middle| \overline{D_o^S \cup D_a^S}\right] P\left[\overline{D_o^S \cup D_a^S}\right] \end{aligned} \right\} f_\eta(\eta) d\eta = \int_0^{\eta_{\max}} \left\{ \begin{aligned} & P\left[D_a^B(\eta) \middle| 1 < \eta \le \eta_a\right] P\left[1 < \eta \le \eta_a\right] \\ & + P\left[D_a^B(\eta) \middle| \eta_a < \eta\right] P\left[\eta_a < \eta\right] \\ & + P\left[D_a^B(\eta) \middle| \eta < 1\right] P\left[\eta < 1\right] \end{aligned} \right\} f_\eta(\eta) d\eta$$

$$p_{fa}^S = \int_0^{\eta_{\max}} P\left[D_a^S(\eta) \middle| \overline{D}_a^B(\eta)\right] f_\eta(\eta) d\eta \qquad p_{fo}^S = \int_0^{\eta_{\max}} P\left[D_o^S(\eta) \middle| \overline{D}_a^B(\eta)\right] f_\eta(\eta) d\eta \tag{3.7}$$

Table 1 Dimension of the pile

Item	Symbol	Unit	Value
Diameter	D	m	0.7
thickness	t	m	0.012
Yield strength	σ_y	kN/m^2	4.90E+05
Elastic modulus	E	kN/m^2	2.06E+08
Pile length	H	m	21.3
Allowable pile capacity	Ra^B	kN	2919
Allowable rotation	θ_a^B	radian	0.02

Table 2 Dimension of the structure

Item	Symbol	Unit	Value
Mean structural response rate	E[S]/S_y	-	5
Cov of structural response	δx	-	0.2
Height of the gravity center	Hs	m	5.229
Typical period of the structure	Ts	sec	1
Mean yield strain	E[ε_o^S]	-	0.005
Mean collapse strain	E[ε_a^S]	-	0.05
Cov of yield strain	$\delta\varepsilon_o S$	-	0.1
Cov of collapse strain	$\delta\varepsilon_a S$	-	0.15

4. NUMERICAL STUDY

The pile dimensions and its material properties are summarized in Table 1. This table shows the dimensions of an original pile, while the same size of a pile is adopted as a reinforcing pile in this study. The allowable pile capacity is used to assess the buckling capability for vertical force of the pile, while the allowable rotation is examined for the rotation of the foundation by rotational moment.

The numerical parameters on the structure are summarized in Table 2. The seismic applied force due to MCE is assumed to be 5 times of the yielding forcing S_y, while the mean values of structural strains, ε_o^S and ε_a^S, are assumed based on that of the steel reinforcing bar.

The coefficient of variations for these values are simply assumed to be 10% to 20%.

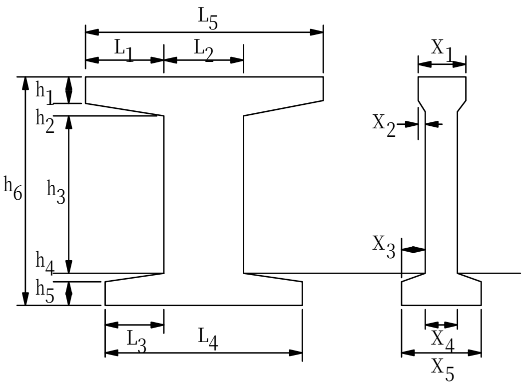
Table 3 Dimension of the soil and ground

Item	Symbol	Unit	Value
Soil strength by SPT	SPT	-	20
Horizontal soil stiffness	k_{Ho}	kN/m^3	373333
Soil stress to the pile	σ_S	kN/m^2	31.4
Typical period of the ground	T	sec	1

4.1. Structural models of existing system

Fig.4 is a structural model of the existing system in which 4 row x 3 line of pile allocation is given as shown in Fig.5 and 6. The detail sizes of the structural profile are listed up in Table 4.

Since the structure has two deformation modes along the axial and transverse directions, discussions will be done for each mode.



(1) View of the axial direction (2) View of the transverse direction

Fig.4 Profiles of the structure and pile foundation system

Table 4 Dimension of the structure and pile foundation system

h_1	h_2	h_3	h_4	h_5	h_6
1	0.96	8.04	0.6	1.4	12
L_1	L_2	L_3	L_4	L_5	
3.4	3.2	1.9	7	1	
X_1	X_2	X_3	X_4	X_5	
2.5	0.25	1.625	2	5.25	

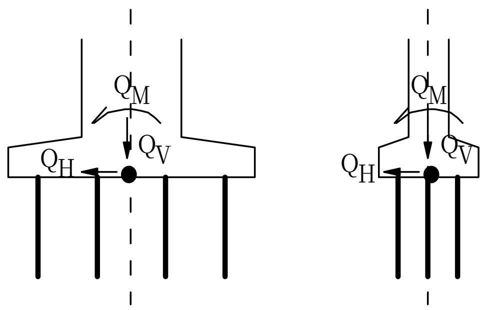


Fig.5 Seismic design loads for the un-reinforced
Pile foundation system

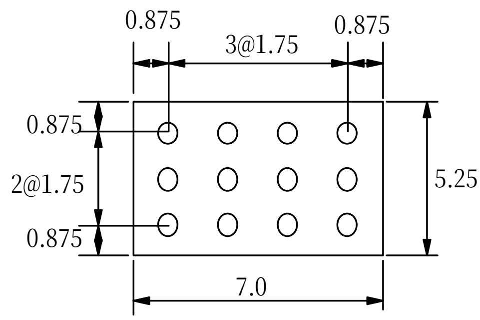


Fig.6 Dimensions of the pile allocations

4.2. Structural models of reinforcing system

The additional reinforcing piles are applied along the outskirt of the original pile allocation as shown in Fig.7. The foundation is also enlarged to make a space for additional pile driving. The detail allocation of the additional reinforcing piles is shown in Figs.8 and 9. The additional piles are assumed to have 3 lines as same as that of the original piles.

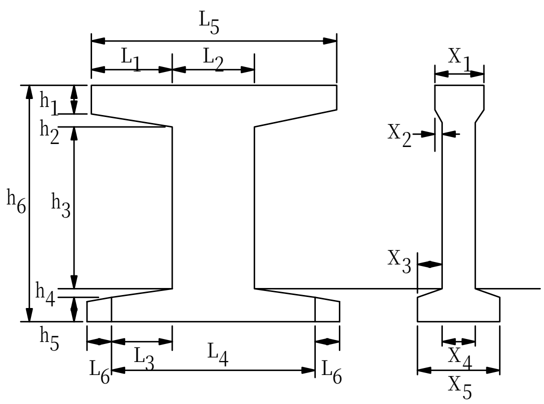


Fig.7 Profiles of the structure and pile foundation system with additional reinforcement

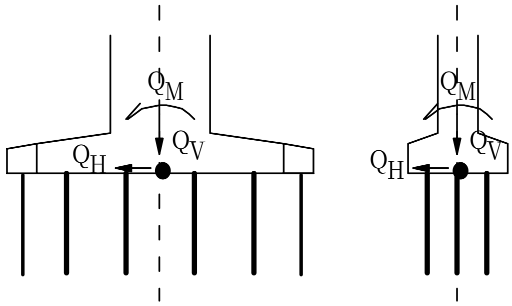


Fig.8 Seismic design loads for the reinforced pile foundation system

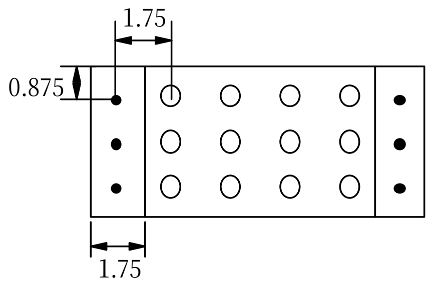


Fig.9 Dimension of pile allocations

4.3. Numerical results

Fig.10 shows the relationship between the yield strength of pile material and the pile displacement to generate fully plastic bending moment by forcing ground response due to Level 2 ground motion. This figure suggests that the pile of smaller yield strength than 350 MN/m^2 is easy to fall into the fully plastic state by the seismic ground response, while the pile of the larger yield strength is difficult to make a fully plastic hinge at the top of the pile by the seismic response.

Figs. 11 and 12 show the effect of reinforcement by comparing the probability of failure before and after the additional reinforcing pile driving. Fig.11 is for the result for the axial direction, while Fig.12 is that for the transverse direction. Fig.11 suggests that the whole system is improved by pile reinforcing. The probability of failure for the pile foundation is decreased, while that for the structure is increased. This trend for the axial direction is applicable to that for the transverse direction, although the probability of failure for the pile foundation is always predominant.

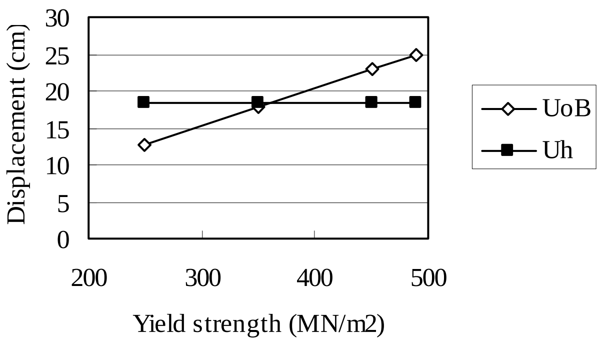
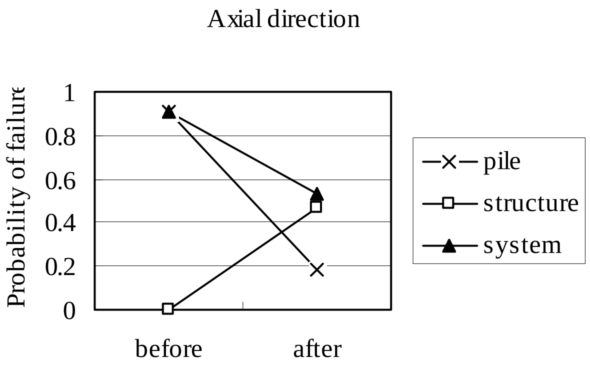


Fig.10 Relationship between the yield strength of pile material and the pile displacement to generate fully plastic bending moment by forcing ground response due to Level 2 ground motion



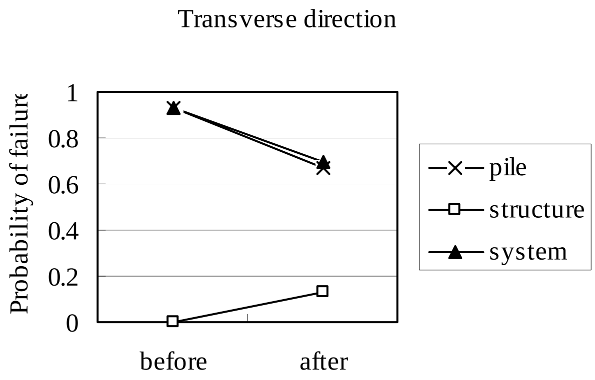
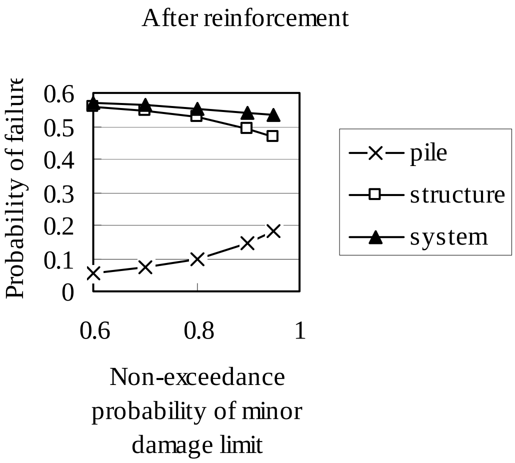


Fig.11 Probability of major damage of a structure with pile foundation system in the axial direction.

Fig.12 Probability of major damage of a structure with pile foundation system in the transverse direction.



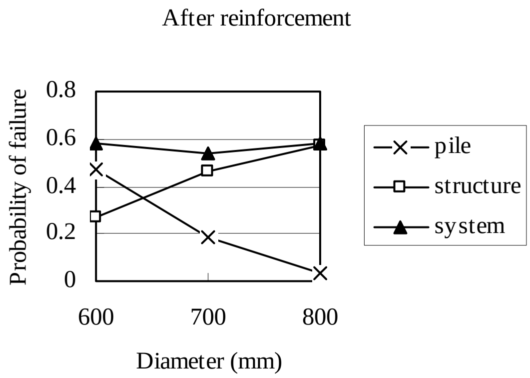


Fig.13 Probability of major damage of the structure with reinforced pile foundation system for the non-exceeded probability of the structure

Fig.14 Probability of major damage of the structure with reinforced pile foundation system for various diameters of piles for the minor damage limit state

Fig.13 shows a probability of major damage of the structure with reinforced pile foundation system for the non-exceeded probability of the structure for the minor damage limit state. This figure means that the probabilities for the major damage depend on the minor damage limit state of the structure, because the yield strength of the structure which can reflect the inelastic response effect through the structural characteristics coefficient, D_s, is designed to comply with the non-exceeded probability of the structure for the minor damage limit state. Larger p_{fi}^S can decrease p_{fa}^S but increase p_{fa}^B.
Fig.14 expresses the effect of pile diameter, in which the minimum value of probability of failure is given in the case of 700 mm diameter. At least, the pile with more than 650 mm diameter can provide smaller probability of failure for the pile foundation than that for the structure.

5. CONCLUSION

This study develops the method to obtain the probability of failure for the reinforcing pile foundation which will be utilized to derive the target probability for the performance-based design of the pile-foundation-structural system.
(1) Reinforcing pile can decrease the probability of failure of the pile-foundation-system.
(2) Larger yield strength of the pile is difficult to make a fully plastic hinge at the top of the pile under level 2 seismic loads.
(3) The optimal diameter of the pile is obtained to decrease the probability of failure of the system by adding the reinforcing piles.

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