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SEISMIC EVALUATION AND RETOFITTING FOR EXISTING PILE-SUPPORTED REINFORCED CONCRETE FRAME PIER

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SUMMARY

This paper discussed about seismic retrofit design method for existing pile-supported reinforced concrete frame pier designed in the pre-1980 design code. The effect of the interaction between pile and soil was examined thought non-linear dynamic response analysis. From the comparison between static and dynamic analysis, it is found that the piles demanded the lateral resistance and residual displacement against earthquake loading by considering inelastic effect, as have been evidenced in the highway bridges in recent strong earthquake. For some situations as retrofitting of existing bridge, an appropriate inclusion by considering soil-structure interaction effects may bring large design cost saving.

1. INTRODUCTION

The bridge designed in accordance with former seismic design code of Japan suffered some serious damages during the Hyogo-ken-Nanbu Earthquake in 19951). Thereafter the shortages in this seismic design code were realized. And the seismic design code for the highway bridge was revised in 1996. Methods of the precise assessment of seismic performance of the existing reinforced concrete frame piers are essentially the same with the methods of design of new bridges in current Japanese seismic design code.

For an existing reinforced concrete pier It is likely that the design lateral force and strength requirements of major structural components in the original design are insufficient based on the current codes. However it is important in the seismic vulnerable assessment to use actual strength of structures and soils rather than the nominal values specified in codes.

At the same time, because of the difficulties for identifying damage degree of piles and for repairing them after such events, the general design philosophy is to ensure that piles response

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should remain in the elastic range to voiding damages. However, from investigation it was found hard for piles to be escaped in the Hyogoken Nanbu earthquake. Retrofitting the piles would be both time and cost prohibitive.

In order to discuss the correct and most economical approach of retrofitting, this paper deals with the non-linear behavior of existing pile-supported reinforced concrete frame pier during earthquake motion. The elevated system was designed in the pre-1980 Design Code with no significant seismic performance requirements. From the comparisons between non-linear static and dynamic analysis, it is found that the piles demanded the lateral resistance and residual displacement against earthquake loading by considering inelastic effect and the interaction effect between the soil and structure, as have been evidenced in the highway bridges in recent strong earthquake. It is possible that the seismic retrofitting program can be completed by retrofitting the columns at some cases.

The assessment is performed by means of a non linear analysis. After a brief review of the models used, the response in terms of load-displacement curves is presented.

2. INVESTIGATED PIER

The prototype studied in this paper is a typical pile-supported reinforced concrete frame pier of highway bridge built following the old seismic design code pre-1980's in Japan. the investigated pier is about 14 meters height and 23 meters in the width as shown in Figure 1 and Figure 2. The cross section of the column of the frame pier is shown in Figure 3, in which the longitudinal reinforcements are of diameter 38 mm and 35 mm.

And the hoops are of diameter 19 mm.

From the provisions of current Japanese code, the pier should be retrofitted following the ductility Design Method based on the equal energy assumption.

$$P_a \geq k_{he} \cdot W \tag{1}$$

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu_a - 1}} \tag{2}$$

$$W = W_u + c_p W_p \tag{3}$$

Eq. (1),(2),(3) should be satisfied, in which Pa is lateral force capacity of the pier, khe is equivalent lateral force coefficient, W is equivalent weight, Wu is weight of super structure supported by the pier, Wp is weight of the pier, khc is lateral force coefficient, μ_a is allowable displacement ductility factor and cp is a coefficient depending on the failure mode.

3. SEISMIC ASSESSMENT FOLLOWING CURRENT JAPANESE CODE

The seismic assessment of the existing reinforced concrete frame pier by the ductility design method following current Japanese code indicated that the pier is not safe both in longitudinal and transverse direction, and would be failed in shear failure mode under earthquake with the intensity comparable to the Hyogo-ken-Nanbu Earthquake in 1995. The transverse direction of the pier is considered herein. The verified result by the static method specified in the seismic design code of 1996 is shown on Table 1. The Equivalent lateral force kheW calculated for the prototype pier is greater than Pa as shown in Table 1. The

pier should be retrofitted for the earthquake level TYPE II. The residual displacement calculated based on the equal energy assumption is less than 1% of the pier height.

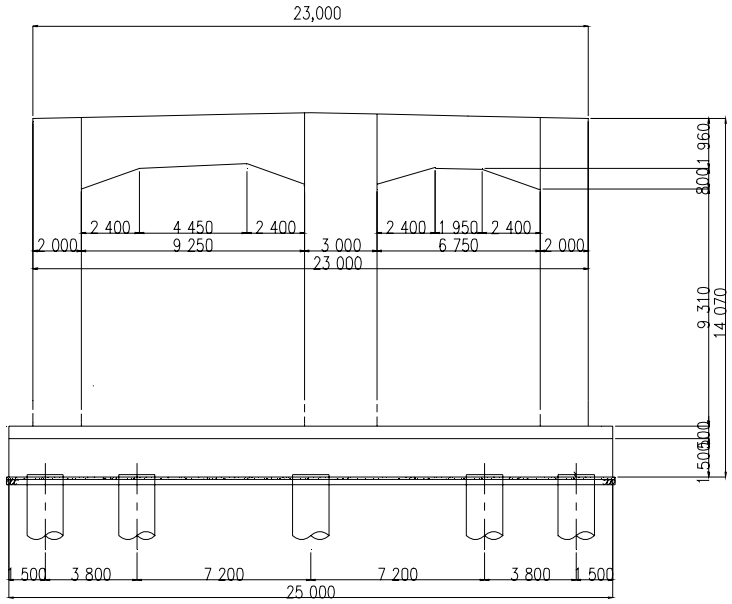

Figure 1 Reinforce Concrete Pier

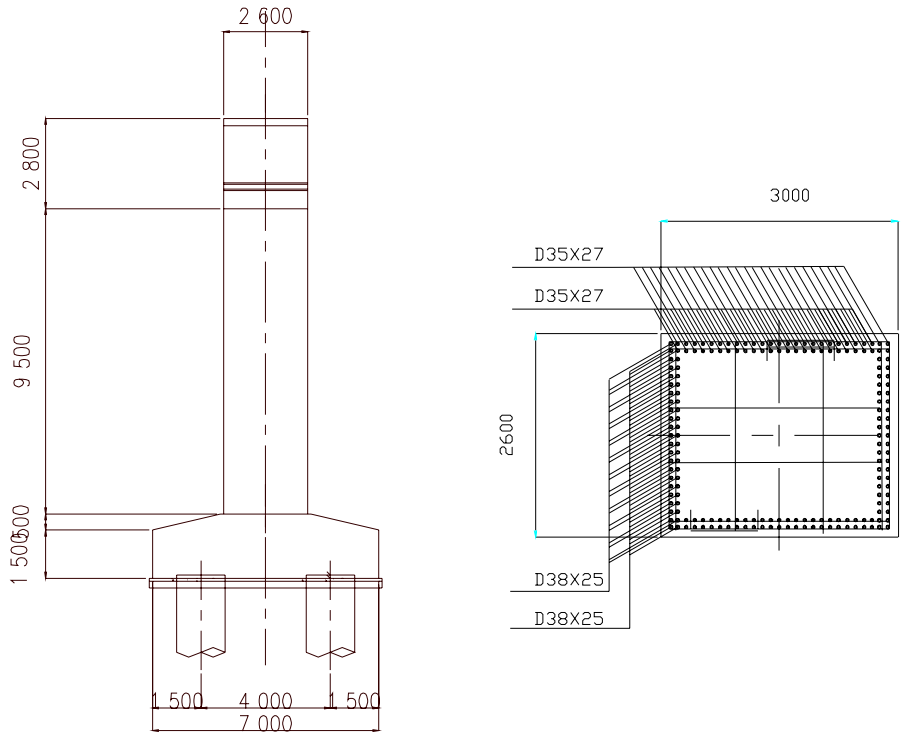

Figure 2 Reinforce details and Pier units

Table 1 Capacity of the pier

Seismic Level	Type I	Type II
Failed mode	Shear	Shear
Yield force (kN)	32926	33073
Ultimate force(kN)	41886	42056
Shear capacity(kN)	13277	13277
Inertia force(kN)	26322	28954

4. RETROFIT BASED ON THE JAPANESE SEISMIC DESIGN CODE

Steel jacket is often used for the retrofit of concrete columns that requires the enhancement of flexural and the shear strengths and ductility capacity. Since retrofit of the column generally results in the increase of moment and shear demand in the foundation, by using anchor bolts the flexural, shear strengths as well as ductility of the column can be controlled within a level. For the retrofitting, 6 mm steel plate was used. The capacity of the retrofitted pier is shown in Table 3.

Push-over analysis was included for design of foundation under safety-evaluation ground motion in the 1996 design code. In the push-over analysis, the foundation is modeled as a structure supported by the ground. For the pile foundation, the piles and footing are idealized to be supported by nonlinear soil spring. The nonlinear deformation of soil around the piles is taken into account. Evaluation of the spring stiffness and strength are presented in the seismic design code in 1996. and the design lateral force for the foundation PF is evaluated form the lateral capacity of the pier Pa by the Eq.(1) as

$$P_F = k_F P_a \tag{4}$$

in which is over strength factor (=1.1).

For the retrofitted pier, the foundation was also verified. The shear capacity was insufficient as shown in Table 2. But the shear capacity is sufficient for the seismic level 1 and seismic intensity as type I. For retrofitting the foundation, it is necessary to increase 8 new piles. However, space for constructing new piles was limited in almost cases and the huge cost in improvement of soils around the foundation. At the same time for an existing pier which was construct 30 years ago, it is necessary to consider the cost effective and verify the actual strength of the pile rather than the nominal values specified in codes. Herein the non-linear dynamic and Pushover analysis methods were used to evaluate the really response during a seismic motion with the intensity level as the Hyogo-ken-Nanbu Earthquake in 1995.

Table 2 Capacity of the pile foundation

Seismic Level	Type I	Type II
Shear capacity (kN)	27800	27800
Shear force (kN)	11865	35931

5. NON-LINEAR ANALYSIS CONSIDERING THE SOIL-STRUCTURE INTERACTION

The dynamic analysis model is shown in Figure 4. the pier and the pile are divided into beam elements, The weight of superstructure supported by the pier is concentrated on the top of the pier.

The interaction of soil and pile was modeled by using the horizontal, vertical springs2)3). The damping factor is taken as 2% for the pier,5% for the pile and 20% for the foundation spring. Takeda moment-curvature relationship is adopted as the material property as known as a tri-linear model. The seismic

motion used for the analysis is the modified recording motion of the Hyogo-ken-Nanbu Earthquake in 1995. The analysis model was also used for the pushover analysis for the comparison of the capacity and responses. The axial load variation is considered.

The capacity of the foundation is 27800 kN as shown before, it is clear that the pile foundation have sufficient shear capacity during the earthquake with the intensity level as the Hyogo-Ken-Nanbu Earthquake in 1995. Nonlinear Moment-Curvature model as shown in Figure 5 is used for the pier and pile beam elements. ϕ_{y} and ϕ_{u} are calculated using conventional procedures that assume axial strain varies linearly across the section. The value of ϕ_{y} for a given section is defined as the curvature when steel first reaches the yielding strain. The ultimate curvature ϕ_{u} is defined as the curvature when the strain of the extreme fiber of the concrete is compression reaches the maximum value ε_{cu}. The stress vs. strain relation of confined concrete is given as

$$\sigma_c = \begin{cases} E_c\varepsilon_c\left\{1-\dfrac{1}{n}\left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right\} & \left(0 \le \varepsilon_c \le \varepsilon_{cc}\right) \\ \sigma_{cc} - E_{des}\left(\varepsilon_c - \varepsilon_{cc}\right) & \left(\varepsilon_{cc} \le \varepsilon_c \le \varepsilon_{cu}\right) \end{cases} \tag{5}$$

in which σ_{cc} =strength of confined concrete, Ec=elastic modules of concrete, Edes=gradient at descending branch and $n = E_c\varepsilon_{cc} / \left(E_c\varepsilon_{cc} - \sigma_c\right)$. The ultimate strain ε_{cu} depends on the type of ground motion.

From the result of analysis, it is obtained that the residual displacement is less than the allowable residual displacement. Maximum response ductility of nonlinear dynamic analysis is about 1.2 and is also less than the allowable value. Maximum shear resistance of the pile is less than the shear capacity of the pile foundation as shown in Table 4. As shown as Figure 4, the shear capacity of the foundation is less than the capacity of the pier, but the maximum shear force is less than the capacity of the pile, the maximum shear resistance for the earthquake with the intensity level as TYPE II is about 96% of the shear capacity of the pile foundation. It is considerable that retrofitting of the foundation is unnecessary. Of cause, seismic motions used for the analysis could not respect the really site effect of the construction area, but considering the large cost for retrofitting the foundation, for a aged existing pier, the result of the dynamic non-linear analysis is a effective and more reasonable result.

For the pile foundation the total horizontal displacement is 1.30 cm, it is in the range of allowable displacement (40 cm). The yielding consequence developed from column to pile foundation.

For the existing pier, after retrofitting the pier, the pile foundation still can resist the earthquake load with the intensity level as the Hyogo-ken-Nanbu earthquake in 1995.

Table 3 Capacity of the retrofitted pier

Seismic Level	Type I	Type II
Failed mode	Flexure	Flexure
Yield force (kN)	32365	32510
Ultimate force(kN)	38821	38979
Shear capacity(kN)	38821	38979
Inertia force(kN)	9570	8144

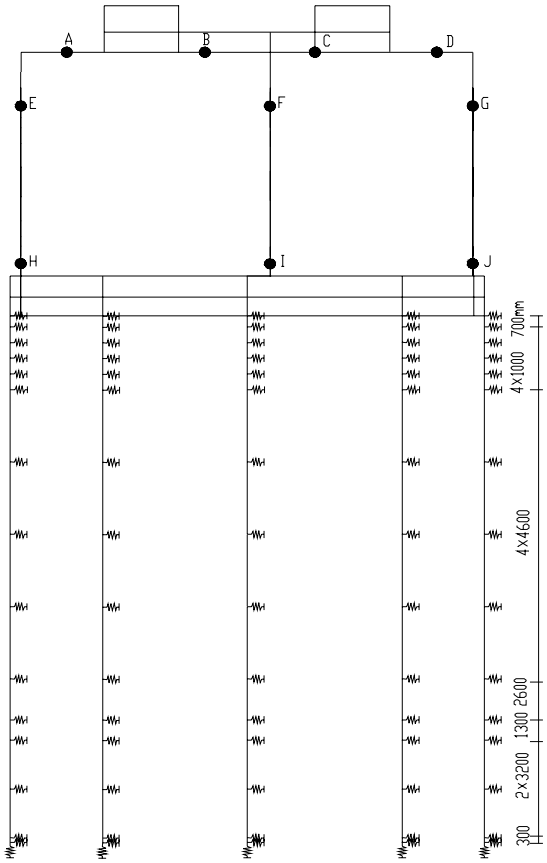

Figure 3 Dynamic analysis model

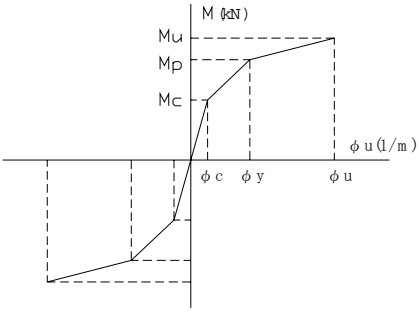

Figure 5 Takeda moment-curvature model

Table 4 Maximum Shear resistance of Pile

Seismic Level	TYPEII
Maximum Shear force of Wave-1 (kN)	26640
Maximum Shear force of Wave-2 (kN)	26200
Maximum Shear force of Wave-3 (kN)	24100
Average Shear force (kN)	25640

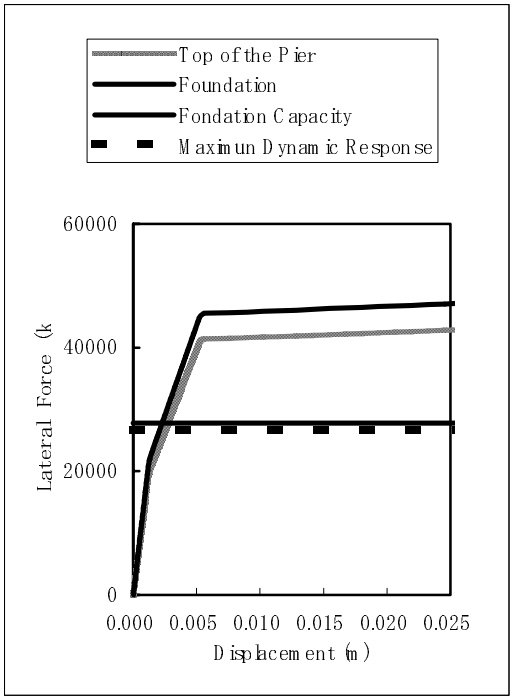

Figure 4 Comparisons of the analysis Result

6. CONCLUSIONS

For Assessing the seismic safety of the existing reinforced concrete frame pier, nonlinear dynamic analysis method should be used by considering the interaction effect between soil and structure.

From the comparisons between non-linear static and dynamic analysis, it is found that the piles demanded the lateral resistance and residual displacement against earthquake loading by considering inelastic effect, as have been evidenced in the highway bridges in recent strong earthquake. For some situations as retrofitting of bridge, an appropriate inclusion by considering soil-structure interaction effects in seismic calculations may bring large design cost saving, by mere recognition and taking effective measures, safety and better performance can be achieved.

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