- Technical Paper -

INFLUENCE OF STRENGTHENING OF BRIDGE PIERS ON SEISMIC BEHAVIOR OF FOUNDATION

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ABSTRACT

This study investigates the possibility of foundation failure in bridges after strengthening of piers. Earthquake responses of bridges before and after strengthening were compared by conducting pseudo-dynamic test including foundation. Two cases of foundations have been selected in order to represent hard soil and soft soil conditions. The failure in foundation was observed in the case of strengthened pier. Also, a remedial measure by soil improvement was investigated by a simple modification of soil parameter.

Keywords: strengthening, pseudo-dynamic test, soil-structure interaction, earthquake

1. INTRODUCTION

After the occurrences of new strong earthquakes, greater peak ground acceleration could always be recorded. As a result, many RC bridge piers have been strengthened to achieve a larger capacity in both loading capacity and ductility. However, as the strengthening of a bridge was always conducted in order to enhance only the seismic performance of the pier, the safety margin of the foundation should certainly reduce.

The target yielding load of pier strengthening is normally kept lower than the yielding load of foundation. As a result, the weakest link of the total system after the strengthening still seems to be the pier, as it possesses the lowest loading capacity. However, this is absolutely correct only if the interaction between pier and foundation is not considered.

In this study, the responses of bridge pier systems including foundation are evaluated by the mean of pseudo-dynamic (PSD) test. A three degree-of-freedom (3-DOF) model is utilized in the PSD test. The piers are selected as the experimental part in order to verify the effect of pier strengthening. In parallel, the responses of foundations for both cases of piers are controlled to follow the same mathematical model. Furthermore, two cases of foundations are selected in order to represent bridges sited on hard soil and soft soil condition.

2. PSD TEST OF BRIDGE SYSTEM

In this study, the responses of the bridge systems were evaluated by PSD test. The movement of a whole bridge including superstructure and foundation was simplified by 3-DOF system as shown in Fig. 1. The three degrees of freedom are lateral displacement at pier top (u_1), lateral displacement at footing level (u_2) and rotation of footing (θ). Further details of the PSD test system with this 3-DOF model can be referred to [1].

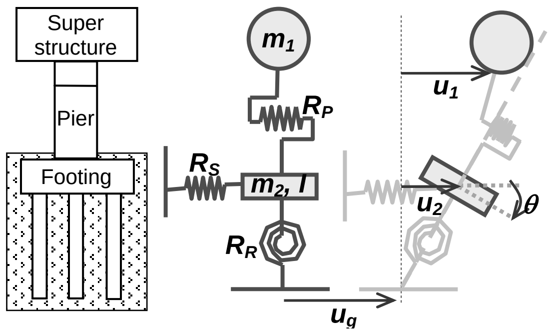

Fig. 1 The 3-DOF model used in PSD test

In this 3-DOF model, the restoring force of pier is modeled by a spring (R_P). While, the restoring force of foundation is represented by sway (R_S) and rocking (R_R) springs. During the numerical time integration, the restoring force of pier was obtained by reversed cyclic test on scaled down pier specimens. In parallel, the restoring forces of sway and rocking springs were calculated conforming to mathematical models. A schematic diagram of the PSD test is shown in Fig. 2.

Hardin-Drnevich model (HD-model) [2] was used as the mathematical model of sway spring, and bi-linear model was used in rocking spring. In order to assign parameters of the mathematical models for both springs, analyses on a 2D beams-springs model of foundation were conducted. The beams-springs models together with boundary conditions and loading configurations are illustrated in Fig. 3. In the analysis of the beams-springs model, moment-curvature

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relationship of beam elements, which represents each row of piles, were assigned to be tri-linear model. Cracking and yielding properties of the piles were also estimated using fiber model technique, in which the pile section was divided into 50 pieces. And also the piles are divided longitudinally into 50 pieces and 150 pieces for hard soil pile and soft soil pile respectively. And for soil springs, bi-linear relationship was applied with the yielding parameters calculated referring to [3].

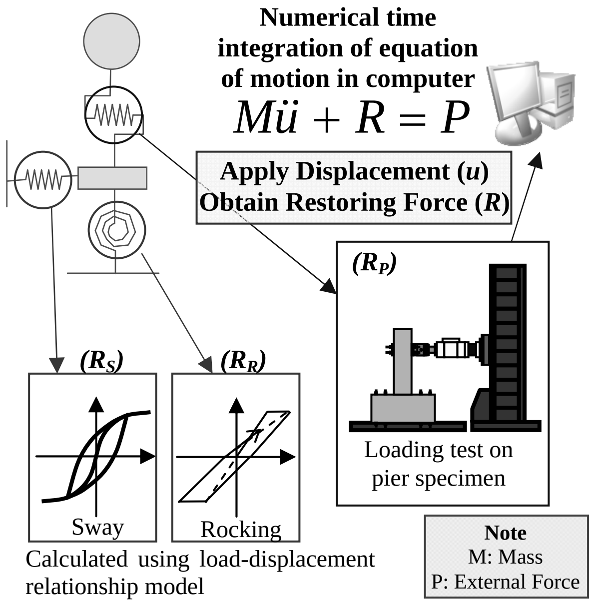

Fig. 2 Schematic diagram of PSD test

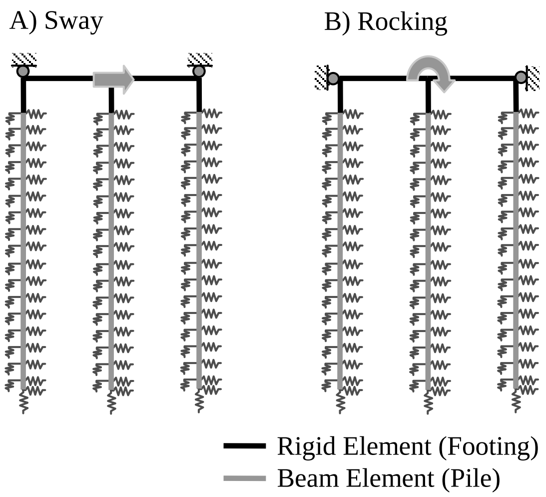

Fig. 3 Foundation model used in the calculation of load-displacement relationship of soil springs

3. OUTLINE OF THE TEST

3.1 BRIDGE PIER SYSTEMS

The study cases along with their configurations are presented in Table 1. The two cases of bridge piers, N and S, were selected in order to represent a pier designed according to around 0.02g peak ground acceleration and its strengthening respectively. The strengthening technique was additional layers of rebar with concrete cover aiming to achieve the yielding load capacity which is compatible to the present designing peak ground acceleration [4]. Furthermore, foundations with soft soil (S) and hard soil (H) conditions were also considered as another parameter in this study. The foundations were designed in associate with the piers of non-strengthened cases. Additionally, a special case for a strengthened pier sited on the soft soil foundation, in which the N value of the top strata was modified from N=2 to be N=10 in order to represent ground improvement (SI), were also tested. Flexural yielding load of piers and foundations for all cases are compared in Table 2, and the relevant drawings are given in Fig. 4 to Fig. 6. The flexural yielding loads of pier means the lateral load that yields the cantilever pier with fixed base. Contrary, the yielding load of foundation denotes the lateral load that yields the beam element representing a row of piles with the loading configuration as shown in Fig. 3A.

All the cases of bridge systems were subjected to the same ground motion of the recorded Kobe earthquake. The ground motion possesses 1,500 steps with an incremental time step of 0.02 second, and the peak ground acceleration of 821 gal.

Table 1 Experimental variables for bridge systems

Case	Pier Condition	Foundation Condition
N-H	Non-Strengthened	Hard Soil
N-S	Non-Strengthened	Soft Soil
S-H	Strengthened	Hard Soil
S-S	Strengthened	Soft Soil
S-SI	Strengthened	Soft Soil with modified N value to represent ground improvement

Table 2 Yielding load of piers and foundations

Case	Yielding Load (MN)		Ratio Found./Pier
	Pier	Foundation	
N-H	5.06	8.49	1.68
N-S	5.06	8.58	1.69
S-H	7.72	8.49	1.12
S-S	7.72	8.58	1.13
S-SI	7.72	15.91	2.06

3.2 Scaled Down Pier Specimens

The scaled down pier specimens, used in the evaluation of pier restoring force in PSD test, are shown in Fig. 7. The load-displacement relationship of the scaled down specimen was mapped onto the one of the actual scale pier by using two scaling factors, load scaling factor and displacement scaling factor. The load scaling factor is the ratio of the flexural yielding load of the scaled down specimen to that of the actual size pier. In the same sense, the displacement scaling factor is the ratio of the flexural yielding displacements of the scaled down specimen to that of the actual size pier.

The flexural yielding loads and displacements of both scaled down specimens and actual size piers were estimated according to fiber model technique as a cantilever beam with fixed base. Concrete section of both actual size pier and scaled down specimen were divided into 50 pieces. And also, 50 pieces for longitudinal direction.

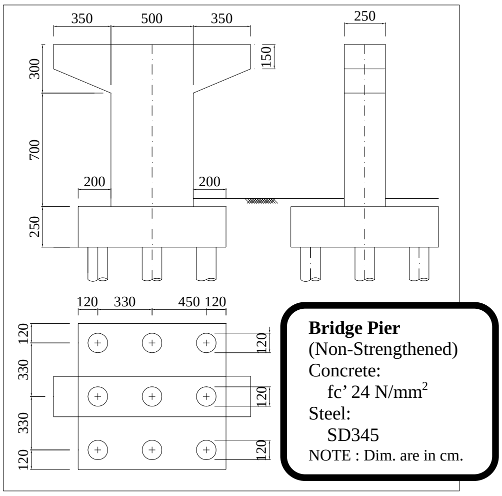

Fig. 4 Configurations of bridge pier and footing

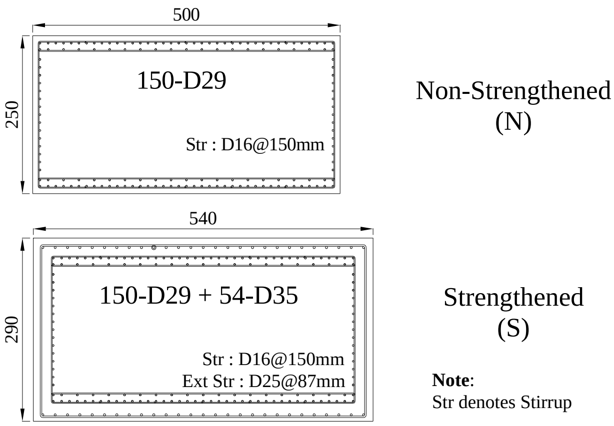

Fig. 5 Details of pier sections

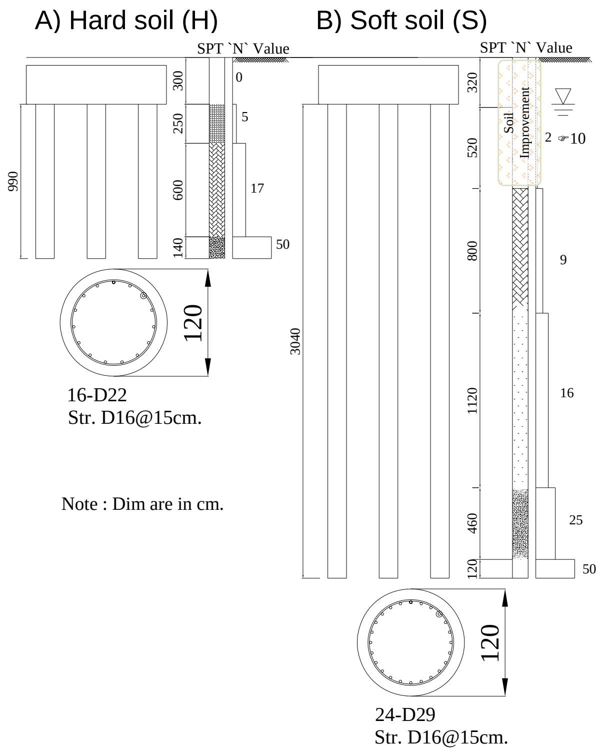

Fig. 6 Details of foundations

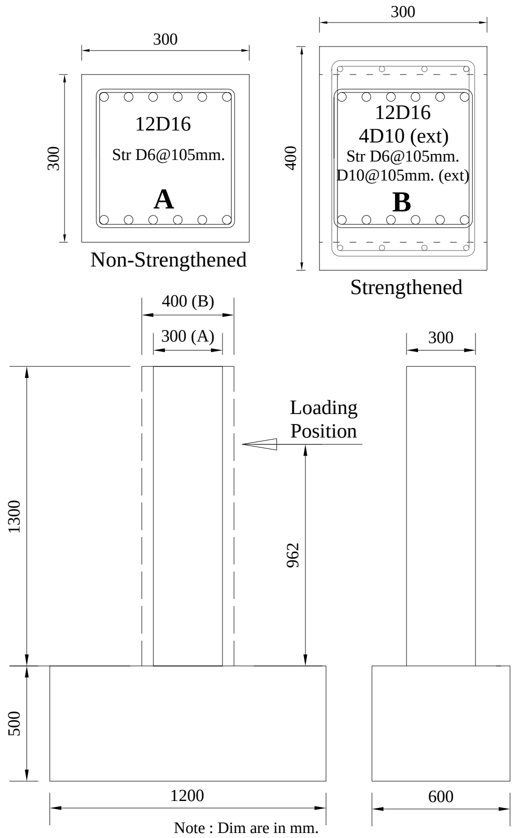

Fig. 7 Details of scaled down pier specimens

Table 3 Properties of scaled down pier specimens

Specimen	Yielding Load [kN]	Yielding Disp [mm]	Shear-Flex. Ratio	Scaling Factor Load	Scaling Factor Disp
A	112.80	9.24	1.25	0.0223	0.302
B	173.81	12.40	2.17	0.0225	0.450

Table 4 Numbers, test cases and material properties of scaled down specimens

Specimen	No.	Test Cases	Material Properties [MPa]	
			Concrete	Steel
A	2	N-H, N-S	36.06	D16 : 386.73 D10 : 377.91 D6 : 350.91
B	3	S-H, S-S, S-SI	35.66	

The constitutive models were the JSCE concrete model for concrete [5] and bi-linear model for reinforcement. On the other hand, the shear capacity of both actual size piers and scaled down specimens were calculated by the JSCE shear equation [5].

Specimen A, represents a non-strengthened pier, was designed to have a low load carrying capacity and also a low ductility capacity. The ratio of shear capacity to flexural yielding load was set equal to 1.25. In contrast, specimen B was fully designed to prevent shear failure, as to represent a strengthened pier. Also, an additional layer of reinforcing bars was added to the non-strengthened specimen in order to enhance the flexural strength. The ratio of the yielding load of strengthened specimen to the yielding load of non-strengthened specimen was set to be identical to that of the actual size piers. Therefore, almost the same load scaling factors were used for specimen A and B.

However, with the experimental facility limitation, the stiffness ratio in elastic range of non-strengthened pier to strengthened pier cannot be kept identically for actual size piers and scaled down specimens. As a result, different displacement scaling factors for specimen A and B were used. On the other hand, the shear strengthening was achieved by supplementary stirrups. The ratio of shear capacity to flexural yielding load of the scaled down specimens, for both strengthened and non-strengthened piers, were set to be equal to the analogous ratio of the corresponding actual size piers. The details of both specimens are displayed in Table 3. And also, Table 4 shows the number of scaled down specimens, testing cases and material properties.

4. RESULTS AND DISCUSSION

The PSD test of the N-H case could proceed only up to 380 steps, as the specimen failed during the test. For the other cases, PSD test were conducted successfully up to 1,500 steps. Analyses of the bridge systems were also made, utilizing the same 3-DOF model. Bi-linear model was used in the calculation of pier restoring force.

Fig. 8 shows the acceleration responses at the pier top for all bridge systems. The responses obtained from PSD test agree well with the analyses. The peak acceleration responses of the non-strengthened pier cases are similar to the one of the strengthened pier cases. This shows that the peak acceleration response at the pier top of a bridge system is not sensitive to the change in pier capacity due to strengthening, even when interaction between pier and foundation is considered.

In another point of view, the same peak acceleration response at the pier top means that the same equivalent seismic force should be used in the seismic design [4] of the piers for both before and after strengthening. Therefore, even the strengthened bridges are expected to be safe from foundation failures, as the yielding loads of foundations are greater than that of the piers for all cases. However, this is not true when the interaction between pier and foundation is considered.
The load-displacement relationships of all restoring springs are presented in Fig. 9. Differences between the responses of pseudo-dynamic test and analyses are observed in pier spring, as softening and hardening of stiffness occurred in the specimens. However, the responses in foundation springs of PSD tests are mostly identical to the analyses.

The pier specimen of case N-H failed during the test, and also, the specimen of case N-S exhibited a large stiffness reduction in the latter steps. In contrast, all the specimens of the strengthened pier cases could sustain their stiffness throughout the PSD test. This ensures that the strengthening of the piers is inevitable in order to make the bridges survive the earthquake.

For hard soil foundation, the peak displacement of pier reduces from 28.82 cm in N-H case to become 6.31 cm in S-H case. On the other hand, the peak responses of both sway and rocking springs increase, in which, the response beyond the yielding point could be observed. The same trend is also observed in the cases of soft soil foundation. The peak displacement of N-S pier reduces from 27.43 cm to be only 11.14 cm in S-S case. However, in the rocking spring of S-S case, even though the response becomes larger comparing to the N-S case, the response is still in an elastic range. As a result, a severe increase in the response of sway spring

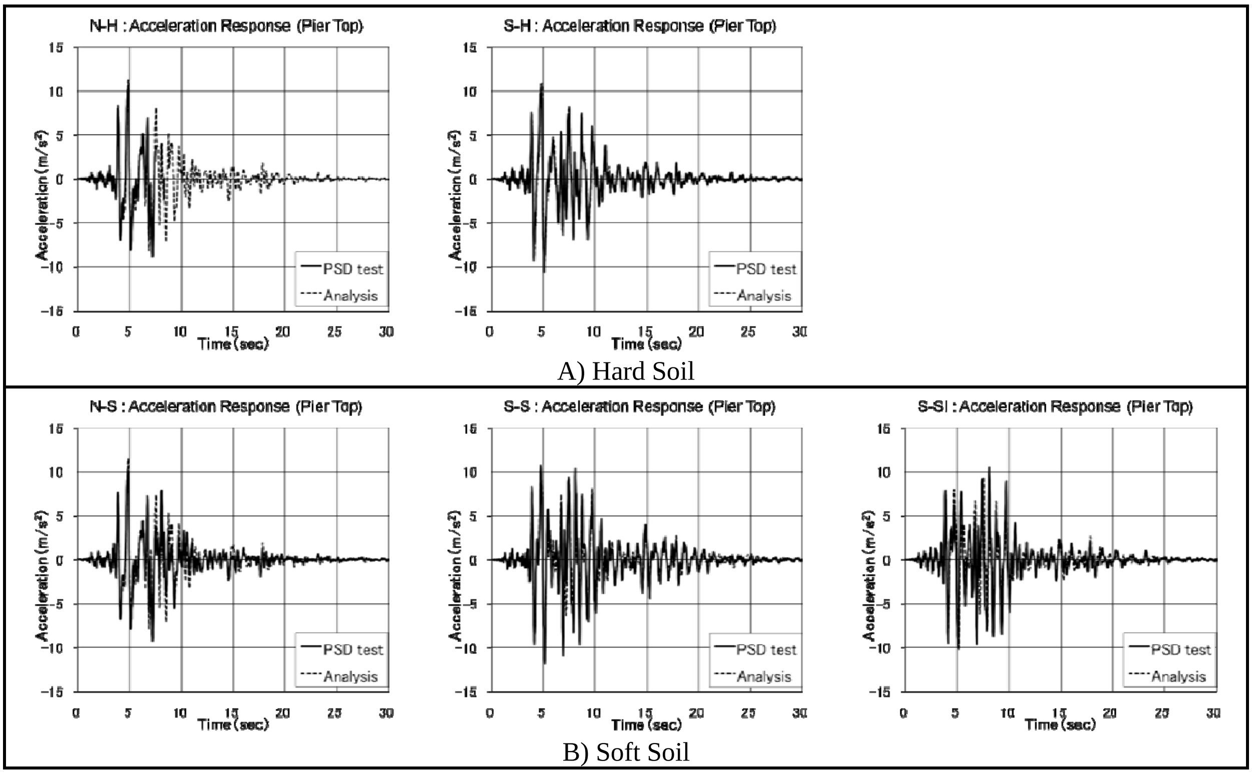

Fig. 8 Acceleration responses at pier top

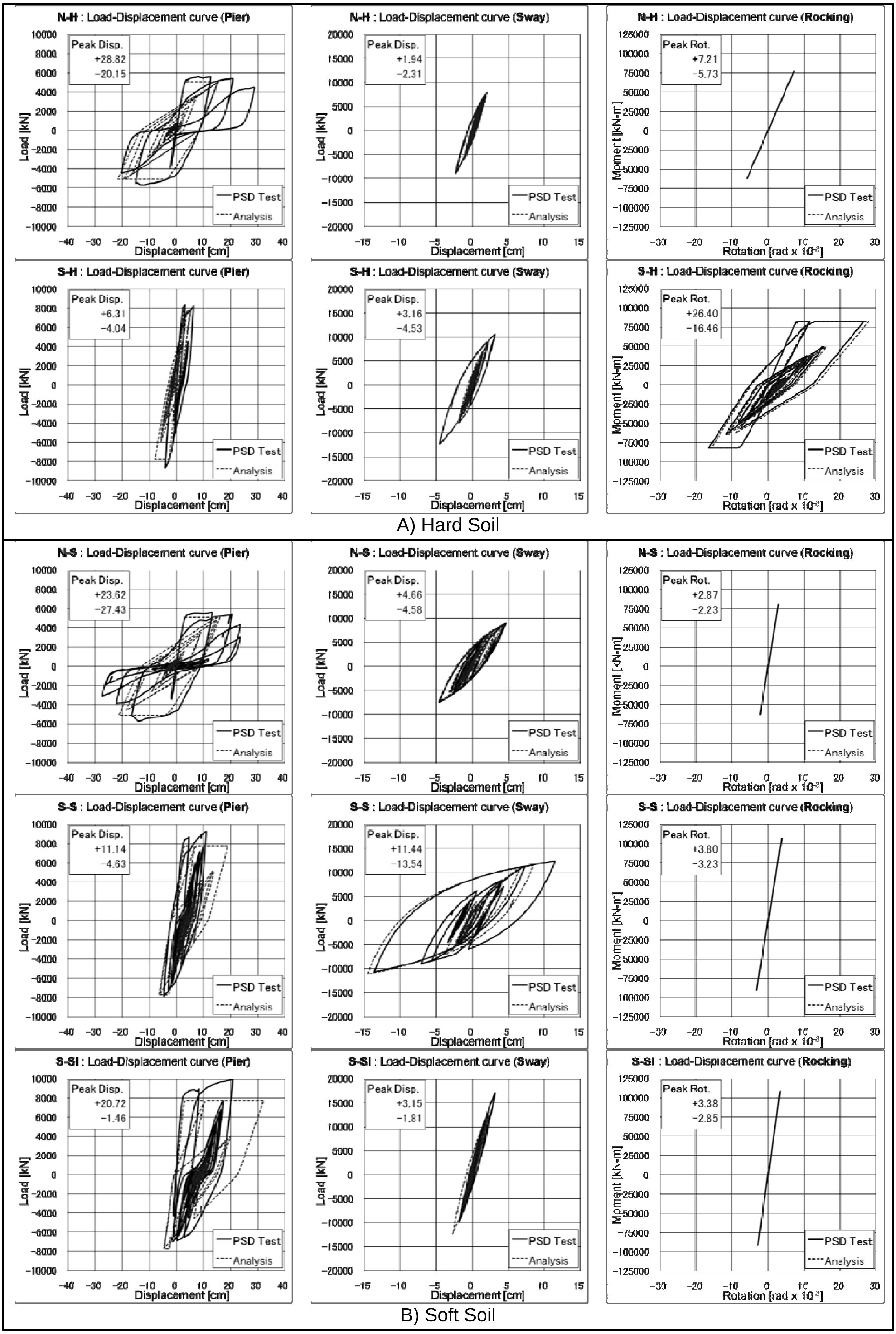

Fig. 9 Load-Displacement responses in restoring springs

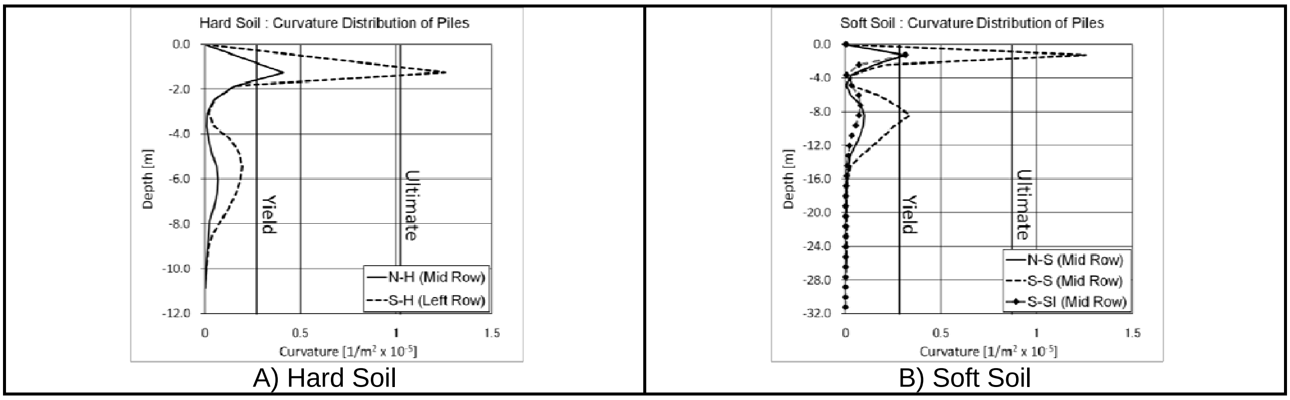

Fig. 10 Peak curvature distribution of piles in the foundation for each case

is observed, as it should be the main source of energy dissipation for the system.

The results of both hard soil and soft soil foundation clearly show the possibility of foundation failure when pier strengthening is applied. The yielding of sway spring relates to the flexural yielding of piles, as well as, the yielding of rocking spring indicates the limitation of end bearing capacity of soil springs.

Therefore, it may be concluded that if pier strengthening is applied, the foundation is prone to be the weakest link in a bridge system. This means that the total collapse of the bridge may occur if it is struck by a strong earthquake in the future.

The foundation of a bridge situated in hard soil may suffer from the yielding of piles as well as the degradation of soil bearing capacity. On the other hand, the foundation of a bridge sited in soft soil may experience only the yielding of piles, as the foundation in soft soil normally possesses a very stiff rotational capacity due to its lengthy depth of piles.

In order to investigate the failure characteristic of piles, a series of analyses, conducted on the beams-springs model of the foundation, were made. The arrays of the displacement responses in sway spring of the PSD test were used as the input. The displacement controlled static analyses on the corresponding beams-springs models of the foundations were conducted, and the distribution of curvatures along the depth of piles were plotted. Fig. 10 shows the peak curvature distribution of piles in each case. The yielding curvature and the ultimate curvature are also given in order to evaluate the damage of piles. The peak curvature responses of all cases locate in the region of pile head. The peak curvatures in piles of the non-strengthened pier cases are observed to be slightly greater than the yielding curvature, in both soft soil and hard soil foundation cases. Oppositely, the curvatures in the cases of pier strengthening go beyond the ultimate curvature in all the rows of piles. With the peak curvature distribution results, the failure of foundation, as the crushing of concrete in piles in the region of pile head, should be observed in the next coming earthquake for bridges with strengthened pier.

Consider the responses of the case S-SI, which represents ground improvement in soft soil foundation, an increase in the peak of pier displacement, comparing to the result of case S-S, is observed together with the reduction in foundation responses. Moreover, the curvature distribution of piles also shows a much smaller peak in comparison to the curvature of the case S-S. Therefore, the soil improvement may help to prevent the failure of foundation in bridges with pier strengthening. However, the result of this study shows just a preliminary possibility, as the soil improvement was assumed to affect only on the standard penetrating value of the soil. The effect of soil improvement on the response of a bridge subjected to an earthquake still need to be investigated in details.

5. CONCLUSIONS

(1) Foundation is in danger if the target level of pier strengthening is set too close to the foundation capacity.

(2) With soil improvement, an increase in foundation stress as a result pier strengthening is possible to be suppressed.

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