- Technical Paper -

SEISMIC BEHAVIOR OF BRIDGE PIER AND FOUNDATION AFTER STRENGTHENING

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ABSTRACT

This study investigates the seismic behavior of bridge with strengthened pier, and also the behavior of bridge when foundation strengthening was imposed. Seismic responses of bridge were compared by conducting pseudo-dynamic test, in which, foundation was selected as an experimental part. Two cases of foundation specimens have been constructed in order to represent a normal foundation and a strengthened foundation. Damage in the piles was observed in the normal foundation with pier strengthening case, whereas the pier was damaged in the strengthened foundation case.

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

1. INTRODUCTION

A number of RC bridges have been strengthened to achieve both greater loading capacity and ductility in order to sustain the strong earthquake in the future. However, the strengthening of a bridge is normally conducted to enhance only seismic performance of pier. By this, the safety margin of the foundation should certainly reduce. However, the target loading capacity of the strengthened pier is certainly kept lowering than the yielding load of the foundation. As a result, the weakest link of the total system after pier strengthening still seems to be at 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. Besides, foundation strengthening has been imposed in bridges using several techniques. Nevertheless, none of an experimental verification for such technique has been conducted, especially for a large scale experiment.

This paper presents an application of pseudo-dynamic test (PSD-test) for seismic evaluation of bridges. PSD-tests are carried out on two foundation cases to study seismic behavior of a bridge with pier strengthening and its foundation strengthening. Analyses based on the PSD-test algorithm are conducted to clarify the influence of pier strengthening on seismic vulnerability of foundation and also the application of foundation strengthening.

2. PSD-TEST OF BRIDGE PIER SYSTEM INCLUDING FOUNDATION

In this study, the behavior of bridges subjected to a ground acceleration were evaluated by conducting a series of Pseudo-dynamic tests (PSD-test), as it combines the merits of both the experimental and analytical study. Dynamic response of idealized structural model is solved while the restoring force terms of doubt was determined from a displacement-controlled test. The parameters of the study were the strengthening of pier and foundation.

2.1 Idealized Model for PSD-Test

Three degrees of freedom (DOF) model, as shown in Fig. 1, was applied as an idealized structural model of a bridge pier system including foundation. In the model, only three displacement components, pier top lateral displacement (u_1, m), footing lateral displacement (u_2, m) and footing rotation (u_3, rad), are considered. The model also consists of three inertia terms - lump mass of superstructure (m_1, kg), lump mass of footing (m_2, kg) and rotational inertia (I, kg-m^2) of the system. The restoring forces of pier, lateral and rotational restoring forces of foundation were replaced by three springs, pier spring (R_p, N), sway spring (R_s, N) and rocking spring (R_r, N), respectively. H denotes pier height. The equation of motion for the system could be formulated as shown in Eq. (1). The damping matrix [C] was set proportionally to the mass and initial stiffness matrix.

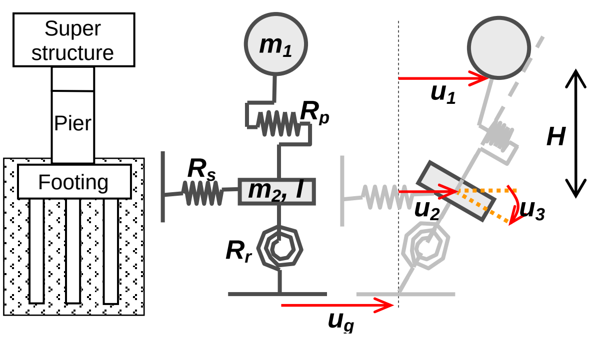

Fig.1 Idealized structural model for PSD-test

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$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & I \end{bmatrix}\begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \end{Bmatrix} + [C]\begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \dot{u}_3 \end{Bmatrix} + \begin{Bmatrix} R_p \\ R_s - R_p \\ R_r - R_p \cdot H \end{Bmatrix} = -\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & I \end{bmatrix}\begin{Bmatrix} \ddot{u}_g \\ \ddot{u}_g \\ 0 \end{Bmatrix} \quad (1)$$

2.2 Evaluation of Restoring Force

Dynamic response of a bridge system subjected to a ground acceleration was evaluated by a numerical solution of the idealized structural model, in which, Newmark's algorithm applying the operator splitting method [1] was employed. At each time step, a predictor displacement of each spring was calculated, and then, applied into either a restoring force-displacement model or a displacement controlled experiment in order to get back the corresponding restoring force. In this study, the foundation (sway and rocking springs) was selected as an experimental part, whereas the pier restoring force-displacement relationship was assigned to be bi-linear model.

2.3 Displacement-controlled Experiment for the Evaluation of Foundation Restoring Forces

In order to obtain the restoring forces for sway and rocking spring, two configurations of the experiments were conducted with the experimental set up as displayed in Fig. 2. Eights steel piles together with steel frame were installed as a supporting system for the hydraulic jacks.

The first configuration utilizes two jacks in order to apply both sway and rocking displacements to the foundation specimen. The lower jack was placed at the same level as the centroid of the footing, and the upper jack was installed 0.75 m above. The displacements of sway (D_{sway}, m) and rocking (D_{rock}, rad) springs could be mapped to the displacements of both hydraulic jacks (D_{upper}, m and D_{lower}, m) conforming to Eq. (2). After applying the displacements to the jacks, sway (R_s) and rocking (R_r) restoring forces could be retrieved from forces on both jacks (R_{upper} and R_{lower}) by Eq. (3)

$$\begin{Bmatrix} D_{upper} \\ D_{lower} \end{Bmatrix} = \begin{bmatrix} 1 & 0.75 \\ 1 & 0 \end{bmatrix}\begin{Bmatrix} D_{sway} \\ D_{rock} \end{Bmatrix} \quad (2)$$

$$\begin{Bmatrix} R_{sway} \\ R_{rock} \end{Bmatrix} = \begin{bmatrix} 1 & 1 \\ 0.75 & 0 \end{bmatrix}\begin{Bmatrix} R_{upper} \\ R_{lower} \end{Bmatrix} \quad (3)$$

For the second configuration, only single jack was installed at the level of footing centroid, and only sway restoring force was involved in the experiment. The displacement of sway spring was input directly as the jack displacement, as well as the restoring force of sway spring was the force on the jack. In parallel, linear model was assigned to be the restoring force-deformation model for rocking spring.

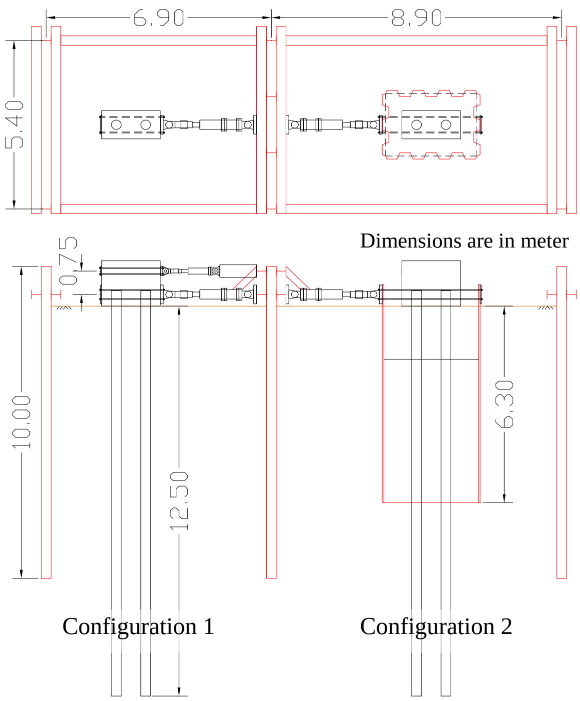

Fig. 2 Experimental set up

3. BRIDGE SYSTEM OF THE STUDY

3.1 Foundation Specimen

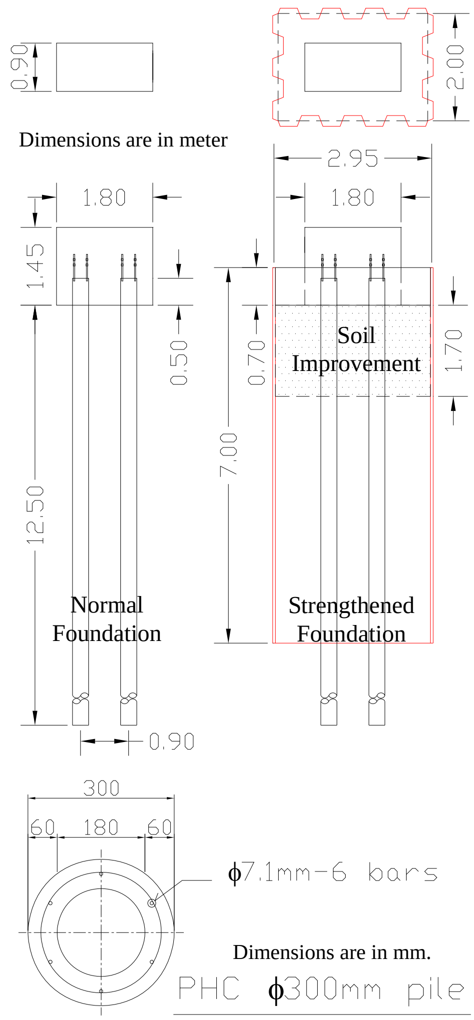

Fig. 3 Foundation specimens

Two foundation specimens used in this study are shown in Fig. 3. The first one is a normal foundation consisting of two ϕ300mm PHC piles with 13.00 m length. The piles were embedded in the soil with 12.50 m depth, and 0.50 m length was anchored into the footing with 6-ϕ22 mm bolt and nuts installed at the pile top in order to ensure the fixity. The footing was of the dimension 0.90x1.80x1.45 m. The pile tip was rested in a sand strata of N=25. Soil profile together with standard penetrating value is given in Fig. 4. The detail of footing part is not necessary here as it was designed to be rigid.

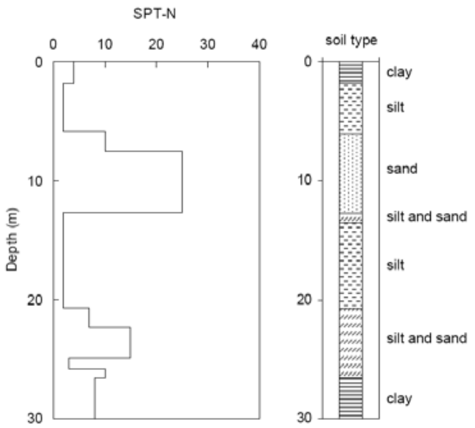

Fig. 4 Soil profile

The second case is a strengthened foundation, according to IN-CAP method [2], which upgrades the foundation capacity by utilizing steel sheet piles together with ground improvement. Steel sheet piles (U-Type, FSP-II) of 7.00 m length were placed surrounding the edge of the foundation with 6.30 m embedding depth. The ground enclosed by the sheet pile up to -1.70 m depth was replaced by improved soil, with compressive strength 1.8 MPa. Gap between the footing and the sheet piles was filled up with concrete.

3.2 Pile Bending Test

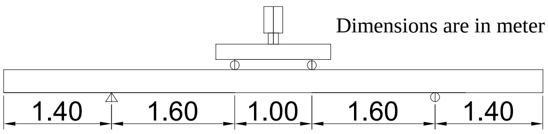

Fig. 5 Pile bending test set up

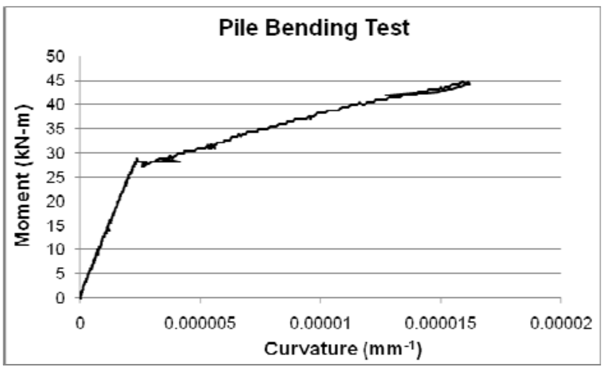

Fig. 6 Moment-curvature of pile

Concrete compressive strength of the pile was 79MPa, and the yield stress of prestressing bar was 1,275 MPa. A bending test of 7.00 m length pile, with the same cross section, was conducted with configuration as shown in Fig. 5. Moment-curvature relationship of the pile obtained from the bending test is given in Fig. 6. The cracking curvature was $2.3x10^{-6}$ $mm^{-1.}$ However, no clearly yielding point could be observed. The failure mode of the pile bending test was the breaking off of the prestressing bar, and the observed ultimate curvature was $16.3x10^{-6}$ mm^{-1}.

3.3 Estimation of Foundation Capacity

Preliminary estimation of the foundation specimen capacity (normal cases) has been made using 2D non-linear finite element analysis code named WCOMD, as shown in Fig. 7a. Further details regarding elements and constitutive models could be referred to [3]. Two loading configurations were applied separately in order to evaluate both sway and rocking capacity. Thickness of soil elements was assumed to be 30 cm, same as the pile section. This was treated as the lower bound of the capacity, and was used as the design foundation capacity for the determination of other parameters for the bridge model (discussed later). The estimated sway and rocking capacity are given in Table 1. The sway capacity governs the overall capacity of the foundation with a pile rupture in a position suddenly below the footing.

Table1 Estimated capacity of normal foundation

	Load	Displacement
Sway	125,850.00 N	0.034 m
Rocking	301,267.50 N-m	0.01 rad

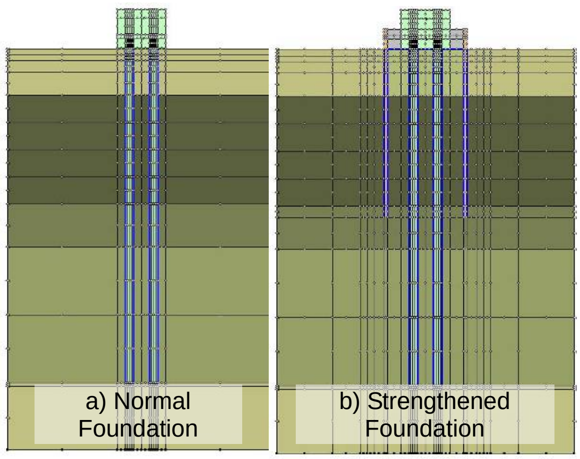

Fig. 7 FEM model of foundation

3.4 Bridge Set Up

In order to set up the mass terms and also the parameters for the pier bi-linear model, the estimated foundation capacity, in the former section, was set to be equal to 1.15 times the top weight. The yielding load for bi-linear model of non-strengthened piers was assumed to be 0.50 times the top weight, as the pier was assumed to be designed by 250 gal peak acceleration, with safety factor for allowable stress design of two. On the other hand, the yielding load of the strengthened pier was set to be 1.00 times the top weight, conforming to the present design code [4] with pier ductility capacity of four. The footing mass, the rotational inertia of the system and also elastic stiffness of the bi-linear model for the pier were selected in order to make this small bridge be able to represent a bridge of large scale. In which, the ratio between the natural frequencies of pier, sway and rocking springs of the small bridge was set equivalent to the one of the large size bridge. To incorporate over-strength in the foundation specimen, over-strength factors for the piers of both normal and strengthened cases were assumed to be 1.20. The values of mass terms, parameters for pier bi-linear model and foundation capacity are summarized in Table 2.

Table 2 Summary of parameters for analysis model

Mass	
M1 (kg)	11,159.30
M2 (kg)	5,021.68
I (kg-m^2)	3,911.85

Pier Model	PY (N)	YY (m)
Normal	65,661	0.0068
Strengthened	131,322	0.0137

Foundation	Pu (N)	Yu (m)
Normal	166,970	0.0814

PY : Yielding load of pier bi-linear model
YY : Yielding displacement of pier bi-linear model
Pu : Ultimate load of foundation (from experiment)
Yu : Ultimate displacement of foundation (from experiment)

3.5 Ground Acceleration

Only 175 steps of ground acceleration were selected from the peak zone of the 1995 Kobe record due to the experimental time limitation. Time scaling and amplitude scaling have been adjusted to make the frequency range of high response match with the small size bridge of this study. The time step size was 0.01 second, and the peak ground acceleration was 1182 gals. The input ground acceleration was plot comparing to the selected original Kobe record in Fig. 8.

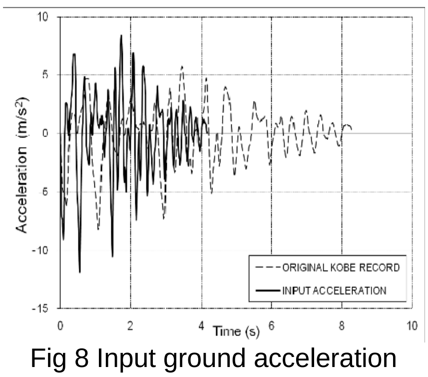

Fig 8 Input ground acceleration

3.6 Parameters and Experimental Cases

Three bridge systems were set up from two cases of piers and two cases of foundation. The first system (N-N) represents a non-strengthened bridge that was designed according to 250 gals peak acceleration. The second system (S-N) is the bridge with pier strengthening. And finally, the third system (S-S) is the bridge with both pier and foundation strengthening. Details of bridge systems and response evaluation method are given in Table 3.

To obtain the response of the N-N system, only an analysis has been conducted. The analysis was conducted like a PSD-test conforming to the first configuration as discussed in section 2.3; however, instead of applying displacements to a foundation specimen, FEM analysis with the model as in Fig. 7a was used for the evaluation of restoring forces for sway and rocking springs. On the other hand, the response of the S-N system was obtained by conducting both PSD-test (configuration 1) and the analysis. However, the thickness of soil elements in the FEM model (in both N-N and S-N cases) have been changed to 40 cm, as it gave a better agreement to the PSD-test result of the S-N system. For the final system S-S, PSD-test with the second configuration as discussed in section 2.3 has been carried out in order to reduce the complexity of loading condition, because the response of this case will be used in a detailed analysis of the strengthened foundation in the future. The rocking stiffness of the strengthened foundation was estimated from the analytical model shown in Fig. 7b, in which a trial modeling of the strengthened foundation has been analyzed. The thickness of soil element used in the model was 2.00 m, same as the width of sheet pile caisson. The sheet piles were modeled using elastic element with the width giving an equivalent sectional inertia. It is to be noted that Rayleigh's damping of 5% was used in all systems.

Table 3 Bridge systems of the study

System	Pier	Foundation	Method of Evaluation
N-N	Normal	Normal	Analysis
S-N	Strengthened	Normal	Analysis, PSD-test
S-S	Strengthened	Strengthened	PSD-test

4. RESULT AND DISCUSSION

The analysis and PSD-test responses of all bridge systems are displayed in Fig. 9. In addition, the ultimate states of the two foundation specimens were verified by conducting cyclic test, after the PSD-test finished. The cyclic test of the normal foundation was terminated when breaking of prestressing bar in pile occurred. However, the cyclic test of the strengthened foundation has been stopped without any manifest damaged.

4.1 PSD-simulation of N-N and S-N systems

As observed in the analysis and PSD-test of S-N case, the analysis using FEM foundation model matches with the real PSD-test up to some extent. The difference in analysis and PSD-test is that the FEM model of foundation gave lower stiffness at the beginning steps, but greater stiffness in latter steps comparing to the specimen. Anyway, this still verifies that the response of the N-N system is reliable.

Consider the analysis result of the N-N system, pier ductility demand of 7.63 is required in order to sustain the earthquake. However, the foundation responses are in low stress range, as there is no yielding observed in the FEM foundation model. Such a behavior could be found in many bridges without strengthening that the pier performs as the weakest link of the system. On the other hand, the ductility demand of the pier in S-N case (analysis) is reduced to be only 2.13, whereas, the larger response in foundation is observed. The yielding in RC element of the FEM model is also observed.

The analysis responses of N-N and S-N systems have confirmed that the weakest link of the system has shift from the pier to the foundation when strengthening of bridge pier is applied.

4.2 PSD-test and Cyclic Test of S-N and S-S systems

PSD-test results of S-N and S-S systems are also displayed in Fig. 9. For the PSD-test result of S-N system, pier response has just reached the yielding point with ductility requirement of only 1.01. However, a clear reduction in stiffness of both sway and rocking responses are observed. The decreasing in the stiffness of sway and rocking springs occurs as a result of damaged in both soil and piles.

In order to investigate the damaged of pile, strain gauges were attached to the prestressing bar inside the piles up to around 6 m depth. Maximum curvature at each position, calculated from strain data, was plotted along the pile length as displayed in Fig. 10. In the left pile, the observed maximum curvature was $11.4\text{x}10^{-6}$ mm^{-1}, and it was $24.9\text{x}10^{-6}$ mm^{-1} for the right hand side pile. Comparing to the ultimate curvature from the pile bending test which was $16.3\text{x}10^{-6}$ mm^{-1}, both piles could be considered to have quite severe damage.

In addition, after conducting the PSD-test, ultimate loading of the foundation specimen was checked using the lower jack only. Loading was continued until the breaking of prestressing bar was observed, and then reversed in direction until another breaking of prestressing bar occurred. The result is also given in the sway response of S-N system in Fig. 9. As could be observed from the figure, the peak displacement of foundation (sway) during the PSD-test was 83% of the prestressing bar breaking displacement.

The above experimental result confirms the danger of foundation in the bridge with only pier strengthening. Even though such a brittle failure will never occur in foundation as the soil possess infinite ductility. However, the investigation of foundation failure after an earthquake, and also the repair of foundation are extremely difficult. Therefore, the foundation is recommended to be designed in a fully safe side.

In the PSD-test result of S-S system, the pier response requires ductility of 4.04. On the other hand, almost linear response is observed in sway spring. The low response level in foundation of S-S system could be also confirmed from both maximum curvature plotted along the pile length and also the reversed-cyclic test after the PSD-test, which has to be separately displayed in Fig. 11 due to the too different loading capacity.

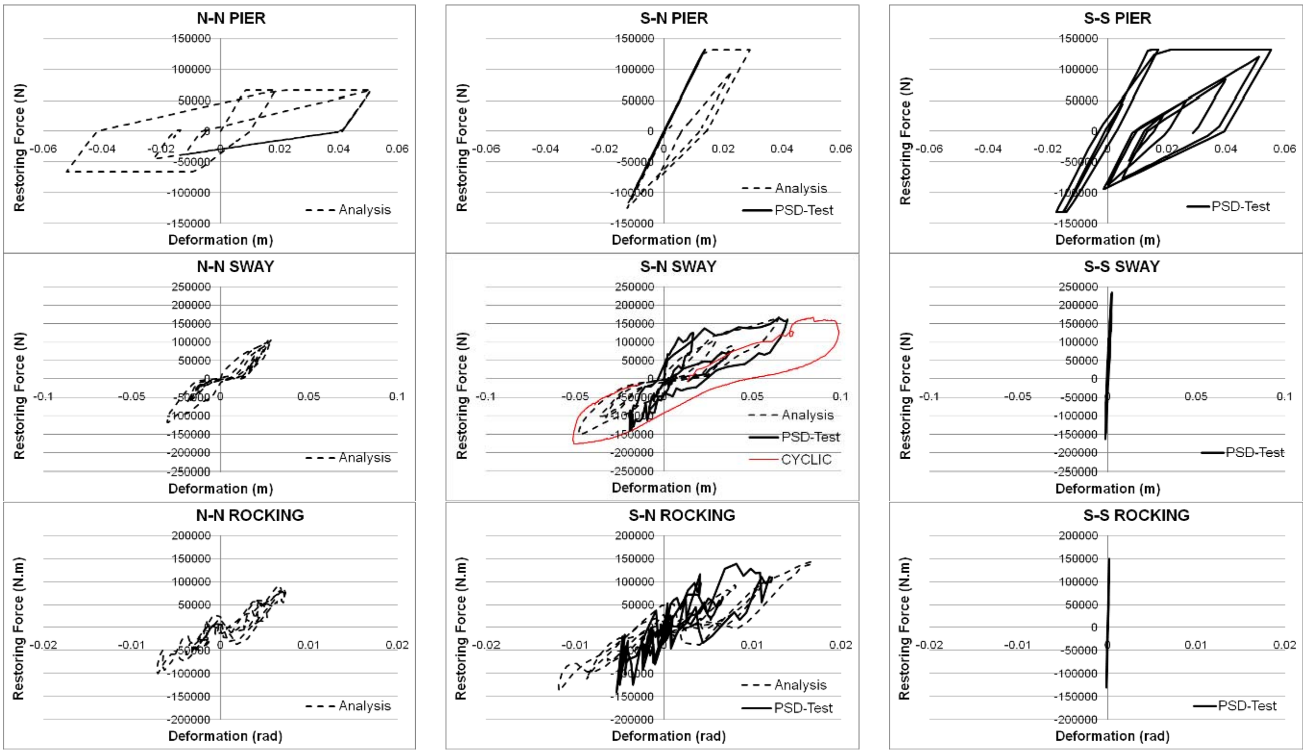

Fig. 9 PSD-Simulation, PSD-test and Cyclic test results

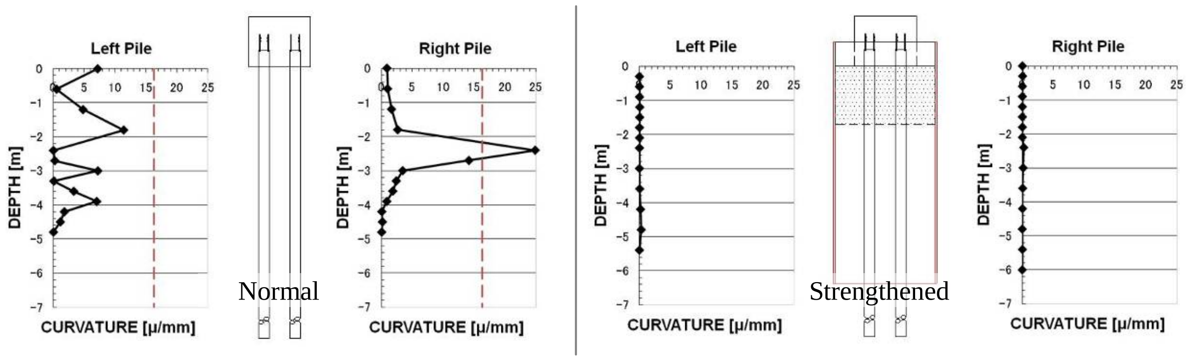

Fig. 10 Maximum curvature plot along pile length

The maximum curvature in piles of strengthened foundation of Fig. 10 narrates extremely low stress level in the piles. From the reversed-cyclic result of strengthened foundation, the foundation started a large stiffness reduction at the load about three times of the peak load during the PSD-test. The greater load carrying capacity of the strengthened foundation occurs mainly due to the surrounding sheet piles which acts like a caisson. Though, too much strengthening of foundation in this study is observed from the test, it is interesting that the maximum load in the foundation during the PSD-test does not increase so much comparing to the normal foundation response, as the maximum load in the system is controlled by either the yielding load of the pier or the foundation. Therefore, in order to get a safe bridge system to sustain a possible stronger earthquake in the future, a foundation could be strengthened up to an extremely high capacity, and the pier is just required to provide enough ductility capacity. And, the pier should be ensured as the weakest link of the system in any means.

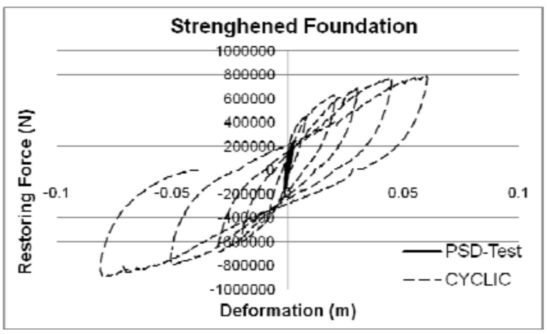

Fig. 11 Reversed-cyclic testing on strengthened foundation after the PSD-test finished

5. CONCLUSIONS

(1) A pseudo-dynamic test of a bridge system considering the interaction between pier and foundation has been developed, using a foundation specimen as an experimental part.
(2) An increase in foundation response as a result of pier strengthening is confirmed based on the analysis result.
(3) From PSD-test result, a foundation with pier strengthening is in danger, even though the target strengthening capacity of the pier is kept lower than the foundation capacity.
(4) Foundation strengthening can mitigate the foundation damage risk according to the pier strengthening. However, it also leads to a larger ductility demand in the pier based on the PSD-test results.

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