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EFFECT OF EARTHQUAKE-PROOF REINFORCEMENT BY GROUND ANCHOR AND DAMPER ON AN EXISTING BRIDGE WITH HIGH PIER

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ABSTRACT

In this study, the oscillation behavior and the seismic reinforcement effect of an existing bridge with high pier are investigated by dynamic analysis method. The bridge pier is retrofitted by ground anchor and damper. The PC cables and the dampers are assumed to be strung between the column of the pier and the ground anchors. Focusing on whether to introduce an initial tension and a damper, the analysis is conducted and, the analysis results show that this reinforcement method has a significant positive effect in improving the aseismicity of the bridges with high piers.

Introduction

Bridges serve as important constituent elements of highway and railway networks and, when damaged by earthquakes, have a direct negative effect on earthquake relief and reconstruction. Of particular note, the recent Northridge Earthquake of 1994, the Hyogoken-Nanbu Earthquake of 1995, the Taiwan Chi-Chi Earthquake of 1999, the Iran Earthquake of 2001, the Chuetsu Earthquake of 2004, and the Wenchuan Earthquake of 2008 caused serious damage to many lifeline facilities, including aseismically-designed bridges. Triggered by such damage, great importance is attached to the seismic reinforcement of the existing bridges and research on the seismic reinforcement becomes active remarkably.

As for the existing bridge, the seismic reinforcement methods can be classified into two types, generally. One type is to replace the bearings such as changing the fixable or movable bearings for elastic supports, isolation supports etc. to improve the aseismicity of the whole structure, and the other is to retrofit the columns and the foundation structures to improve the proof strength of the substructure. The jacketing method such as reinforcement concrete, steel plate or FRP jacketing etc. is widely used to retrofit the pier columns, however, this reinforcement method is often add a further load to the fundamental structure at the same time as improving the earthquake resistance of the columns. Recently, to string PC cables between the columns of the pier and the walls of the abutments namely PC & PA method is developed to improve the earthquake resistance of the columns in Japan. The effect of reinforcement is limited due to the displacement and the proof strength of the abutments. When the bridge is reinforced

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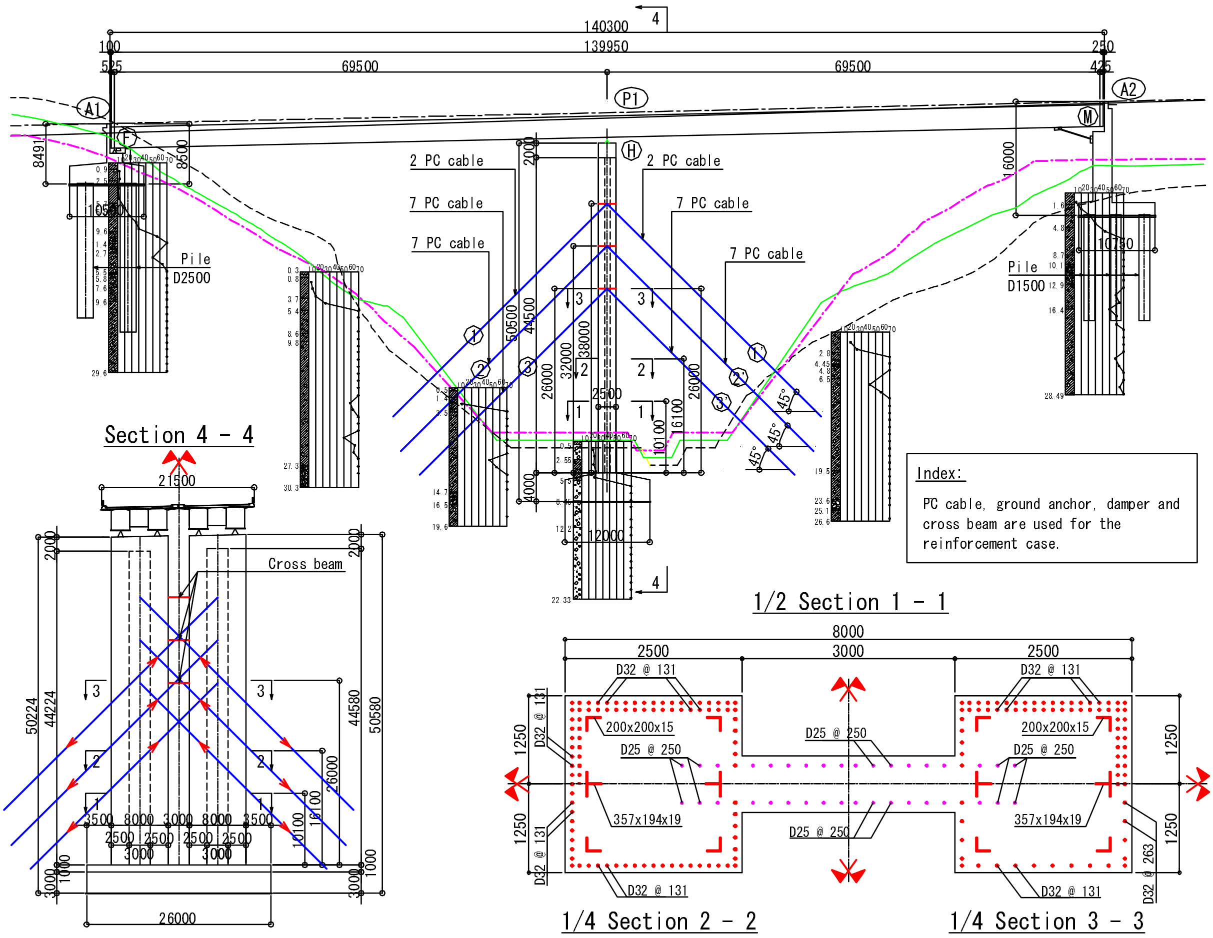

Figure 1. Outline of the target bridge.

by stringing the PC cables between the ground anchor and the superstructure, the column or the foundation structure, it is thought that the effect of reinforcement is considerable and the influence to the fundamental structure resulting from the seismic reinforcement is small because the ground anchor is directly anchored to the ground and the ground anchor hardly deforms. In addition, in case that a damper is also introduced into the seismic reinforcement, the effect of reinforcement is even more achieved due to the earthquake energy absorbed by the hysteresis attenuation. In this paper, as an example, the effect of earthquake-proof reinforcement by ground anchor and damper on a certain existing bridge with high pier is verified by dynamic analytical method.

Target Bridge

The bridge considered for this study is a two-span continuously bridge located in a steep V-type valley as shown in Fig. 1. It was constructed in the 80's based on Japanese design code. The span length is 69.50 m (per span) and the total length of the bridge is 140.30 m. The bearing support condition in the longitudinal direction is fixable at A1, hinge at P1 and movable at A2. Above the footings, the bridge is a separation structure of up and down line. The superstructure of each line is composed of a two-box steel girder and reinforced concrete slab. Meanwhile, the pier is consisted of two columns with an I-type section and a spread foundation. The height of each

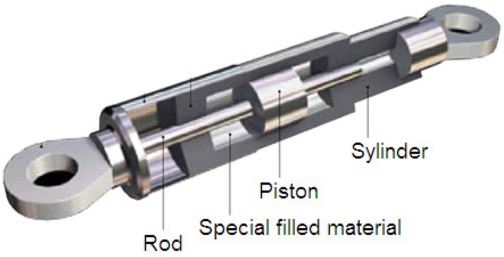

Figure 2. Outline of the damper.

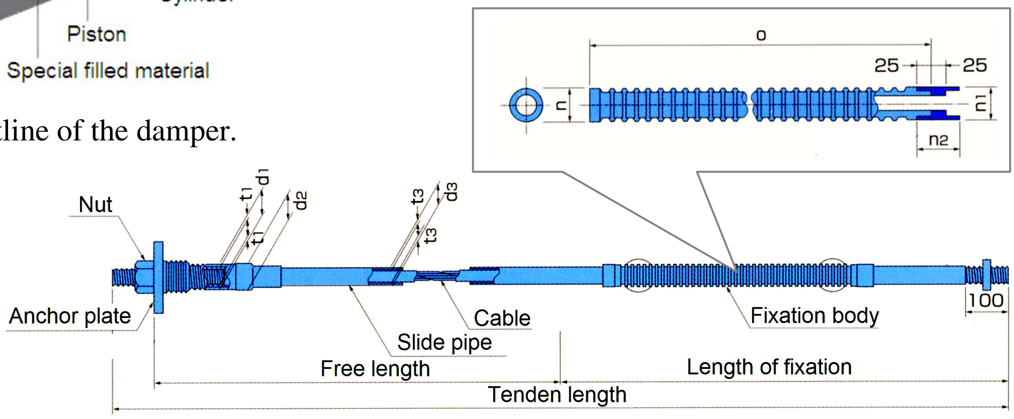

Figure 3. Outline of the ground anchor.

column is 46.50 m and, the width of its section in the longitudinal direction is 2.50 m, the ratio of the height to the column width reaches to 18.60. The columns are made of steel-frame reinforced concrete (SRC). The strength of the column is 24 MPa, the yield strength of the reinforcement bar is 295 MPa (SD295) and that of the steel-frame is 235 MPa (SM400). The longitudinal reinforcement is terminated at column mid height three times, the first time is at the height of 10.10 m from the column bottom where the reinforced bar arrangement is decreased from two layers to one layer, the second time is at the height of 16.10 m where the space of the reinforced bar is increased from 131.5 mm to 263.0 mm, and the third time is at the height of 26.0 m where the cross section of the steel-frame is decreased.

The surface ground at the site is a weathering conglomerate layer, and the base ground for seismic design is a conglomerate whose mean N-value is over than 70. The characteristic value calculated by Equation-1 is 0.1 s at pier site. The ground type for seismic design is Type I as specified in Seismic Design (Specifications for Highway Bridges, Japan, hereinafter referred to as Specifications V).

$$T_G = 4 \Sigma H_i/V_{si} \tag{1}$$

where T_G, H_i, and V_{si} are the characteristic value of ground (s), thickness of the i^{th} soil layer (m) and average shear elastic wave velocity of the i^{th} soil layer (m/s), respectively.

Outline of Reinforcement and Calculation Case

The damper considered here is assumed a frictional type one as shown in Fig. 2. The capacities of the horizontal force and the stroke are 400 kN and 100 mm, respectively. A compression frictional type ground anchor as shown in Fig. 3 is assumed to be used for this work. The diameter and the length of the fixation body are 120 mm and 4700 mm, respectively. The PC cable is consisted as 19 × ϕ 12.7, the cross sectional area is 1875.5 mm^2, and the yield tensional strength is 2964 kN.

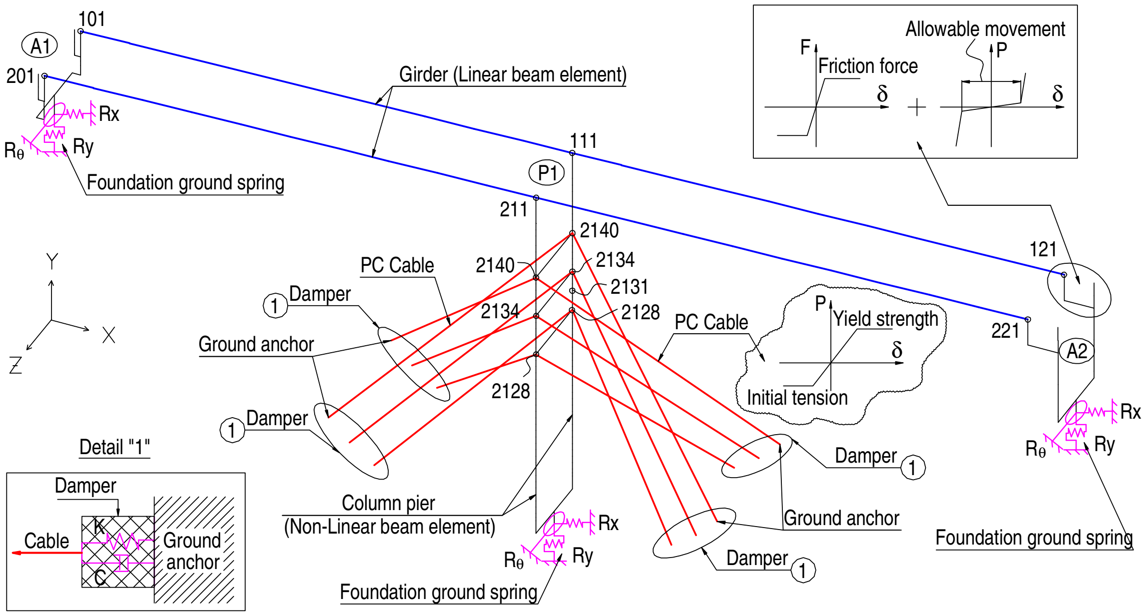

Figure 4. Calculation model.

The PC cables are arranged symmetrically to the longitudinal and transversal direction of the bridge by 3 layers at the height of 26.0 m, 32.0 m and 38.0 m from the column bottom (referred Fig. 1). The ground anchor is set at an angle of 45 degrees with the horizontal direction. The damper is inserted between the anchor and the PC cable. In addition, to improve the reinforcement effect of transversal direction, a cross beam is assumed to be introduced at each height where the PC cable is fastened between the two columns.

In this work, the aseismicity of the existing bridge is firstly investigated, then focusing on whether to introduce an initial tension and a damper, the reinforcement effect is verified. The calculation cases are assumed as Case 1: existing bridge, Case 2: reinforced by anchor + damper with no initial tension, Case 3: reinforced by anchor without initial tension and Case 4: reinforced by anchor with initial tension. The initial tension of the PC cable is assumed to be 890 kN (30% of the yield strength) here.

Analytical Idealization and Analytical Method

The target bridge is modeled to a 3-D frame in this work. The superstructure, the walls of the abutments and the footings are modeled to 3-D linear beam elements and, the bases are modeled to linear spring elements. The bearings except A2 in longitudinal direction are modeled to rigid springs and that of A2 in longitudinal direction is modeled to two nonlinear springs, one is a bilinear spring to take consideration of the friction force and the other is a nonlinear elastic spring to express the sliding and limit the displacement of the bearing. In order to take account of the crack of concrete and the yield of the reinforcement, the relations between the moment and curvature of the column is modeled to the Takeda model. Since the PC cables cannot resist compression, nonlinear elastic springs are used to express the PC cables. As to the dampers, bilinear springs are used to express the relations between the force and the displacement. The calculation model is shown in Fig. 4. Moreover, as to the damping ratio of structure elements, the superstructure, the reinforcement elements and the PC cables are assumed to be 0.02, 0.05 and 0.03, respectively.

To investigate the vibration behaviour and evaluate the seismic performance of the bridge, nonlinear dynamic response analysis method is adopted in this work. In order to take consideration the nonlinearity of the structural element accurately, the time history nonlinear dynamic analyses are performed by the direct integral calculus, the integration is taken as the Newmark beta method, and the time interval of integration is 0.001 s. In addition, the Rayleigh type damping coefficient calculated by strain energy damping ratios is used in the analysis.

Input Ground Motions

The acceleration time history is used as the earthquake input, and only horizontal excitation is in consideration for this study. The earthquake inputs for the analyses are selected as the three waves of the Level 2 Type II seismic motion (inland earthquake) on Type I ground as specified in Specifications V, which are based on acceleration strong motion records actually obtained at ground surface during the Hyogo-ken Nanbu Earthquake of 1995. Two of the three records were obtained at the Kobe Meteorological Observatory (The maximum accelerations were 8.12 m/s^2 − Kobe Wave 1 and 7.66 m/s^2 − Kobe Wave 2, respectively) and the other one was obtained around the Inagawa Bridge (The maximum acceleration was 7.80 m/s^2 − Inagawa Wave 3). The wave forms used in the analyses are shown in Fig. 5.

Eigen-Value and Vibration Mode

Eigen-value is calculated by subspace method, and until the 50th vibration mode is calculated. As an example, the results of the longitudinal direction are taken up to present. The principal natural modes of vibration and natural frequencies of the bridge of the calculation case 1 and Case 2 are shown in Fig. 6. As to the fundamental mode, either Case 1 or Case 2 is a translation (the 1st mode of P1: flexural vibration) coupled with vertical vibration (the 1st mode of the superstructure: unsymmetry) mode. The natural frequency of Case 1 is 1.102 Hz and that of Case 2 is 1.128 Hz, the ratio of Case 2 to Case 1 is 1.024. As to the participation factor, the ratio of Case 2 to Case 1 is 0.900 Hz. It means that the vibration of Case 1 (existing bridge) contributes to the fundamental mode more than that of Case 2 (bridge retrofitted).

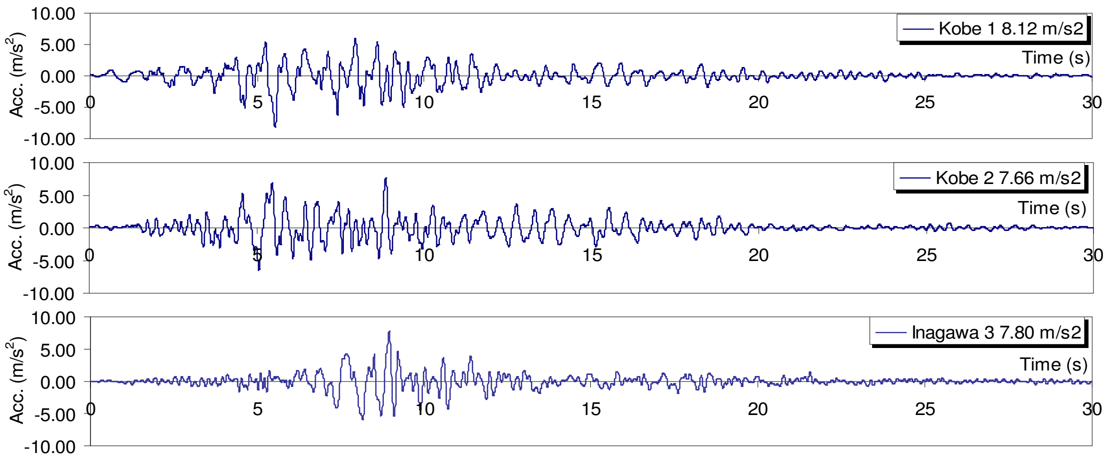

Figure 5. Input earthquake wave.

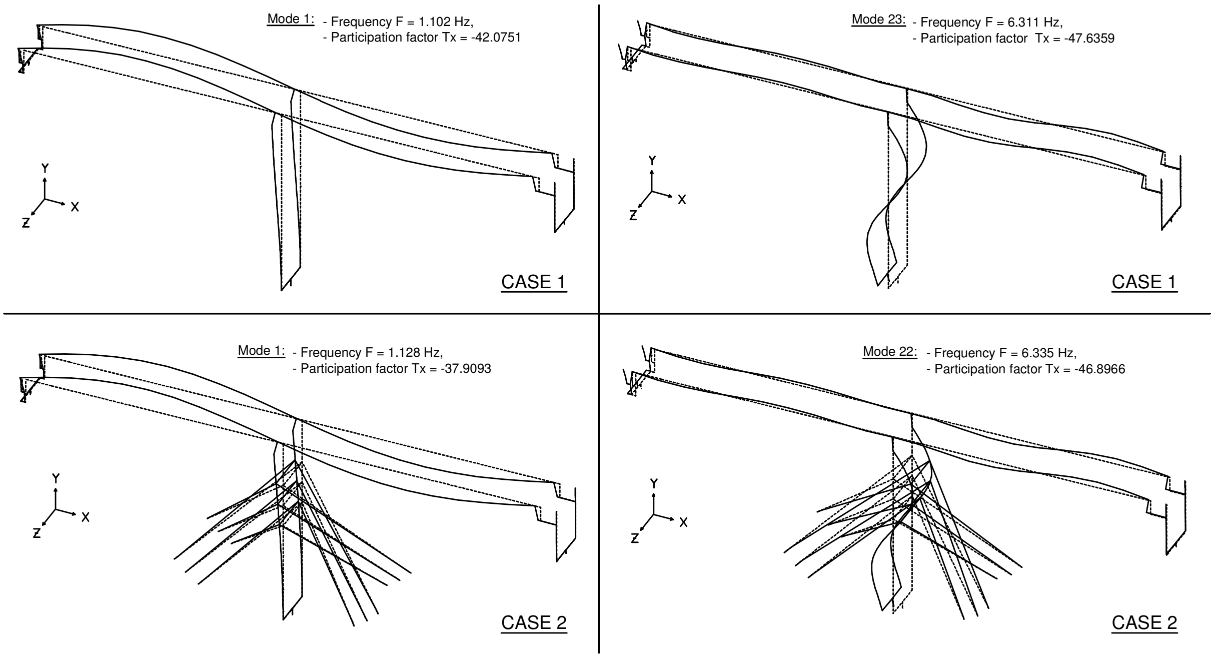

Figure 6. Vibration mode.

Table 1. Eigen-value.

Case 1 (Existing)				Case 2 (Retrofitted)			
Mode	f (Hz)	h (%)	P. F. -X	Mode	f (Hz)	h (%)	P. F. -X
1	1.102	2.788	42.1	1	1.128	2.590	37.9
5	1.635	4.545	49.5	4	1.650	4.378	47.3
7	2.191	5.698	63.2	6	2.280	7.799	69.3
9	2.592	7.175	14.5				
14	4.143	19.200	50.6	14	4.143	19.202	50.6
16	4.761	4.013	14.7	16	4.762	4.015	14.8
23	6.311	5.919	47.6	22	6.335	5.913	46.9
30	9.061	8.790	13.3	30	9.071	8.689	13.1
31	9.485	6.138	27.9	31	9.494	6.098	27.9
35	10.850	2.996	10.7	34	10.852	2.995	10.8
42	14.407	5.433	12.0	42	14.415	5.438	12.0

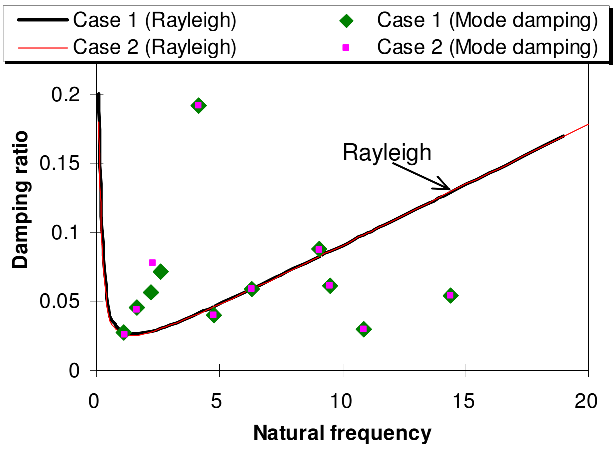

Figure 7. Mode damping.

The 23rd vibration mode of Case 1 is the 4th mode of P1 (flexural vibration) with a natural frequency of 6.311 Hz. As to Case 2, the same vibration mode with Case 1 is the 22nd mode and the natural frequency is 6.335 Hz, the ratio of Case 2 to Case 1 is 1.004 Hz. Table 1 shows the eigen-value. When the natural frequency is higher than 5.0 Hz (higher than 14th mode), there is little difference between the Case 1 and Case 2. Meanwhile, the natural frequency increased and the participation factor changed a bit for the low-order vibration mode.

The Rayleigh damping coefficient is calculated by Equation 2 using the 1st and the 23rd vibration mode for Case 1, and using the 1st and the 22nd vibration mode for Case 2 as shown in Fig. 7. As to the mode damping coefficient, although in case that the natural frequency is higher than 5.0 Hz, the mode damping almost becomes the same, there is a little difference between Case 1 and Case 2 for the low-order vibration mode.

$$h = (\alpha + \beta \times \omega^2)/(2\omega) \tag{2}$$

where h and ω are the mode damping ratio (%) and the angular frequency (rad/s), respectively and, α , β are the Rayleigh damping coefficients.

As to the eigen-value, since the stiffness of the PC cable is small, the influence on eigen-value for high-order mode is limited due to the reinforcement, while there is somewhat difference for the low-order mode that greatly influences the aseismicity of the bridge.

Vibration Behaviour

Focusing on the response displacement and acceleration, the vibration behaviours of the bridge before and after reinforcement are verified by the results of the longitudinal direction. The time history of the response horizontal displacement of the superstructure at P1 calculated by Kobe Wave 1 is shown in Fig. 8-1. As to the wave form of the displacement time history, each case after reinforcement is very similar, and between reinforcement before and after, except around the 10 s and 20 s the form is almost similar. The maximum (minimum) displacement of Case 1, Case 2, Case 3 and Case 4 are 0.085 (0.078) m, 0.096 (0.072) m, 0.097 (0.071) m and 0.095 (0.075) m, respectively. The maximum displacements of the reinforcement cases increase a little (about 10%) compared with the existing bridge, while the changes among the reinforcement cases are just a little. Fig. 8-2 shows the distribution of the maximum and minimum displacement of the column. The displacements are the mean values by the three earthquake motions. The displacements at the mid height of the column are bigger than that at the column top. The distribution shape is between the 1st vibration mode and the 2nd vibration mode. Consequently, besides the 1st vibration mode, the higher mode also dominates the vibration of the bridge with high pier. Since ground anchor and damper are installed in the reinforcement cases, the displacements at the mid height of the column decrease greatly. Because of the installing of initial tension, the PC cable might resist a compression and the stiffness of reinforcement is larger than that of the other reinforcement cases, the decrease quantity of Case 4 is the largest. In addition, as to Case 2, owing to the installation of a friction type damper, the displacement nearly 10% decreases than Case 3 at the height of 33.0 m from the column bottom where the displacement is largest. On the other hand, compared with the mid height of the column, the displacement at the column top hardly decreases due to the reinforcement. It is thought that the pier column is restricted by the superstructure, and the superstructure is supported by A1 and A2 abutments which stiffness is large.

The time history of the horizontal accelerations of the superstructure at P1 calculated by Kobe Wave 1 is shown in Fig. 8-3. As to the maximum acceleration, Case 1 is 1.77 m/s^2, Case 2 is 3.34 m/s^2, Case 3 is 3.43 m/s^2 and Case 4 is 3.02 m/s^2. Compared with the existing bridge, the reinforcement makes the maximum acceleration of the superstructure increase near two times, while the changes of the maximum acceleration among the reinforcement cases are just a bit. Fig. 8-4 shows the distribution of the maximum and minimum acceleration of the column. The accelerations are the mean values by the three earthquake motions. As for the bridge before and after reinforcement, the maximum accelerations calculated at the mid height of the column present the similar tendency to that of the superstructure. It is thought that because of the yield of the reinforcement of the column section at the mid column for the existing case (Fig. 9-1), the earthquake energy is dissipated by the yield sections and, as for the reinforcement cases, the yield of the reinforcement is not allowable. Among the reinforcement cases, due to the installation of a friction damper, the acceleration of Case 2 is the smallest.

The Fourier spectrums of the horizontal acceleration by Kobe Wave 1 are shown in Fig. 8-5 and Fig. 8-6. As shown in Fig. 8-5, the vibration of the superstructure of either the existing bridge or the retrofitted bridge is dominated by two vibration modes, the frequency of the first mode is 1.46 Hz, and that of the second mode is 2.44 Hz. It is thought that the influence on vibration characteristic of the superstructure by the reinforcement is small. The spectrum shown in Fig. 8-6 is obtained at the column mid height of 30.0 m. Like the superstructure, there are two prominent vibration modes with the frequency of 1.46 Hz and 2.20 Hz, respectively.

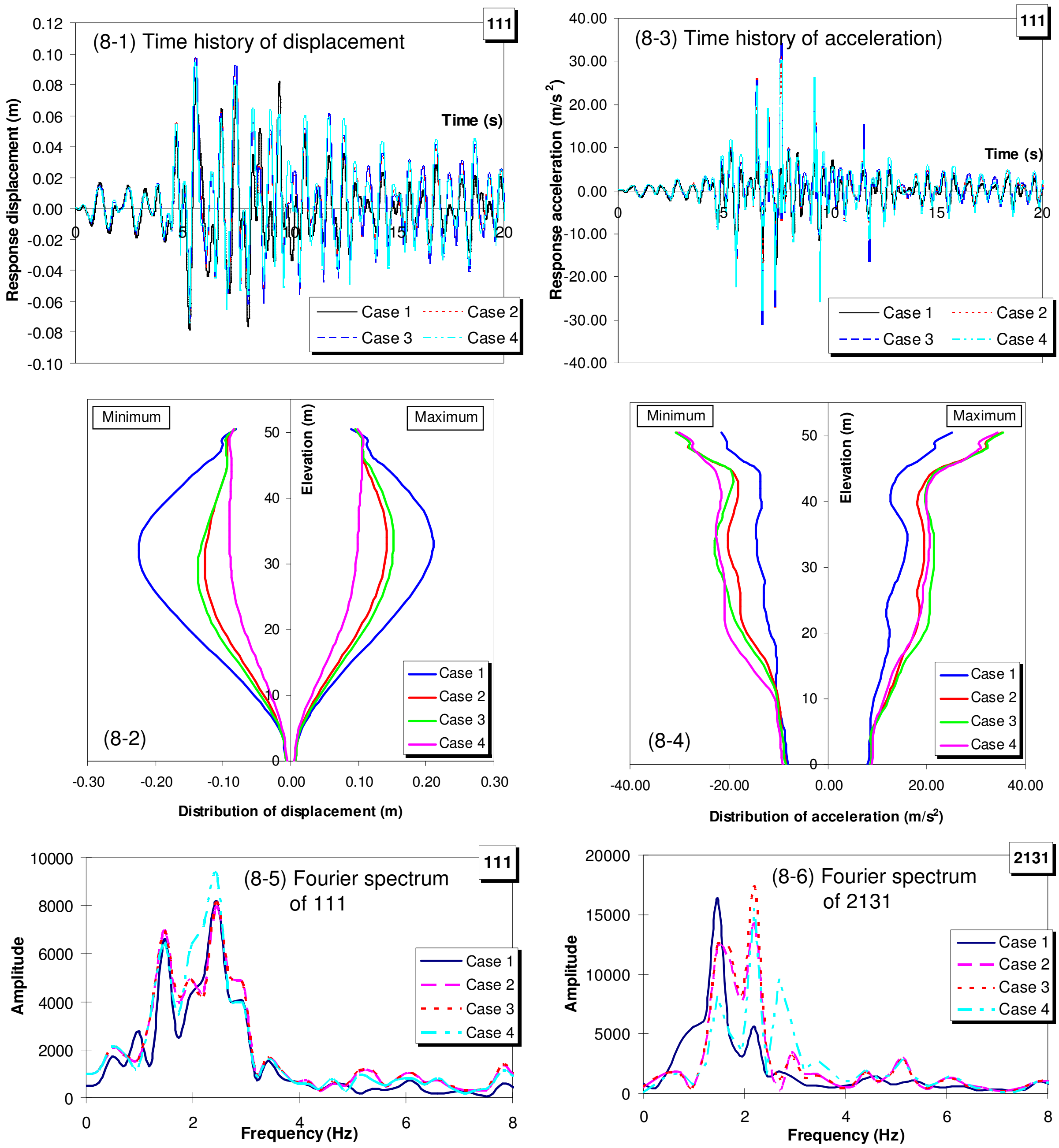

Figure 8. Response displacement and acceleration.

The frequency of the 2nd mode agrees with that of the 7th natural vibration mode of the existing bridge and the 6th mode of the bridge after reinforcement. The contribution rate to these two modes changed greatly between the reinforcement before and after. As to the existing bridge, the vibration is dominated greatly by the 1st mode, while that is dominated by the 2nd mode as to the bridge after reinforcement. Because the superstructure depends mainly on the support of the abutments with large rigidity in the longitudinal direction, it is thought that the influence on the vibrational property of the superstructure by the reinforcement of the pier is limited, while that on the pier is big.

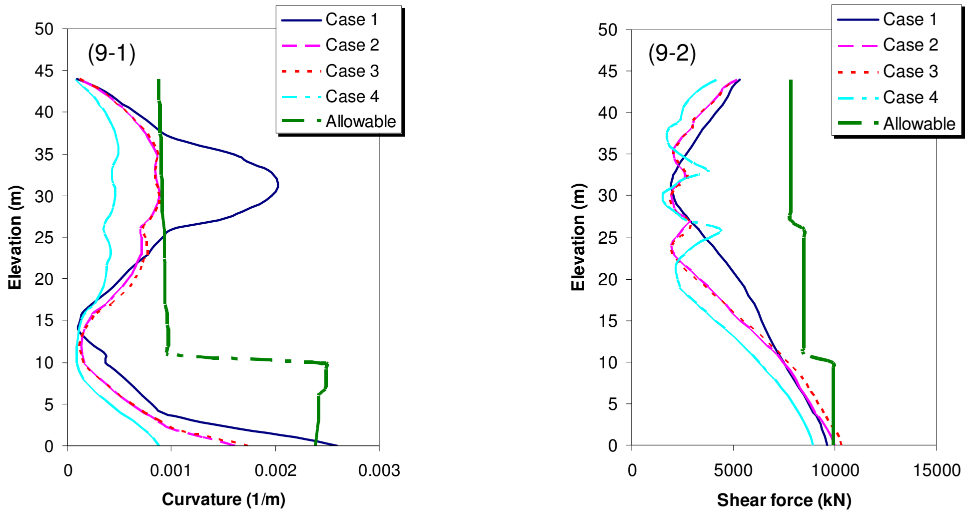

Figure 9. The deformation and the shear force of the pier column.

Table 2. Reaction force of ground anchor.

		1-A1 Side	2-A1 Side	3-A1 Side	1-A2 Side	2-A2 Side	3-A2 Side
Case 2	②	561	641	695	645	726	735
Case 3	③	634	815	958	809	964	1016
	③/②	***1.13***	***1.27***	***1.38***	***1.25***	***1.33***	***1.38***
Case 4	④	1417	1464	1495	1405	1439	1471
	④/②	***2.52***	***2.28***	***2.15***	***2.18***	***1.98***	***2.00***

Table 3. Reaction force of the foundation.

		H (kN)	V (kN)	M (kNm)
Case 1	①	40982	31609	315024
Case 2	②	43174	45608	310107
	②/①	***1.05***	***1.44***	***0.98***
Case 3	③	43541	48004	323298
	③/①	***1.06***	***1.52***	***1.03***
Case 4	④	42501	43265	264157
	④/①	***1.04***	***1.37***	***0.84***

Earthquake Resistance

As an example, the results of the longitudinal direction are used to verify the earthquake resistance. The distribution of the deformation and the shear force of the pier column are shown in Fig. 9. The maximum responses are the mean values by the three earthquake motions. The bend curvature is improved greatly by the reinforcement, especially at the bottom and the mid height where the PC cables are fixed. As for the existing bridge, the curvatures at the bottom and mid height are 0.0026 1/m and 0.002 1/m, respectively. Both of them exceed the allowable values. Here the allowable curvature at the mid height is the curvature when the reinforcement yields owing to the reinforcement being terminated at the mid height. When reinforced by Case 2, the curvatures decrease to 0.0016 1/m and 0.0008 1/m at the bottom and the mid height, respectively. As to the Case 3, the result is almost same as Case 2. When reinforced by Case 4, the curvatures decrease to 0.0009 1/m at the bottom and 0.0004 1/m at the mid height. All the curvatures are settled in the allowable values when reinforced. As shown in Fig. 9-2, while the reinforcement effect can not be expected very much at the column bottom and the cable mounting vicinity, the shear force decreases except for the column bottom and the cable mounting vicinity. According to Fig. 9, it is clear that the initial tension of PC cable has an effect in improving the aseismicity of the bridge.

The reaction forces of the foundations are shown in Table 2. Either of the reinforcement case, except that the vertical reaction force increase from 37% to 52% by the vertical force component of the ground anchor, the horizontal reaction force and the rotation moment hardly change. The reaction forces of the ground anchors are shown in Table 3. The anchor reaction forces can decrease from 13% to 38% when the dampers are installed. The adoption of the dampers can improve the performance of the ground anchor considerable. On the other hand, the initial tension makes the reaction force of the anchor increase over than twice, the demand for the performance of the ground anchor becomes higher.

Conclusions

In order to investigate the oscillation behaviour and the seismic reinforcement effect of the existing bridge that retrofitted by ground anchor and damper, a dynamic analysis was conducted on an existing bridge with high pier. The PC cables and the dampers are assumed to be strung between the column of the pier and the ground anchors. This analysis clarified the following:

1) The introduction of the PC cables and the ground anchors influences the low-order vibration mode of the bridge somewhat.

2) As to the vibration behaviour of the bridge, because the PC cable is assumed to be fixed at the column, the influence on the substructure is bigger than that on the superstructure.

3) This reinforcement method has a significant positive effect in improving the bend deformation of the pier column and, there is hardly addition burden to the foundation structures.

4) The damper makes the reaction forces of the ground anchor decrease and improves the performance of the bridge.

5) The initial tension of the PC cable has an effect in improving the aseismicity of the bridge.

The validity of the reinforcement method by is confirmed by dynamic analysis in this study. It is thought that the introduction of ground anchor and damper will allow the execution of more rational seismic retrofit design.

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