

EVALUATION OF SEISMIC PERFORMANCE AND DAMAGE CONTROL FOR HIGH WAY WITH PLURAL FIXED SUPPORTS

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In this paper, mitigation of structural damage of the highway bridge using the viscous damper after the large earthquake is described. Generally, in order to evaluate the effect of viscous dampers, iterative calculation using total system model of a bridge is needed. So, properties of the viscous dampers were estimated using single degree of freedom system. Viscous dampers were adapted to the highway bridge with plural fixed supports. Dynamic response analysis of the highway bridge was considering the effect of viscous dampers. From the results, feasibility of bridge piers damage control using viscous dampers is confirmed.

Key words: *Damage control design, ductility factor, dynamic response analysis, high way bridge, viscous damper*

1. INTRODUCTION

Destructive effects of Hyogo-ken Nanbu earthquake especially occurred as extreme damages of highway bridges. Through the tough experience of this severe earthquake, the seismic energy dissipation methods are developed not only based on increasing capacity of structural elements but also taking into account the whole structural energy absorption during an earthquake. By limitation of the structural demand, the stability during seismic excitation can be provided. The structure with limited-seismic responses by providing the behavior of main structure stay in elastic range due to even Level 2 type severe earthquakes is generally mentioned as damage-limited structure. The seismic

energy dissipation of the bridge structures may be supplied by additional braces or dampers. The earthquake-resistant of superstructures of bridges have been achieved by base isolation devices so far. However in the last decades, the velocity-dependent viscous dampers¹⁾ are initiated to get wide application area in the retrofitting studies. The plastic deformation of the piers of a damaged bridge due to major earthquakes such as Level2 Hyogo-ken Nanbu earthquake is examined to determine repairing and retrofitting costs. The repairing costs and delay at traffic flow during reconstruction make economic loss in any case. In order to reduce the economical loss, retrofitting by viscous dampers is investigated in terms of financial respect²⁾.

In this paper, the viscous damper which will be installed to the abutments of bridge structure is analyzed by a simplified method, single degree-of-freedom system (SDOF), associated with the modal characteristics of the bridge. Based on the concept of structural utilization continuity, the behavior of piers is desired to be in elastic behavior range. Moreover, the overall economical analysis is conducted to figure out the cost of the retrofiting process with viscous dampers.

2. BRIDGE MODEL AND ANALYSIS CONDITIONS

2.1 Overview of the Bridge

The bridge investigated in this paper is a five-span girder bridge with the length of 199 m as seen in **Figure 1**. The ground type is Zone A- Type II according to the Japan Highway Bridge Specifications. The boundary conditions are movable and fixed at the abutments and piers, respectively. Piers are prestressed concrete (PC). Additional internal forces and deflections of superstructure (creep, dry shrinkage, prestress) are neglected. Ductility ratio of piers are taken as $\mu=3.0\sim3.5$. In **Figure 2** shows the cross-sections of piers, **Table 1** shows bending moment and rotation angle of pier foundations.

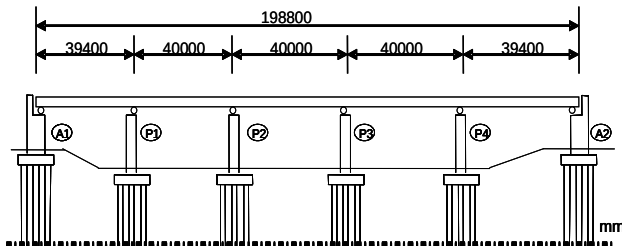


Figure 1. Five-span Girder Bridge

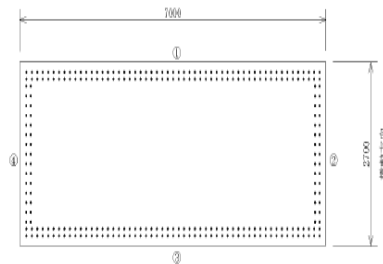


Figure 2. Cross-section of pier

Table 1 Bending moment and rotational angle of pier foundation

	Crack	Yield	Ultimate	
			Type I	Type II
Bending Moment	25839	99608	122762	122752
Rotational Angle	93.4	1184	19425	37580

2.2 Bridge Model

The finite element model of the bridge is generated as seen in **Figure 3**. The piers have fixed support whereas the connection between deck slab and piers are pin connections. The horizontal and vertical elements are rigidly attached to the main elements. For numerical analysis the piers and abutments are modeled as nonlinear two-dimensional beam element. Pier foundations are represented as spring elements. Then the beam model is Takeda beam model. Damping coefficient are 3%, 2%, 5% and 10% of girder, piers, abutments and foundations respectively. In the dynamic response analysis, structural damping matrix is considered by Rayleigh damping. According to the eigenvalue analysis the first mode of natural frequency is 1.702 Hz and strain energy proportional damping is 8.8% while α and β are 1.8474774, 0.00030 respectively. Numerical solutions are put into practice by Newmark β method with the time interval of 0.002 seconds. **Table 2** shows the earthquake waves which are used in this research.

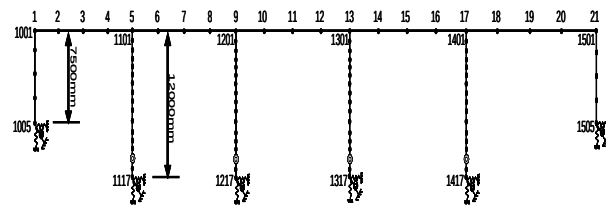


Figure 3. Finite element model of the bridge

Table 2 Earthquake waves

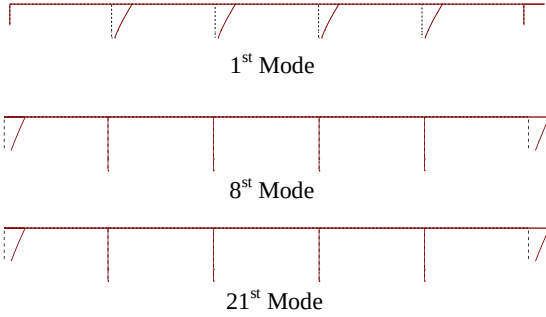
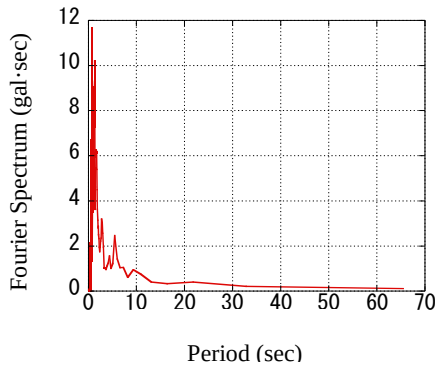
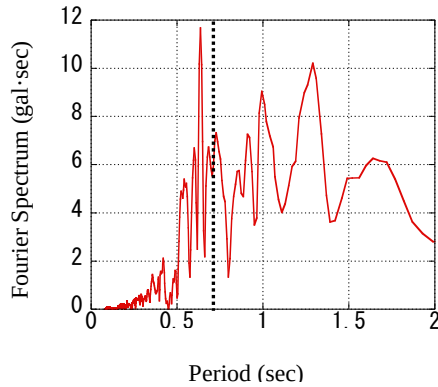
Earthquake Type	Ground Type	No.	Earthquake	Direction
Type II	II	II-II-1	1995 Hyogo-ken Nanbu	NS
		II-II-2	1995 Hyogo-ken Nanbu	EW
		II-II-3	1995 Hyogo-ken Nanbu	N27W

2.3 Eigenvalue Analysis

The modal characteristics (**Table 3**) and mode shapes (**Figure 4**) are obtained by subspace method. In the first 32 modes the effective mass ratio reaches 100%. According to the Table 4 the fundamental mode is 1st mode with the frequency of 1.701, the period of 0.588 and effective mass ratio of 70%. **Figure 5** and **Figure 6** show the Fourier spectrum of the dynamic responses due to Type II-II-3 earthquake. From the figure the fundamental period is detected as 0.636sec. The difference between eigenvalue and Fourier spectrum analysis arises from the plastic deformations occurred in the piers due to seismic waves. Therefore the period is longer in Fourier analysis.

Table 3 Modal characteristics

Mode	Frequency (Hz)	Period (sec)	Effective Mass Ratio (%)
1	1.701	0.588	70
8	6.340	0.157	14
21	13.148	0.076	13
24	14.992	0.067	1
32	22.998	0.043	2

**Figure 4.** Predominant vibration modes**Figure 5.** Fourier Spectrum of the velocity response**Figure 6.** Fourier Spectrum of the velocity response

2.4 Retrofitting Cases

Retrofitting methods include various ways such as changing boundary conditions, reinforcing of piers etc. However in this research viscous damper is investigated since their growing application area in practical site. Herein the dampers will be attached to the abutments of A1 and A2. Target displacements should be determined firstly to obtain damping force capacity. Four cases are studied for the target displacements of 0.10 m, 0.075 m, and 0.05 m as

seen in **Table 4** in order to make effective selection of dampers by comparing to the results for each case.

Table 4 Cases studies

Case 1	Undamped structure
Case 2	Damped structure with the target displacement of 0.10 m
Case 3	Damped structure with the target displacement of 0.075 m
Case 4	Damped structure with the target displacement of 0.05 m

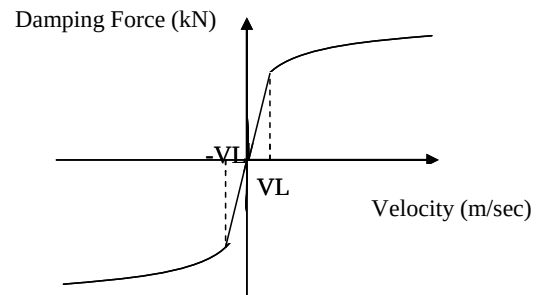
3. ESTABLISHMENT OF DAMPER PROPERTIES USING NONLINEAR DYNAMIC RESPONSE ANALYSIS

3.1 Characteristics of Viscous damper

Figure 7 shows the relationship between damping force and velocity. The nonlinearity and linearity are represented by the Equation (1) and (2), respectively. F and C indicate the damping force and damping coefficient of nonlinear viscous damper device. $C1$ is the damping coefficient value when V , the relative velocity between damper ends, is 1.0 m/sec. $C3$ is the slope of linear line. α takes real positive exponent ranging between 0.1 and 1 characterizing the nonlinearity. In this paper it is taken into account as 0.1 for nonlinear viscous damper and for the sake of convergence from $-VL$ to VL is considered as linear.

$$F = C1 \times V^\alpha \quad (V < -VL, VL < V) \quad (1)$$

$$F = C2 \times V \quad (-VL < V < VL) \quad (2)$$

**Figure 7.** Damping force-velocity relationship

3.2 Analysis Condition

The simplified representation of the abutments with dampers as SDOF system is depicted in **Figure 8**. M is the mass of the bridge, $C1$ is damping coefficient of supplemental damper, $C2$ and $K1$ is structural damping and stiffness of the bridge. $C2$ is generally assumed as 4% and $K1$ is established according to the fundamental period (0.588 sec of 1st mode). The

general formulation for viscous dampers is $F = C1 \times V^\alpha$, whereas the α is 0.1 for nonlinear damper. The dynamic time response analysis is performed to find out the responses of the bridge under earthquake waves. The damper allows the bridge to move in the range of target displacements to resist the structure from damages. Herein this allowable displacement value is called as target displacement. Generally target displacement value is equal to the maximum displacement response which might be induced by earthquake and exceeding this value is considered to cause big damages on the structure. In this research the aim is to make the piers stay in elastic range during a severe earthquake such as Level 2 type. Thus target displacement is decided as small values. Each case for different target displacements is conducted carefully so that no plastic deformation occurs. **Figure 9** depicts the flow chart for determination of damper characteristics.

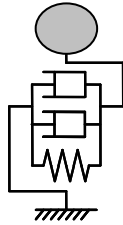


Figure 8. SDOF system

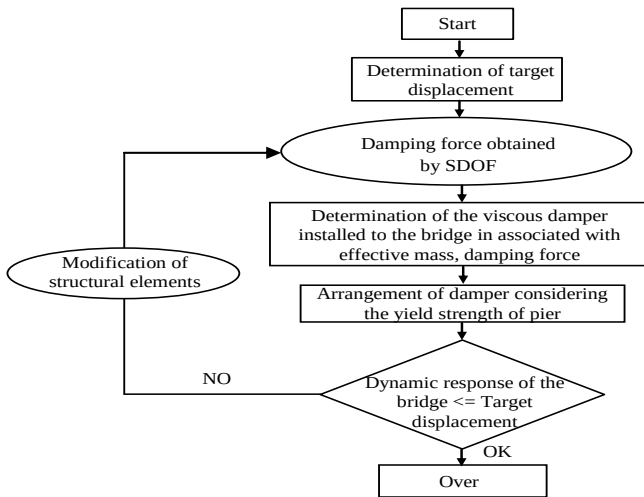


Figure 9. Flowchart of retrofitting steps

3.3 Analysis Method

Dynamic analysis is conducted by the numerical integration method of Newmark β method ($\beta=0.25$) ($\Delta t=0.002$ sec). **Figure 10** shows the change of damping force, F in accordance with the gradual increase of damping coefficient, C . The time point where the error between determined target displacement and calculated displacement response stay in 2% shows us the damping force for the target

displacement. The damping force is normalized by the weight of the mass point of SDOF system (yield seismic coefficient) so that the damper can be applicable for other structures which have different weights.

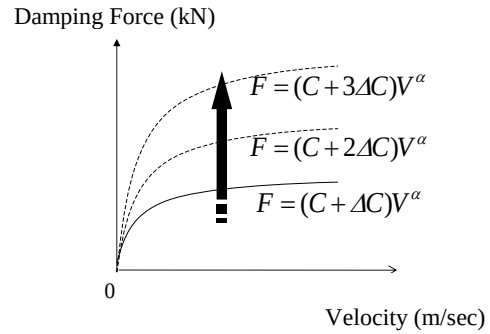


Figure 10. The relationship between damping force and velocity

3.4 Analysis Results

The damping forces and yield seismic coefficients are presented in **Table 5** for each earthquake types. The damper capacity for whole bridge can be determined based on the results obtained from SDOF system for each target displacement for each earthquake wave. The equation for conversion is given in Equation (3). $C1$ is viscous damping coefficient, R is damping force, Y is effective mass ratio of fundamental mode, W is the weight of bridge, N is the amount of damper.

$$C1 = R \times Y \times W \div N \quad (3)$$

Table 5 Viscous damping coefficients

Earthquake Wave	Target Displacement	0.10	0.075	0.05
Type II-II-1	Yield seismic coefficient	0.126	0.180	0.345
	Viscous damping coefficient (kNs/m)	4638	6626	12699
Type II-II-2	Yield seismic coefficient	0.135	0.200	0.215
	Viscous damping coefficient (kNs/m)	4969	7362	12920
Type II-II-3	Yield seismic coefficient	0.094	0.194	0.501
	Viscous damping coefficient (kNs/m)	3460	7141	18441

4. COSIDERATION OF THE RESULTS

The dynamic analysis is conducted for the structure with and without dampers to evaluate the effectiveness of dampers. Comparisons are done between the time responses of the pier cap and pier

foundation.

4.1 The Response of Pier Cap

The dynamic responses of the pier cap (P1) are shown in **Figure 11, 12**. In **Figure 11**, it can be seen that Case 1 gives maximum response in the negative direction due to Type II-II-2. This situation can be explained by earthquake characteristics. Also Case 2, 3, 4 have negative maximum displacement values under the Type II-II-1 and Type II-II-3. In overall look to the figures, one can detect the effectiveness of damper in terms of reducing dynamic responses. The error percentage between target displacement and calculated responses is indicated in **Table 6, 7, 8**. The responses satisfied the target displacement of 0.10 m and 0.075 m. However for the target displacement of 0.05 m, the responses exceed the allowable displacement value with the large percentage errors likewise all responses induced by Type II-II-3. This can be explained based on the selection of target displacement value very small. In general, 0.30 m response may be allowed to the highway bridges. But the bridge investigated in this paper has 0.10 m distance between the deck slab and abutments. Therefore the damping force was obtained very big related to the small target displacements.

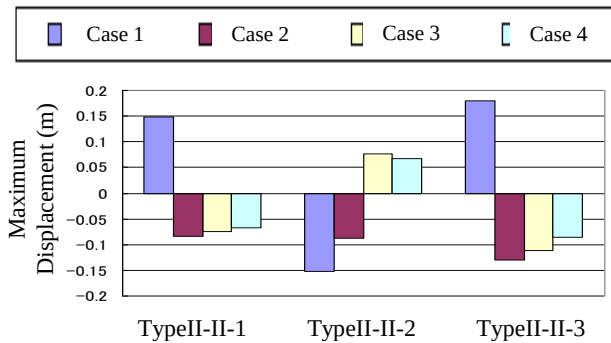


Figure 11. Displacement of Pier cap (positive/negative value precision)

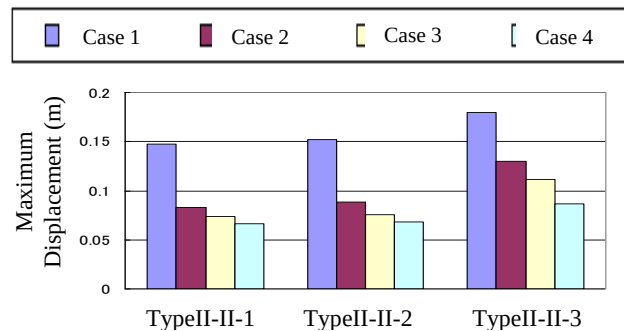


Figure 12. Displacement of Pier cap (absolute values)

Table 6 Comparison of responses and target displacement (0.10m)

Earthquake Wave	Response	Error (%)
Type II-II-1	0.083	20.5
Type II-II-2	0.088	13.6
Type II-II-3	0.13	23.1

Table 7 Comparison of responses and target displacement (0.075m)

Earthquake Wave	Response	Error (%)
Type II-II-1	0.074	1.4
Type II-II-2	0.076	1.4
Type II-II-3	0.112	33

Table 8 Comparison of responses and target displacement (0.05m)

Earthquake Wave	Response	Error (%)
Type II-II-1	0.066	24.2
Type II-II-2	0.068	26.5
Type II-II-3	0.087	42.5

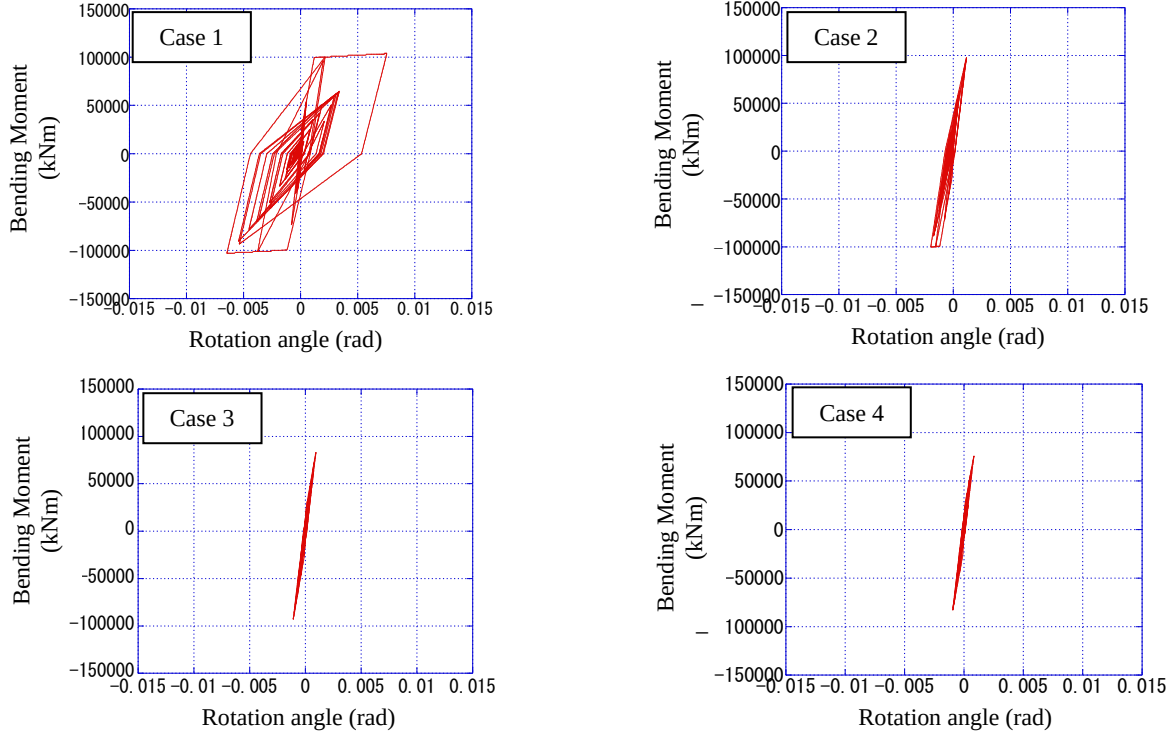
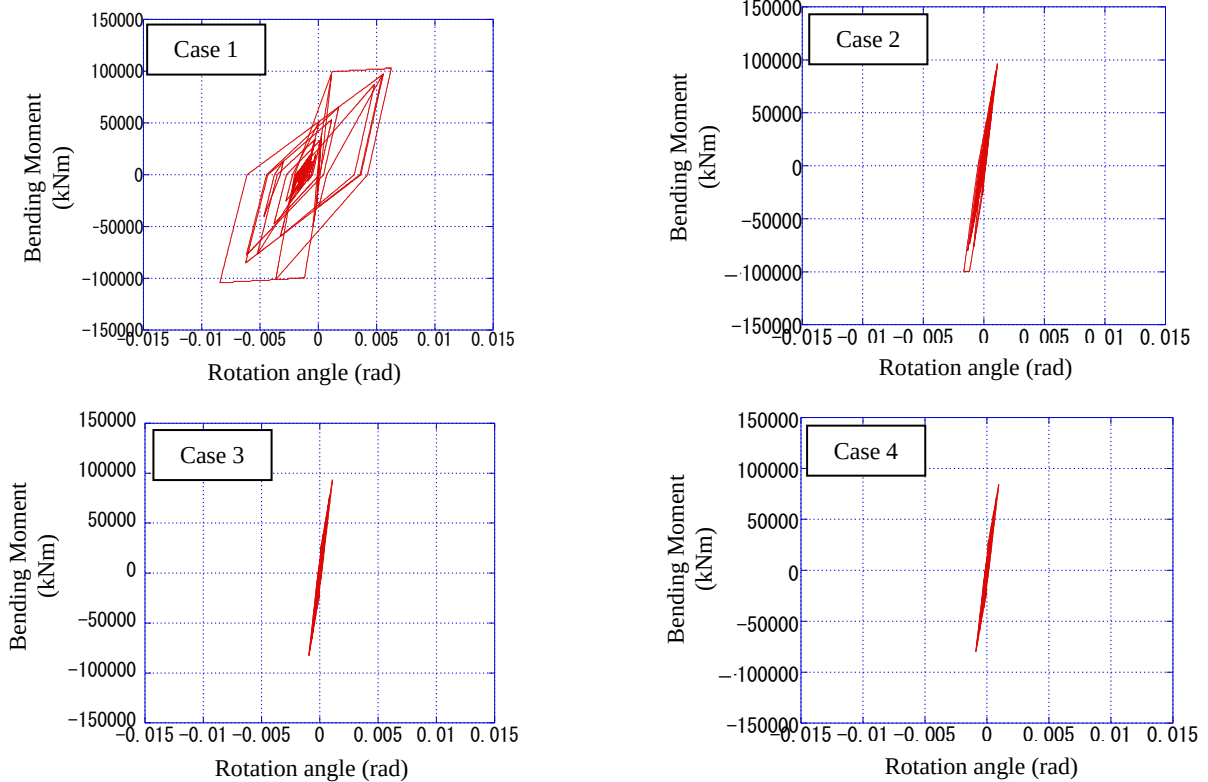
4.2 Relation between Bending Moment and Rotation Angle of Pier Foundation

The bending moments and rotation angles are depicted by **Figures 13, 14, 15**. From the figures, in Case 1 the plastic deformations at the pier foundations and large energy absorption in addition to the big rotation angles can be observed. During Type II-II-1 and Type II-II-3 earthquakes, maximum bending moment and rotation angle at P1 pier occurs in the positive side in contrast with the Type II-II-2.

Table 9 gives the maximum bending moments and rotational angles for each case and earthquake wave. **Figure 13** shows the maximum bending moments of P1 pier due to Type II-II-1 are 103655 kN, 99660 kN, 92844 kN and 82488 kN for the Cases 1-2-3-4, respectively. Errors are found according to the Case 1 results as 3.9%, 10.4%, 20.4% for the target displacement of 0.10m, 0.075m and 0.05 m, respectively. For Type II-II-1, the pier foundation shows elastic behavior, likewise Type II-II-2 presented in **Figure 15**. **Figure 15** shows the maximum bending moments of P1 pier due to Type II-II-3. For case 1-2-3, the pier foundation shows plastic behavior, however for Case 4 pier foundation has elastic behavior.

Table 9 Maximum bending moments and rotational angles

Earthquake Waves	Type II-II-1		Type II-II-2		Type II-II-3	
Cases	Max. Bending Moment (kN)	Rotation Angle (rad)	Max. Bending Moment (kN)	Rotation Angle (rad)	Max. Bending Moment (kN)	Rotation Angle (rad)
Case 1	103655	0.00755402	-104221	-0.00844493	105802	0.0109311
Case 2	-99660	-0.00127302	-99924	-0.00168731	-102877	-0.0063312
Case 3	-92844	-0.00108406	93173	0.00108892	-101623	-0.00435935

**Figure 13.** Bending moment-rotation angle relationship at pier foundation (P1) (Type II-II-1)**Figure 14.** Bending moment-rotation angle relationship at pier foundation (P1) (Type II-II-2)

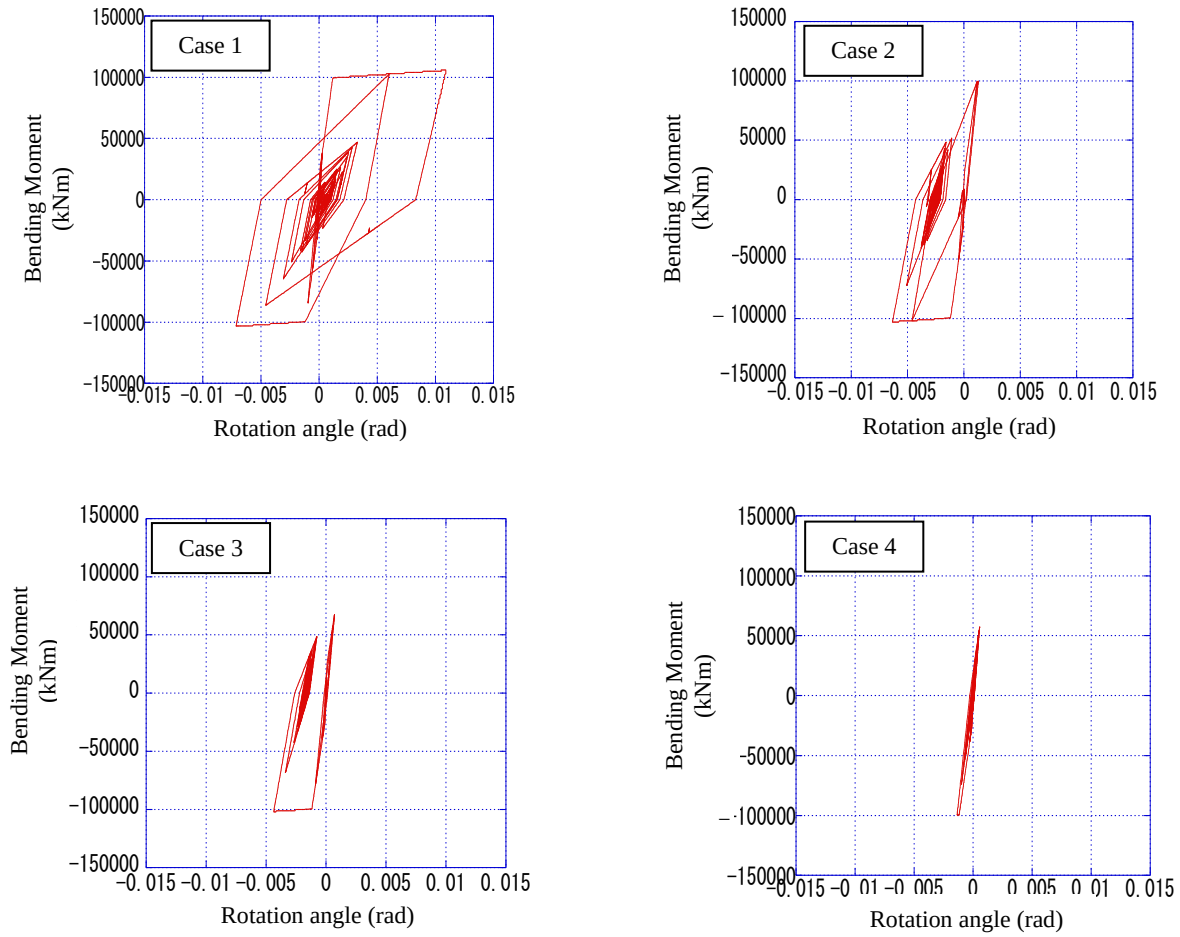


Figure 15. Bending moment-rotation angle relationship at pier foundation (P1) (Type II-II-2)

5. CONCLUSION

In this paper the seismic upgrading of a highway bridge structure is investigated. Firstly the eigenvalue analysis was conducted to find out the modal characteristics of structure. To figure out the efficiency of viscous dampers the characteristics obtained by single degree-of-freedom system was applied to the bridge structure. The dynamic analysis is processed for severe earthquake waves of Level 2. It is aimed that bridge responses stay in elastic range. The dampers were installed at the abutments. The responses were measured at the piers. Finally, economic feature of damper was investigated in brief.

- 1) A simplified method associated with the effective mass of fundamental mode and probable maximum displacement response of the bridge structure is proposed and verified so that repetitive steps of retrofitting process are avoided.
- 2) The bridge with viscous dampers show elastic behavior according to the dynamic responses at the piers for Cases 2, 3, 4 due to Type II-II-1

and Type II-II-2, likewise for Case 4 due to Type II-II-3.

- 3) The displacement responses of the bridge satisfied the target displacements of 0.10 m and 0.075 m. However for the target displacement of 0.05 m, the responses of the bridge exceeded this allowable value as explained before.

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