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SEISMIC PERFORMANCE EVALUATION OF RETROFITTED BRIDGE BY ISOLATION BEARINGS

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Abstract. The seismic performance evaluation and retrofit process are very important in existing bridges. If the result is not appropriate, then retrofit process are required. Among various retrofit methods, the seismic isolation is a very useful method, because it can be applied by replacing old bridge bearings. In this study, the effectiveness of seismic isolation is rationally verified. For this purpose, two seismic isolations used widely are selected and non-linear static and non-linear dynamic analyses are performed. The responses of existing bridges are compared with those of retrofitted bridge by seismic isolation bridge for earthquake of target level, and seismic performances are evaluated.

Keywords: seismic performance evaluation, retrofit, existing bridges, seismic isolations, non-linear analysis, target level.

1. Introduction

Most of existing bridges already display developed agedness and their seismic performance in case of potential earthquakes are being questioned. Bridges that do not satisfy standard seismic performance level need installations of effective seismic retrofit, and the most popular methods would include retrofit bridge bearings, piers and abutments, foundations and underlying soil, and using seismic isolation bearings (Juozapaitis *et al.* 2008; Zavadskas *et al.* 2008; Dulinskas *et al.* 2008; Sivilevičius *et al.* 2008; Kashevskaya 2007; Masanobu *et al.* 2003; Kamiński, Trapko 2006).

Particularly, the seismic isolation bearings are recommended not only for the seismic design of newly constructed bridges, but also for the seismic retrofits of existing bridges, and are increasingly being applied to constructions. The method of using the seismic isolation bearings can improve the seismic performance without retrofit existing piers or foundations by reducing the inertia force generated in case of earthquakes (Han *et al.* 2008; Park, Han 2004). In particular, it minimizes extra construction expenses because it utilizes the seismic isolation bearings to replace the aged bridge bearings. As the necessity of seismic isolation bearings are being recognized and its construction cases are being increased, the process of accurately inspecting its capacity of seismic performance improvement is required. The inspection process of the performance of seismic isolation bearings is important as it tests whether it actually accomplishes the aimed level of seismic performance of the seismically reinforced bridges. Recently, several valuable research studies have been conducted to evaluate the performance of various isolated structural system (Bakir *et al.* 2007; Chehab, El Naggar 2003; Komodromos *et al.* 2007; Nagarajaiah, Narasimhan 2007; Grigorjeva *et al.* 2008; Kala 2008). However, these research studies are broad and do not particularly address the practical issues concerned with the seismic performance of seismic isolated bridges.

This study inspects the improvement of the bridges seismic performance in case when the seismic isolation bearing is applied, through an analysis method. For this purpose, two types of most popularly applied seismic isolation bearings were selected and actually applied to a non-seismically constructed existing bridge. The static and dynamic non-linear analysis had been implemented to measure the responses of the bridge's seismic performance before and after the retrofit, and the properties were compared for the analysis. Also, as the result of non-linear analysis, the capacity spectrum is organized and the performance points of the bridges estimated displacement is measured. Non-linear time-history analysis is implemented to compare the responses of the performance points to inspect the accuracy of the capacity spectrum method, which is an illustrational seismic performance evaluation method for bridges reinforced with seismic isolation bearings.

2. The application of seismic isolation and the analysis model

The seismic design for existing bridges increased the resistance capacity toward transverse-directional loads and was designed to absorb the seismic energy with the inelastic behaviors of the structural members. However, this method had the potential to develop gradual failure due to the inelastic deformation of the structural members, to display considerable horizontal displace- ment, and to cause fragility failure and amplification of response due to overloading and putting excessive expectation of resistance on the bridge joints. Currently, by using the seismic isolation bearings, the horizontal seismic force that is transferred from the superstructure to the substructure in case of earthquakes is reduced to achieve not only safety but also economical efficiency. The seismic isolation bearings can replace the aged bearings of existing bridges that the installation process is very simplified.

The major purpose of the seismic isolation bearings is to reduce the destructive energy of earthquakes that is transferred to the structures. Many different types of methods were proposed to achieve this purpose. For the analysis of this study, two most popularly applied seismic isolation bearings were selected.

2.1. Rubber bearing

Rubber bearing (RB) is mostly made out of laminated rubber with more than one reinforcing steel sheets inside. It is the most broadly studied and applied seismic isolation bearing, because it represses the expansion situation of the rubber in case of compressive deformation in order to improve the load resistance capacity. For the basic structure, rubber is used as the major material for flexible bending rigidity and the steel sheets are inserted to reinforce the horizontal stiffness. In the case of applying RB to actual bridges, the max range of displacement should be designed to be less than the displacement capacity of RB. The formation of RB, the analysis model and the theoretical hysteresis curve are as shown in Fig. 1. As indicated in the figure, RB is idealized to perform full elasticity behavior.

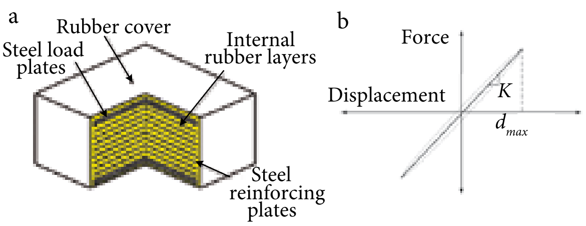

Fig. 1. Formation (a) and analysis model and theoretical hysteresis curve (b) of the RB

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2.2. Lead lubber bearing

Lead rubber bearing (LRB) is designed to improve the weakness of RB. RB has low damping and displays huge deformation for static loads. Therefore, LRB inserts a lead plug in RB, as shown in Fig. 2, to provide damping to response to earthquakes and to resist static loads. Because the lead plug inserted into the LRB is characterized by almost full elasto-plastic hysteresis loop, the rigidity of the bearing after the yielding of the lead is equal to that of RB, and the capacity of LRB can be determined by vertical load, random horizontal reaction, and scale of construction expansion. Generally, in the case of designing a LRB, the diameter of the lead plug that is determined by the coefficient of horizontal reaction and should be ideally small. The formation of LRB, the analysis model and the theoretical hysteresis curve are as shown in Fig. 2.

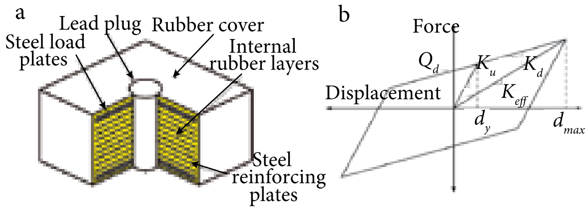

Fig. 2. Formation (a) and analysis model and theoretical hysteresis curve (b) of the LRB

3. The evaluation of seismic performance

3.1. Evaluation of seismic performance according to the capacity spectrum method

By comparing the seismic performances of an existing non-seismically bridge and a seismically reinforced bridge with seismic isolation bearings, the improvement achieved by the seismic isolation bearings can be inspected. To evaluate the seismic performance of the bridges, various aspects should be considered. Whether or not it guarantees standard seismic performance level to resist the earthquakes with max seismic force should be evaluated reasonably. For the bridges, installed with seismic isolation bearings, the force response of the pier generally decreases, but the relative displacement of the superstructure and substructure increases. All piers of the bridges with seismic isolation bearings resist the seismic load and redistribute the load that it is different than the existing bridges that resist the seismic load with one fixed pier. It is not reasonable to evaluate the performance of the bridge system with the response of only one pier. Therefore, the bridge should be considered as a separate vibrant unit from the contract point prior to evaluating each vibrant system. The capacity spectrum of a structure is a method of directly comparing the capacity and seismic demand of each vibrant system, and the capacity is measured by non-linear static analysis. The capacity of earthquake endurance can be organized by converting the response spectrum to fit the aimed performance level. The capacity spectrum method, in particular, is one of the most useful methods in evaluating the performance of existing concrete structures and in designing the reinforcement, and is widely used to evaluate the seismic performance of structures in performance-based seismic design.

3.2. Capacity and demand estimation

In order to estimate the capacity while considering the non-linear response of the bridges, pushover analysis is implemented. In case that the stress on one part of an indeterminate structure reaches the yielding point, the nodal point yields; and the load is increased, the stress remains at the yielding point. But another nodal point that had not reached the yielding point stays within the range of elasticity. The stress redistribution allows this system structure to possess the redundant force, so that it can resist the load until it reaches unstable state. Also the relationship of load and displacement of a structure as the load increases can be estimated. In the case of implementing the pushover analysis, an increase of load is continuously applied until the structure reaches unstableness due to gradual yielding. For the pushover analysis, the material behavior is assumed to be bilinear. In this study, the pushover analysis is implemented by using the bilinear material model. The load-displacement relation-ships estimated by the pushover analysis causes base shear and top displacement of the structure.

The capacity demand for earthquakes becomes the design response spectrum to fit the aimed level of capacity. The design response spectrum is defined. The values of the seismic coefficients, C_A and C_V, are determined by the inspected region and its geological conditions. The situation model of the material and the design response spectrum are shown in Fig. 3 (Han *et al.* 2004).

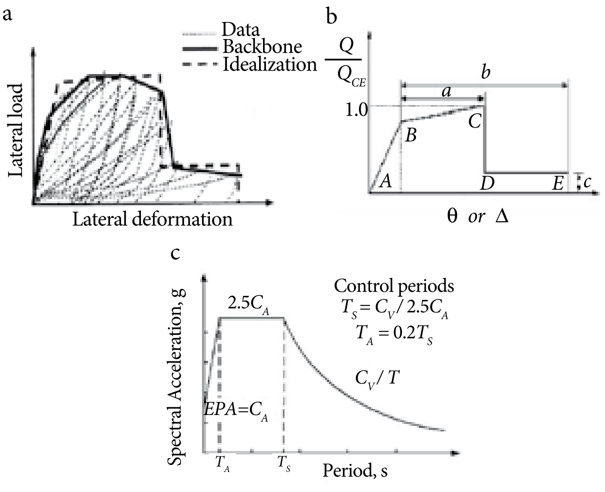

Fig. 3. Plastic behavior (a), bilinear material model (b) and linear response spectrum (c) of the reinforcement concrete

The design response spectrum designed to estimate the capacity demand toward earthquakes should consider the non-linear responses of the structures and be expressed as non-linear design response spectrum. Generally, the non-linear spectrum can be estimated by being reduced using the reduction coefficient while considering the ductility toward linear spectrum. The method of organizing the non-linear spectrum using equivalent damping of a structure instead of ductility is used. However, the influence of damping in the non-linear spectrum organization is almost non-effective in case that the natural period of the structure is exceptionally long or short. In addition, as the non-linear displacement increases after yielding, that is as the ductility of a structure increases, the effectiveness of the damping reduces significantly. In the case of converting to capacity spectrum, iteration process is required to calculate the performance point representing the max linear capacity of a structure. In this case, sometimes the conversion cannot be converged. Therefore, the ductility of a structure should be considered in order to organize the actual non-linear spectrum (Peter 1999).

3.3. Organization of capacity spectrum

After estimating the capacity of a structure through the pushover analysis and the capacity demand through the non-linear design response spectrum, the capacity and capacity demand are converted into same format and illustrated on a graph to organize the capacity spectrum. The capacity of a structure is converted into the capacity curve and the capacity demand into the demand curve (Chopra, Goel 1999). The conversion formula is as follows.

(1) conversion formula into capacity curve:

$$S_a = \frac{\left(\frac{V}{W}\right)}{\alpha_1}, \tag{1}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \times \Phi_{1,roof}}, \tag{2}$$

where S_a – spectral acceleration; S_d – spectral displacement; Δ_{roof} – top displacement, V – bottom shear force; W – total load. $\Phi_{1,roof}$ – first mode shape at top roof. α_1 and PF_1 represent the modal mass coefficient for the first natural mode and the modal participation factor of the mode 1, in respective.

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} m_i \varphi_{i1}\right]^2}{\sum_{i=1}^{N} m_i \sum_{i=1}^{N} m_i \varphi_{i1}^2}, \tag{3}$$

$$PF_1 = \frac{\sum_{i=1}^{N} m_i \varphi_{i1}}{\sum_{i=1}^{N} m_i \varphi_{i1}^2}, \tag{4}$$

where m_i – mass at level i; φ_{i1} – ordinate of mode shape i at level 1.

(2) conversion formula into demand curve:

$$S_d = \frac{1}{4\pi^2} S_a T^2, \tag{5}$$

where T – period.

4. Example of improvement in seismic performance of bridge

4.1. Selection of a bridge for analysis

The bridge selected for the analysis is a PSC-Beam bridge that had been designed using POT bearings with longitudinal length of 150 m and transverse length of 11.5 m, and 5 girders with girder spacing each 2 m. The wall-type piers with rectangular sections are made out of concrete-filled reinforcement concrete and reinforced by horizontal steel bars and cross-ties. The steel tendons in the piers miss 2-ply joints, the bridge is assumed to be positioned on a stable soil. The evaluation is implemented for the case of an earthquake that is included in 1000-years period of collapse prevention standard. The responses before and after the seismic retrofit are compared. The characteristics of selected bridge are illustrated as in Fig. 4.

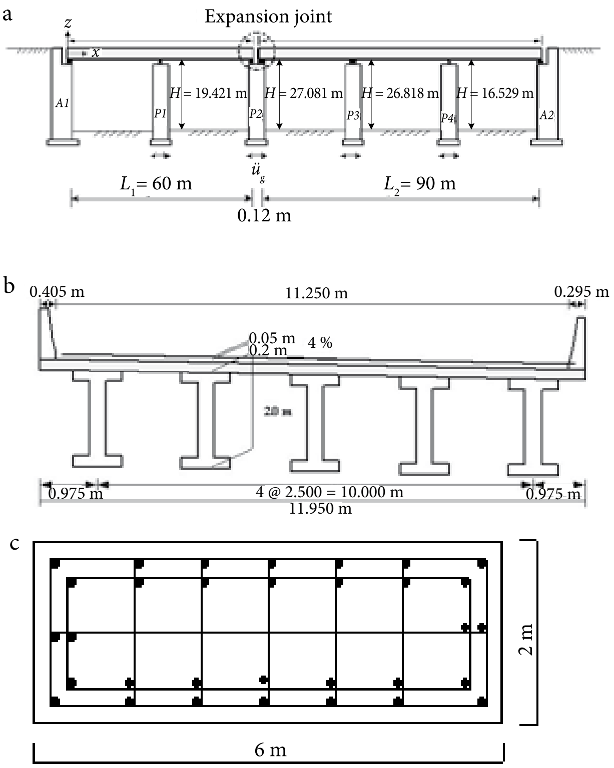

Fig. 4. Longitudinal section (a), cross-sectional view (b) and steel reinforcements of piers (c) of sectional view of the selected bridge

4.2. Finite element model of the selected bridge

The finite element model (FEM) for the analysis of the selected bridge is as shown in Fig. 5. The superstructure and substructure used frame elements, and rigid link elements were used to connect the superstructure with the top portion of the seismic isolation bearing. The materials and sectional constants of the selected bridge are shown in Tables 1 and 2.

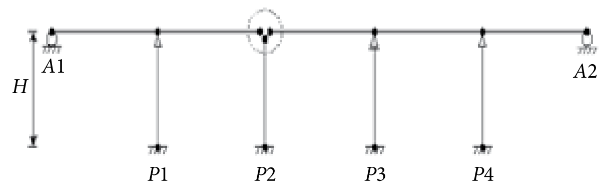

Fig. 5. Finite element model of the selected bridge

Table 1. Materials used in the selected bridge

Concrete	Girder	Bottom panel	Piers	Pavement
Unit weight, kN/m^3	25	25	25	23
Compressive strength, MPa	40	270	270	-
Elastic modulus, MPa	28 000	24 648	24 648	-
Steel bar	Yielding stress (f_y= 400 MPa)			
PS strand (Φ 15.2 mm)	Sectional area (A = 1 387 cm^2) Tensile strength (f_{pu} = 1 900 MPa)			

Table 2. Section properties of the selected bridge

Girder	Sectional area (A = 6.24 m^2)
	Moment of inertia (I_3 = 4.5 m^4)
Piers	Sectional area (A = 12 m^2)
	Moment of inertia (I_3 = 4 m^4)

4.3. Selection of seismic isolation bearings and modeling

For the selection of seismic isolation bearings, the superstructure is modeled separately to estimate the design reaction and expansion and to select adequate seismic isolation bearings. The properties of the selected seismic isolation bearings are shown in Tables 3 and 4. The bearings are modeled using non-linear link elements. RB is modeled to move linearly with no influence of damping and the LRB is idealized using non-linear model to fit the properties of the selected bearing.

Table 3. Properties of RB

	Design load, kN	W×L, mm	Thickness, mm	Height, mm	Expansion, mm	K_v	K_h
Outer	1750	300×500	72	105	50	62461	191.3
Inner	3000	450×600	60	89	42	278000	413.0

Table 4. Properties of LRB

Location	Design load, kN	Static displacement, mm	Seismic displacement, mm	Height, mm	Characteristic properties				
					D_y, cm	Q_d, kN	K_u, kN/cm	K_d, kN/cm	K_v, kN/cm
Out	2000	110	250	276	0.8	3.3	5.7	1.5	242.2
In	3000	80	250	271	0.8	4.3	7.3	2.0	318.9

4.4. Determination of aimed performance standard

For the selected bridge, the aimed standard is determined to be the earthquakes of 1000-years period of collapse prevention standard. For the seismic load, El Centro (NS, 1940) earthquake with its effective peak acceleration altered to 0.224 g is used to fit the 1000-years recurrent interval. This is determined based on the coefficient of soil, C_A = 0.224 and C_V = 0.322. Fig. 6 shows the shape of El Centro earthquake.

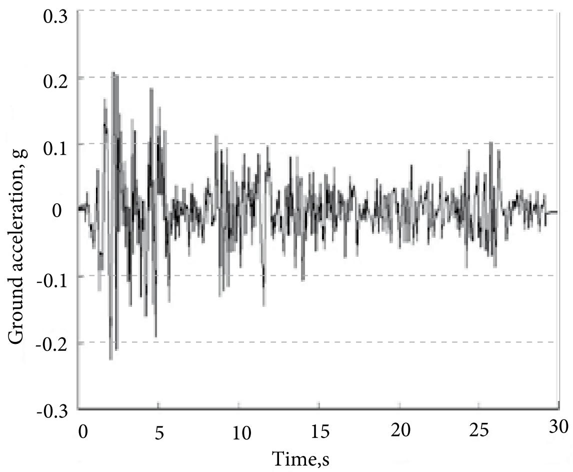

Fig. 6. El Centro earthquake (NS, 1940, *EPA* = 0.224 g)

4.5. Changes in shearing force and moment

The max section forces are compared for an existing non-seismic bridge and a bridge with seismic isolation bearings installed according to the response spectrum analysis and non-linear time-history analysis. Fig. 7 shows the max shearing force and moment for each pier as studied by the response spectrum analysis of 3-span part. In case of ordinary POT bearings, the section force due to seismic load is shown to be concentrated on the hinged pier (P4). For the seismic retrofit piers with RB and LRB, the seismic load is distributed to all piers to resist it equally. The reason why the response at the hinged pier (P4) is lower than that at other piers is understood to be because the height of P4 is relatively lower.

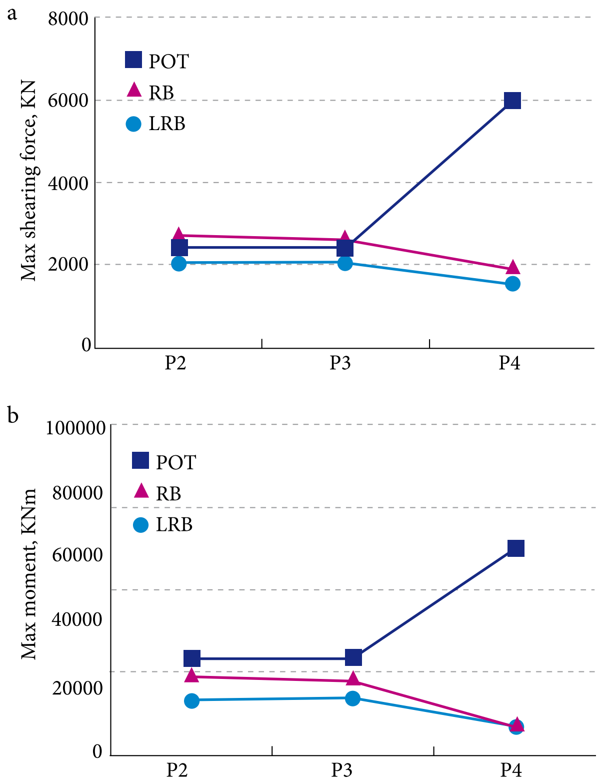

Fig. 7. Max shearing force (a) and max moment (b) in comparison of sectional forces at each pier (response spectrum analysis)

Fig. 8 shows the max shearing force and moment of each pier according to the non-linear time-history analysis of 3-span part. Similar to the result of response spectrum analysis, the max shearing force and moment of seismically reinforced bearings with seismic isolation bearings are estimated to be higher than those of ordinary POT bearings.

4.6. Seismic performance evaluation

The capacity spectrums were configured for the selected bridge before the seismic retrofit and after the seismic retrofit with RB and LRB. The capacity of the selected bridge was estimated by implementing the pushover analysis and the capacity demand by non-linear spectrum. Methods, such as Peter (1999), were used to set up the non-linear spectrum.

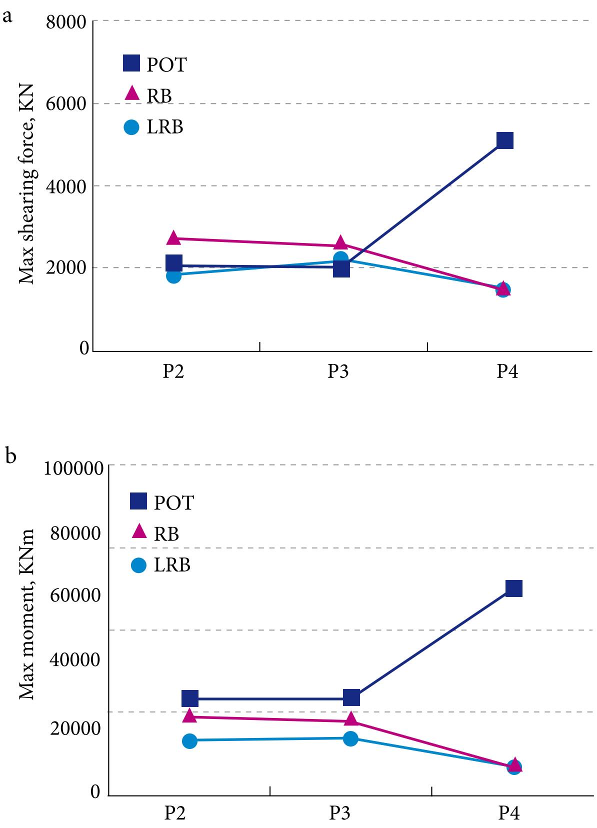

Fig. 8. Max shearing force (a) and max moment (b) in comparison of section force of each pier (non-linear time history analysis)

4.6.1. Configuration of capacity spectrum

The pushover analysis with controlled displacement was implemented for the existing POT bearings, RB and LRB. With the outcome, the relationship between the top displacement and the base shearing force of the bridge is calculated. The distribution shape of load is generally important in the case of implementing the pushover analysis. For the standardized structures such as bridges, the longitudinal mode 1 is most influential and generally the mass of the piers is excluded for the analysis that the simple increase of load was applied to the topmost part of the bridge for the pushover analysis. The material model used for the analysis was idealized by the non-linear material models in consideration of the cost of materials such as steel bars. The capacity demand becomes non-linear design response spectrum using displacement ductility factor. Based on the design response spectrum of elasticity, the seismic coefficients C_A and C_V were modified and used. The coefficients were $C_A = 0.224$ and $C_V = 0.322$. The process of configuring non-linear response spectrum implies altering the factors of horizontal and yielding displacements for each span of aimed displacement ductility factor in order to draw out the satisfying target ductility factor (Masanobu *et al.* 2002). In this case, the non-linear design response spectrum can be estimated by applying the coefficients related to the displacement ductility factor where the acceleration, velocity and displacements should be treated delicately. After configuring the capacity curve using the results of the pushover analysis, the conversion Eqs (1) and (2) are applied to convert the outcomes into the capacity curve. The dynamic characteristic properties of mode 1 of the selected bridge required for the equation can be calculated by analyzing the eigenvalues. The results are shown in Table 5.

Table 5. Dynamic characteristics of mode 1 piers according to the eigenvalue analysis

Type of bearing	Natural vibration period, s	Modal participation factor	Modal mass coefficient
POT	0.897	1.079	0.376
RB	1.189	1.137	0.404
LRB	1.165	1.157	0.553

After converting the non-linear design response spectrum into the demand curve by using the relationship between spectrum displacement and period of Eq (5), the capacity spectrum is configured by illustrating the capacity and demand curves that had been converted into Acceleration Displacement Response Spectrum (ADRS) form, on a same graph. Fig. 9 shows the capacity spectrum of each type of pier. With the existing non-seismic POT bearing, the piers were expected to yield when the estimated displacement exceeds the yielding displacement in case of collapsing prevention standard earthquakes. In case that RB or LRB were applied, the piers performed elastic behavior without yielding. Also, in case of LRB installation, the lead yielded very soon due to its low yielding strength and after its yielding, the rubber deformation occurred, as in RB. Moreover, the bearing spans, expressed as the inclination of the capacity curve in the capacity spectrum, proved that the seismic isolation bearings contributed to the lengthening of the bridge bearing span, when compared to that before the retrofit (non-seismic POT bearings).

To calculate the estimated displacement and performance point of each bearing, the ratio of horizontal and yielding dispositions was altered for each span of target ductility factor in order to find the satisfying target ductility factor.

The top displacements, base shear, effective periods, and other at the performance points are shown in Table 6. Compared to the case of existing POT bearing, RB contributed to an increase of the displacement, the declination of the degree of shearing force, and eventually the lengthening of the life span achieving the design concept of the seismic isolation bearings.

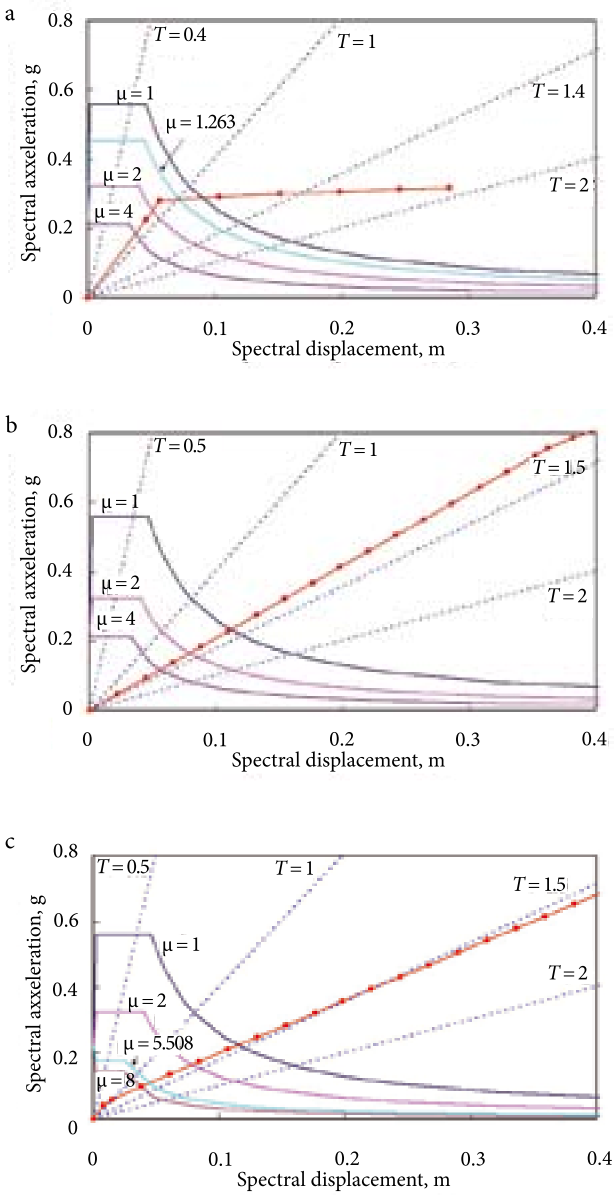

Fig. 9. POT (a), RB (b) and LRB (c) of capacity spectrum of the selected bridge

Table 6. Comparison of performance points of each bearing (collapsing prevention)

	Equivalent SDF model		MDF model		Effective period, s	Demanded ductility
	S_d, m	S_a, g	Top displacement, m	Base shear, kN		
POT	0.072	0.284	0.077	4379.21	1.010	1.263
RB	0.111	0.230	0.126	3810.65	1.394	1
LRB	0.0444	0.110	0.051	2481.10	1.263	5.508

4.6.2. Comparison to the result of non-linear time-history analysis

Fig. 10 shows the result of non-linear time-history analysis using El Centro (NS, 1940) with controlled effective max acceleration of 0.224 g to match the design response spectrum of the capacity spectrum. The result revealed that the displacement response becomes the biggest in case that RB was installed and it is smaller LRB is applied. LRB is understood to have a reduced degree of displacement with the influence of lead insertion.

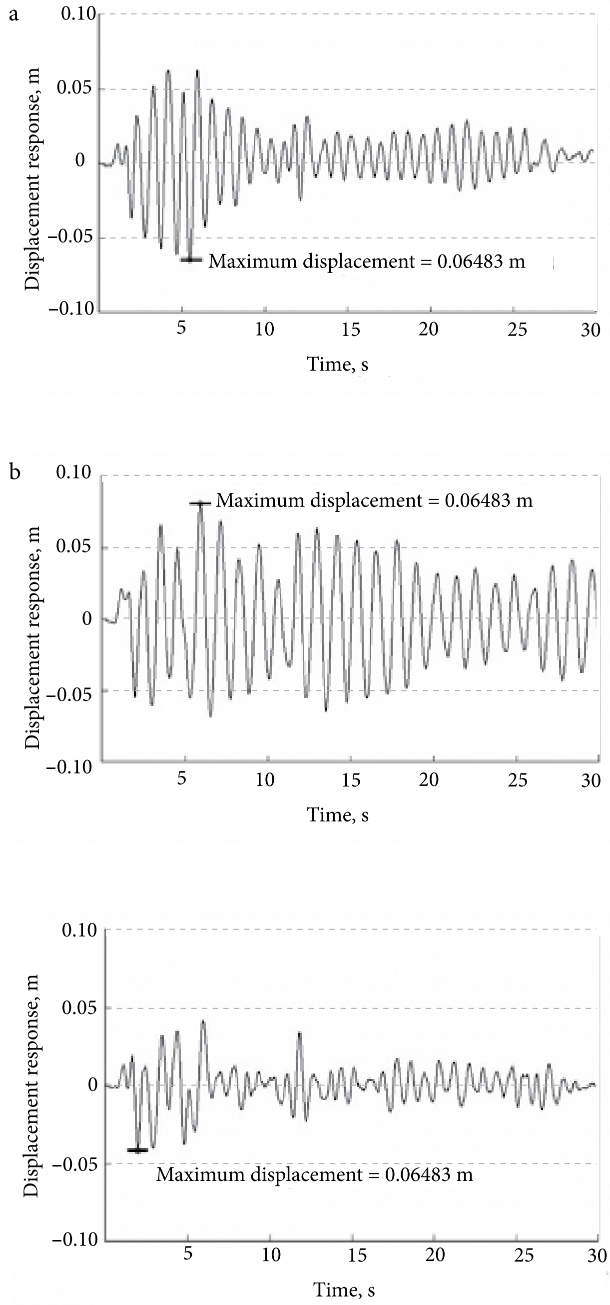

Fig. 10. POT (a), RB (b) and LRB (c) of result of non-linear time-history analysis for each bearing

Table 7. Comparison of displacement responses of capacity spectrum and nonlinear time-history analysis

	Capacity spectrum method		Non-linear time-history analysis
	Equivalent SDF model	MDF model	MDF model
	S_d, m	Topmost displacement, m	Top displacement of hinged pier-P4, m
POT	0.072	0.077	0.0648
RB	0.111	0.126	0.0806
LRB	0.0444	0.051	0.0421

The results of non-linear time-history analysis were compared to the responses at each performance point in the capacity spectrum. The outcomes are shown in Table 7. Although the displacement responses of non-linear time-history analysis and capacity spectrum are similar, the displacement response of capacity spectrum is calculated to be slightly bigger. This should be because the design response spectrum is used for the configuration of capacity spectrum.

5. Conclusions

In this study, the method of seismic retrofit using seismic isolation bearings for the existing non-seismic bridges whose evaluations revealed that they do not satisfy the capacity demand. Using the results of comparing the seismic performances of an existing bridge before and after the seismic retrofit with the seismic isolation bearings, it is identified that the seismic isolation bearings support effective seismic performance improvement.

The implementations of response spectrum analysis and non-linear time-history analysis revealed that the seismic retrofit with the seismic isolation bearings contributed to the significant decrease of section force, such as shearing force and moment, while the degree of displacement on the piers increased in case of RB installation and decreased for LRB. This decrease of displacement for LRB is understood to be because of the influence of the lead insertion. LRB improves the increased displacement of RB and reduces the seismic energy with the plastic deformation of the lead plug; these performances were also proved by the analysis results. In addition, the application of RB and/or LRB performed increased flexibility and lengthened bearing span, compared to the existing POT bearings. Moreover, as the displacement of the performance point, expressing the expected response to the target earthquakes, as shown in the capacity spectrum, is similar to the displacement response of the non-linear time-history analysis, the estimation of the displacement response using the capacity spectrum method is determined to be accurate to a certain degree.

The configuration of capacity spectrum proved that the existing bridges with POT bearings could not satisfy the capacity demand, whereas the seismic retrofit with seismic isolation bearings achieved all the demanded standards. The piers solely perform elastic behavior in case when the seismic load is completely resisted by the seismic isolation bearings. This fact accords to the design concept of bridges installed with seismic isolation bearings. In conclusion, the capacity spectrum illustratively proved that the installation of seismic isolation bearings onto existing non-seismic bridges accomplished the improvement of overall seismic performance of the bridge. It can be understood that the installation of seismic isolation bearings onto the piers without additional reinforcements can achieve the expected level of seismic performance.

It is unreasonable to rely on the seismic performance of a single pier to evaluate the overall performance of a seismically isolated bridge, but the redistribution of loads by the seismic isolation bearings should be considered in order to evaluate the overall performance of the bridge system accurately. Moreover, when evaluating the safety performance of the seismically isolated bridge in case of earthquakes, the shocks on the expansion joints, the failure of piers and abutments, and allowable displacement of the bearings should be considered additionally.

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