The Effectiveness of Restrainers to Enhance the Seismic Performance of Bridges with Rocking Foundations

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Abstract

Rocking is a form of base isolation that can be adopted in bridge foundations to reduce the overall seismic demand on a structure. By allowing the foundation to uplift under severe earthquake loading, inelastic behaviour that would typically occur at the piers is significantly reduced and as a result, a safer structure is achieved. However, reduction in the foundation stiffness can lead to significant deck displacements causing damage to abutments, deck, bearing supports and in severe cases, result in deck unseating. This study seeks to investigate the effectiveness of restraining devices for reducing deck displacements of bridges with rocking foundations. The restraining devices investigated in this study are lead-rubber bearings, fluid viscous dampers, and cable restrainers. The devices are compared in identical bridge configurations and modelled using three dimensional numerical finite element software with consideration of soil-structure interaction and material non-linearity. Non-linear time history analyses are performed using four earthquakes scaled to the appropriate site hazard. The seismic performance of the bridges are compared in terms of the structural actions and displacements experienced by the deck, piers, abutments, and piles. The results show that the restraining devices are effective in reducing the deck displacements, however, come at the cost of transferring larger forces to the abutment, requiring larger pile sections and further design provisions.

Keywords: Rocking foundations, lead rubber bearing (LRB), viscous damper, cable-restrainer, soil-structure interaction, deck displacement, non-linear time history analysis

1. INTRODUCTION:

Conventional bridges are designed to resist severe earthquake shaking through the development of plastic hinging at the pier ends through adequate detailing. This non-linear behaviour dissipates the energy from the earthquake and prevents the structure from collapsing (Mander 1983). However, the damage resulting after a large earthquake often leads to lengthy and costly repairs, traffic disturbances and in severe cases, demolition of the entire bridge (Priestley et al. 1996). As a result, researchers and engineers are actively seeking a more sustainable and cost effective design solution.

Over the past few decades, the motion of rocking has been investigated and studied for its superb performance during earthquake shaking (Housner 1963). Researchers have concluded that rocking is an effective earthquake isolation method that can be used in structures to improve their seismic performance (Dimitrakopoulos and Giouvanidis 2015). This is because rocking increases the natural period of the structure and as a result, generally reduces its seismic demand (Xu and Fatahi 2019). Several researchers have implemented this technique in seismic bridge design using two main approaches, rocking piers and rocking shallow foundations. These rocking techniques have been studied in various analytical, numerical and experimental work. Agalianos, Psychari et al. (2017) numerically compared the seismic performance of rocking piers and rocking shallow foundations on a motorway bridge. Furthermore, Zhou, Han et al. (2019) conducted shake table tests for a bridge with post-tensioned rocking piers whereas Antonellis, Gavras et al. (2015) conducted shake table tests of piers supported on rocking shallow foundations. It was found that rocking piers experience inelastic concrete spalling damage at the base of the piers when large drifts are experienced. In addition, rocking shallow foundations experience excessive settlements and rotations of the soil which is not only unfavourable but also prohibited in current seismic standards due to the difficulty of repair (Xu and Fatahi 2018). Antonellis and Panagiotou (2013) proposed rocking foundations on piles (rocking pile foundations) which solves the aforementioned issues, opting for an elastic performance of the bridge. However, due to the reduced lateral stiffness of the rocking foundation/pier, and the absence of energy dissipation that would otherwise develop due to the inelastic behaviour at the pier, the bridge deck experiences significant deck displacements. These displacements are sufficient to cause pounding damage between the decks and the abutments, damage to bearing supports and in some cases, lead to deck unseating (Hao and Daube 2012). This research aims to investigate the use of energy dissipating and restraining devices, namely lead rubber bearings (LRB), fluid viscous dampers and cable retainers, to reduce the seismic displacements of the deck that would otherwise lead to unfavourable damage. These devices will hereby be referred to as seismic restrainers.

Seismic restrainers are used by engineers as part of the earthquake resisting system of conventional bridges (Hassoun and Fatahi 2019). The seismic restrainers are often connected between the deck and abutment, and function by transferring the inertial forces from the deck to the abutment and soil foundation. Both the LRB and fluid viscous dampers have been proven to provide energy dissipation to bridges and structures (Robinson 1982). However, the fluid viscous damper’s resistance is velocity dependant and out-of-phase with the primary direction of bending, having the added advantage of reduced seismic loading (Pacheco et al. 1993). Moreover, the stiffness of both these devices contributes to the transfer of inertial forces from the deck to the abutment and piles as they resist the deck displacements. The cable restrainers in contrast provide no energy dissipation as they are designed to remain elastic and as a result, transfer the full loading from the deck to the abutments and surrounding soil (DesRoches et al. 2003). Despite its inability to dissipate energy and reduce the inertial actions caused by the deck, the cable restrainers are more effective in controlling the displacements when compared to the energy dissipating devices as discussed later in this paper.

2. BRIDGE DESCRIPTION

The bridges adopted in this study represent a typical highway bridge located at a high seismic region. A total of five bridges are investigated in this study: (1) bridge with a conventional fixed base foundation (FB) designed to develop plastic hinging at its piers, (2) bridge with rocking pile foundation (RP) designed to have its piers remain elastic, (3) RP bridge with lead rubber bearings (RP-LRB), (4) RP bridges with fluid viscous dampers (RP-VD), and (5) RP bridges with cable restrainers (RP-CR). The bridges are designed to comply with Eurocode-8 (European Committee for standardization 2005) and created with geometries similar to existing bridges in the seismic regions. Moreover, the aforementioned bridges are compared with identical bridge configurations, as presented in Figure 1.

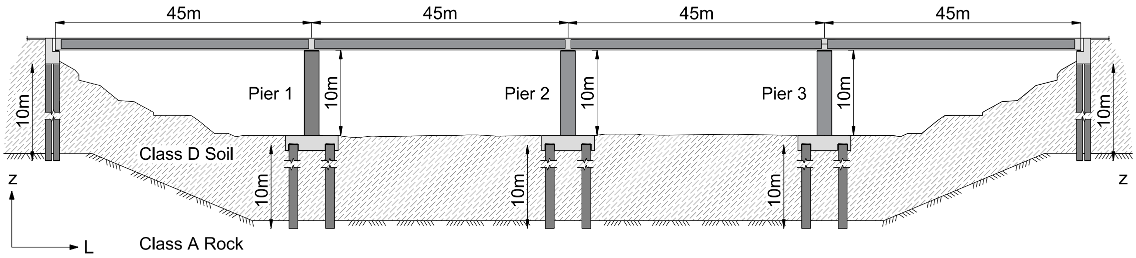

Figure 1: Longitudinal section of the bridge adopted in this study

The bridge encompasses a continuous concrete box girder section and has a total of four equal spans at 45 m length. The concrete used in this bridge has a compressive strength of 50 MPa and a unit weight of 25 kN/m^3. Additionally the reinforcing steel has a yield strength of 500 MPa and unit weight of 77 kN/m^3. The bridge deck is supported by three intermediate single pier bents all having equal heights of 10 m. The bridge piers consist of a concrete hollow rectangular section of 2.6 × 4.8 m with wall thickness of 0.45 m and longitudinal steel reinforcement of 2%. The piers are connected to the underside of the bridge deck using a pin connection. The piers are supported by pile caps that have a plan area of 7 × 7 m and a pile cap thickness of 1.9 m. The pile cap is supported by four piles each having a diameter of 1.2 m and protruding 0.4 m into the pile cap for the adequate transfer of shear for the case of the rocking pile foundations. The concrete deck is supported by four square elastomeric bearings at the abutments with a seat width of 500 mm. The elastomeric bearings are 200 mm thick with plan dimensions of 500 × 500 mm. For the RP-LRB bridge, the elastomeric bearings are replaced with four LRBs having thickness of 200 mm, plan area of 500 × 500 mm and a lead core diameter of 250 mm. The Abutment is supported by eight 0.8 m diameter piles. All piles have longitudinal reinforcement of 1.5% and a total length of 10 m with 1m socket into Class A strong rock. Moreover, the piles are founded in soft clay with properties characterised by Class D type soil as defined in in Eurocode-8 (European Committee for standardization 2005).

3. NUMERICAL MODEL

The finite element software SAP2000 V20.1 was used to develop the three dimensional numerical model of all the bridges and perform non-linear time history analyses. Figure 2 presents the numerical model of the bridge. The bridge deck is modelled as a spine through the use of multiple frame elements and lumped masses as suggested by Kappos et al. (2012). Frame elements were used to model the bridge deck, piers, and piles. The frame elements are characterised to capture biaxial bending, shear, axial and torsional actions experienced by the member. The bridge deck was modelled and designed with dead and live loads of 120 kPa and 60 kPa, respectively.

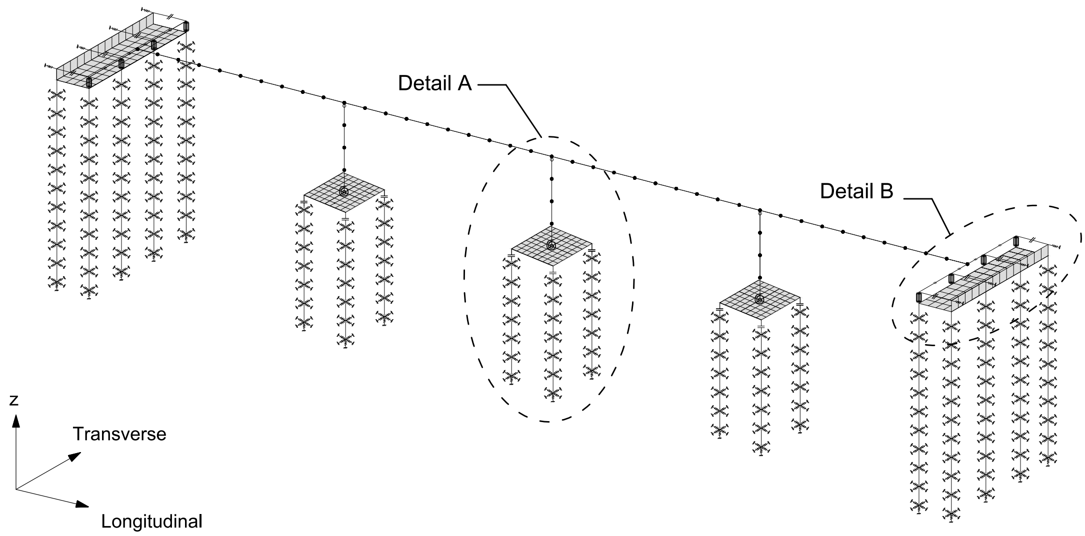

Figure 2: Numerical Model of the bridges.

The intermediate piers form pin connections to the underside of the deck to allow rotation while restraining lateral movements. All piers are assigned with flexural plastic hinges (P-M2-M3) at the base of the piers as displayed in Figure 3a. The flexural hinges capture possible inelastic behaviour that could develop at the piers. Moreover, Eurocode-8 (European Committee for standardization 2005) necessitates a reduction of 35% in bending stiffness to the piers to capture possible cracking of concrete in the tensile regions of concrete under seismic loading.

The abutments of the bridge are modelled using shell elements whereas the piles are modelled as frame elements as can be seen in Figure 3b. The connection between the abutment and the piles are fixed. The passive capacity of the soil behind the abutment back wall is modelled using linear springs as per Caltrans recommendations (Caltrans 2010). The linear springs are spaced at 2 m intervals with a compressive stiffness of 28.7 kPa/mm/m and upper bound limit of 6.2 kN. These springs are only activated during compression when the deck makes contact with the abutment back-wall and closes the expansion joint. The expansion joint is modelled using a gap element with a compressive stiffness equal to the stiffness of the bridge deck and an opening of 300 mm. Moreover, the abutment shear keys are modelled transversely using multi-linear springs adopting a tri-linear force displacement relationship designed to resist an ultimate capacity of 1000kN.

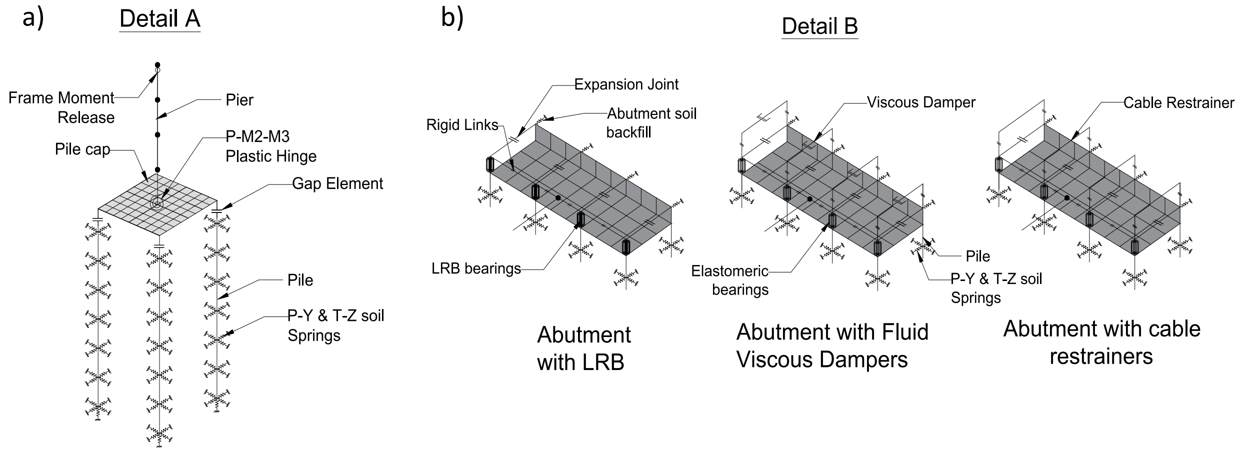

Figure 3: a) numerical model of the column and rocking foundation interface b) Numerical model of the abutments and the subsequent seismic restraining devices.

The elastomeric bearings are modelled using multi-linear springs with a horizontal stiffness given by Equation (1) (Antonellis and Panagiotou 2013):

$$k_h = \frac{G_r A}{t_r} \quad (1)$$

Where A is the plan area of the bearing, G_r is the shear modulus of the rubber taken to be 0.9 MPa and t_r is the total height taken as 200 mm. Moreover, Eurocode-8 defines the maximum strain of the bearing as 200%. On the occasion that the bearing experiences a horizontal displacement equal to twice its height, the bearing will rupture and its stiffness reduces to zero. The LRB are also modelled using multilinear springs with bi-linear force displacement relationship. The horizontal stiffness is given by Equation (2) (Antonellis and Panagiotou 2013):

$$k_h = \frac{G_r A}{t_r} + \frac{G_L A_L}{h} \quad (2)$$

Where G_L is the shear modulus of lead taken to be 150 MPa, A_L and h are the plan area and height of the lead core. The fluid viscous dampers are modelled using a non-linear links with their force-velocity relationship given by Equation (3) (Hassoun and Fatahi 2019):

$$f_c = c.v^{\alpha} \quad (3)$$

Where c is the viscosity coefficient set to 2500 kN.s/m and its power α is set to 0.15. Moreover, the cable restrainers are modelled using cable tension only elements and designed to withstand an ultimate force of 20 MN at each abutment. The abutments and corresponding restraining devices are displayed in Figure 3b.

The pile cap of the rocking pile foundation is modelled as a shell element whereas the piles are modelled as frame elements. The rocking interface is modelled using a gap element between the pile and pile cap. The gap element has a compressive stiffness set to the stiffness of the pile and a gap opening of 0 mm. This will allow the pile cap to uplift under earthquake loading while only transferring shear and compressive forces to the pile. Pile group effects were not considered as the spacing of the piles were greater than 3 × D (pile diameter D = 1.2 m at the piers and D = 0.8 m at abutments).

The behaviour of the soil surrounding the piles are modelled through various spring properties. The lateral resistance of the clay is modelled using a plastic spring with a P-y backbone curve as suggested by Welch and Reese (1972). The P-y springs can only take compressive forces and are modelled on opposite sides of the pile to model the possible separation between the pile and soil during cyclic lateral loading. The skin friction of the pile due to presence of soil is modelled using plastic springs with a T-z force-displacement backbone curve as suggested by the API (2000). Both the P-y springs and T-z springs are modelled at 1 m intervals along the depth of the pile. The pile toe resistance is modelled using a linear spring with compressive stiffness equal to the stiffness of the rock. Furthermore, all the soil springs account for hysteretic damping with a kinematic hysteresis model.

4. SITE CHARACTERISTIC AND SEISMIC HAZARD

The bridges investigated in this study are hypothetically located at a high seismic region. Thus, the following earthquakes were selected in this study: 1994 Northridge, 1995 Kobe, 1976 Friuli and 1971 San Fernando. The bridges are founded on a 10 m deep layer of clay with properties consistent with Class D type soil underlined by Class A rock (Eurocode-8). The European seismic hazard map was used to determine the PGA for the maximum considered earthquake (2% probability of exceedance in 50 years) taken to be 0.6g. The earthquakes are then scaled to the Eurocode-8 target response spectrum displayed in Figure 4, for a damping ratio of 5% between the periods of T=0.05 s and 3 s. Moreover, non-linear time history analyses were used to simulate the earthquake loadings. Rayleigh's mass and proportional stiffness damping was used with a 5% damping ratio for the first and second fundamental periods of the bridges (Xu and Fatahi 2018).

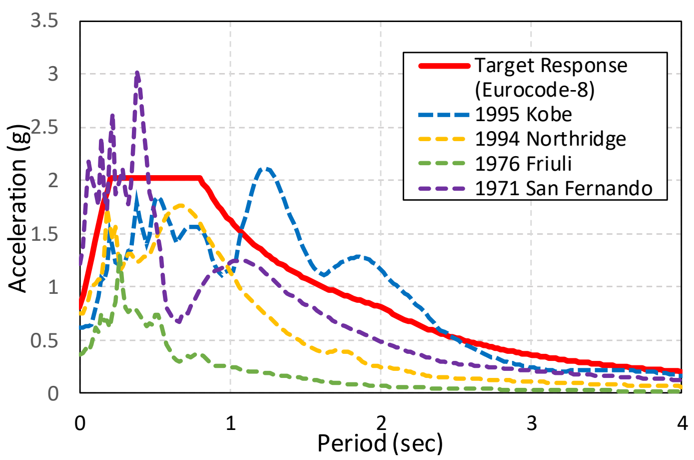

Figure 4: Response spectrum of the original earthquakes.

5. RESULTS AND DISCUSSION

5.1 MODAL ANALYSIS

A modal analysis was conducted for the different bridges using Ritz vectors with a 98% mass participation ratio. The fundamental periods of the bridges were then computed and presented in Table 1.

Table 1: The First and second Fundamental periods of the bridges

Bridge Types	Natural Period in the longitudinal direction	Natural Period in the Transverse direction
FB	0.45	0.39
RP	1.8	1.35
RP - LRB	1.5	1.35
RP - VD	1.48	1.35
RP - CR	1.1	1.35

*Note FB represents the bridge with the fixed base foundation, RP represents the bridge with the rocking pile foundation, RP-LRB represents the RP bridge with the LRBs, RP-VD represents the RP bridge with the viscous dampers and RP-CR represents the RP bridge with the cable restrainers.

It can be observed that the first fundamental periods of the bridges are in the longitudinal direction followed by the second fundamental period in the transverse direction with the exception of the rocking pile bridge with the cable restrainers (RP-CR). The natural period of the bridges in the longitudinal direction for the RP, RP-LRB, RP-VD, RP-CR, and FB bridges are 1.8 s, 1.5 s, 1.48 s, 1.1 s and 0.45 s, respectively. The rocking pile (RP) bridge has the largest period in the longitudinal direction due to the reduction of lateral stiffness of the piers and foundations when compared to the fixed base bridge (FB). The supplementary use of the LRBs and viscous dampers provide added stiffness to the RP bridge in the longitudinal direction and hence decreased its natural period. The period of the RP-LRB and RP-VD are comparable as they function similar to each other, allowing the deck to displace while adding resistance to the displacement of the deck and transferring the inertial forces to the abutment. Moreover, the use of cable restrainers significantly lowers the natural period of the bridge from 1.8 s to 1.1 s in the longitudinal direction. The restrainers reduce the displacement capacity of the deck which therefore contributes to a smaller natural frequency. This great increase in stiffness causes the longitudinal displacement of the bridge to become the second fundamental period. The FB bridge has the smallest natural period of 0.45 s due to the contribution of stiffness of the pier and foundation in reducing the displacements of the deck. The bridge with the largest period will tend to have the smallest dynamic response whereas the bridge with the smallest period will tend to have the largest dynamic response. Indeed, this very much depends on the shape of the response spectrum of the earthquake (Choi and Stewart, 2005). With the addition of seismic restraining devices, it can be expected that the dynamic response will increase due to the decrease in the natural period of the structure.

In the transverse direction the restraining devices have no effect on the natural period of the bridge. This is because the transverse displacements of the bridge are impacted by the presence of concrete shear keys, lateral stiffness of the piers, and bending stiffness of the deck. Thus, the longitudinal response of the bridges is the primary focus of this study, in order to assess the effectiveness of the seismic restrainers.

5.2 NON-LINEAR DYNAMIC TIME-HISTORY ANALYSIS

Non-linear dynamic time history analyses were performed on the bridges with different restraining devices. As explained earlier, four earthquake time-histories were selected and scaled, namely 1994 Northridge, 1995 Kobe, 1976 Friuli and 1971 San Fernando earthquakes. These earthquakes were applied to the longitudinal direction of the bridges and their structural response are reported in Table 2.

Table 2. Dynamic response of the bridges for the longitudinal direction

Earthquake	Bridge Type	Maximum Pier Bending Moment (MN.m)	Design Ratio (%)	Maximum Deck Displacement (mm)	Maximum Bearing Strain (%)	Abutment Pounding Force (MN)		Maximum Abutment Pile Bending Moment and Shear Force	
						Right	Left	(MN.m)	(MN)
1994 Northridge	FB	225	225%	179	90%	0	0	2.3	1.4
	RP	49.7	50%	602	301%	19.7	19.7	20	7.6
	RP - LRB	41.7	42%	411	206%	6.6	6.6	11.6	5.2
	RP - VD	41.5	42%	435	218%	8.5	7.2	12.6	5.5
	RP - CR	39.6	40%	329	165%	1.9	1.5	25.2	8.9
1995 Kobe	FB	300	300%	120	60%	0	0	2.5	1.4
	RP	43.7	44%	525	263%	15.7	14.1	15.2	6.4
	RP - LRB	37	37%	391	196%	5.5	5.5	8.4	4
	RP - VD	38.9	39%	367	184%	4.2	4.2	5.2	2.7
	RP - CR	38.1	38%	310	155%	2.5	2.5	28.1	9.5
1976 Friuli	FB	236	236%	94	47%	0	0	2	1.4
	RP	46.9	47%	573	286%	18.3	14.1	18	7.2
	RP - LRB	37.9	38%	447	223%	8.7	8.7	5	2.6
	RP - VD	38.3	38%	439	219%	8.7	8.7	5	2.6
	RP - CR	38.3	38%	325	162%	2.5	2.5	28.4	9.5
1971 San Fernando	FB	188	188%	110	55%	0	0	2.4	1.4
	RP	50.2	50%	537	269%	15.9	12.7	16.5	6.7
	RP - LRB	48	48%	412	206%	6.6	3.6	11.6	5.2
	RP - VD	48.5	49%	412	206%	7	0.57	10.8	4.9
	RP - CR	43.4	43%	372	186%	4.9	1.8	25.4	8.8

*Note FB represents the bridge with the fixed base foundation, RP represents the bridge with the rocking pile foundation, RP-LRB represents the RP bridge with the LRBs, RP-VD represents the RP bridge with the viscous dampers and RP-CR represents the RP bridge with the cable restrainers.

5.2.1 PIER BENDING MOMENTS

All three bridge piers experienced identical results due to the symmetrical configuration of the bridge. Hence, Table 2 and Figure 4 only report the seismic response of the central pier (Pier 2). Figure 4 displays the maximum bending moment experienced by piers for all the bridges. It is evident from Table 2 and Figure 4 that the FB bridge experienced the largest pier bending moments when compared to the rocking foundation bridges. Moreover, the design ratio (ratio between the experienced bending moment and ultimate bending capacity, M_{ult} = 100 MN.m) confirms the piers of the FB bridge have exceeded their bending capacity by up to 300% as can be seen for the 1995 Kobe Earthquake reported in Table 2. The plastic hinge results assigned to these piers reveal that the collapse hinge state was reached for all four earthquakes resulting in the total collapse of FB bridge. This is due the fact that the FB bridge had a small natural frequency, which attracted larger inertial forces to the pier as a result from the response spectrum of the earthquake. Additionally, the high stiffness piers have been forced to deform due to the large displacement of the deck and therefore generated larger bending moments.

Referring to Table 2 and Figure 4, the bridges with the rocking foundations experienced relatively small bending moment in their piers for all the earthquakes applied, requiring smaller pier sections. The supplementation of seismic restrainers did not significantly affect the pier bending moments despite the observed change in natural periods reported in Table 1. This is because by allowing the foundation to uplift, the piers do not resist the deck displacements via bending. Moreover, referring to Figure 4, it can be seen that by adding dissipative devices such as viscous dampers and lead rubber bearings, the pier moments only reduced slightly. Indeed, the addition of the cable restrainers were the most effective for reducing the pier moments. This is because by reducing the large deck displacements and subsequent transient drift, the secondary moment effects of the piers were also reduced.

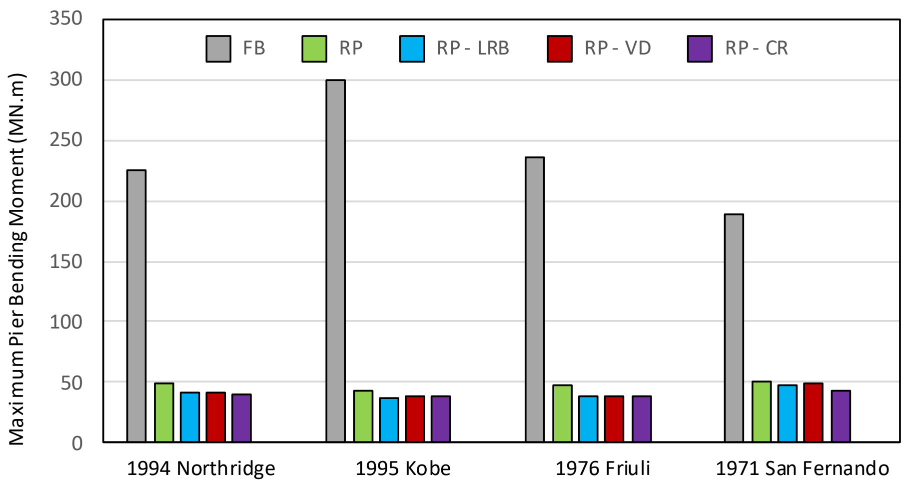

Figure 5: Maximum bending moments experienced by the piers

5.2.2 DECK DISPLACEMENTS

Figure 5 presents the deck displacement time histories for the bridges subjected to the four earthquakes. The maximum deck displacements experienced by each bridge under each earthquake are also reported in Table 2. When observing the results for the 1994 Northridge earthquake as an example, the deck displacements experienced by the RP, RP-LRB, RP-VD, RP-CR, and FB bridge were 602 mm, 411 mm, 435 mm, 329 mm, and 179 mm, respectively. It can be observed from both Figure 5 and Table 2, that the deck displacements belonging to the FB bridge are significantly smaller when compared to the rocking foundation bridges. This is due to the higher stiffness in the foundation and piers contributing to the smaller natural frequency of the bridge. In contrast, the RP bridge experienced the largest deck displacements, due to the reduced bending stiffness in the foundation and pier. This not only led the bridge to have the largest natural period of all the other bridges, but also the largest deck displacements. With reference to Figure 5, the deck displacements of the RP foundation exceeded 500 mm (i.e. corresponding to the seat width at the abutment) for all the earthquakes and as a result, the bridge decks pounded into the abutments and eventually unseated from the opposite end, resulting in the total collapse of the bridge.

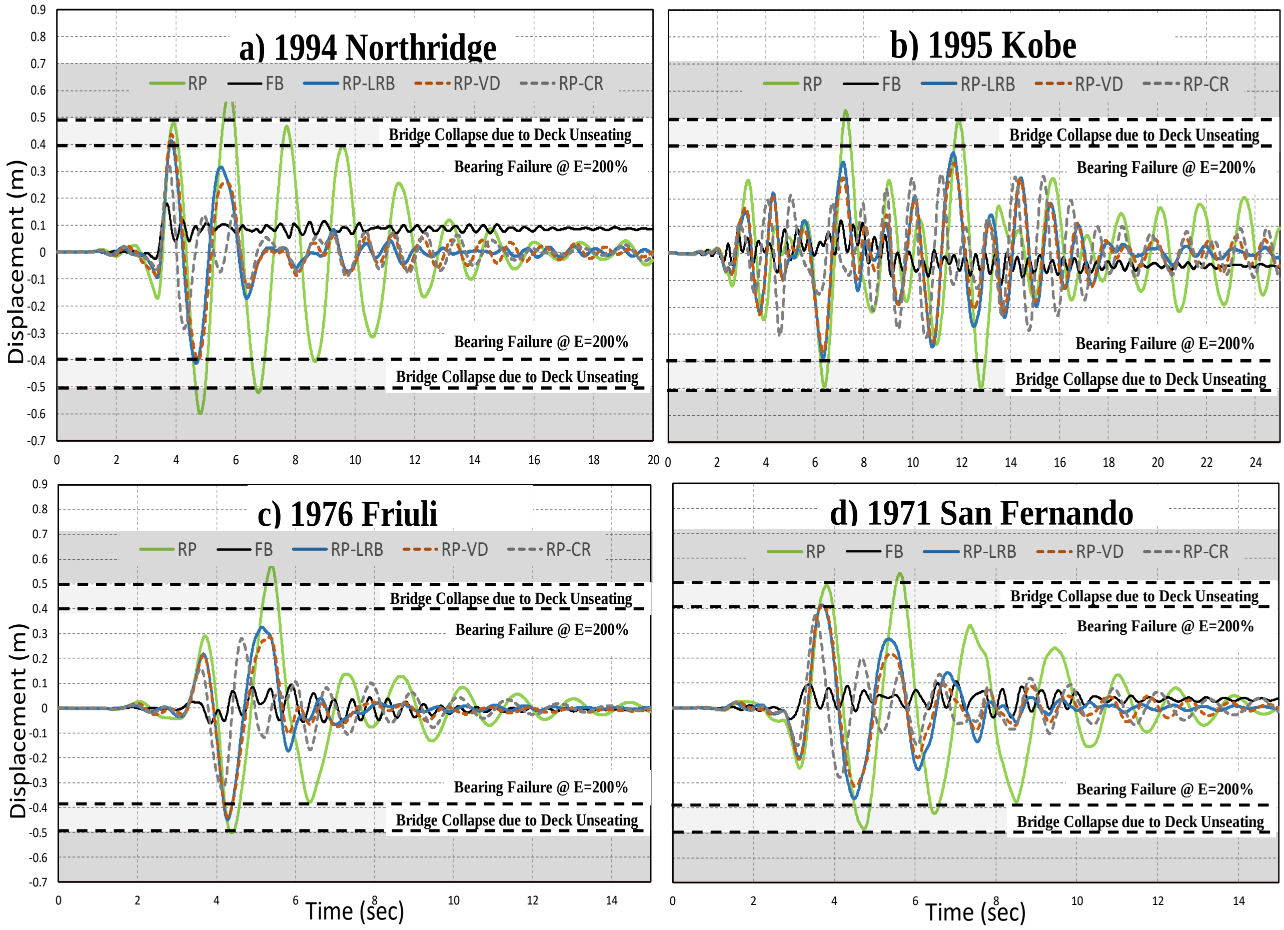

Figure 6: Deck Displacement-time history for the bridges under a) 1994 Northridge b) 1995 Kobe c) 1976 Friuli and d) 1971 San Fernando

The rocking bridges with the LRBs (RP-LRB) and viscous dampers (RP-VD) performed very similar to each other as they had similar natural periods. Throughout the four earthquakes, it can be seen from Figure 5 that the RP-LRB and the RP-VD bridges followed similar deck displacement history patterns despite the two devices functioning with different mechanisms. The energy dissipation provided from both devices were rather similar and hence yielded a similar response.

The LRBs and viscous dampers were effective in dissipating energy from the bridge to reduce the deck displacements. Nevertheless, the cable restrainers were most effective for reducing the seismic displacements of the deck. The RP Bridge with the cable restrainers, as displayed in Figure 5, yielded the lowest deck displacements out of all the rocking foundation bridges. Additionally, the RP-CR was the only bridge besides the FB bridge that experienced deck displacements smaller than the ultimate strain of the bearings. The observed patterns in deck displacements of the RP-CR bridge (see Figure 5) are in line with the pattern of natural periods (see Table 1), showing the lowest natural period and deck displacements out of all the rocking foundation bridges.

Furthermore, Figure 6 displays the maximum pounding force experienced at the abutment due to the deck displacements. It can be seen that with the addition of restraining devices, the decrease in deck displacements leads to the subsequent decrease in the pounding force experienced at the abutment. It is evident that the cable restrainers are most effective when restraining the peak deck displacements of bridges with rocking foundations. This is because the additional stiffness introduced by the cables are not only restraining the deck displacements but also affect the period of the bridge, which corresponds to the smaller deck displacements.

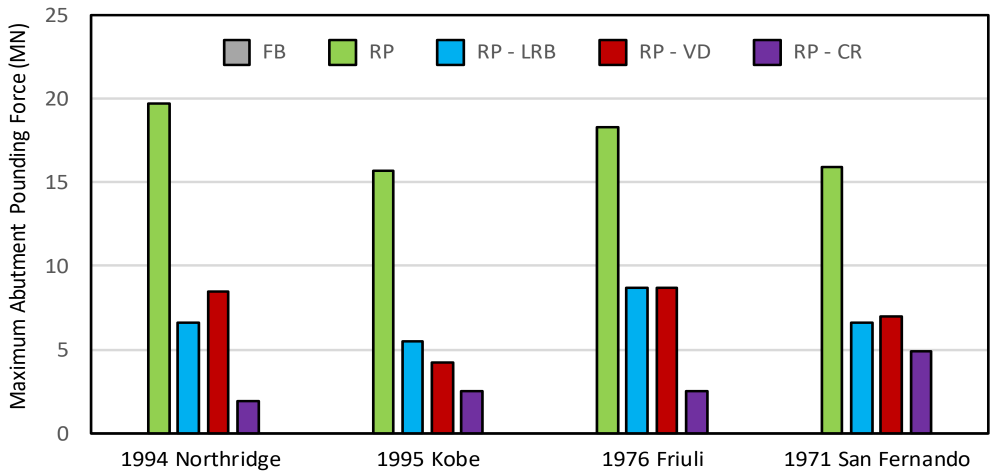

Figure 7: Maximum Abutment pounding force

5.2.3 ABUTMENT AND PILE RESPONSES

Figure 7 displays the maximum bending moment experienced by the piles at the abutment. It is evident from Figures 5 & 6 and Table 2 that the cable restrainers are most effective for reducing deck displacements and subsequent abutment pounding forces in bridges with rocking foundations. However with further examination into the transfer of the forces, it is evident from Figure 7 that the cable restrainers are transferring the full force from the deck displacements into the abutment and piles. When viewing the bridge responses from the 1976 Friuli earthquake, the maximum pile bending moments at the abutments in the RP, RP-LRB, RP-VD, RP-CR, and FB bridges are 18 MN.m, 5 MN.m, 5 MN.m, 28.4 MN.m and 2 MN.m, respectively. The bending moment in the abutment piles are the largest when cable restrainers are added to the bridge with rocking foundations. Indeed, as the cable restrainers resist the deck displacements, they transfer the full seismic force to the pile and surrounding soil. This leads to the bridge requiring larger pile sections at the abutment in order to resist additional forces. However, this takes away from the advantage of having reduced member sizes at the piers.

As shown in 7, in the absence of any restraining system, the RP bridge experiences quite significant bending moment in the abutment piles, as a result of the pounding forces generated from the excessive deck displacements which were transferred into the piles. With the addition of the LRBs and the viscous dampers, it can be seen that they did not only reduce the deck displacements, but also decreased the pile bending actions at the abutment. This is because part of the seismic energy was dissipated by the viscous damper and yielding of the LRBs. For this reason, the RP-LRBs and the RP-VD were the most effective in terms of enhancing the the overall response of bridges with rocking foundations. Moreover, it is important to note that if the deck would have not pounded into the abutment, The RP bridge would have had the smallest seismic actions at the abutment piles whereas the restrainers would have all increased the seismic actions. This is a result of the restrainers being directly connected to the abutments and transferring the inertial forces to the abutment and piles.

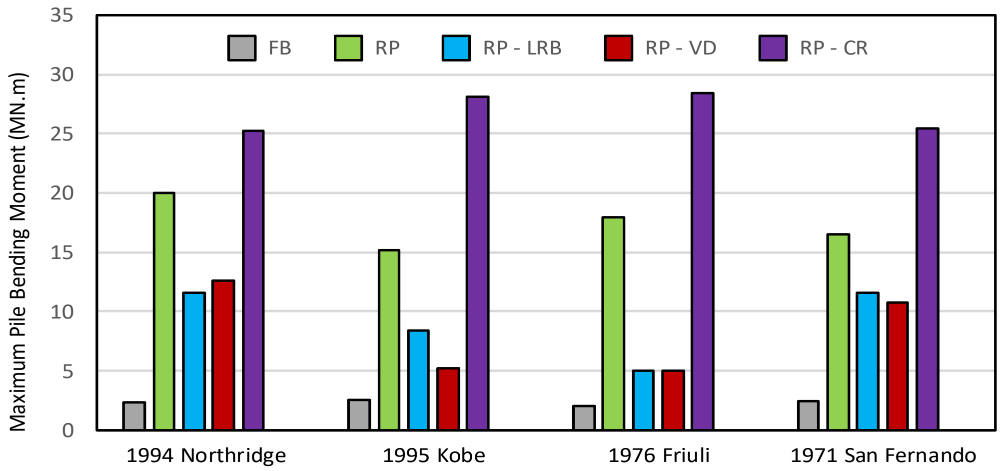

Figure 8: Maximum pile bending moment at the abutment

6. CONCLUSIONS

This paper investigated the effectiveness of various restrainers on the seismic performance of bridges with rocking foundations. Lead rubber bearings, viscous dampers and cable restrainers were all modelled in identical bridge configurations and subjected to non-linear time history analyses. The dynamic response of the bridges subjected to four earthquakes were then compared in terms of the structural actions and displacements of the piers, deck, abutment and piles. It was observed that the seismic restrainers had minimal effect on reducing the bending actions on the piers, but significantly reduced the displacements of the deck. The cable restrainers were most effective in reducing and restraining deck movements. On the contrary, the cable restrainers transferred the largest inertial forces to the abutment piles which could potentially cause substructural failure. The LRBs and viscous dampers both performed similarly despite their different mechanisms. It was observed that they were able to reduce the seismic deck displacements without transferring all the forces to the abutment making them very efficient options for bridges with rocking foundations. Nevertheless, when considering these seismic restrainers, design provisions are necessary to account for the significant forces that are transferred to the abutments and piles.

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