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Improved seismic response of multi-span bridges retrofitted with compound restrainers

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Abstract. This numerical study aims to evaluate the seismic performance of bridges retrofitted with a new compound restrainer, and the sensitivity of their responses to likely changes in the characteristics of the restrainer's components. The compound restrainer was introduced as a retrofit tool for improving the seismic response of multi-span bridges in terms of forces and displacements. The compound restrainer is mechanically an assembly of several elastic and plastic springs. To this end, real 2-span and 3-span simply supported plate girder bridges have been used for the case study. Nonlinear time history analyses of detailed three-dimensional models have been performed under seismic excitations in order to assess the performance of the existing and retrofitted bridges restrained by conventional and compound restrainers, numerically. The results show that the compound restrainer is very sensitive to the characteristics of its components. Moreover, while past earthquakes have shown the deficiencies of the conventional restrainers, the compound restrainer seems to be successful in dissipating hysteresis energy in bridges, as well as in reducing the internal forces imposed on the substructures.

1. Introduction

One of the most common types of failure in bridges is unseating. If the seat width provided at the joint is less than the relative joint opening, the bridge superstructure becomes unseated, which leads to potential collapse due to the lack of support [1]. Such failures usually cannot be repaired and, hence, the collapsed spans should be demolished and reconstructed. Because of the catastrophic consequences of the loss of support, and the perception that they could easily be prevented at relatively low cost, early retrofitting programs are focused on preventing such failures. These programs, which were first undertaken by Caltrans, involved the addition of longitudinal restrainer cables and bars to limit the relative movements at expansion joints [2].

Many bridges retrofitted with restrainer cables failed in both the 1989 Loma Prieta and 1994 Northridge earthquakes, and similar failures of Japanese restraining devices were observed during the 1995 Kobe earthquake. These restrainer failures drew further attention to the need to study their characteristics and behavior in more detail, with the aim of improving their design provisions [2].

Restrainers must not only be stiff and strong enough to prevent the joints from separating, but the other elements of the bridge must also be able to resist the forces developed in the restrainers that are eventually transferred to them. The restraining devices may also transmit larger forces to other bridge components, such as the bearings and columns, which may result in their failure if not properly designed [2].

Parametric studies have indicated that maximum hinge displacement is a function of the frame period

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ratio, frame ductility and the characteristics of the ground motion [3].

Results from another study indicate that the restrainer cables are effective for Multi-Span Simply Supported (MSSS) bridges, yet not for Multi-Span Continuous (MSC) bridges. A less invasive and less costly retrofit measure using steel restrainer cables may be an acceptable alternative for MSSS bridges, but is not as effective for MSC bridges. In MSSS steel bridges, the restrainer cables cut the mean peak expansion bearing deformations below the levels of expected bearing toppling, and reduce the column demands to levels nearing yield or potential cracking, which may be tolerable. However, the restrainer cables are less effective for MSC bridges, which have higher bearing deformations and inertial loads that tend to yield the cables [4].

Other passive control technologies have been proposed for limiting joint displacement, including metallic dampers, viscoelastic (VE) solid dampers and fluid viscous dampers [1,5,6]. However, some of the proposed devices, such as metallic dampers and fluid viscous dampers, lack the re-centering capability that is necessary for reducing possible permanent damage and displacement, while other devices, such as VE dampers, are highly dependent on the frequency content of the ground motion, and, hence, are not effective for a broad range of input characteristics.

Many studies have investigated the effectiveness of Shape Memory Alloy (SMA) restrainer bars to reduce the seismic vulnerability of bridges [1,7-9].

The SMA-based restrainers are effective in protecting abutments and bearing devices from damage [8].

For multi-span simply supported deck bridges, the overall objective of rehabilitation is to control the deck horizontal displacements. The SMA-based device, therefore, plays the role of a seismic restrainer. For continuous deck bridges, the overall objective is to control the seismic forces transmitted to all the piers. The SMA-based device, therefore, plays the role of a seismic absorber [8].

The super-elasticity characteristic of shape memory alloys is highly dependent on ambient temperature. This would be of major concern in cases of structures that are directly subjected to the surrounding environment, such as bridges and coastal structures. The results of studies on a 6-frame MDOF bridge showed that SMA restrainers are more effective in limiting the hinge opening at higher ambient temperatures [9].

In applications where superelastic behavior is desired, the temperature of the SMA has to be kept above the austenite finish temperature, where the development of austenite (superelasticity) in the alloy is 100% complete, otherwise the SMA will experience residual deformation [9].

For some SMAs, such as Nitinol (NiTi SMA), the phase change can be stress-induced at room temperature (22°C) if the alloy has the appropriate formulation and treatment [7].

With several technologies available to limit joint opening in bridges, bridge engineers are faced with the task of determining the most effective measure to restrict unseating and also eliminating damage to adjacent bridge structures due to relative responses, such as pounding.

The restrainers are found to reduce the relative displacements efficiently, lowering the probability of span failure due to pounding [10].

In a study, the effect of cable restrainers in MSSS steel girder bridges with elastomeric bearings was evaluated. While isolation provided by elastomeric bearings limits the forces in the columns, the added flexibility results in pounding between the decks. When restrainer cables are used with elastomeric bearings, the restrainer cables negate the isolation effect of the bearings [11].

Bearing in mind some weaknesses of the above mentioned restrainers, including the transmission of excessive forces to substructures, the weak capability of energy dissipation during severe earthquake, and high sensitivity to ambient temperature, while respecting the need for decreasing the probability of damage occurrence between adjacent bridge structures due to pounding, a compound restrainer device was introduced by the authors in an earlier study (which is under review), where the characteristics of this restrainer and its force-deformation relation built through mathematical equations were examined. However, since this paper is not an extension of the previous paper, its content can be studied independently. This study aims to evaluate the improvements in the seismic response of multi-span bridges when compound restrainers are added as a retrofit tool. The performance of two existing bridges, which suffer from deficiencies in their current condition, is evaluated through numerical simulation. To this end, the bridges are then retrofitted by means of conventional cable restrainers, as well as the new compound restrainers, separately. Various nonlinear time history analyses using detailed three-dimensional models are performed to assess the performance of the existing and retrofitted bridges during seismic excitations. Assessment of the sensitivity of the compound restrainer's performance to changes in variables is another aspect of this study.

2. Mechanics of compound restrainer

The main concept of the compound restrainer was previously introduced by the authors. Inspired by the modeling of shape memory alloy behavior, the compound restrainer's aim is to present a restrainer element

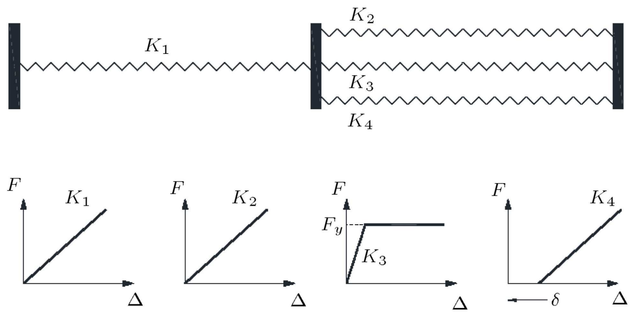

Figure 1. Phenomenological model of the compound element.

which has a considerable hysteretic energy dissipation capacity, as well as a large working displacement.

The phenomenological model of this element is shown in Figure 1. This element is, in fact, a compound of several elastic and plastic springs. Springs, K_1 and K_2, represent elements with high yield stress and moderate to high elastic modulus that must remain elastic during severe seismic motion. Spring K_4 is similar to the previous springs, but with an initial gap that shifts its active performance to ranges where strong excitations of ground motion apply. Spring K_3 is used to account for the energy dissipation capacity of the compound device and represents an elastoplastic element with comparatively moderate yield stress and a very high elastic modulus, as well as extensive plastic strain capacity. The K_1 element is put in series with the other springs, working in parallel under the same horizontal displacement. It has been supposed that all the elements depict tension-only behavior. The axial force-displacement cyclic behavior of the compound element is shown in Figure 2. The force-deformation curve, increasing elastically, experiences a yield at point A due to the rise of plastic behavior in spring K_3. It continues a gradual increase to point B, where the displacement of the parallel springs equates the gap of spring K_4. Then, displaying a sharp increase in force-deformation action, the curve bounces back at point C, where unloading takes place. It is obvious that all the springs except K_3 will remain elastic during the loading and unloading procedure. The highest element stiffness occurs during unloading from point C to D, where all the springs are in an elastic unloading condition. The curve slope diminishes after point D, where the residual displacement of spring influences the curve trace. Point E depicts the point where spring K_3 meets its initial gap. Finally, the compound element returns to its initial point in a gradual decline from point E to O.

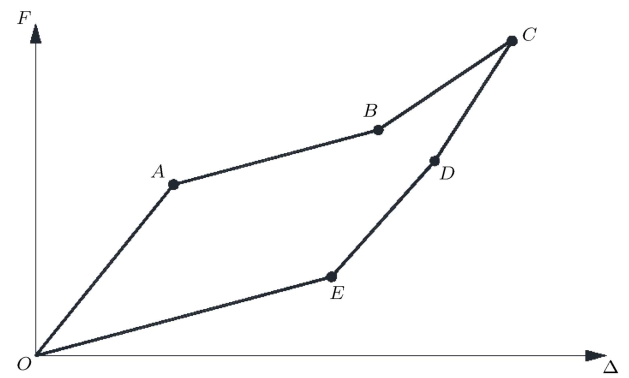

Figure 2. Force-deformation curve corresponding to the compound element.

This curve depicts two main characteristics of the compound element; first, the capability to dissipate a considerable amount of energy and, second, ability to recover its initial shape (to a respectable extent). Consequently, this element can dissipate substantial energy while exerting less force to adjacent structural elements in comparison with its equivalent elastic model. Moreover, the elastic springs of the compound element can recover their initial shape, despite the residual displacement, due to the plastic behavior of spring K_3.

3. Case studies

Two case study problems were conducted to assess the ability of the compound restrainer in improving the seismic capacity of bridges. The existing and retrofitted bridges were numerically restrained by conventional restrainers and compound restrainers. Using nonlinear time history analyses, the sensitivity of the compound restrainer is evaluated due to changes in the properties of its components. Moreover, the performance of the bridges under existing and retrofitted conditions is assessed to examine the reliability of the retrofit measures. A description of the case studies is presented in the following subsections.

3.1. Modelling of bridges

The bridges selected for this case study are Dalichai Bridges located in Firoozkooh, Tehran, Iran, which are two and three-span steel girder bridges. Dalichai-2 consists of two adjacent decks with one intermediate joint, while Dalichi-3 has three adjacent decks with two intermediate joints. Both bridges have two abutments supporting the far ends of the exterior decks. Figure 3 shows a side view of the bridges. The decks are

Figure 3. View of the Dalichai bridges: (a) Dalichai-2; and (b) Dalichai-3.

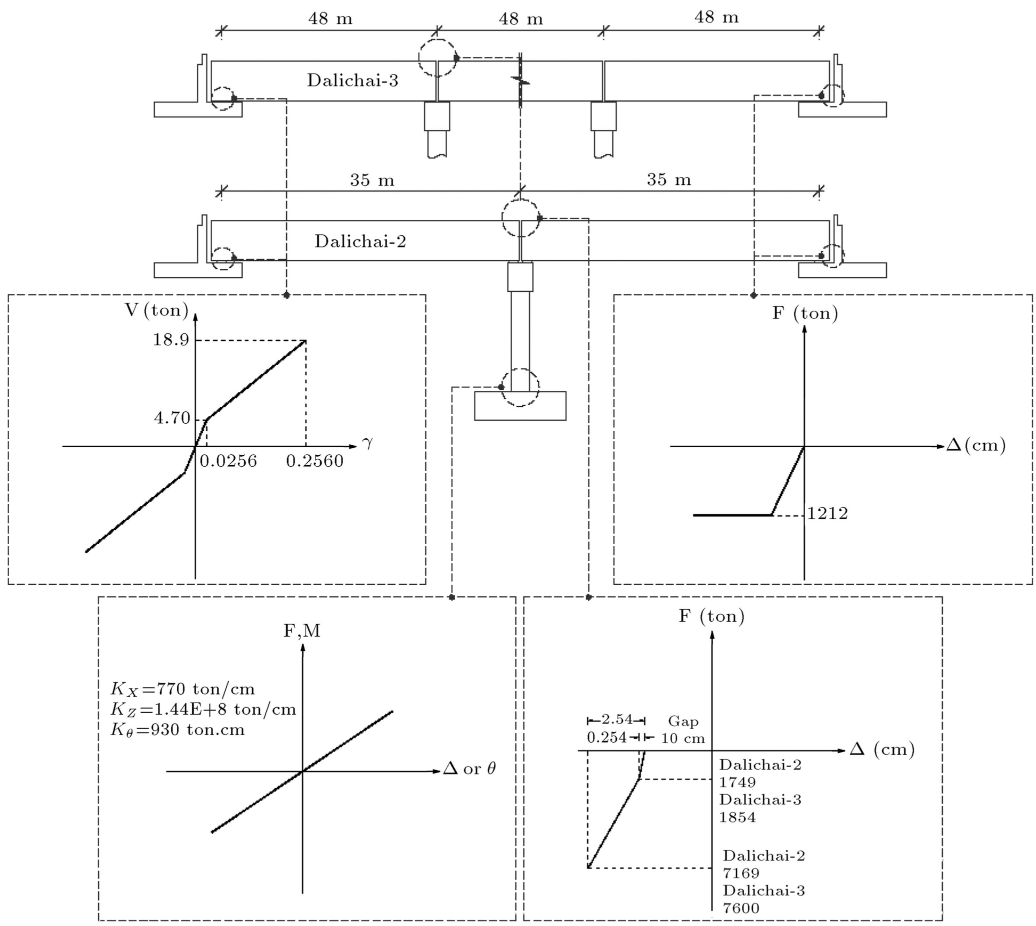

Figure 4. Nonlinear analytical model of a steel girder bridge including nonlinear elements used for abutments, bearings, columns and pounding.

supported by interior bents supported by columns. Steel girders are installed on the bents and abutments by means of elastomeric bearings.

Since the bridges consist of elements that may exhibit highly nonlinear behavior (elastomeric bearings, columns, abutments, impact), three dimensional nonlinear analytical models of the bridges were developed. Figure 4 presents a schematic of the models of the bridges used in the case study, together with the mechanical behavior of each of the elements used in the models. All the abutments are of a seat abutment type.

Since the superstructure is expected to remain linear under longitudinal earthquake motion, it is modeled using linear elements. The bridge deck was modeled using an elastic slab element. The deck was assumed to remain elastic during the analysis. An average height of 16.6 m was assumed for the columns, based on available data. The column's nonlinear behavior was modeled using P-M-M hinge. The abutments were modeled using a series of linear spring elements which represent the passive and active resistances of the abutments. The foundation supporting the column was also modeled using two linear and a rotational spring, which represent lateral and rotational stiffness, respectively. Several numerical modal analyses were conducted on the bridges, where the natural periods of the bridges were determined as 1.3 s and 1.8 s for Dalichai-2 and Dalichai-3, respectively.

3.2. Input earthquakes

The analytical models were analyzed for seven earthquake acceleration records. Ground motion was applied in the longitudinal direction of the bridges. The peak ground accelerations were scaled to 0.35 g. Many earthquake records were considered and used as input. The following seven earthquakes were selected because they have different characteristics and, in other studies, combinations (most times 3 or 4 of them) have also been used. Their selection has been mainly based on their amplitude, frequency content, and duration characteristics: (1) Tabas, 1978; (2) El Centro, 1940; (3) Northridge, 1994; (4) Bam Earthquake, 2003; (5) Landers, 1992; (6) Imperial Valley, 1979; and (7) Kobe Earthquake, 1995. The elastic acceleration spectra for the earthquakes are shown in Figure 5. It can be seen that bridges with a wide range of periods would

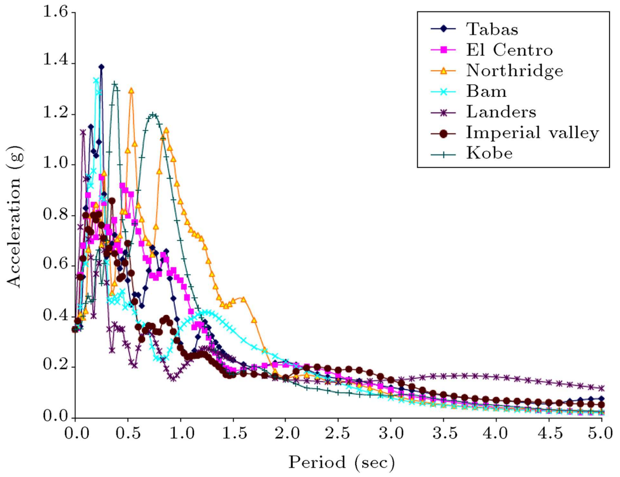

Figure 5. Elastic spectra for input earthquakes.

be excited by the collective effect of the earthquake records.

3.3. Sensitivity analysis of compound restrainer

Nonlinear time history analyses were used to evaluate the sensitivity of the restrained bridge performance to changes in the properties of the components of compound restrainers. Accordingly, variations of four parameters, including the bridge maximum displacement, the column maximum moment, the maximum force imposed on abutments by the restrainers, and the inelastic energy dissipated in the structure (hysteresis energy), have been assessed by altering the properties of the components (Figures 6 to 10).

The line-graphs, as well as bar charts, highlight the influence of change in component stiffness and yield stress on the aforementioned parameters. As can be seen in "Column Maximum Moment" and "Bridge Maximum Displacement" bar charts, stiffness augment in any component leads to a decrease in the force transferred to the columns, as well as the structure's longitudinal displacement. Despite this desirable effect, reviewing the bar charts for the "Maximum Force on Abutments", reveals that an increase in the value of all the components, except K_1, has led to a rise in the forces transferred to the abutments. Depending on the earthquake intensity and bridge characteristics, an increase in K_1 may result in an increase (Figure 6(b)) or decrease (Figure 6(a)) in the force transferred to the abutment. A reverse pattern is observed in the variation of hysteresis energy dissipated in the structures ("Hysteresis Energy" bar charts). That is, an increase in any component quantity leads to a growth of inelastic energy, which is then dissipated in the bridges. The K_1 component shows unusual behavior again. To sum up, the following points can be categorized:

a) A change in K_4 does not show any considerable influence on the four aforementioned parameters (Figure 9). This may arise from this fact that, due to the use of slacked cables in the K_4 element, this component may not engage completely during seismic excitations.

b) Evaluating all the figures, it can be perceived that both Dalichai-2 and Dalichai-3 bridges display similar seismic performance, that is, they have

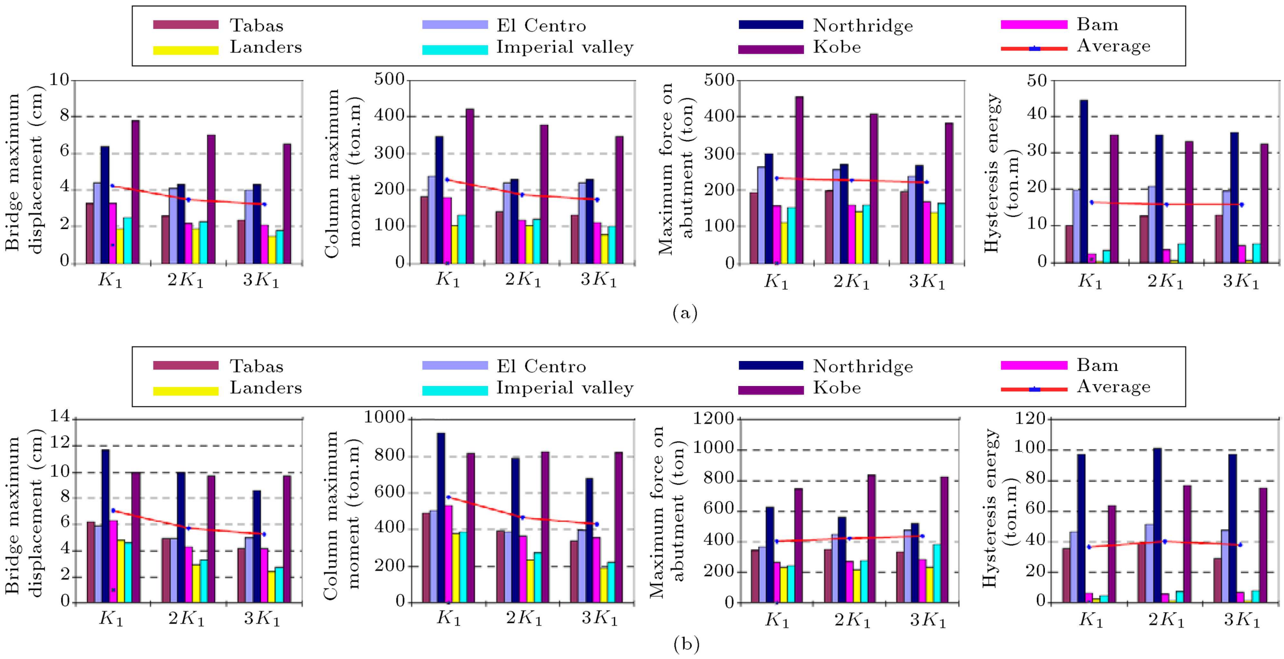

Figure 6. Sensitivity of bridges seismic responses considering progressive increase in the stiffness of spring K_1: (a) Dalichai-2; and (b) Dalichai-3.

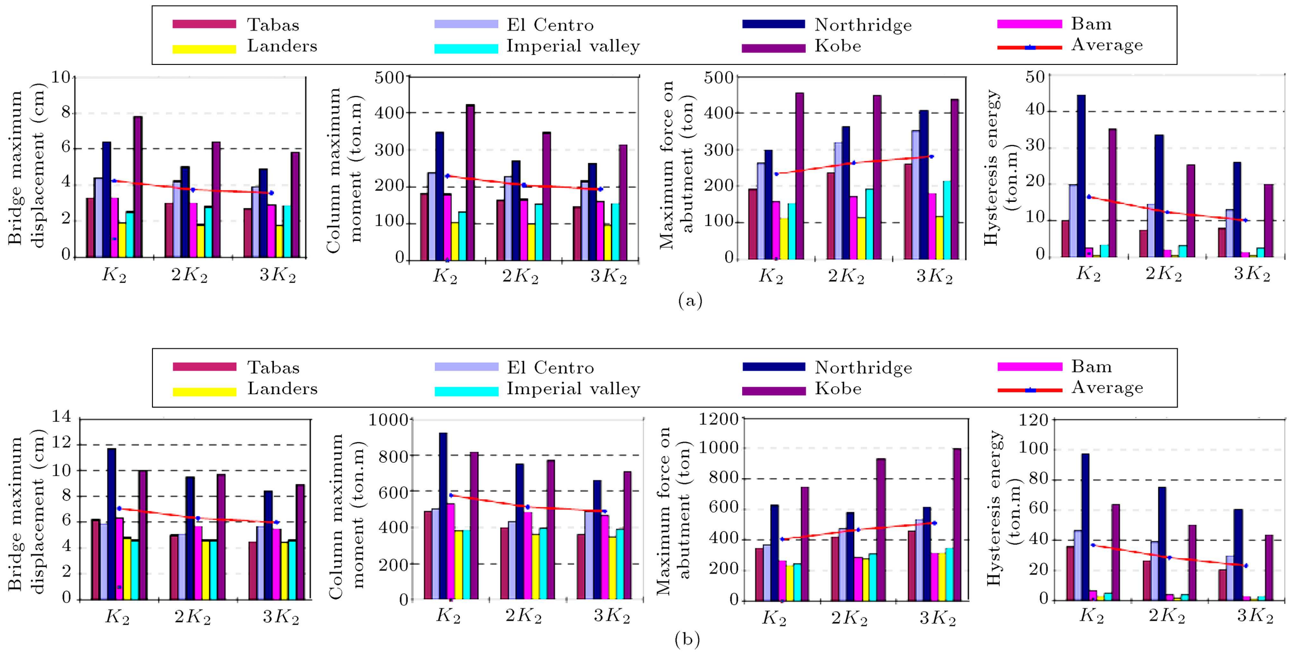

Figure 7. Sensitivity of bridges seismic responses considering progressive increase in the stiffness of spring K_2: (a) Dalichai-2; and (b) Dalichai-3.

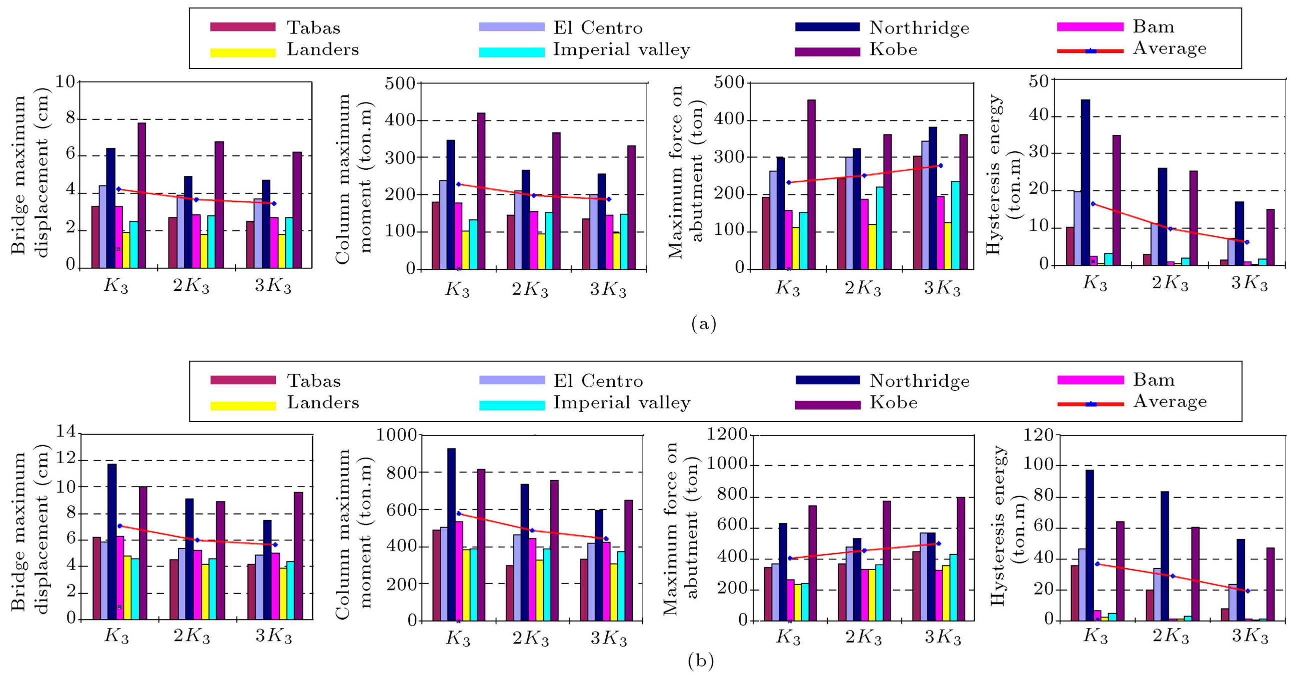

Figure 8. Sensitivity of bridges seismic responses considering progressive increase in the stiffness of spring K_3: (a) Dalichai-2; and (b) Dalichai-3.

represented a similar ascending or descending trend in their corresponding curves.

c) Although all the peak ground accelerations were scaled to an identical quantity (0.35 g), the maximum responses due to ground motion do not seem to be similar (or approximately equal). It may be concluded that the seismic performance of a bridge, and specifically the bridges studied here, is a function of more parameters than just the earthquake PGA. Such parameters may logically include the characteristics of ground motion and the natural period of the structure.

d) The stronger an earthquake is, the more explicit the change in the response trend can be, when the

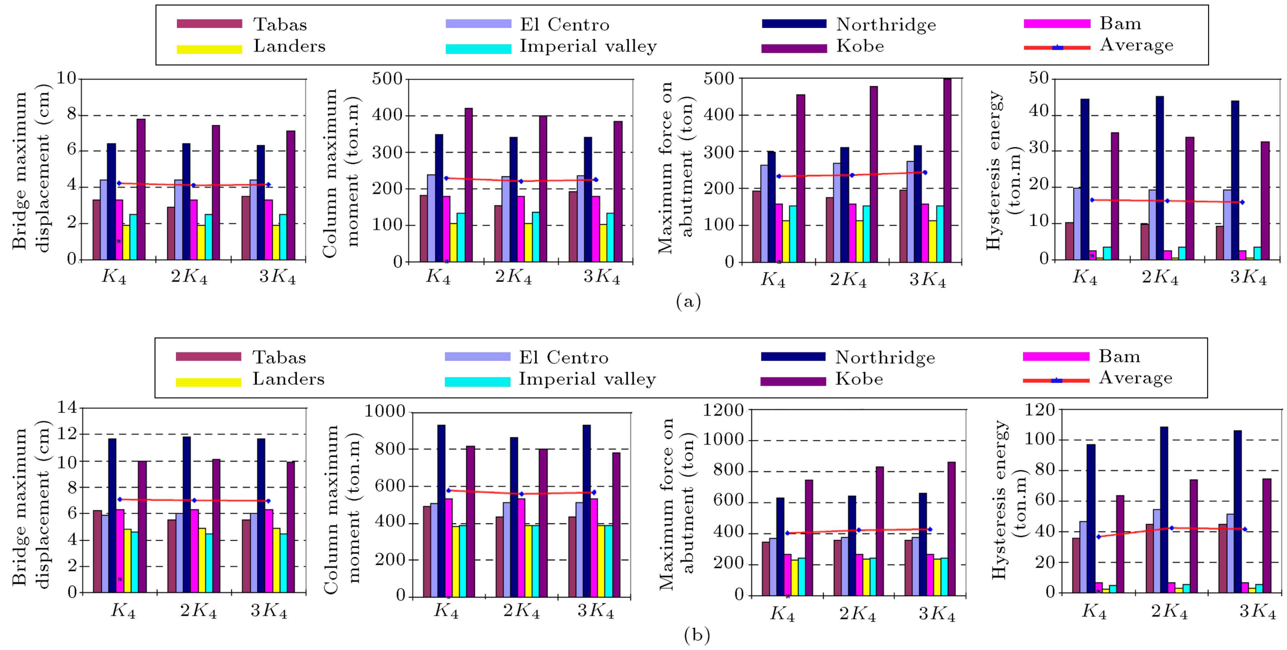

Figure 9. Sensitivity of bridges seismic responses considering progressive increase in the stiffness of spring K_4: (a) Dalichai-2; and (b) Dalichai-3.

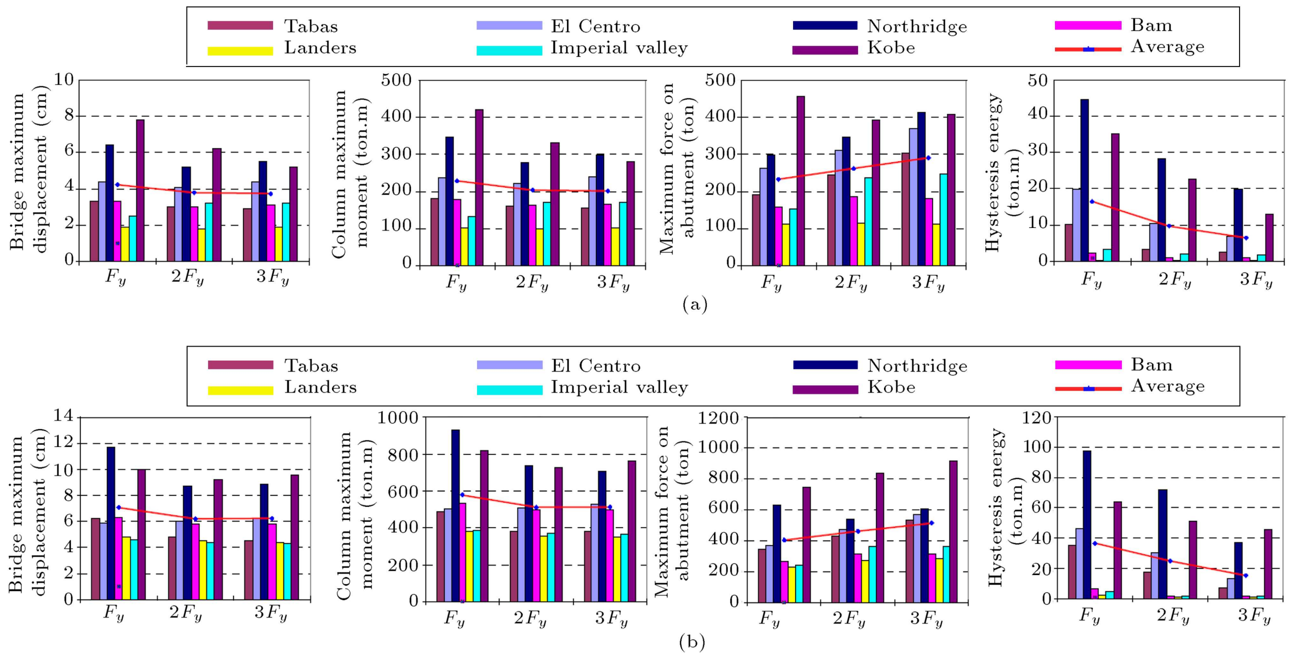

Figure 10. Sensitivity of bridges seismic responses considering progressive increase in F_y: (a) Dalichai-2; and (b) Dalichai-3.

properties of restrainer components are changed. In other words, the response is more sensitive to the restrainer properties for stronger earthquakes. As an example for verification, referring to Figures 6 to 10, the hysteresis energy of the two studied bridges when subjected to the Kobe earthquake is drastically changed when the parameters of the restrainer have been modified, as compared to the Landers and Imperial Valley earthquakes. One can conclude that the rate of change is greater in the plastic zone than in the elastic region. It is obvious that plastic behavior is more likely to occur under stronger earthquakes.

3.4. Condition assessment and retrofit study

The structural behaviors of the existing and retrofitted bridges were numerically investigated. The engineering drawing documents for the bridges (prepared more

Table 1. Seismic response of the Dalichai-2 bridge (existing condition).

Earthquake name	**Pounding force (ton)**	**Top of column displacement (cm)**	**Column moment (ton.m)**	**Column shear force (ton)**	**Hysteresis energy (ton.m)**	**Bearing D/C**
Tabas	0	10.2	552	33	10.3	7.8
El Centro	656	8.6	485	29	10.9	8.8
Northridge	154	13.6	765	46	11.8	6.4
Bam	678	12.3	659	40	7.36	5
Landers	640	16.4	886	53	11.49	7.4
Imperial Valley	390	12.1	652	39	8.9	8
Kobe	402	9.8	533	32	10.7	6.3

Table 2. Seismic responses of the Dalichai-3 bridge (existing condition).

Earthquake name	**Pounding force (ton)**	**Top of column displacement (cm)**	**Column moment (ton.m)**	**Column shear force (ton)**	**Hysteresis energy (ton.m)**	**Bearing D/C**
Tabas	0	9.5	808	52	19.3	9.5
El Centro	0	8.2	700	45	19.7	8.9
Northridge	1118	11	942	60	27.8	6.9
Bam	1044	12.1	963	59.5	15.1	4.3
Landers	978	11.1	881	54.3	20.8	14.8
Imperial Valley	1422	11.5	982	62.8	21.9	9.6
Kobe	170	8.4	666	41.3	17.2	8.7

than 30 years ago) mention that the strength of the concrete was 30 MPa, the yield strength of the rebar was 400 MPa and the yield strength of the steel profiles was 360 MPa. These values were used in the analyses in this research.

Tables 1 and 2 show the seismic responses of the bridges along their longitudinal direction. It can be observed from the tables that in all cases of ground motion, the elastomeric bearing failure has resulted in a total collapse of the bridges. Table 1 corresponds to Dalichai-2 and Table 2 to Dalichai-3.

It is clear from the last columns of the tables that the elastomeric bearings experience a high level of displacement demand in contrast to their capacity. Because of the weak performance of the elastomeric bearings, the other elements do not have a chance to show plastic behavior. Hence, eventually, the columns and girders remain elastic during the seismic motion. The amount of column moments and shears (which registered a maximum moment of 982 ton.m, less than the yield moment of about 1120 ton.m, as well as a maximum shear of 62.8 ton, much less than the yield shear of about 1100 ton) can be the acceptable proof.

From the zero value for the pounding force in the tables, it can be concluded that, despite the strong excitation from the El Centro earthquake, influencing the Dalichai-3 and Tabas earthquakes and both bridges, the displacements have not exceeded the gap width between the decks and, thus, no impact has occurred. This may prove the idea that the impact between the decks is highly dependent on ground motion characteristics, as well as the natural period of the bridges.

The primary factors affecting the pounding response in adjacent frames are identified as the frame stiffness ratio or period ratio and the ground motion effective period ratio (ratio of period of frame to period of ground motion). Pounding reduces the frame response when vibration occurs near the characteristic period of the ground motion. Investigations of two-sided pounding using MDOF models have shown a favorable post impact response for the flexible frame and a detrimental effect for the stiff frame demand for all period ratios [12].

In summary, evaluation of the seismic responses of the bridges showed significant vulnerabilities in the bridges responses. Furthermore, the decks displacement in the longitudinal direction resulted in the elastomeric bearing deformation demands that generally exceeded the limit.

To assess the performance of the proposed restrainer element and to compare it with the performance of the ordinary restrainers, the bridges were subjected to the same above mentioned ground motion records after being retrofitted with conventional as well as compound restrainers.

Typically, the restrainers used in the retrofit studies are 3/4 inch (19 mm) diameter steel cables with an area of 143 mm^2, made of 6×19 strands, galvanized with a wire strand core, a right regular lay, and made of improved plow steel. The restrainer assembly is composed of cables with swaged fittings, studs, nuts and turnbuckles all of which should be 25% stronger than the cable. Under cyclic loading, the cables have shown a yield strength of 174 kN, which corresponds to a yield stress of 1210 MPa and an initial modulus of elasticity of 69,000 MPa. The ultimate strength per cable is 235 kN [12].

In this study, 20 foot long, 3/4 inch diameter cables that stretch approximately 4.22 inches at yield are considered as the conventional restrainer. The slack of the cables is ignored. The restrainers are modeled at the intermediate and external hinge locations, using spring elements that resist only tensile forces.

Tables 3 and 4, respectively, show the seismic responses of the Dalichai-2 and Dalichai-3 bridges retrofitted by conventional restrainers. Because of the considerable weakness seen in the bearings, the elastomeric bearings acceptance criterion governs the number of restrainers required to put the bearing's deformation within allowable limits, in all cases of ground motion.

It is clear that restrainers have a considerable influence on reducing bridge seismic responses, such as bridge drift, column moment and shear forces. Tables 1 and 3 show the minimum and the maximum diminution, about 49 and 88 percent, respectively, for maximum displacement, column moment and shear in the Dalichai-2 bridge for the retrofitted structure in comparison with their equal responses under existing conditions. Considering Tables 2 and 4, similar comparison for the Dalichai-3 bridge results in a minimum and maximum decrease of about 0 and 71 percent, respectively. It also inhibits the pounding between decks. Reviewing the tables for "Maximum Force on Abutments", it is noticeable that the need for numerous restrainers to restrict the bearing deformation imposes considerable forces on the abutments. By evaluating the ductility ratios of restrainers, it is obvious that they remain elastic during seismic motion; somehow registering a maximum ductility ratio of about 0.55. Although this satisfies the main objective of restrainer

Table 3. Seismic responses of the Dalichai-2 bridge retrofitted using conventional restrainers.

Earthquake name	Top of column displacement (cm)	Column moment (ton.m)	Column shear force (ton)	Maximum force on abutment (ton)	Hysteresis energy (ton.m)	Bearing D/C	Restrainer ductility ratio
Tabas	3.6	195	11.7	270	1.9	0.62	0.21
El Centro	4.2	226	13.6	390.4	4.7	0.8	0.29
Northridge	6.3	342	20.5	540	9.6	0.98	0.4
Bam	2.3	122	7.3	216	1	0.46	0.17
Landers	2	110	6.6	131	0.3	0.25	0.12
Imperial Valley	4.1	222	13.3	273.6	2.2	0.6	0.22
Kobe	5	275	16.5	378.2	9	0.82	0.3

Table 4. Seismic responses of the Dalichai-3 bridge retrofitted using conventional restrainers.

Earthquake name	Top of column displacement (cm)	Column moment (ton.m)	Column shear force (ton)	Maximum force on abutment (ton)	Hysteresis energy (ton.m)	Bearing D/C	Restrainer ductility ratio
Tabas	6	476	29.5	472	3.5	0.65	0.31
El Centro	7	593	38	546	7.7	0.83	0.35
Northridge	12	954	59	756	11.6	1	0.49
Bam	4.8	412	26.4	331.6	1.4	0.46	0.25
Landers	4.5	357	22.2	252	0.77	0.44	0.197
Imperial Valley	3.7	295	18.2	284	1.05	0.42	0.22
Kobe	11.4	974	62	950	13.8	1	0.55

Table 5. Seismic responses of the Dalichai-2 bridge retrofitted with compound restrainers.

Earthquake name	Top of column displacement (cm)	Column moment (ton.m)	Column shear force (ton)	Maximum force on abutment (ton)	Hysteresis energy (ton.m)	Bearing D/C	Restrainer ductility ratio
Tabas	3.3	181	10.9	192	10.2	0.67	2.45
El Centro	4.4	238	14.3	262	19.8	0.85	3.68
Northridge	6.4	347	20.8	298	44.5	1	4.18
Bam	3.3	179	10.8	157.8	2.4	0.44	1.82
Landers	1.9	103	6.2	111.6	0.32	0.29	1.00
Imperial Valley	2.5	132	8	153.6	3.3	0.56	1.73
Kobe	7.8	420	25.3	455	35	1	6.36

Table 6. Seismic responses of the Dalichai-3 bridge retrofitted with compound restrainers.

Earthquake name	Top of column displacement (cm)	Column moment (ton.m)	Column shear force (ton)	Maximum force on abutment (ton)	Hysteresis energy (ton.m)	Bearing D/C	Restrainer ductility ratio
Tabas	6.2	490	30.2	346	35.6	0.95	3.05
El Centro	5.9	504	32.3	368	46.6	0.96	3.36
Northridge	11.7	928	57.2	630	97.3	1	5.55
Bam	6.3	534	34.3	268	6.6	0.74	2.09
Landers	4.8	381	23.5	234	2.7	0.56	1.64
Imperial Valley	4.6	388	24.9	245.4	5.1	0.52	1.82
Kobe	10	817	52.4	746	63.8	1	6.55

design to perform elastically, it substantially decreases the capability of the structure to dissipate the hysteresis energy during earthquakes. It can be proved by comparing the amount of energy dissipated in the bridges retrofitted by conventional restrainers with the dissipated energy in existing bridges.

In another attempt to improve the seismic performance of the Dalichai-2 and Dalichai-3 bridges, they were retrofitted with the compound restrainers explained before. Hence, the cable restrainers (similar to the ones used in conventional restrainers) have been assembled in such a way that they form a device like the compound restrainer introduced in the previous sections. The same number of cables is used to construct the springs, K_1, K_2 and K_4, each ten feet long. The amount of slack for spring K_4 was assumed 1 in. In order to model spring K_3, several steel cables of grade ST37 (F_y = 240 MPa, E = 200000 MPa) were used to account for the energy dissipation capacity of the device and to represent elasto-plastic elements with comparatively moderate yield stress and very high elastic modulus, as well as considerable plastic strain capacity. The number of steel cables was determined in a process of trial and error through dynamic analyses until the seismic performance of the bridges could satisfy the acceptance criteria, particularly the criterion on the allowable deformation of bearings. It resulted in an augment of about 30 percent in the total length of the cables needed for restrainers.

Tables 5 and 6, respectively, show the seismic responses of the Dalichai-2 and Dalichai-3 bridges retrofitted with the compound restrainers. It can be observed that in addition to the capability of the compound restrainers to dissipate more seismic energy, compared to conventional restrainers, their use has also resulted in a reduction in forces imposed on structural elements, like abutments. Comparing Tables 3 and 4 with Tables 5 and 6, respectively, it can be perceived that the maximum diminution of force applied on the abutment is about 25 percent for Dalichai-2 bridge, and about 20 percent for Dalichai-3 bridge when a comparison is made between bridges retrofitted with compound restrainers and bridges retrofitted with conventional restrainers. By evaluating the ductility ratios of the compound restrainers, it is obvious that they experience plastic deformation during seismic motion. Due to the contribution of the plastic part of the compound restrainers in the seismic performance of the bridges, the amount of energy dissipation increases considerably, especially during strong ground motion. For example, during the Northridge earthquake, it recorded hysteresis energy of about 45 ton.m for the Dalichai-2 bridge, and about 97 ton.m for the Dalichai-3 bridge. It is very desirable to decrease the load burden of columns, abutments and foundations, because, when conventional methods of retrofitting substructures are applied, such as jacketing and extending, not only does it make retrofitting hardly manageable, it also encounters difficulties in handling the problems of conducting or controlling the stream during the execution phase. The retrofitting with compound restrainers can compensate for the expenditures associated with the allocation of additional elasto-plastic cables.

4. Conclusions

This paper presents the results of a numerical simulation and parametric study for seismic evaluation and retrofit of two multi-span simply supported steel girder bridges. The retrofit plan is to use compound restrainers to restrain the decks of the bridges against excessive displacement which results in their sliding off their supports and causes catastrophic failure. Several nonlinear time history analyses of analytical models of the bridges for seven earthquake acceleration records were conducted and the results were used to evaluate the sensitivity of the performance of the restrained bridges to changes in the properties of components of the compound restrainers, and assess the reliability of the retrofit measures. The seismic performance of the compound restrainer is sensitive to changes in the stiffness and yield stress of its components. Stiffness augment in any component leads to a decrease in the force transferred to the columns (contrary to the abutments), as well as the structure's longitudinal displacement. In conclusion, moreover, to the changes in the properties of a component, its seismic performance is a function of the characteristics of ground motion and the natural period of the bridge. As an illustration, despite the strong excitation from the El Centro and Tabas earthquakes, the displacements have not exceeded the gap widths between the decks and, thus, no impact has occurred in the analyses.

The effectiveness of the compound element in dissipating seismic energy and reducing forces which are induced within the structural elements of the bridges under study, including their columns and abutments, in comparison with conventional restrainers, has been illustrated in two case studies of the Dalichai-2 and Dalichai-3 bridges. It can be perceived that the maximum diminution of the force applied on the abutment is about 25 percent for the Dalichai-2 bridge and about 20 percent for the Dalichai-3 bridge, when comparison is made between the bridges retrofitted with compound and conventional restrainers. Conversely, using compound restrainers, the amount of energy dissipation increases considerably, especially during strong ground motion. In summary up, noticing the results summarized in the tables and figures in this paper, it can be concluded that the proposed system offers a feasible solution that is simple and practical to implement for seismic retrofitting of existing bridges. The main advantage of using the proposed compound restrainers is to effectively reduce the load on the substructure of bridges.

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Appendix A

Property definition for bridge components

Elastomeric bearings. The elastomeric bearings are modeled using a bilinear element based on Kelly's model, with three parameters; elastic stiffness (K_1), strain hardening stiffness (K_2) and characteristic strength (Q), as shown in Figure A.1. Experimental tests on elastomeric bearings have shown that the ratio of K_1/K_2 is around 3.0 [13].

The effective stiffness of the bearings can be calculated as:

$$K_{\text{em eff}} = \frac{\text{GA}}{h}, \tag{A.1}$$

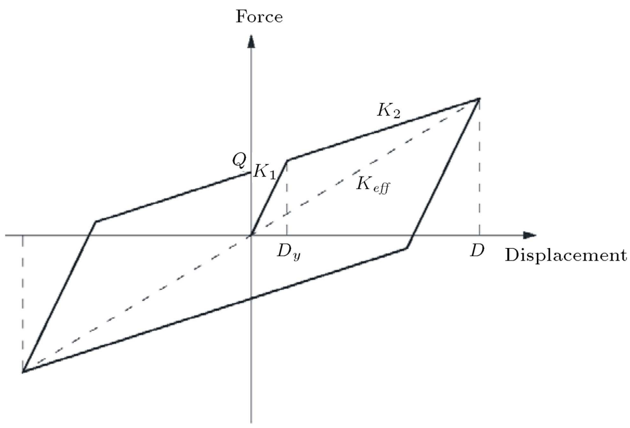

Figure A.1. Bilinear model for elastomeric bearings [13].

where A is the area of the elastomeric bearing, G is the shear modulus of the elastomer taken as 100 psi, and h is the height of the elastomer. The effective stiffness can be related to other parameters, as shown below:

$$K_{eff} = K_2 + \frac{Q}{D}, \tag{A.2}$$

where D is the maximum design deformation in the bearing, typically taken equal to the height of the elastomer. The yield displacement can be expressed in terms of the primary parameters as [13]:

$$D_y = \frac{Q}{K_1 - K_2}, \tag{A.3}$$

The yield displacement is typically taken equal to one-tenth the maximum deformation (D) [13]. Thus, all the primary parameters can be calculated from Eqs. (A.1)-(A.3) given the bearing dimensions. In this study, the elastomeric bearings are modeled at the intermediate hinge and abutment locations. Table A.1 presents the properties of the elastomeric bearings used herein.

Pounding. Because the characteristics of expansion joints have a major influence on the seismic response of bridge structures, they must be correctly modeled. The existence of the gap introduces nonlinearity into the seismic analysis of the structure. An analytical model of the expansion joints that takes account of the effect of pounding is developed. The external nodes of adjacent segments are linked by nonlinear gap elements to model the impact forces resulting from collision. The model recommended by Muthukumar has been used in this study [14]. The force-deformation characteristics of such an element are shown in Figure A.2.

The derivation of the parameters, K_{t1} and K_{t2}, uses the energy dissipated upon impact in comparison with the area in the hysteresis. By equating these two values and assuming a maximum deformation, δ_m, these stiffnesses are calculated. In this study, the maximum deformation or penetration, δ_m, is assumed to be 25.4 mm, and δ_y is assumed to be $0.1\delta_m$. Following this assumption, K_{t1} = 68.8 ton/m and K_{t2} = 23.7 ton/m for the Dalichai-2 bridge, and K_{t1} = 73 ton/m and K_{t2} = 25.1 ton/m for the Dalichai-3 bridge.

The parameters, K_{t1}, K_{t2}, δ_y and δ_m, for the impact model are calibrated to the total expected energy loss, ΔE, during an impact event. Using

Table A.1. Properties of elastomeric bearings.

Dimension (cm)									
L	B	h	G (kg/cm^2)	D (cm)	D_y (cm)	K_{eff} (kg/cm)	K_1 (kg/cm)	K_2 (kg/cm)	Q (kg)
52	52	12.8	7	12.8	1.28	1478	3697	1232	3155

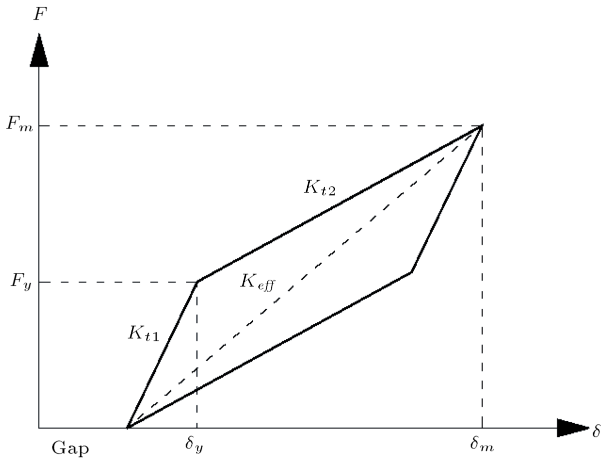

Figure A.2. Analytical model of impact between decks.

a stereo-mechanical approach, the energy dissipated during impact can be derived and written as [14]:

$$\Delta E = \frac{k_h \delta_m^{n+1}(1-e^2)}{n+1}, \tag{A.4}$$

n is the Hertz coefficient, typically taken as 3/2, and e is the coefficient of restitution with a typical range of 0.6-0.8 [14]. ΔE is energy dissipated, and k_h is the impact stiffness parameter = 1770000 kg.cm$^{-3/2}$ for the Dalichai-2 bridge and 1880000 kg.cm$^{-3/2}$ for the Dalichai-3 bridge, which are calculated following the Hertz model.

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