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YIELDING STEEL DAMPERS TO REPLACE CONCRETE SHEAR KEYS FOR SEISMIC RETROFIT OF BRIDGES WITH UNEQUAL HEIGHT PIERS

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Abstract

Irregular multi-span continuous bridges with unequal height piers are commonly adopted in mountainous areas when crossing steep-sided valleys. As common practice, economical unbonded laminated rubber bearings and concrete shear keys are usually installed in such bridges as superstructure-to-substructure connections. In strong earthquakes, the sliding of rubber bearings and the shear key failure are mostly expected, which may weaken the seismic resistance of the irregular bridges. The current study is mainly divided into three parts: the first one investigates the effect of concrete shear keys on the transverse seismic performance of the irregular bridges by incorporating the potential sliding of laminated rubber bearings; the second part proposes the concept of using yielding steel dampers to replace shear keys for mitigating the irregular seismic responses of the unequal height bridges with the practical design method being developed; the third part demonstrates the retrofit effectiveness by conducting nonlinear time-history analyses. The results indicate that the existence of concrete shear keys increases the bridge irregularity by inducing a significant in-plane superstructure rotation response around the vertical axis, posing nonuniform displacement demands at the substructures. The shear keys also contribute to the unbalanced ductility demands on the unequal height substructures, rendering the taller piers more vulnerable to the seismic damages. This differs from the case with fixed bearings, where the shorter piers are more subjected to damages. Using yielding steel dampers that are appropriately designed as per the proposed method, the seismic rotations of the bridge superstructure can be considerably mitigated with a relatively uniform superstructure movement maintained. The steel dampers are also effective in distributing the inertia forces among the unequal height piers. The effectiveness of yielding steel dampers replacing concrete shear keys in unequal height irregular bridges is successfully verified by the results from the numerical simulation.

Keywords: irregular bridges; unequal height piers; concrete shear keys; yielding steel dampers; seismic irregularity mitigation

1. Introduction

Bridges constitute a substantial portion of national wealth in most countries worldwide. They are critical parts of transportation systems by providing effective ways to overcome obstacles like rivers, valleys and intersecting roads. In mountainous areas, multispan continuous bridges with unequal height piers are commonly built since they can be well suited for the rugged terrain. As common practice, the unequal height piers are usually designed with identical cross sections for aesthetic considerations [1]. This renders the shorter piers more vulnerable than the taller ones to earthquake damages since a larger stiffness attracts more force demand, particularly when fixed girder-pier connections (e.g., fixed bearings) are assumed. In actual cases, however, economical unbonded laminated rubber bearings and shear keys are installed in parallel at bridges in order to provide such a fixed constraint. This practice is reasonable when the shear key strength is adequately large to resist the inertia force of superstructure. The seismic performance of shear keys and laminated rubber bearings has been examined during the past major earthquakes like the 1999 Chi-Chi earthquake [2] and the 2008 Wenchuan earthquake [3]. In the Wenchuan earthquake, the sliding of laminated rubber bearings and the shear key failure were extensively observed, leading to excessive superstructure displacements (Fig. 1). Most investigated bridges failed to behave as fixed-constraint systems as expected during the earthquake. Instead, these bridges showed quasi-isolated seismic responses, protecting the substructure from extensive damages. For irregular bridges with unequal height piers, the fusing of rubber bearings and shear keys may pose some effect on the bridge irregularity during earthquakes, which may differ from the common cases like those with fixed bearings. It is essential to reveal such effect and propose novel retrofit measures to mitigate the seismic irregularity of the unequal height bridges.


(a) (b) (c)

Fig. 1 – Typical bridge damages in 2008 Wenchuan earthquake: (a) bearing sliding, (b) shear key failure, and (c) excessive superstructure displacement

This paper investigates the effect of the conventional design practice on the seismic response of irregular bridges with unequal height piers and proposes new design strategies for irregularity mitigation, which is divided into three parts. First, a six-span continuous highway bridge with unequal height piers located in the mountainous area of China is selected as prototype. By establishing the refined FEM of the bridge, the effect of concrete shear keys and rubber bearings on the bridge irregular seismic responses is investigated by conducting nonlinear time history analyses. Second, considering the seismic deficiencies of the current design practice, yielding steel dampers are proposed to replace conventional shear keys at the irregular bridges as transverse restraining devices along with laminated rubber bearings. The design criteria and steps are developed specifically for the new restraining systems. Third, the effectiveness of using yielding steel dampers in mitigating seismic irregularity of unequal height bridges is analyzed. The efficiency of the proposed design method is also examined.

2. Selected prototype and its modelling

The selected prototype is a six-span continuous highway bridge with an identical span length of 25 m (Fig. 2). The bridge superstructure consists of five prestressed concrete T-shaped girders that are supported upon five laminated rubber bearings at each abutment or pier. Note that the intermediate piers are equipped with conventional laminated rubber bearings, whereas the abutments are installed with polytetrafluoroethylene (PTFE) sliding bearings. The substructure is composed of two end abutments and five unequal height double-column piers. The effective heights of the unequal piers from P1 to P5 are 5 m, 9 m, 9 m, 6 m, and 3 m, respectively. The minimum ratios of effective lateral stiffness between any two piers and between adjacent piers are estimated as $(3/9)^3$=0.037 and $(3/6)^3$=0.125, respectively, much smaller than the suggested values of 0.5 and 0.75 as specified in AASHTO [4] and Caltrans [5]. Thus, the bridge prototype can be regarded as an irregular bridge according to the evaluation criteria of AASHTO and Caltrans. In the transverse direction, exterior shear keys are constructed at both sides of the cap beams to provide lateral restraint for the superstructure. The laminated rubber bearings are directly inserted between the superstructure and the substructure without connections detailed except friction at the contact surfaces. This indicates that the potential bearing sliding is very likely to occur under strong earthquakes, especially after the shear keys failure. The bridge substructures rest on rock strata through pile foundations.

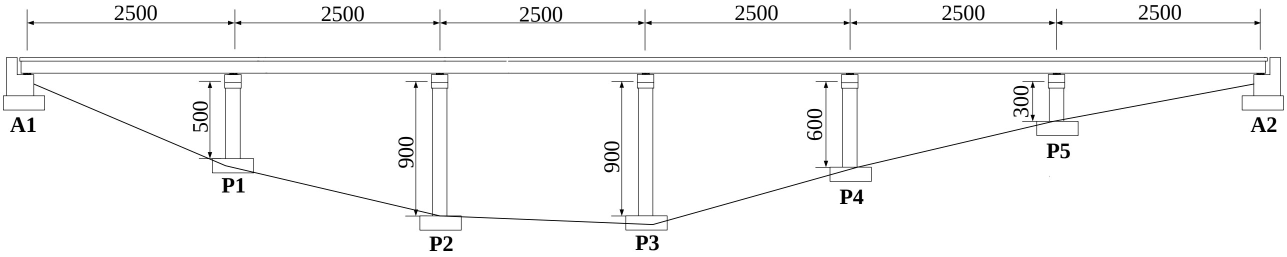
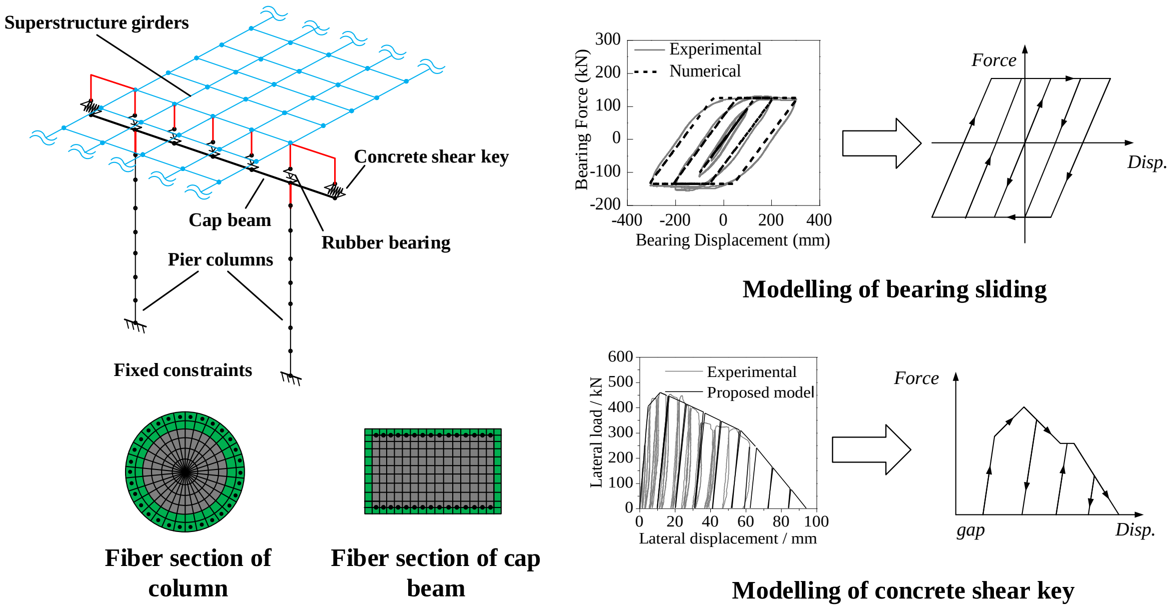


Fig. 2 – Selected prototype: an irregular continuous bridge with unequal height piers

Finite element model of the bridge prototype is generated in OpenSees by incorporating various component nonlinearities from RC pier columns, laminated rubber bearings, and concrete shear keys (Fig. 3). Since it can be expected that the superstructure including T-girders and diaphragms behave elastically during earthquakes, its seismic behavior is modelled using elastic elements. The pier columns and cap beams are essential earthquake resistant members that are prone to nonlinear deformations. The seismic behaviors of such components are modelled by the fiber-based nonlinear elements in OpenSees. The cross sections are divided into numerous small fibers representing the uniaxial stress-strain behaviors of unconfined concrete, confined concrete, and reinforcing steel bars. The constitutive behaviors of both unconfined and confined concrete are simulated using the Concrete01 material in OpenSees, except that the confinement effect is incorporated in the model for the confined concrete. The behavior of the steel reinforcement is modelled using the Steel02 material that is the Giuffré-Menegotto-Pinto model with isotropic strain hardening. The potential sliding response of laminated rubber bearings is modelled using a bearing sliding model, which is defined by initial stiffness, vertical load and coefficient of friction. For a single prototype bearing, the initial stiffness and the coefficient of friction are taken as 2125 kN/m and 0.35 at intermediate piers, and 2125 kN/m and 0.06 at end abutments. When the bearing displacement is smaller than the critical sliding displacement, the bearing shows linear elastic responses. After the critical sliding displacement is exceeded, the bearing sliding occurs with the response hysteresis becoming wide, showing substantial energy dissipation. For modelling concrete shear keys, the conventional practice is that the effect of shear keys is either neglected or regarded as linear elastic response without failure. Although this simplifies the numerical analysis, underestimation or overestimation of seismic responses may be induced, especially under strong earthquakes, where the shear key damage or failure can easily occur. Hence, the current study utilizes a multi-linear hysteresis model to capture the shear key response from pre-peak to post-peak until total collapse [6]. The nonlinear models of the bearing sliding and the shear keys were successfully validated by the previous experimental data.



Modelling of bearing sliding

Fiber section of column

Fiber section of cap beam

Modelling of concrete shear key

Fig. 3 – Finite element modelling of prototype bridge in OpenSees

3. Earthquake ground motions

In order to be compatible with the design earthquake intensity, a total of 20 earthquake records are first randomly selected from the PEER database and then modified for spectrum matching. Fig.4 shows the spectral accelerations from the individual records, mean record, and the design earthquake. It can be seen from Fig. 4 that the mean spectrum from the modified records matches well with the design spectrum, indicating that the mean responses from the individual records can well represent the bridge performance under design-level earthquakes. Since the main purpose of this paper is to investigate the effect of transverse shear keys on the seismic performance of irregular bridges, the selected records are input in the bridge model only in the transverse direction.

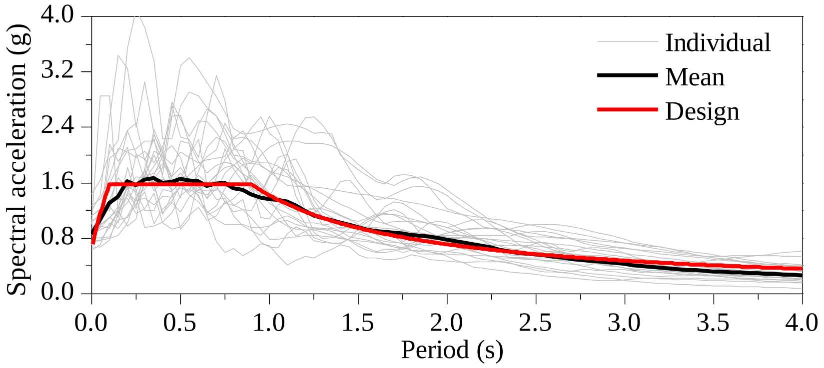


Fig. 4 – Design acceleration spectrum compatible real earthquake records

4. Effect of shear key failure on bridge irregular response

To demonstrate the effect of concrete shear keys on the irregular responses of the unequal height bridge, the shear key strength at the intermediate piers is chosen as a variable for parametric analysis. The strength of the shear keys is normalized by the dead-load reaction (DLR) at the piers, which is taken as 20%, 40%, 60%, 80%, 100%, 120%, 140%, 160%, 180%, 200% DLR, individually. To provide the upper and lower bounds for the parametric analyses, two special cases are added, which are the ones without shear keys at piers and with elastic shear keys at piers. These two cases are commonly assumed in actual practice when the shear keys are designed as weak fuses and as strong enough, respectively.

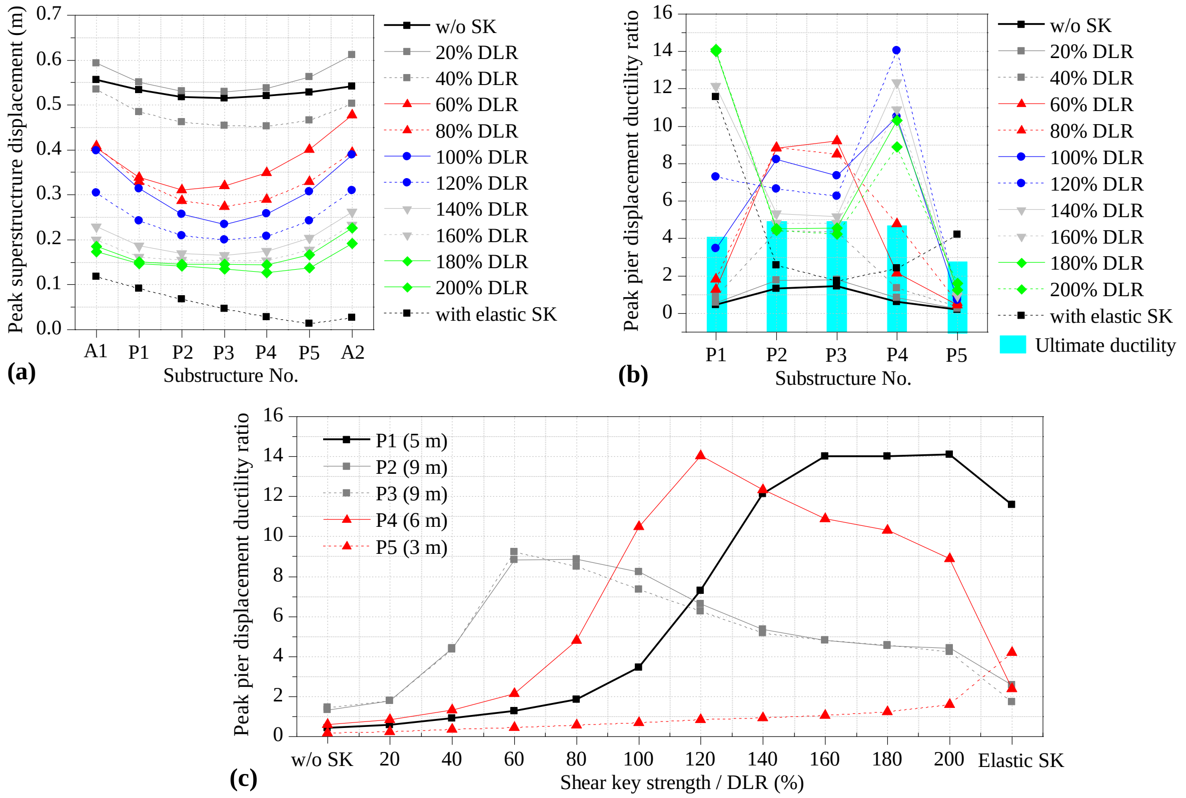


Fig. 5 – Effect of shear key strength on bridge seismic responses: (a) peak superstructure displacement, (b) and (c) peak pier displacement ductility ratio

Fig. 5 shows the variation of the peak seismic responses of the irregular bridge averaged from the selected ground motions with the variation of shear key strength at intermediate piers. The peak girder displacement and the peak pier ductility ratio are given since they are essential seismic responses for a bridge structure. It can be seen from Fig. 5a that for all the analyzing cases with and without shear keys and with elastic shear keys, the superstructure girders more or less display nonuniform peak displacements at the supports from A1 to A2, which indicates that the in-plane rotation of the superstructure around the vertical axis always occurs under the transverse earthquake excitations. Taking the case of 60% DLR as an example, it is shown that smallest peak displacement occurs at P2 whereas the largest one is found at A2. The difference of superstructure displacement between P2 and A2 reaches 53% (0.311 m versus 0.478 m). The superstructure rotation response poses unbalanced risks of unseating at different supports by inducing unbalanced displacement demands. It is shown from Fig. 5a that with elastic shear keys, the abutment at the side of the taller piers (A1 for this prototype) displays larger displacements than the abutment at the other side (A2). This makes sense since more displacement demands will be concentrated at the side of more flexible piers when the girder-pier connections are taken as elastic. For the case without shear keys, the superstructure seems to have a slight rotation with minor displacement differences at different supports. When installed with concrete shear keys, significant rotations of superstructure occurs, especially at the shear key strengths of 60%-100% DLR. This demonstrates that the existence of shear keys may intensify the seismic responses of superstructure in-plane rotation. Fig. 5b shows the peak pier ductility ratios of different piers for different shear key strengths. It is seen from the plot that without shear keys, all the piers remain un-collapsed with the peak ductility ratios lower than the ultimate ductility. With the assumption of elastic shear keys, the two shorter piers (P1 and P5) totally collapse while the taller ones including P2, P3, and P4 do not fail. This just verifies the common sense that shorter piers are more prone to failure than taller ones as they attract more force demands due to the stiffer characteristics. With shear keys of various strengths, the failure modes of bridge piers vary with the magnitude of shear key strength. For low shear key strengths like 20% and 40% DLR, the piers show a same damage mode as the case without shear keys, where the ductility ratios of all the piers remain lower than the ultimate ductility capacities. For medium shear key strengths from 60% to 80% DLR, the taller piers including P2 and P3 collapse whereas the others not. For the shear key strengths from 100% to 160% DLR, all the piers except the shortest one (P5) suffer from failure. When the shear strength reaches up to 180% and 200% DLR, the two taller piers (P2 and P3) and the shortest pier (P5) do not collapse while the two medium-height piers (P1 and P4) show much larger ductility ratios than the ultimate ductility.

The variation of failure modes of bridge piers with the strength of shear keys can be well illustrated by the force redistribution mechanism. Fig. 5c plots the variation curves of the pier ductility with the increase of shear key strength. It can be seen from Fig. 5c that the ductility ratios of all the bridge piers except the shortest P5 first increase and then decrease but with different phases. The taller piers show a faster variation rate with the peak ductility ratio arising earlier than those of the shorter piers. The different phases of the pier ductility ratios just demonstrate the variation of force redistributions. For instance, after exceeding 60% DLR of shear key strength, the ductility ratios of P2 and P3 start to decrease, while simultaneously, the values of P1 and P4 increase significantly. This indicates that more seismic demands are being redistributed from P2 and P3 to P1 and P4. When reaching up to 200% DLR, all the piers except P5 begin to display a decreasing ductility ratio. Oppositely, the ductility ratio of P5 increases in order to make up for the reductions of the ductility ratios at the taller piers. But during the whole range of shear key strengths, the ductility ratio of the shortest pier (P5) is much less sensitive than the other bridge piers. This differs from the case with elastic pier-girder connections, where the shorter piers are generally more susceptible to seismic loads. Such a force redistribution mechanism can be attributed to the coupling effects from bearing sliding, shear key failure, as well as the irregular configuration of bridge piers.

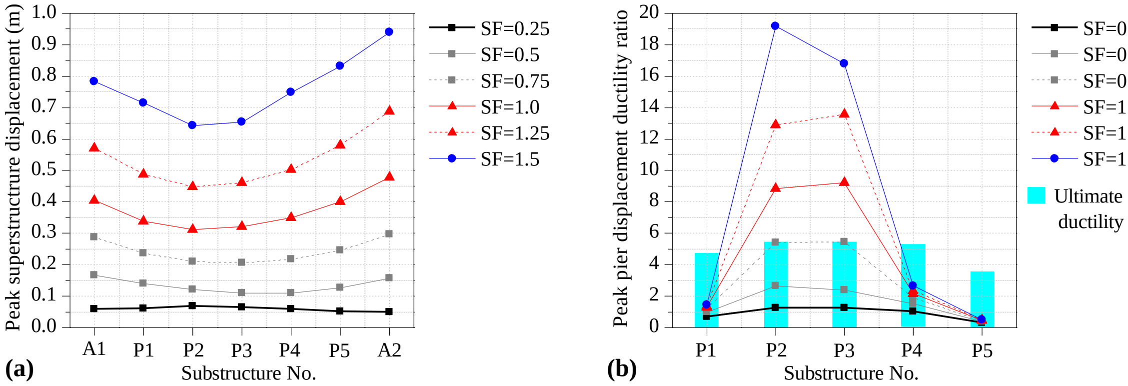
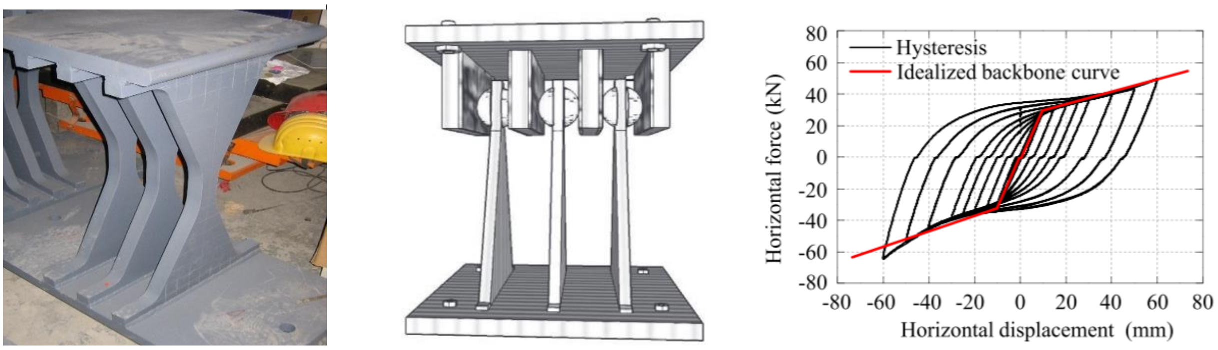


Fig. 6 – Effect of earthquake intensity on bridge seismic responses (taking 60% DLR as an example): (a) peak superstructure displacement, (b) and (c) peak pier displacement ductility ratio

In order to investigate the effect of earthquake intensity on the irregular seismic responses of the bridge with sacrificial concrete shear keys, a range of scale factors (SFs) are applied to the design-level earthquake to generate a coarse incremental dynamic analysis [7], where SF=1.0 indicates the design earthquake as given in Fig. 4. Fig. 6 shows the seismic responses of the irregular bridge at different levels of earthquake intensity. It can be seen from Fig. 6a that with the increase of the earthquake intensity, the differences of superstructure displacement at different supports become remarkable, indicating that the earthquake intensity can intensify the superstructure rotation responses of the irregular bridge with concrete shear keys. For instance, the difference of superstructure displacement between P2 and A2 is 16.6 cm at SF=1.0, whereas it reaches up to 29.6 cm at SF=1.5. Regarding the peak pier ductility, it is shown in Fig. 6b that the taller piers are more sensitive to the earthquake intensity than the shorter ones. With the increase of the SFs, the peak displacement ductility ratios of P2 and P3 increase faster than those of P1, P4, and P5, exceeding the ultimate ductility capacities when the SF becomes larger than 0.75. For the shorter piers like P1, P4, and P5, the piers do not collapse even when the earthquake intensity reaches to 1.5 times of the design level. This may be attributed to the fusing characteristics of shear keys and laminated rubber bearings, which can effectively isolate the shorter piers from extensive earthquake damages. Such a fusing mechanism occurs more easily at those shorter piers. It can be summarized from Fig. 6 that the increase of earthquake intensity can generally increase the seismic irregularity of the unequal height bridge with conventional shear keys.

5. Design of yielding steel dampers to replace concrete shear keys

The analysis results presented in the previous section just demonstrate that conventional shear keys are unable to balance the seismic demands of irregular bridges with unequal height piers, especially when the fusing characteristics of shear keys and laminated rubber bearings are considered. The progressive damage of shear keys makes them inconsistent in transmitting the inertia forces of the bridge superstructure, resulting in the significant rotations of superstructure and the unbalanced damage or failure of substructure piers. Hence, it is necessary to propose novel retrofit devices to overcome the limitations of conventional shear keys. Yielding steel dampers (Fig. 7), which are also called added damping and stiffness (ADAS) devices, are designed through the flexural yielding deformations of mild steel plates to dissipate earthquake energy and then mitigate the seismic demands of structures. Some of the distinct advantages of yielding steel dampers to replace concrete shear keys are summarized as: 1) initial stiffness of steel dampers can provide additional lateral supports the superstructure; 2) yielding of steel dampers does not affect the vertical load capacity of bridges since they are design separately as part of lateral resistant system; 3) steel dampers possess stable energy dissipation and force transmission capacities. Using yielding steel dampers to balance the seismic demands among different supports, the seismic irregularity of the bridges with unequal height piers can be considerably mitigated.



(a) (b) (c)

Fig. 7 – Typical yielding steel dampers and force-displacement hysteresis: (a) X-shaped, (b) triangular [8], and (c) tested hysteresis [9]

There are two essential issues required to be considered in the design of yielding steel dampers as a replacement of concrete shear keys: first is to maintain a relatively uniform superstructure movement to balance the displacement demands at different supports; second is to appropriately distribute the seismic demands among different substructures in accordance with their own resistant capacities.

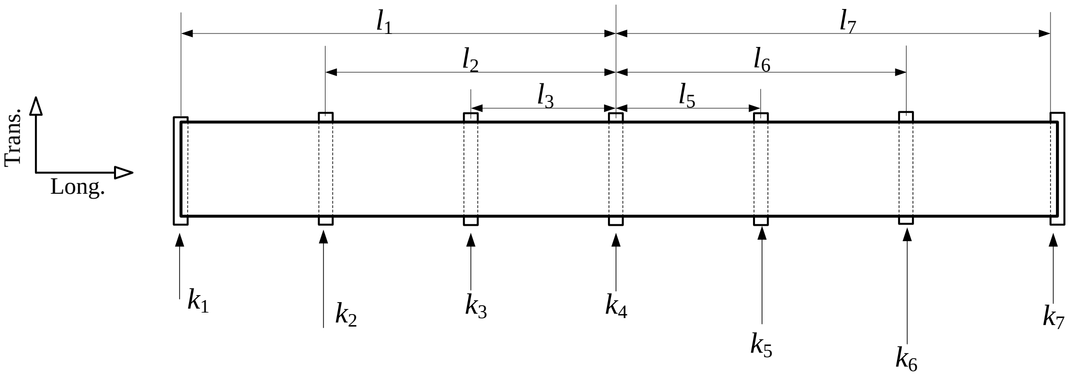


Fig. 8 – Graph of basic requirement for maintaining a uniform superstructure movement

It is assumed that the transverse rigidity of the bridge superstructure is strong enough that the superstructure moves like a rigid body under the transverse earthquake excitations. Since the superstructure is supported at two end abutments and five intermediate piers, the critical point for maintaining a uniform superstructure movement is to correlate the support stiffnesses of the abutments and the unequal piers. Fig. 8 plots the rigid superstructure and its support conditions. If the support stiffnesses are taken as elastic and the superstructure mass is uniformly distributed along the longitudinal direction of the bridge, the basic requirement for maintaining a uniform superstructure movement is that the mass centroid coincides with the stiffness centroid, as illustrated in detail in the study of Xiang and Li [10]. In other words, the stiffnesses of the supports should be well adjusted to ensure that stiffness centroid is located at the centroid of mass. The following equation should be satisfied for the current bridge prototype:

$$\sum_{i=1}^{3} k_i l_i = \sum_{j=5}^{7} k_j l_j \tag{1}$$

in which k_i and k_j are stiffnesses at different supports; l_i and l_j are distances between supports and mass centroid. Note that Eq. (1) is derived based on the assumption of elastic supports. For yielding steel dampers that are intended to deform in plastic ranges, Eq. (1) can be modified to account for this issue. It is obviously not appropriate to use the elastic stiffness of steel dampers in Eq. (1) since steel dampers are deemed to yield during earthquakes and enter post-yield response ranges. In this regard, effective secant stiffness is proposed to incorporate both the pre-yield and the post-yield responses, where Eq. (1) can be rewritten as:

$$\sum_{i=1}^{3} \frac{F_i}{d_i} l_i = \sum_{j=5}^{7} \frac{F_j}{d_j} l_j \tag{2}$$

where F_i and F_j are design forces at different supports, which are those maximum achieved displacements during earthquakes; d_i and d_j are design displacements at different supports. Since it is the design objective that the superstructure displacements at all the supports are approximately identical, d_i can be regarded equal to d_j. This leads to the eventual pattern of equation:

$$\sum_{i=1}^{3} F_i l_i = \sum_{j=5}^{7} F_j l_j \tag{3}$$

Another respect for designing steel dampers is to incorporate the individual capacities of each substructures. The design forces of steel dampers should be determined based on the lateral resistance of abutments and piers. For abutments, according to Caltrans, the design strength of abutment shear keys shall be no more than 100% of the dead-load reaction at abutments when spread abutment footings are adopted. For piers, the maximum transmitted force from superstructure to substructure shall be less than the yield force of piers. In the current study, the sum of the bearing sliding forces and the forces of the restraining devices like steel dampers are design as 85% of the yield force of the piers, according to Priestley et al. [11].

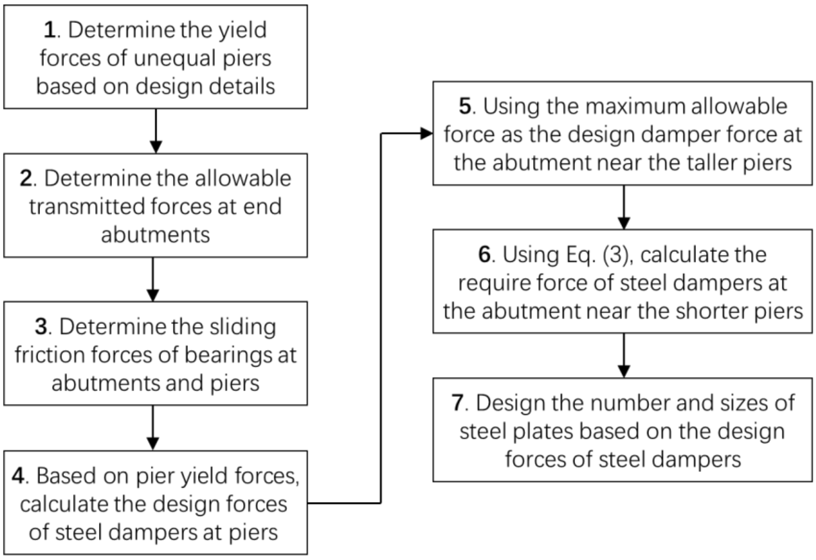


Fig. 9– Design chart of steel dampers for seismic irregularity mitigation of bridges with unequal height piers

Fig. 9 shows the design chart of yielding steel dampers in an irregular bridge with unequal height piers. The chart provides step-by-step design guidelines for how to easily select the appropriate values of damper parameters to mitigate the irregular seismic responses of bridges. The more detailed description of the design steps can be found in Xiang and Li [10], where a design example was selected and the design process was presented. For the current bridge prototype with six spans, specific design steps will not be given for the sake of page limitation. Using the design method presented in Fig. 9, the design yield forces of the yielding steel dampers at A1, P1, P2, P3, P4, P5, and A2 are calculated as 1776 kN, 1831 kN, 346 kN, 293 kN, 1188 kN, 3189 kN, and 450 kN, respectively. The yield displacement and post-yield hardening ratio of the steel dampers are taken as 0.02 m and 3%, respectively. The bilinear hysteretic model in OpenSees is utilized to model the seismic behavior of the steel dampers, whose accuracy was verified through the previous experimental study [9].

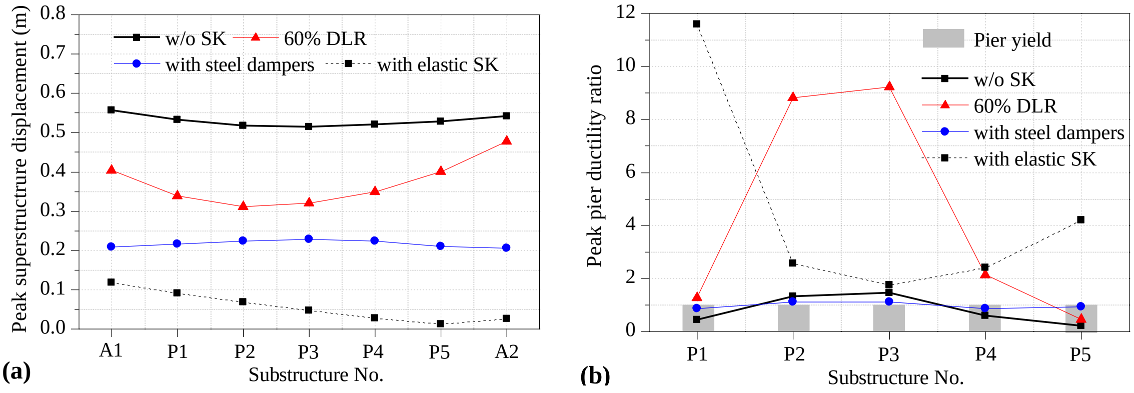


Fig. 10 – Seismic response comparison between the case with steel dampers and the other cases: (a) peak superstructure displacement, and (b) peak pier ductility

Fig. 10 shows the comparison of seismic responses between the case with properly designed steel dampers and other previously mentioned cases including those without shear keys, with conventional shear keys, and with elastic shear keys. Compared with the case without shear keys, the installation of steel dampers can considerably reduce the displacement demands of the bridge superstructure with an average reduction of 59%. In both the cases without shear keys and with steel dampers, the resulted ductility ratios of all the piers are found close to unity, indicating an essentially elastic seismic responses of piers. When compared with the case with elastic shear keys, although the superstructure displacements are increased by implementing the steel dampers, the ductility demands of piers can be well balanced and controlled around the yielding limit. The bridge with elastic shear keys shows extremely large ductility demands at the shorter piers. When the conventional shear keys and the steel dampers are compared, it can be found that both the superstructure displacements and the pier ductility demands can be uniformly distributed among different substructures when the irregular bridge is designed with the yielding steel dampers. The conventional shear keys considering failure characteristics during earthquakes may impose more ductility demands on those taller piers, which differs from the case with elastic shear keys.

The coarse IDA analysis mentioned in Section 4 is also applied to the case with steel dampers, where the IDA results are presented in Fig. 11. It is seen from Fig. 11 that the uniform superstructure movement can be basically maintained at different levels of earthquake intensity although the displacement increases with the increase of earthquake SFs. The peak pier displacement ductility ratios are also increased as the earthquake intensity increases. With the increase of earthquake intensity, the nonuniformity of pier displacement ductility ratios becomes more significant, with the larger ductility concentrated on the taller piers like P2 and P3. However, compared with the case with concrete shear keys shown in Fig. 6b, the ductility ratios are generally much lower with a maximum value of 1.85 for P2 at SF=1.5. In general, however, the installation of yielding steel dampers with properly designed parameters shows effectiveness in mitigating the seismic irregularity of the bridge with unequal height piers. The distribution of superstructure displacement and pier ductility demand can be well balanced among different supports by the steel dampers. Using the steel dampers, the unequal height piers can make full use of their individual capacities to resist earthquakes. For example, the shorter piers are expected to sustain more inertia loads than the taller ones due to the larger lateral force capacity of the shorter piers. The steel dampers can be well designed to distribute the inertia forces to the unequal height piers in accordance with their capacities.

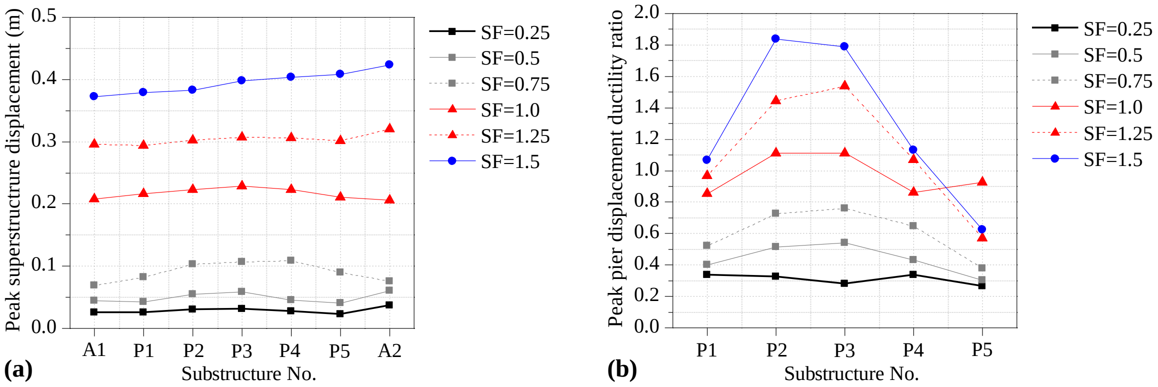


Fig. 11 – Effect of earthquake intensity on bridge seismic responses (taking 60% DLR as an example): (a) peak superstructure displacement, (b) and (c) peak pier displacement ductility ratio

6. Conclusions

This study investigates the possibility of using yielding steel dampers to replace conventional concrete shear keys for seismic irregularity mitigation of bridges with unequal height piers. A six-span T-girder continuous bridge with five unequal intermediate piers is selected as the prototype for analysis. The effect of conventional shear keys is first examined using parametric sensitivity analysis by focusing on the effects of shear key strength and earthquake intensity. Based on the demonstrated deficiencies of the shear key systems, yielding steel dampers are proposed as possible alternatives. The practical design method of the steel dampers for bridge seismic irregularity mitigation is developed, followed by nonlinear dynamic analyses as verifications. The current study points to the following conclusions:

(1) When the shear key failure and the bearing sliding are considered, significant in-plane rotation responses of superstructure can be induced, which poses unbalanced unseating risks at different supports.

(2) For medium shear key strengths, the shear keys may impose more ductility demand on the taller piers, which differs from the common sense that the stiffer shorter piers are more subjected to ductility demand.

(3) With the increase of the shear key strength, the ductility demand re-distribution mechanism can be observed at the unequal height piers. The taller piers are found to reach the peak ductility demands earlier than the shorter ones as the shear key strength increases.

(4) Using the yielding steel dampers to replace the conventional shear keys, the superstructure rotation responses can be significantly mitigated with a relatively uniform superstructure movement. The properly designed steel dampers are also able to help distribute the ductility demands among different supports according to their different lateral capacities.

(5) The advantages of the steel dampers over the conventional shear keys can also be found at different levels of earthquake intensities, where the uniform superstructure movements can be basically maintained and the peak displacement ductility demands of piers can be well controlled even under 1.5 times of the design-level earthquake.

7. Acknowledgements

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