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UNSEATING PREVENTION STRATEGY FOR BRIDGE SUPERSTRUCTURES-CURRENT STATE OF PRACTICE

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ABSTRACT

Past earthquakes worldwide have demonstrated without doubt that extensive bridge collapses are caused during earthquake due to unseating or dislodgement of spans. This has led to improvement in the seismic design codes for bridges in the recent past where codes now highlight the necessity of implementing effective unseating prevention measures. Current practice worldwide is to resort to passive unseating prevention measures for highway bridges against earthquakes, which primarily include minimum support length requirements and use of unseating prevention restrainers. Important aspects of these strategies in terms of their effectiveness, limitations, and future research needs in various national and international codes are summarized and discussed in this paper.

KEYWORDS: Minimum Support Length, Unseating, Pounding, Restrainers, Shear Key, Dampers

INTRODUCTION

Bridges constitute a substantial portion of national wealth. India has the second largest road network in the world, covering nearly 6 million kilometres. It is roughly estimated that on an average there will be 2 bridge/ culvert structure per kilometre, thereby a rough estimate of number of highway bridges in India is around 12 million [1]. In addition, India has the fourth largest railway network in the world by size, with a route length of 68,155 km and there will be a sizable number of railway bridges also existing. These bridges are playing a critical role in providing transportation passage over physical obstacles like rivers, valleys, intersecting roads, or railway lines. Failure of these bridges may pose a serious threat to public safety during extreme events like earthquakes since these bridges are life-line structures and are supposed to provide important links to hospitals, schools, and office buildings in the post-disaster scenario.

Past earthquakes have demonstrated that bridges suffered from a variety of structural damages, e.g., unseating, pounding, failure of bearings and connection devices, substructure collapses, etc. Among all these damage patterns, unseating of the span could be one of the most catastrophic damages, leading to a span falling or collapse. This highlights the necessity of implementing effective unseating prevention measures, which is intended for minimizing the possibility of bridge superstructure collapse as a result of unseating.

The effectiveness of bridge unseating prevention measures was first recognized by Japanese engineers when investigating the bridge damages in the 1964 Niigata earthquake. Unseating prevention seismic restrainers were then developed and introduced in the seismic retrofit of bridges, and incorporated in the 1971 Japanese bridge design code. After that, such a practice has been adopted all around the world. There are generally two kinds of passive measures taken to prevent seismic unseating for highway bridges. These are:

a. Minimum support length (MOL) requirements and

b. Unseating prevention restrainers.

By designing a minimum support length, the movement of bridge superstructure during earthquakes can be accommodated without the span falling. However, this measure is expected to be effective for minor to moderate earthquakes where superstructure displacements are not very large. When subjected to stronger than expected earthquakes, an excessively large displacement is expected, requiring a much larger support length. It is not feasible to construct an oversized substructure with large support lengths due to aesthetical and economic considerations. Passive unseating prevention restrainers are proposed to overcome the limitations of excessive support lengths by restraining seismic displacements.

This paper presents an overview of typical examples of bridge unseating in the recent earthquakes, guidelines on unseating prevention measures available in modern bridge codes, and characteristics of various unseating prevention restrainers. The primary objective of this paper is to provide readers with a good understanding of the state-of-the-art development of passive unseating prevention measures for bridges.

MINIMUM SUPPORT LENGTH REQUIREMENTS

1. Overview of Bridge Unseating in Past Earthquakes

Major earthquake events worldwide in last six decades witnessed substantial damages to bridges, where span unseating was recognized as one of the most severe damage patterns. Some of these major earthquake events are:

a. 1964 Niigata earthquake (Japan)
b. 1994 Northridge earthquake, California (USA)
c. 1999 Chi-Chi earthquake (Taiwan)
d. 2008 Sichuan earthquake (China)
e. 2010 Chile earthquake (South America)
f. 2011 Tohoku earthquake (Japan)
g. 2011 Christchurch earthquake (New Zealand)
h. 2013 Kashmir earthquake (India-Pakistan)
i. 2015 Nepal earthquake
j. 2016 Italy earthquake
k. 2017 Mexico earthquake

Fig. 1 1964 Niigata earthquake - unseating of deck and collapse of Showa Bridge due to liquefaction [19]

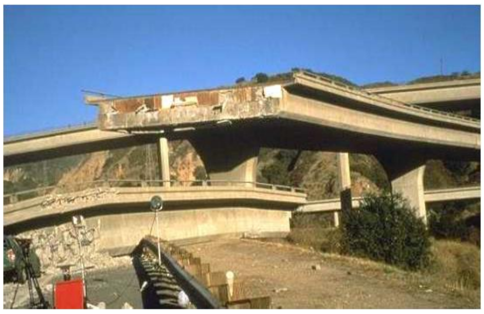


Fig. 2 Unseating at in-span hinge during 1994 Northridge earthquake [14]

Fig. 3 Bearing sheared resulting in displacement of the deck (1999 Chi-Chi earthquake) [15]

The structural configuration of collapsed bridges in many cases comprise of frame structures with in-span hinges, simply-supported structures, and continuous structures. On the basis of study of these earthquakes, researchers and academicians have concluded that most of the collapses in these catastrophic events were attributed to the span unseating. Simply-supported bridges were more vulnerable to unseating than other types of bridges during the earthquakes due to span discontinuity. The unseating of series of spans in Showa bridge, during 1964 Niigata earthquake is shown in Figure 1 [13]. Figure 2 shows unseating of articulation of a suspended span during the 1994 Northridge earthquake [14]. Figure 3 shows transverse displacement of deck due to shearing off of the bearings in 1999 Chi-Chi earthquake [15]. Figure 4 shows unseating of Ming-tsu bridge in 2008 Sichuan earthquake, China. This was a multiple-span simply supported structure with cast-in-place reinforced concrete (RC) girders. The fault rupture passing the south end of the bridge induced large superstructure displacements as well as severe pounding between the superstructure and abutment back-wall. Figure 5 shows a photograph of toppling of bridge in 2017 Mexico earthquake.

Fig. 4 Unseating of Ming-tsu bridge in 2008 Sichuan earthquake [9]

Fig. 5 Toppled span during 2017 Mexico earthquake [20]

2. Specifications on Unseating Prevention in Modern Bridge Codes

Since unseating of spans is common in past major earthquake events, which resulted in catastrophic bridge collapses, implementing immediate measures in existing flock of bridges are essential to preventing such a failure mode, as a part of seismic retrofit measure. Effective unseating prevention measures should also be developed and implemented in the seismic design of new bridges. This has been increasingly highlighted and presented in modern bridge design codes. Generally, codes provide design guidelines regarding unseating prevention measures mainly from two perspectives: a) ensuring a minimum support length and b) implementing unseating prevention restrainers. Specifications on bridge unseating prevention measures in following codes around the world are summarized and compared and given in Table 1 below:

a. AASHTO Guide Specifications for LRFD Seismic Bridge Design [2]
b. CALTRANS [3]
c. Japanese Design Specifications for Highway Bridges-Seismic Design [4]
d. Eurocode 8 [7]
e. Indian Code for Highway Bridges (IRC: SP 114) [5]

Minimum Overlapping Length

A deck supported on a pier or abutment via a horizontally movable device (slider, elastomeric bearing, or special isolation device) should be prevented from dropping off by means of a minimum horizontal overlapping of the underdeck and the top of the supporting abutment or pier in any direction along which relative displacement between the two is physically possible. The same applies at a movement joint separating the deck between adjacent piers or a pier and an abutment or where one part of the span is vertically supported on the other (normally the shorter on the longer of the two). Hence, minimum overlapping length (MOL) is always the primary consideration in the seismic design of bridges, which has been extensively specified in modern bridge codes.

Different codes in different countries adopt different principles for determining MOLs depending on various influencing factors. It can be seen from Table 1 that the code specified MOLs are formulated by considering various influential factors, such as:

a. Structural sizes and configurations.
b. Seismic demands, etc.

In AASHTO, the bridge MOLs located in low and medium seismic zones (e.g., Seismic Design Category (SDC) A, B, C) are determined using the proposed empirical expressions as a function of span length, pier height, as well as alignment skewness. This simplifies the seismic design procedure. Complex dynamic analysis is not necessary for determining an appropriate MOL in such cases. For SDC D bridges with high seismicity excluding single-span bridges, the seismic displacement demand of the more flexible frame under design earthquakes is utilized to calculate an MOL. This indicates that rigorous analysis is required for SDC D bridges to determine an MOL that may be smaller than the results from the empirical formulations [8]. As expected, empirical formulations provide a more conservative estimate of MOLs.

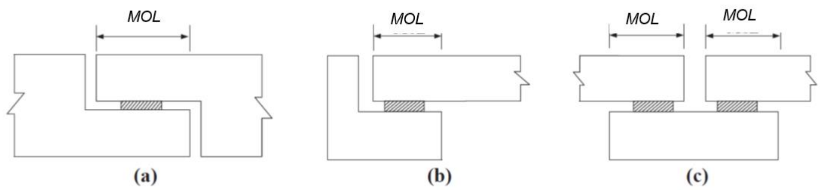
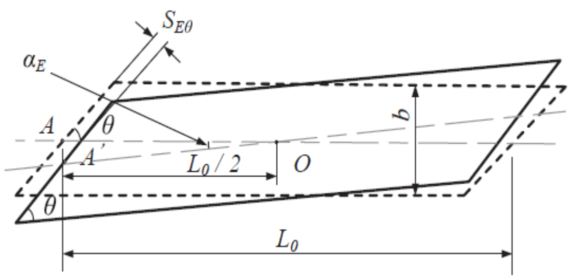
CALTRANS, intended for Californian bridges, recommends a similar MOL formulation as in AASHTO. However, the CALTRANS formulation combines the seismic demands of the two adjacent frames to calculate MOLs at in-span expansion joints, while the AASHTO formulation only considers the demand of the more flexible frame multiplied by a factor of 4.19.

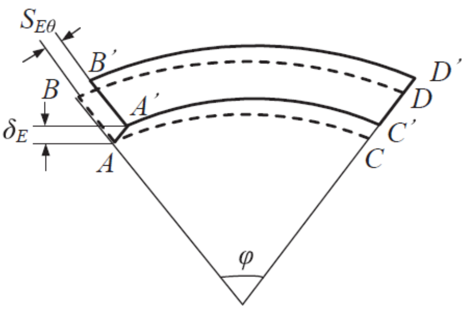
Compared with AASHTO and CALTRANS, the Japanese code provides more rigorous MOL formulations. Since span unseating cases induced by intense soil liquefaction and lateral spreading were observed in past earthquakes in Japan, such effects have been incorporated in the updated code to calculate MOLs. The code also specifies MOLs for skewed and curved bridges by recommending threshold values of the movement. The seismic ground strain, as included in the Japanese code, represents the seismic deformation of ground along the span axis, which may account for a substantial portion of MOLs in the cases with long spans and complex site conditions.

Such an effect is also included in Eurocode 8 to determine MOLs, but it is referred to as the spatial variation of ground motions. It is also pointed out in Eurocode 8 that the MOLs are implemented to ensure the normal function of supports under extreme seismic displacements. By investigating the MOL formulations in these codes, it is seen that all the formulations provide a minimum threshold value irrespective of the seismic demand values. The threshold values are 61 cm in AASHTO, 61 cm and 76 cm subtracting time-dependent deformations (thermal, creep and shrinkage, prestressing) for in-span hinges and abutments, respectively in CALTRANS, 70 cm in Japanese code, and 40 cm in Eurocode 8. IRC code stipulations are similar in concept as Eurocode 8. Indian railway code has simplistic provision and no rigorous calculation is needed to get the MOL [6]. The threshold MOLs are required to maintain the safe transmission of vertical reaction at supports.

Table 1: Code specifications on minimum overlap requirement between deck and support

Design code	Influential factors	Specifications
AASHTO	• Span length • Pier height • Skewness • Seismic demand	• SDC A with $A_s < 0.05$: $MOL = 0.75 \times (20.32 + 0.02\,L + 0.08\,H) \times (1 + 0.000125\,S^2)$ (1) • SDC A with $A_s \geq 0.05$: $MOL = 1.0 \times (20.32 + 0.02\,L + 0.08\,H) \times (1 + 0.000125\,S^2)$ (2) • SDC B and C, and SDC D with single span bridges: $MOL = 1.5 \times (20.32 + 0.02\,L + 0.08\,H) \times (1 + 0.000125\,S^2)$ (3) • SDC D: $MOL = (20.32 + 4.19\,\Delta_{earthquake}) \times (1 + 0.000125\,S^2) \geq 61$ (4) in which *SDC* = Seismic Design Criteria; *MOL* = minimum overlap length (cm); A_s = acceleration coefficient; *L* = length of the bridge deck to the adjacent expansion joints (cm); *H* = average pier height (cm); *S* = angle of skewness in degrees measured from normal to span; $\Delta_{earthquake}$ = seismic displacement of the more flexible frame (cm). • Minimum overlap length of bridges refer to the overlap between the girder and the support, as shown below:

		 Fig. A Illustration of support length at different locations: (a) in-span hinge, (b) end abutment, and (c) intermediate piers
CALTRANS	• Skewness • Seismic demand	• Straight bridges: $MOL = \Delta_{earthquake} + 10.16$ (5) • Skewed bridges: $MOL = (\Delta_{earthquake} + 10.16) / \cos\theta$ (6) in which θ is skew angle; $\Delta_{earthquake}$ is relative longitudinal seismic displacement demand (cm): for in-span hinges, it is calculated as the SRSS combination of the seismic demands of two adjacent frames, and for end abutments, a zero abutment displacement is assumed; MOL is expressed in unit of cm.
Japanese code	• Seismic demand • Soil liquefaction • Seismic ground strain • Skewness • Curvature • Span length	• The minimum threshold MOLs are 61 cm and 76 cm at in-span hinges and end abutments, respectively. • Ordinary straight bridges: $MOL = \varepsilon_G L + u_G \geq S_{EM} = 70 + 0.005 \times l$ (7) in which u_G = maximum relative seismic displacement (cm) considering soil liquefaction; S_{EM} = minimum allowable support length (cm); l = span length (cm); ε_G = 0.0025-0.005 is the seismic ground strain; L = distance (cm) between two substructures for determining the support length. • Skewed bridges (see Figure B below): $MOL = (L_\theta/2) \times (\sin\theta - \sin(\theta - \alpha_E))$ (8) in which L_θ = length of a continuous superstructure (cm); θ is skew angle in degrees; α_E = 5° is threshold of unseating rotation angle in degrees.  Fig. B Support length definition of skewed bridges • Curved bridges (see Figure C below): $MOL = (70 + 0.005 \times \varphi) \times \sin\theta/\cos(\theta/2) + 30$ (9) in which φ = fan-shaped angle of a continuous superstructure in degrees; δ_E = displacement of superstructure toward the outside direction of the curve (cm).

		 Fig. C Support length definition of curved bridges
Eurocode 8	• Seismic demand • Spatial variation of ground motions • Span length	• At end abutments: $MOL = l_m + d_{eg} + d_{es}$, $d_{eg} = \varepsilon_e L_{eff} \leq 2d_g$, $\varepsilon_e = 2d_g/L_g$ (10) in which l_m is minimum support length ensuring a safe transmission of vertical load reaction that is no less than 40 cm; d_{eg} is seismic ground displacement induced by the spatial variation of ground motions (cm); d_{es} is effective seismic displacement demand (cm); L_{eff} is the effective length of deck (cm); L_g is distance beyond which ground motions can be regarded completely uncorrelated (cm); d_g is design ground displacement (cm). • At in-span hinges: $MOL = \sqrt{MOL_1{}^2 + MOL_2{}^2}$ (11) in which MOL_1 and MOL_2 are minimum support lengths calculated for the two adjacent structures using Equation (10). • At intermediated piers: $MOL = MOL_0 + D_p$ (12) in which MOL_0 (cm) is calculated from Equation (10), and D_p is maximum seismic displacement at pier top for design earthquake (cm). • Code has provision for seismic links, holding down devices, and shock transmission units.
Indian Code for Highway Bridges (IRC: SP 114)	• Seismic demand • Spatial variation of ground motions • Type of sub-soil • Span length	• $MOL = l_m + d_{eg} + d_{es}$ (13) $d_{eg} = \varepsilon_e L_{eff} \leq 2d_g$ (14) $\varepsilon_e = 2d_g/L_g$ (15) where l_m is the minimum support length = 40 cm; d_{eg} is the effective displacement of the two parts due to different seismic ground displacement; L_g is the distance beyond which ground motion may be considered uncorrelated and is taken as 500 m; d_g is the design value of the peak ground displacement: $d_g = 0.025\, \alpha_g \times S \times T_C \times T_D$ (16) where α_g is the ground acceleration; S is the soil factor; T_C is the upper limit of the period of the constant part of the spectral acceleration = 0.4 for Type I (Rock or Hard Soil) $N > 30$; = 0.5 for Type II (Medium Soil) and = 0.65 for Type III (Soft Soil) $N < 10$; T_D is the value defining the beginning of the constant displacement response range of the spectrum = 2.0.

		d_{es} is the effective seismic displacement of the support due to the deformation of the structure, estimated as follows: For decks fixed at piers either monolithically or through fixed bearings, $d_{es} = d_{ED}$ (17) where d_{Ed} is the total longitudinal design seismic displacement, $d_{Ed} = d_E + d_g + 0.50\ d_T$ (18) where d_E = design seismic displacement; d_g = *long* term displacement due to permanent and quasi-permanent actions (e.g., post-tensioning, shrinkage and creep of concrete); d_T = displacement due to thermal movements; ψ_2 = combination factor for quasi-permanent value of thermal action. For decks connected to piers or to an abutment through seismic links with slack equal to '*s*': $d_{es} = d_{Ed} + s$ (19)
Indian Railway Standard Seismic Code	• Seismic demand • Spatial variation of ground motions • Type of sub-soil • Span length	• The bearing seat width S_E in mm, between the end of girder and edge of substructure, and minimum S_E between the ends of girder at suspended joint should be not less than the following values: S_E = 203 + 1.67 L + 6.66 H for seismic Zones II and III (20) S_E = 305 + 2.50 L + 10.0 H for seismic Zones IV and V (21) where L = length of superstructure to the adjacent expansion joints or to the end of superstructure. In case of bearings under suspended spans, it is the sum of the lengths of two adjacent portions of the superstructure. In case of single span bridges, it is equal to the length of the superstructure, in *m*, and H = average height of all columns or piers supporting the superstructure to the next expansion joint. For bearings at column or piers, it is the height of two adjacent columns or piers.

UNSEATING PREVENTION RESTRAINERS

Various types of passive restrainers have been developed, which are mainly divided into three categories:

a. Stiffness based restrainers,

b. Energy dissipation restrainers, and

c. Self-centering restrainers.

1. Stiffness Based Restrainers (e.g., Tie-Plate Steel Restrainers, Concrete Shear Keys, Steel or Fiber-Reinforced Polymer (FRP) Cables, Stiffened Side Restrainers)

Stiffness-based unseating prevention restrainers are those providing sufficient stiffness for bridges to restrain seismic displacements. Tie plate restrainers directly prevents the falling out by connecting a girder to another girder or a pier. In normal service, the tie plate does not carry load; it works only when the girders undergo excessive displacement. Figure 6 shows arrangement of a typical tie plate steel restrainer.

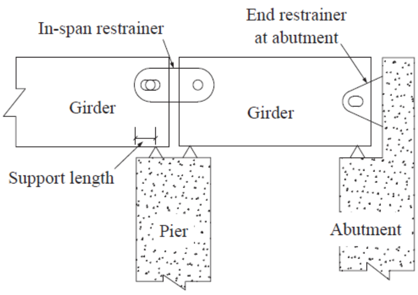


Fig. 6 Tie plate steel restrainer [9]

Fig. 7 Concrete seismic stopper [10]

Concrete seismic stoppers/shear keys are quite common in India. They are routinely implemented in highway bridges. These stoppers can be placed in the interior of the deck, which is not visible from outside or it can be exteriorly placed at both ends of pier cap/abutment cap. Figure 7 shows typical use of seismic stoppers in concrete in India [10]. Unlike concrete shear keys in the transverse direction, cable restrainers are normally implemented in USA, Japan, and many other countries in the longitudinal direction of bridges. They can be installed at either end abutments, intermediate piers, or in-span hinges. Figure 8 and Figure 9 show application of cable restrainers with girder to girder arrangement, with shear key [16,17].

Fig. 8 Cable restrainer with shear key [16]

Fig. 9 Restrainer rods at expansion joints [17]

Stiffness-based restrainers reduce displacements by providing additional stiffness to bridges. Under service- and small earthquake loads, the added stiffness prevents over displacements of superstructure, ensuring a desirable ride comfort. In moderate and large earthquakes, it is inappropriate to design an extremely strong stiffness-based restrainer to control seismic displacements, since excessively large seismic forces can be attracted by the strong restrainers, which poses a high risk of substructure failure. If the restrainers are designed with a low capacity, their effectiveness in restraining displacements may be considerably reduced.

2. Energy Dissipation Restrainers (e.g., Yielding Metallic Dampers, Viscous Dampers, Viscoelastic Dampers)

The limitations of stiffness-based restrainers can be satisfactorily overcome by using energy dissipation restrainers. Energy dissipation restrainers are designed to absorb substantial energy to reduce superstructure displacements without necessarily imposing much seismic demand on substructure, especially under moderate and large earthquakes. Since stiffness-based restrainers are generally not expected to dissipate much earthquake energy, restrainers with substantial energy dissipation capacities have been developed and implemented in highway bridges. Plastic deformation of steel is one of the most effective mechanisms available for the dissipation of energy, from both economic and technical point of view. The idea of utilizing steel hysteretic dampers (SHDs) within a structure to absorb large portions of seismic energy began with the conceptual and experimental work in the 1970s. Figure 10 shows a typical compact steel damper [18]. Their strong points: high reliability, functionality independent of temperature and applied speed, high aging resistance, no maintenance required, and limited costs. There are however some disadvantages as well, which is their limited ability to take large displacements.

Fig. 10 Typical compact steel damper from Germany (Maurer) [18]

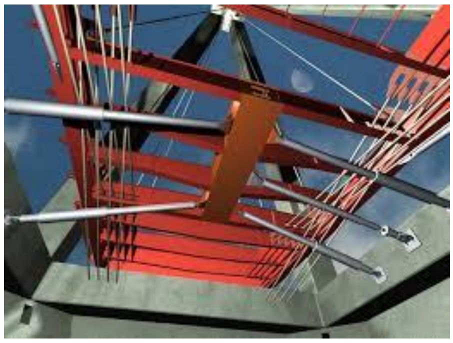


Fig. 11 Use of fluid viscous dampers-Rion-Antirion CS Bridge, Greece [11]

Fluid viscous dampers (FVD) characteristics of velocity dependency constitute another major form of energy dissipation, which have been widely implemented in bridges. Use of FVD helps by shifting the fundamental frequency of a structure away from the dominant frequencies of earthquake ground motion. The isolation system helps to provide an additional means of energy dissipation, thereby reducing the transmitted acceleration into the superstructure. A high velocity normally induces a large damping force in viscous dampers, which makes them relatively attractive in seismic mitigation of bridges and structures. Figure 11 and Figure 12 shows typical use of fluid viscous dampers in Bridges [11].

Despite the inherent advantages of using energy dissipation restrainers, they have some demerits as well. Common energy dissipation restrainers lack re-centering mechanisms, which may result in large residual displacements following a strong earthquake. This makes such energy dissipation restrainers unfavorable when rapid post-earthquake rehabilitation is regarded as a priority for decision-makers.

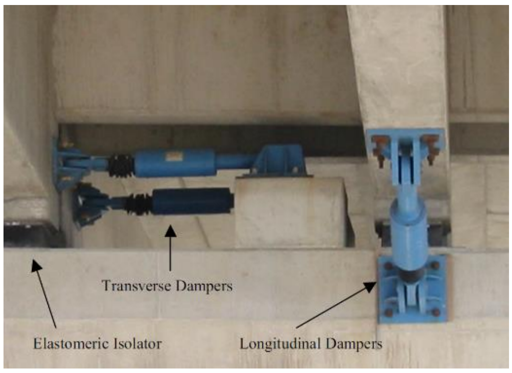


Fig. 12 Use of FVD in approach viaducts of Rion-Antirion Bridge, Greece [11]

3. Self-Centering Restrainers (Super Elastic Shape Memory Alloy (SMA), Hybrid Self-Centering)

Self-centering restrainers are better options over elastic restrainers in minimizing bridge residual displacements without inducing excessive force demand. With the advancement of performance-based seismic design, residual displacement of a structure following an earthquake has become an important consideration, considering post-earthquake rehabilitation. For bridges equipped with unseating prevention restrainers, a desirable scenario is that bridge superstructure returns to its original position after an earthquake with little permanent offset. Although this can be achieved by using elastic restrainers like high-strength steel cables, excessively large seismic force may be induced, imposing high demand on the substructure and foundation. Superelastic shape memory alloy (SMA) and hybrid devices are common self-centering restrainers.

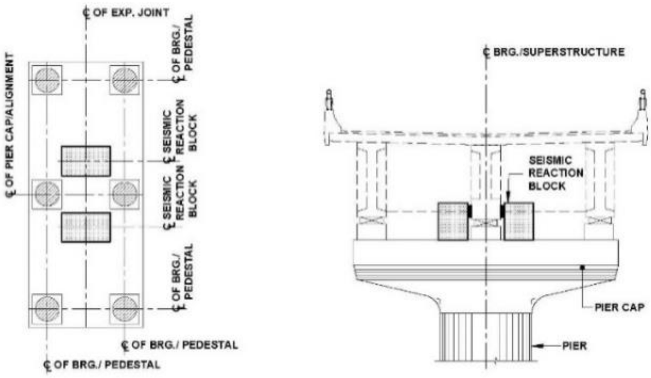
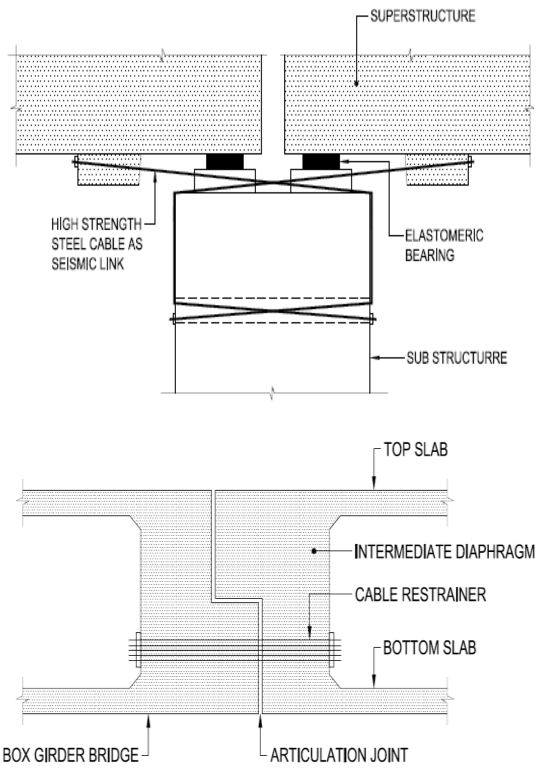
Self-centering restrainers possess the ability of re-centering and energy dissipation, making them superior to other types of restrainers in unseating prevention. However, there are still several limitations existing for self-centering restrainers. First, for the SMA-based restrainers, the high cost and the demanding machining of SMAs are the primary restraining factors to mass-scale implementations of SMA self-centering restrainers.

Table 2 below gives the detailed comparison of provisions available in various codes, in a tabular form. It can be seen from Table 2 that AASHTO, CALTRANS, and Japanese code explicitly distinguish the design requirements between longitudinal and transverse restrainers, while Eurocode 8 and Indian Highway as well as Railway code does not distinguish.

Table 2: Code specifications on unseating prevention restrainers

Design code	**Specifications**
AASHTO	• Longitudinal Restrainers: - An adequate gap/slack in restrainers is required to ensure they are engaged when the design displacement of bridge superstructure is exceeded. Friction-type devices are not recommended as effective restrainers due to the lack of restore forces. - Cable restrainers are commonly-used bridge restrainers, which can be designed using an iteration method, as specified in FHWA's Seismic Retrofitting Manual for Highway Structures. • Transverse Concrete Shear keys: - Shear keys shall be designed to remain essentially elastic at lower than designed earthquakes. - Fused or sacrificial when design-level earthquakes strike. - The interface shear friction mechanism specified in AASHTO LRFD Bridge Design Specifications can be utilized to determine the nominal shear key capacity. - An over-strength factor of 1.5 is adopted to design the shear key capacity-protected members.
CALTRANS	• Longitudinal Restrainers: - More suitable for seismic retrofit of existing deficient bridges. - An approximate one-step method specified in CALTRANS Bridge Design Aids can be used to design the size and number of cable restrainers without iterations. - Sacrificial bolts are designed to ensure that cable restrainers are not working under service-level loads. • Transverse concrete shear keys: - Specifically implemented at end abutments. - The capacity of exterior shear keys at abutments can be determined as. - For abutment on piles: $F_{sk} = \alpha X (0.75V_{piles} + V_{ww})$ (22) - For abutment on spread footings: $F_{sk} = \alpha X P_{dl}$ (23) in which α = 0.5-1.0; V_{piles} is sum lateral capacity of piles; V_{ww} is the shear capacity of one wing wall; P_{dl} is the superstructure dead load reaction at abutment plus the self-weight of abutment and footings. • Configurations of isolated and non-isolated shear keys are provided, where the required amount of vertical reinforcement is determined - For isolated shear key: $A_{sk} = \frac{F_{sk}}{1.8 \times f_{ye}}$ (24) - For non-isolated shear key: $A_{sk} = \frac{1}{1.4 \times f_{ye}}(F_{sk} - 0.4A_{cv})$ (25)

	in which $0.4 \times A_{cv} < F_{sk} \le \min(0.25 \times f_{ce'} 1.5A_{cv})$ (26) $A_{sk} \ge \frac{0.5 \times A_{cv}}{f_{ye}}$ (27) in which A_{sk} is required area of vertical interface shear reinforcement (in^2); A_{cv} is area of concrete engaged in the interface shear transfer (in^2); f_{ce} and f_{ye} are the expected compressive strength of unconfined concrete, and expected yield strength for steel reinforcement, respectively (ksi). • The amount of horizontal reinforcement in abutment stem walls shall be two times of the corresponding vertical shear key reinforcement for isolated shear keys, while for non-isolated shear keys, it shall be taken as the larger one between $2A_{sk}$ and the ratio of F_{sk} to f_{ye}. Sufficient development length of horizontal bars shall be ensured.
Japanese Code	• Longitudinal restrainers: - Unseating prevention restrainers shall be designed without interrupting normal functions of bearings. - Restrainers shall be movable in the transverse direction and also can alleviate seismic impacts. The ultimate force (H_F) and displacement (S_F) of longitudinal restrainers: $H_F = 1.5R_d$ (28) $S_F = c_f \times MOL$ (29) where R_d is dead load reaction at bearing support; MOL is the support length specified in Equations (7)-(9); c_f is equal to 0.75. • Transverse restrainers: - Working as excessive displacement stoppers. - Possible scenarios requiring a transverse restrainer include: skewed bridges with large skewness (satisfying $sin2\theta/2 > b/L$, in which b and L are width and length of a continuous superstructure), curved bridges with large curvature (satisfying $115/\varphi \times (1 - cos\varphi)/(1 + cos\varphi) > b/L$), bridges with a narrow substructure in longitudinal direction, bridges with limited bearings on substructures, and bridges showing excessive transverse displacements subjected to ground soil liquefaction. - Sufficient slack/gap of transverse restrainers shall be provided to ensure an uninterrupted function of bearing systems. The design capacity (H_t) of a transverse restrainer is recommended as: $H_t = 3k_h R_d$ (30) in which k_h is design horizontal seismic coefficient of the first level earthquake hazard specified in the Japanese code; R_d is dead load reaction.
Eurocode 8	• Displacement restrainers are implemented when the requirements of MOLs are not met. • Friction devices are not considered as effective seismic restrainers. • The capacity design principle is adopted to design restrainers installed at deck-to-pier connections.

	• For deck-to-deck connections at intermediate joints, the restrainer capacity shall be taken 1.5 times of the seismic force of decks.
Indian Highway Code (IRC: SP 114)	• Seismic reaction blocks or anti-dislodgement devices are to be provided between adjacent sections of superstructure. Figure D shows the typical arrangement of seismic reaction blocks recommended in the code. • Seismic links are usually considered as a part of retrofitting measure. Various forms of seismic links are proposed in the code as shown in Figure E. • Vertical holding down devices are to be used where FOS against uplift is more than 2. Figure F shows typical arrangement for holding down device.  Fig. D Typical seismic reaction blocks  Fig. E Seismic links at supports and at in-span joints

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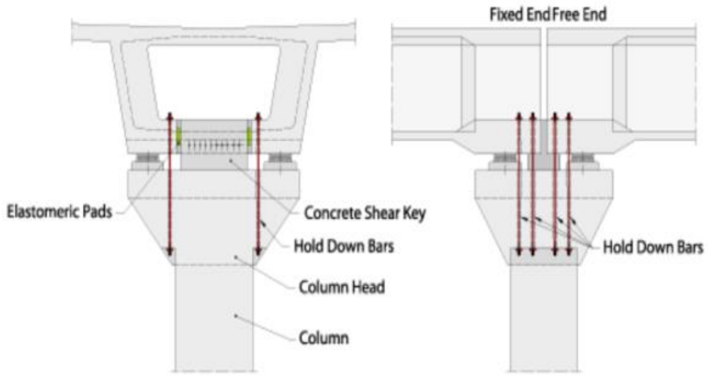
<tr><td></td><td>

Fig. F Holding Down Device</td></tr>
<tr><td>Indian Railway Standard Seismic Code</td><td>

- Anti-dislodgement elements shall be provided between adjacent sections of the superstructure at supports and at expansion joints. Anti-dislodgement elements like reaction blocks and seismic arrestors shall be designed for at least twice the seismic force.
- The linkages, if provided, shall be designed for at least, elastic seismic acceleration coefficient, A_h times the weight of the lighter of the two adjoining spans or parts of the structure as in the case of suspended spans.
- If the linkage is at location where relative deformations are designed to occur, then sufficient slack must be allowed in the linkage so that linkages start functioning only when the design relative displacement at the linkage is exceeded and linkage becomes effective, after overcoming the designed slack in the linkage.
- When linkages are provided at columns or piers, the linkage of each span may be connected to the column or pier instead of to the adjacent span.

</td></tr>
</table>

For longitudinal restrainers, both AASHTO and CALTRANS recommend practical design methods for these devices, although detailed design steps are presented elsewhere. The design methods are developed mainly for steel cable restrainers that constitute a substantial portion of restrainer implementation in modern highway bridges. However, the Japanese code just recommends the reqirements of the ultimate capacity of longitudinal restrainers rather than the specific design steps. It requires that the ultimate strength of restrainers shall be designed as 1.5 times of the dead load reaction. The longitudinal restrainers shall be still capable of holding the unseated side of the span from falling in extreme cases (refer Figure 13) [9]. IRC code provisions are greatly influenced by the provisions of Eurocode.

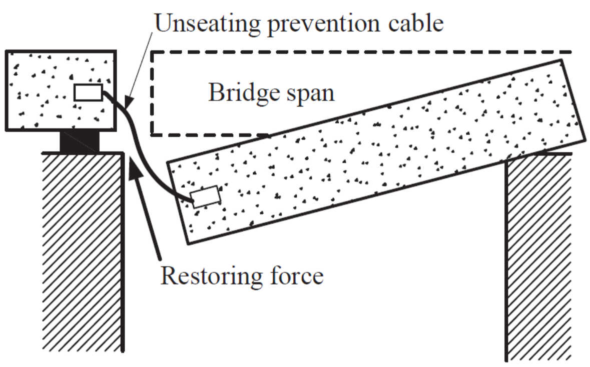


Fig. 13 Role of unseating prevention cable restrainers in case of span unseating - Japanese code [9]

CONCLUSIONS

The paper presents an overview of the current practices about various devices used to mitigate pounding and unseating damages on bridges structures. The interest within the structural engineering community in applying these devices in retrofit of existing structures and in new structures appears to be growing. Based on extensive review of current practice worldwide, presented in this paper, the following general conclusions are drawn.

a. Past earthquakes clearly indicate that most unseating damages in bridges were caused by dislodgement due to insufficient seat width. Bridges with irregular configurations, skewness, curvature, irregular geometry were found to be more susceptible to unseating due to added complexities of in-plane superstructure rotation.

b. Seismic design codes address the issue of providing minimum seat width and also provides for unseating prevention measures in the form of displacement restrainers. Amongst all the national/ international codes, the Japanese code is more stringent and rigorous in this respect since it incorporates factors like the effects of curvature and soil liquefaction in determining the minimum overlap length. Indian highway code is quite advanced in terms of provisions for seismic restrainers and minimum overlap length and is at par with the provisions of Eurocode.

c. Both AASHTO and CALTRANS provide specific design procedures for longitudinal cable restrainers, whereas the Japanese code specifies the restrainer ultimate capacity requirements to hold spans from falling in extreme situations. As commonly-used transverse restrainers, detailed design issues of concrete shear keys are presented in CALTRANS, including the capacity and reinforcement requirements for improved seismic performance.

d. Using devices capable of dissipating the energy could provide protection against pounding and unseating failures. However, some of these devices have no re-centering ability and are frequency dependent, which make it difficult to control the force during the design [12]. Moreover, some of these devices could require regular inspection.

e. Self-centering restrainers (i.e., either shape memory alloy or hybrid ones), appear to be a better option over energy dissipation restrainers in enhancing structural re-centering performance, and are superior to stiffness based restrainers by dissipating a certain amount of energy. Nevertheless, the high costs and the limited energy dissipation capacities are still primary restraining factors to a wider implementation of self-centering restrainers.

f. Recent researchers have focused on the application of hybrid devices that combine the advantages of two or more materials or devices to mitigate pounding and unseating damages in bridge structure. Application of rubber bumpers with restrainers, combination of dampers with the restrainers and modular expansion joints (MEJ) with damping devices have received considerable attention to reduce the damages as well as improve the serviceability of the bridge. These research findings should result in inclusion of more advanced unseating prevention restrainers in the codes and practices, in addition to the steel cable restrainers and shear keys.

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