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SEISMIC ISOLATION OF HIGH-SPEED RAILWAY BRIDGES

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Abstract

The demand for sustainable and resilient transportation infrastructure in India has necessitated the need to develop high-speed railway (HSR) corridors to improve connectivity and support economic growth. The first of these HSR corridors is being constructed between Mumbai and Ahmedabad in collaboration with Japan. Several such corridors are planned for future in India in regions of moderate to high seismicity. Seismic safety of HSR bridges is a major factor governing its design. The stringent seismic performance goals for HSR bridges are difficult and uneconomical to achieve through conventional earthquake resistant design. Seismic isolation using elastomeric bearings has the potential to significantly reduce the seismic demands to HSR bridges.

In this paper, the feasibility of seismic isolation to HSR bridges is investigated and the implementation challenges are addressed. The performance goals of HSR bridges are adopted from the HSR projects under construction or implemented in other countries (e.g. Japan, Taiwan, USA). A benchmark four-span continuously supported bridge on the HSR route proposed between Mumbai and Ahmedabad is considered for this study. The superstructure deck is seismically isolated using lead-rubber (LR) bearings. The seismic performance of the seismically isolated bridge is compared against the corresponding non-isolated (integral) bridge using three-dimensional nonlinear models. The bridge models include the material nonlinearities and explicitly considers the track-structure interaction. A series of nonlinear response-history analysis is performed to obtain the critical response parameters. It is observed that seismic isolation significantly reduces the force demand in the sub-structure, pier drifts and deck acceleration. However, the design is controlled by the deck rotations at expansion gaps and rail stresses, which are increased due to incorporation of seismic isolation. Moreover, the characteristic strength of the isolation system is governed by the braking loads in HSR bridges as opposed to seismic loads. The critical points of rail failure are in the vicinity of the expansion gaps at the abutments due to increased displacement and rotations in transverse direction. It is recommended based on this study that the isolator design of a seismically isolated HSR bridges should be performed using deformations and stresses in the rails as the primary design objective.

Keywords: High speed rail; seismic isolation; elastomeric; bullet train, bridges

1. Introduction

India has announced several high-speed railway (HSR) corridors between major cities to meet the mobility needs of the growing population. In general, a HSR network is defined where train speeds are higher than 200 km/h. Table 1 presents the characteristics of some of the HSR operating around the world and the details of the proposed HSR corridor between Mumbai and Ahmedabad in India. Due to the dynamic effects associated with high speeds, the criteria for the design of bridges for a HSR network are different compared to regular railway bridge projects. The design of the HSR bridges en route should allow the train running safely and comfortably at high speed. These criteria should be fulfilled not only under normal conditions but also under extreme loading conditions like an earthquake shaking.

Seismic response of a bridge is a crucial aspect in bridge analysis and design. The acceptable mechanism for the traditional force-based seismic design approach is to dissipate energy through the formation of plastic hinges in a stable manner to prevent collapse during an earthquake. The damaged regions of plastic hinges need to be retrofitted, or the piers need to be replaced after earthquakes which has far-reaching consequences in terms of the downtime loss, additional cost to restoring the structure etc. The risk acceptance to a damage state to achieve a performance goal (e.g., life safety) ensures economic design. The performance goals are much more stringent (e.g., operational with no damage) for critical bridges due to the necessity for post-earthquake recovery and due to economic investments. The performance requirements of HSR bridges are concerned with the limitations on the structural deformations, accelerations and rail stresses. The structural response of the HSR bridges is amplified due to the dynamic effects of high-speed moving trains. Excessive deformations and accelerations can lead to numerous issues, including unacceptable changes in vertical and horizontal track geometry, excessive rail stresses, dynamic amplification of loads and passenger discomfort [1]. The criteria for serviceability of HSR bridges are much more stringent as compared to regular railway bridges. These performance goals become difficult and uneconomical to achieve through conventional seismic design approach in regions of high seismicity. Seismic isolation using elastomeric bearings could be a viable strategy to achieve the strength and serviceability goals of HSR bridges. In the past, several highway bridges were retrofitted using the seismic isolators where the performance goals were difficult to achieve through traditional seismic design. More recently, seismic isolation has been used on non-critical bridges because of construction cost savings with reduced seismic forces on foundations. Seismic isolation has not been used for HSR bridges. With the announcement of HSR on Mumbai-Ahmedabad route in India, the seismic response of the bridges and viaducts en-route is one the major challenge to the structural engineers.

Table 1 – Major HSR projects around world [2]

Country	Total network length (kms)	Peak recorded speed (kmph)	Average operating speed (kmph)
China	19369	394	313
France	2036	574	272
Japan	2664	443	256
Taiwan	336	315	245
India	508	n.a.	320

In the past twenty years, a broad range of seismic response control devices and technologies have been used in the earthquake-resistant design of civil structures, which include seismic isolators, viscous and friction dampers. Seismic isolation has been used in many existing and new bridges around the world; there are more than 200 seismically isolated bridges in North America [3]. Seismic isolation has not yet been implemented to HSR bridges. There are limited studies available for seismic isolation of HSR bridges. There is also limited research available on this topic due to complexities associated with the implementation of HSR bridges. Sarno [4] presented a study on seismic isolation of railway bridges with lead rubber bearings and highlighted issues with serviceability requirements. Li and Conte [5; 6] performed studies on seismic isolation of a prototype California HSR bridge. The effect of seismic isolation was studied through comparison between the seismic response of the bridge with and without isolation without the presence of the train. It was found out that seismic isolation significantly reduces the force demand in the substructure and increases the deck displacement. The peak stresses in rails increase because of the increased deck displacement and rotations. The paper recommended a performance-based evaluation of the behavior of the seismically isolated HSR bridges.

The current study investigates the feasibility of seismic isolation of HSR bridges and focuses on characterizing the behaviour of HSR bridges seismically isolated using Lead Rubber (LR) bearings. The beneficial and detrimental aspects of using seismic isolation over conventional earthquake resistant design would be ascertained and recommendations would be provided to address challenges associated with the seismic isolation of the HSR bridges.

2. Performance goals and limits

The performance goals and limits for HSR bridges are developed to ensure safe operation at peak operating speeds under conditions associated with its design loads. High speed railway bridges are designed considering two primary performance goals [1]:

(A) Resistance and structural durability (generally through ultimate limit state criteria)
(B) Serviceability (serviceability limit states) that can also be divided into two criteria:
   i) Safety of train operations on the bridge, that requires sufficient track stability and permanent contact between rails and wheels;
   ii) Comfort inside the trains.

The strength limit states are specified based on performance desired for different load intensities expected during the service life of the bridge. The serviceability requirements are very critical for safe and reliable operation of high-speed trains and typically governs the designs of substructure and superstructure of HSR bridges. The serviceability limit states are prescribed in terms of deflection and rotations of the superstructure deck, rail stresses and vertical deck accelerations. The serviceability limits specified by the California High Speed Rail Authority [7] and Taiwan High Speed Rail Corporation [8] are followed here. THSRC [8] and CHSRA [9] specify load cases to check response limits in the vertical and lateral direction. Some of those load cases, which would be later used for this study, are presented in Table 2.

Table 2 - Load cases defined for Taiwan and California HSR

Taiwan [10]	**California [7]**
Group V: $L_1 + I_1 + LF_1 + EQ_{II}$	Group 4: $(LL + I)_2 + LF_2 \pm T_D$ Group 5: $(LL + I)_1 + LF_1 \pm 0.5T_D + OBE$
L = live load I = impact load LF = longitudinal force from live load (braking & acceleration) LF_1 = longitudinal force from one train live load (braking) EQ_{II} = type II earthquake	$(LL+I)_{1,2}$= one (two) track of train load plus impact $CF_{1,2}$ = centrifugal force – one (two) track I = vertical impact factor from LL LF_1 = braking forces (apply braking to one track) LF_2 = braking and acceleration forces (apply braking to one track, acceleration to the other track) T_D = Temperature load OBE = operating basis earthquake

2.1 Seismic performance goals

The earthquake hazard levels for which the HSR bridges are designed and the associated performance expectations for the HSR projects around the world are summarized in Table 3.

Table 3 – Design earthquake and associated performance expectation

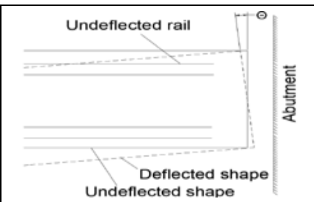
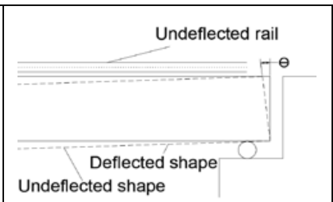
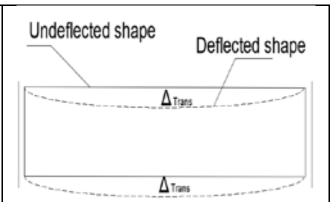
Country	Earthquake	Probability of exceedance/ PGA	Performance expectation
Taiwan [8]	Type I	10% in 100 years, 950 years return period	Repairable damage, inelastic response within acceptable ductility demand
	Type II	PGA 1/3rd of Type I earthquake	No yielding permitted, design for serviceability limit state at peak speed
USA [7]	MCE	10% in 100 years, 950 years return period	Significant damage but minimum loss of vertical load capacity, operation can be resumed after significant repair or replacement
	FBE	18% in 100 years, return period of 500 years	Yielding of rebars and cracking of concrete at plastic hinge locations. Serviceability must be maintained after short term repairs.
	OBE	86% in 100 years, return period of 50 years	No yielding or permanent deformations permitted
Japan [11]	Level I	Return period of 50 years	Deformation may exceed the yield limit
	Level II	Spectrum I (Magnitude 8, source to site distance between 30 to 40 km and Spectrum II (statistical analysis)	Significant damage permitted with collapse prevention
India [11]	DBE	Design earthquake as per IS1893[12]	Structure would undergo deformation beyond its yield limit.
	MCE	2 times the DBE	The structure might undergo severe damage, but total collapse avoided

FBE: functional basis earthquake, DBE: design basis earthquake, MCE: maximum considered earthquake

2.2 Deflection and rotation limits

Table 4 gives deflection and rotation limits specified by different codes for single loaded track. The parameter L is the span length in meters.

Table 4 – Table of limits for the benchmark bridge

Document	Load case			
		In-plane rotation (°)	Vertical rotation (°)	Transverse deflection (mm)
THSRC [8]	Load case V	0.092[1], 0.081[2]	0.098[1], 0.086[2]	$L/2.2 = 11.36$[1], 15.9[2]
CHSRA [7]	Load case 3	0.12	0.15	$L^2 / 84.365 = 7.4$[1], 14.52[2]

1: for 25 m span length, 2: for 35 m span length

2.3 Rail stresses

In addition to the design requirements for the structural components of the bridge system, the CHSTP design criteria restrict the track response (e.g., rail stress) for train operation requirements under operational basis events, while they allow derailment of the train under maximum considered earthquake events with containment walls maintaining the train within a specified area of the bridge deck. The total normal stress in a rail is given by the following equation:

$$\sigma_{rail} = \frac{P}{A} \pm \frac{M_2}{Z_{22}} \pm \frac{M_3}{Z_{33}} \qquad (1)$$

where axial stress, P/A, and bending stresses, M_2/Z_{22} and M_3/Z_{33}, are defined for axes shown in Figure 1.

Permissible additional axial rail stresses are specified excluding stresses due to vertical wheel loads, stresses due to relative displacements at structural expansion joints, and stresses due to rail temperature and preheat during rail installation. The CHST project document TM 2.10.10 [9] gives the permissible additional axial rail stresses (σ_{rail}) as per Table 5 for load cases corresponding to Group 4 and 5.

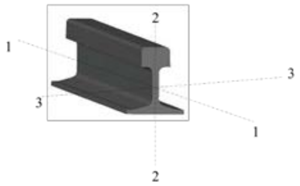


Figure 1 - Reference axes of the rail

Table 5 - Permissible additional axial rail stress limits

Group	Range of σ_{rail} (MPa)	
	Non-ballasted Track	Ballasted Track
4	$-96 \leq \sigma_{rail} \leq +96$	$-82 \leq \sigma_{rail} \leq +96$
5	$-158 \leq \sigma_{rail} \leq +158$	$-144 \leq \sigma_{rail} \leq +158$

The permissible additional axial rail stress limits exclude any bending stress (M_2/Z_{22} and M_3/Z_{33}). The total normal stress (due to P, M_2, and M_3) at the OBE hazard level must be smaller than the factored yield strength of the rails. CHSRA [9] uses standard rail (141RE) with a minimum yield strength of 510 MPa (74.0 ksi) and an ultimate tensile strength of 982.5 MPa (142.5 ksi). This corresponds to factored yield strength of 365.4MPa, as reported in Li and Conte [5]. For the Indian HSR, it is proposed to have the UIC 60 (90 UTS FF UIC) continously welded rail (CWR) over the complete stretch [2]. These rails have ultimate tensile strength and the yield strength of 900 MPa and 468 MPa, respectively.

2.4 Acceleration limits

The passenger comfort inside the train depends on the vertical acceleration of the deck. A maximum vertical acceleration of 2.0 m/s^2 is recommended by IRICEN [2] for acceptable level of comfort. THSRC [8] prescribes a limit of 0.35g for vertical accelerations.

3. Benchmark HSR bridge

Proposed Ahmedabad – Mumbai HSR route in India contains more than 20 river crossings. Kim River (Chainage 293.30 km from Mumbai) crossing out of these have been considered as a site for HSR bridge study. A four-span continuous box girder bridge of total length 120 m is considered for this study. The details of the proposed bridge are summarized in Table 6 and the bridge spans are shown in Figure 2. Bridge sub-structure consists of three interior piers of equal heights and two seat type abutments (A1 and A2). Pile foundations with circular concrete piles are assumed to provide fixed boundary conditions at the base of the piers and soil-structure-interaction is not considered.

Table 6 – Details of HSR bridge site

Location	**Chainage (kms)**	**Bridge Length(m)**	**Main structure type**	**Seismic zone**	**Soil site**
Kim River	293.5	120	PC- Box girder	Zone IV	Medium stiff

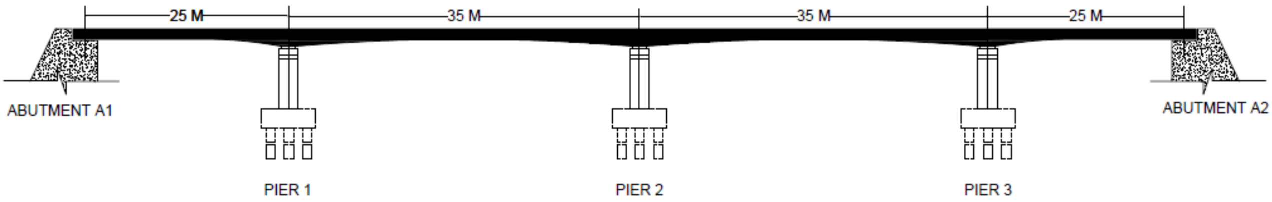
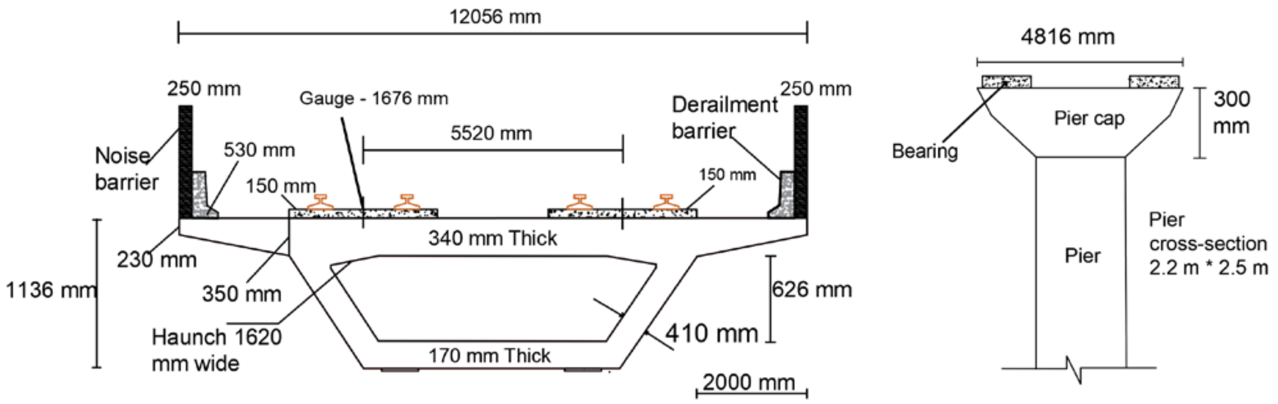


Figure 2 – Longitudinal section of the benchmark HSR bridge

Preliminary dimensions of the superstructure box girder are obtained by following the guidelines given by US department of transportation[13] and the piers were designed considering all the vertical and lateral loads acting on them. The girder and pier cross-section are as shown in Figure 3.

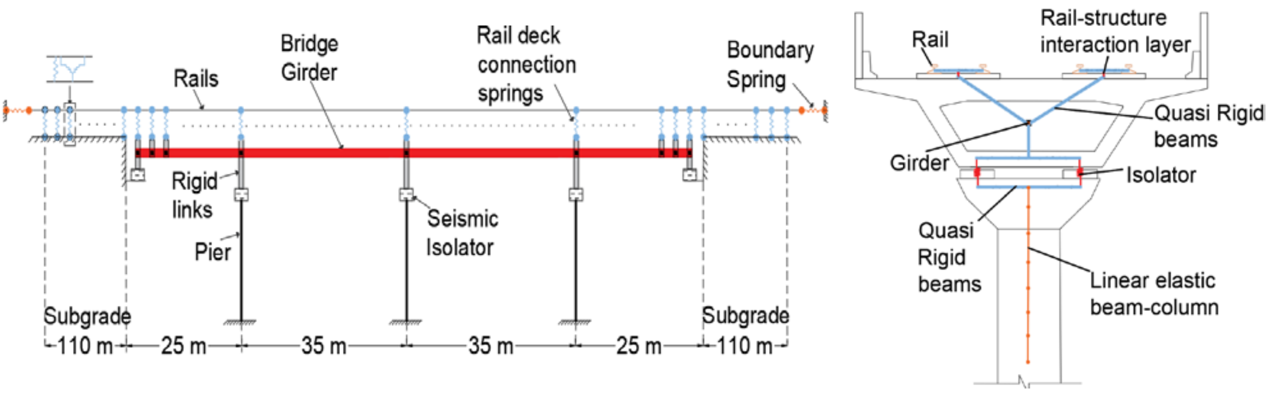


(a) Girder cross-section (b) Pier

Figure 3 – Geometry of the benchmark bridge

4. Numerical model

Three-dimensional nonlinear FE models of Mumbai-Ahmedabad prototype bridge were developed. Two models are created: 1) bridge with integral pier and deck connection, 2) seismically isolated bridge with lead rubber bearings. The non-uniform section of the deck superstructure along the length is modeled with linear elastic beam-column elements by discretizing it into small elements with equivalent section properties. Rigid links (linear elastic beam-column elements with very high stiffness properties) are used to model the rigid offsets between the centroidal axis of the box girder and the seismic isolators. The mathematical model of the bridge is presented in Figure 4.

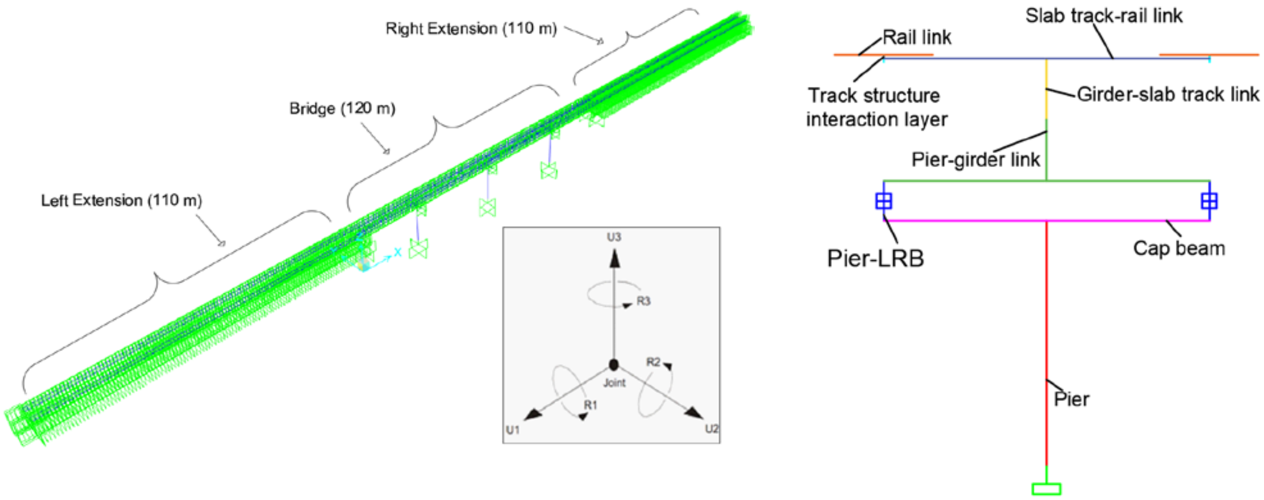


(a) Components of the HSR bridge (b) Pier cross-section

Figure 4 – Mathematical model of the bridge

6

CHSRA [9] recommends track-structure-interaction analysis for the performance evaluation of bridges or aerial supporting structures in the HSR system. The rails on bridge are explicitly modeled and extended partially beyond both the abutments up to 110 m. A nonlinear longitudinal boundary spring is modelled and provided at the end of extensions to account for a longitudinal support provided by infinitely long continuously welded rails (CWRs). The UTS FF UIC rail section (60 kg/m) is adopted for the Mumbai-Ahmedabad HSR. The rail-slab track connection is represented by elastic plastic springs with stiffness 2.87 kN/m. The rail structure interaction is represented by two linearly elastic springs with stiffness 21.5 kN/mm and 191.5 kN/mm in two horizontal directions. Further details on modeling the track-structure interaction are provided in additional references (e.g.,[6], [9], [14]). The numerical models of the integral and base-isolated bridges are created in SAP2000 [15] and shown in Figure 5. The numerical models explicitly consider the track-structure interaction to assess the seismic performance of bridge and the track system.



(a) 3D FE model of the benchmark bridge (b) Cross-section view of FE model

Figure 5 – Numerical model of the benchmark bridge in SAP 2000

4.1 Isolation system

For the present study, lead rubber bearing has been selected with idealized bilinear inelastic force-deformation behavior. All the isolators used are of the same size and the design of seismic isolators guarantees that it will not yield under the train operation. Each isolator is modelled as a zero-length element with two uncoupled bilinear inelastic springs in each horizontal direction. The characteristic properties of seismic isolator are tabulated in Table 7. The bearing arrangement in the non-isolated bridge is shown in Figure 6. Seismic isolations are placed below the superstructure at all pier and abutment locations.

Table 7 – Characteristic properties of an isolator

Material type	Initial stiffness (kN/mm)	Yield force (kN)	Post-yield stiffness (kN/mm)
Bilinear elastic	31.41	138.54	3.4

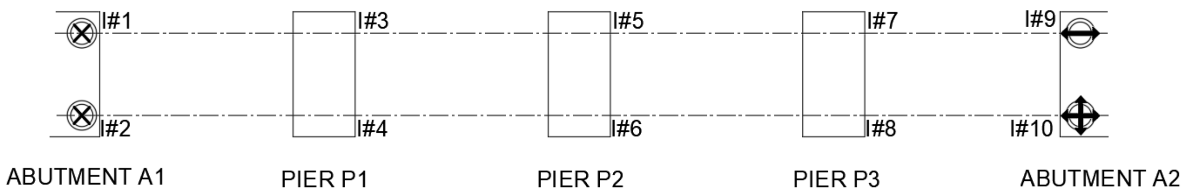
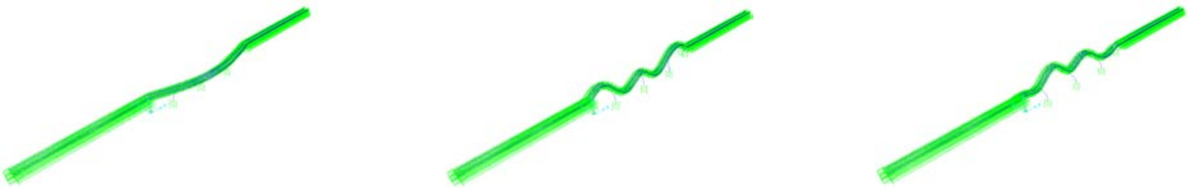


Figure 6 – Bearing arrangement in non-isolated bridge (top view)

4.2 Modal properties

Mode shapes and modal participation factors for both isolated and non-isolated models are summarized in Figure 7 and Figure 8, respectively. The fundamental mode of vibration is the in-plane lateral deflection of the integral and the base-isolated bridge spans.



1st Transverse mode	Mixed mode	1st longitudinal mode
T = 0.73 s	T = 0.42 s	T = 0.39 s
U2 = 62.06%	U1=21.82, U3=3.87	U1 = 56.33%

Figure 7 – Modal properties of the integral bridge

1st Transverse mode	Mixed mode	1st longitudinal mode
T = 1.66 s	T = 0.98 s	T =0.75 s
U2 = 68.6%	R3 = 20.2%	U1=75.9%

Figure 8 – Modal properties of the seismically isolated bridge

4.3 Axle load

The axle load considered for the analysis is shown in Figure 9.

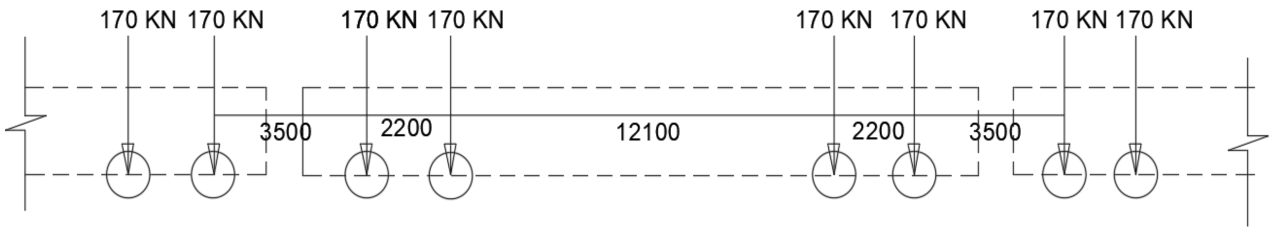


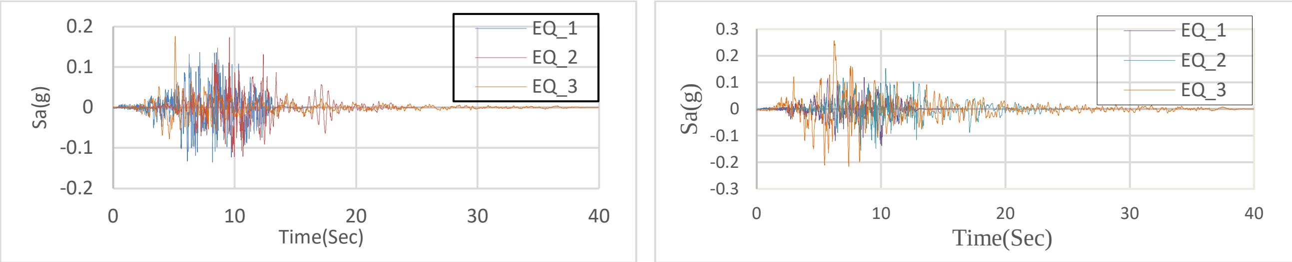
Figure 9: E5 Shinkansen high-speed train axle configuration [16]

5. Seismic hazard

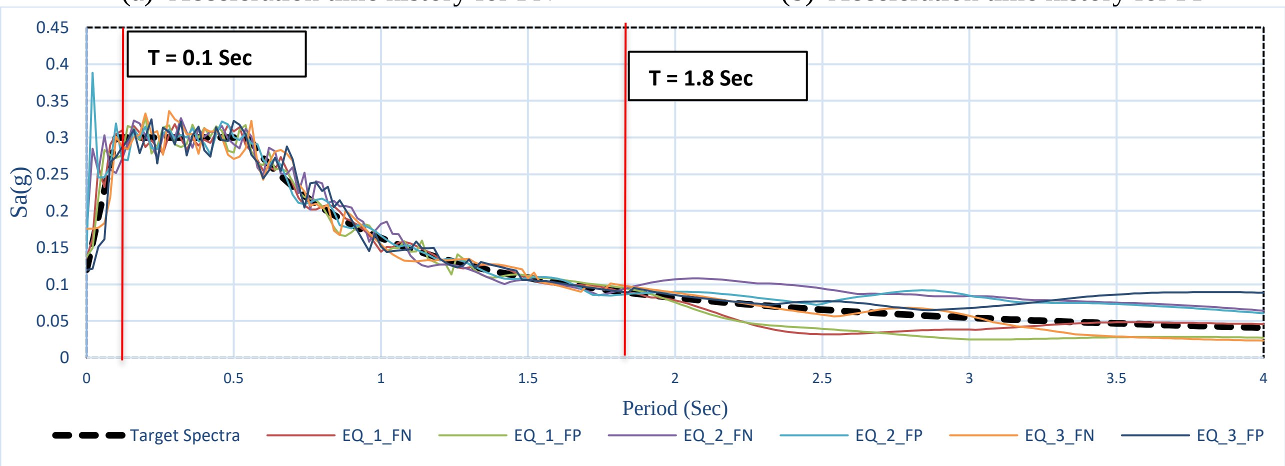
Seismic hazard for different HSR projects around the world were summarized in Table 3. The response of the integral and the base-isolated bridges are obtained by conducting response history analysis. The spectra specified in IS1893 [12] for seismic zone IV and soil type II (medium stiff) is used. Three ground motion sets are selected from PEER NGA ground motion database [2] and scaled to match the uniform hazard spectra at DBE hazard level. The properties of the selected ground motions are presented in Table 8. Figure 10 shows the unscaled acceleration histories of the ground motions in the two perpendicular horizontal directions. The FP and FN denote the parallel and normal components of ground motion.

Table 8 – Selected ground motion records

Name	Year	Ground Motion	Magnitude	PGA (g)
EQ 1	1989	Loma Prieta-LGPC	6.93	0.94
EQ 2	1994	Northridge - Jensen Filter Plant	6.69	1.07
EQ 3	1995	Kobe, Japan	6.90	0.69



(a) Acceleration time history for FN (b) Acceleration time history for FP



(c) Response spectrum

Figure 10 – Target response spectra and ground motion properties

6. Response history analysis

The response of the integral and the base-isolated bridges are obtained subject to ground excitation presented in the previous sections. The response quantities are reported along the directions shown in Figure 11.

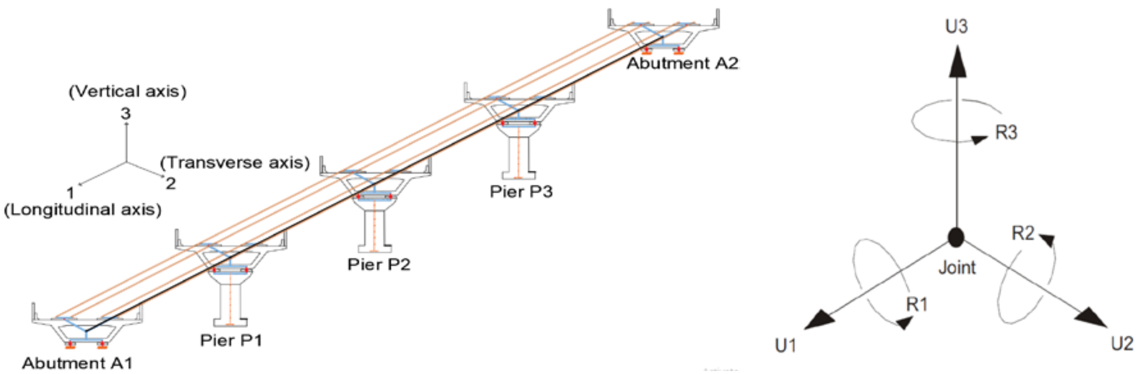


Figure 11 - Displacement degrees of freedom in coordinate system

The response history analysis is conducted with three live load case: i) both track loaded, ii) one track loaded, and iii) no track loaded. The peak response for three live load cases and the three ground motions (total nine combinations) are presented in the subsequent sections.

6.1 Force and moments at pier base

The forces and the moments at the base of the base of the pier P1 are summarized in Table 9. There is a substantial reduction of around 65% in peak shear and transverse moment due to introduction of seismic isolation to the HSR bridge. The significant reduction in design force and moments for the piers would directly result in enhanced safety, better economy and ease of constructability.

Table 9 – Peak forces and moments at the base of pier P1

	F1 (kN)	F2 (kN)	F3 (kN)	M1 (kN-m)	M2 (kN-m)	M3 (kN-m)
Non-isolated	1332	4137	33070	62221	15827	1168
Isolated	1079	1368	32770	21818	13604	154
% reduction	19	66.9	0	64.9	14	86.8

6.2 Displacements and rotations

The displacement response is obtained at four locations: 1) abutment, 2) top of the pier, 3) deck level at the location of the pier, and 4) mid-point of the central span. The rotations are obtained at the abutments. The values are presented in Table 10 through Table 12. The transverse and the longitudinal displacements at the abutment are in Table 10 increased expectedly for the isolated bridge and rail stresses need to be calculated to check if limits are exceeded. The peak displacements at top of deck and mid span (relative to span chord) in Table 12 are also increased due to seismic isolation but remain within permissible limits specified by the HSR guidelines. Peak rotations and displacements at the top of pier P1 in Table 11 are significantly reduced.

Table 10 – Rotation and displacement at Abutment A1

	R2 (°)	R3 (°)	U1 (mm)	U2 (mm)
Response limit	0.092	0.098	N.A.	N.A.
Non-isolated	0.044	0.088	23.9	2.2
Isolated	0.03	0.075	42	23.1
% reduction	-31.8	-14.7	+75.7	+954.8

Table 11 – Peak rotations and displacements at top of pier P1

	R2 (°)	R3 (°)	U1 (mm)	U2 (mm)
Non-isolated	0.12	0.11	17.3	52.8
Isolated	0.04	0.10	11.3	18.3
% reduction	66.6	9.1	34.4	65.3

Table 12 – Peak displacements of the deck

	Top of deck at P1		Mid span with respect to span chord	
	U1 (mm)	U2 (mm)	25 m span	35 m span
Response limits	n.a.	n.a.	11.4	15.9
Non-isolated	20.6	65.7	5.0	6.1
Isolated	34.3	75.7	6.8	11.3
% increase	66.5	15.1	36.0	85.2

6.3 Accelerations

The peak acceleration of the deck top at pier P2 in the integral and the seismically isolated bridge are tabulated in Table 13. Seismic isolation of HSR bridge reduces the peak accelerations along three-directions and the vertical acceleration is reduced below the limit 2.0 m/s^2 (= 0.2g) recommended by IRICEN [2].

Table 13 – Deck accelerations over pier P2

	$\ddot{U}_1(g)$	$\ddot{U}_2(g)$	$\ddot{U}_3(g)$
Non-isolated	0.28	0.58	0.21
Isolated	0.14	0.35	0.12
% decrease	50	39.6	42.8

6.4 Rail stresses

Peak rail stresses (both tensile and compressive) due to the three ground motions are presented in Table 14. High bending stresses in the rails are developed due to increase in transverse deck displacement. The average combined stress value is 365.5 MPa which is smaller than the factored yield strength of 374 MPa (considering a FOS of 1.25) of the UIC 90 rail proposed to be used for the Indian HSR.

Table 14 – Peak rail stresses in outermost rail

	Axial Stress (MPa)	Bending Stress (MPa)		Combined Stress (MPa)
	(P/A)	(M_{22}/Z_2)	(M_{33}/Z_3)	(P/A)+ (M_{22}/Z_2)+ (M_{33}/Z_3)
Non-isolated	117.7	11.25	27	155.9
Isolated	185.3	98.2	81.9	365.5
% increase	58.4	772	203	134

Table 15 – Additional axial stress due to TSI

	Group 4 - $(LL + I)_2 + LF_2 \pm T_D$	**Group 5 - $(LL + I)_1 + LF_1 \pm 0.5T_D + OBE$**
Response limit	$-96 \leq \sigma_{rail} \leq +96$	$-158 \leq \sigma_{rail} \leq +158$
Non-isolated	15.2	103.1
Isolated	21.8	147.3

7. Conclusions and recommendations

The research work presented in this paper investigates the feasibility of seismic isolation of HSR bridges to reduce the seismic demand and satisfy the serviceability requirements. A comprehensive literature review was conducted to determine the strength and serviceability limits for safe and reliable performance of HSR bridges. A benchmark HSR bridge on the proposed route between Mumbai and Ahmedabad was considered for this study. Three-dimensional models of non-isolated (integral) and isolated bridges were created in SAP2000. The models include the effect of material nonlinearity with explicit consideration of track-structure interaction. Three sets of bi-directional horizontal ground motions were selected and scaled to match target design response spectra at DBE hazard level in seismic zone IV as per IS1893. The structural response quantities of interest were obtained for the isolated bridge and compared to the non-isolated bridge. The results allow users to understand issues associated with seismic isolation of HSR bridges and make design decisions to achieve the performance objectives. The key conclusions of this study are:

1. The characteristic strength to weight ratio (Q_d/W) of the isolation system of a seismically isolated HSR bridge is controlled by the braking load, which is quite large for railway bridges.
2. The introduction of seismic isolation to a HSR bridge substantially reduces the force demand in piers (around 65%) and pier drifts at the DBE hazard level for seismic zone IV.

3. Seismic isolation increases the displacement demand in the superstructure of the isolated HSR by 47% due to increased deformation in the isolators.
4. The horizontal and vertical rotations at the abutment of the seismically isolated HSR are reduced for all seismic zones.
5. Pier drifts in the seismically isolated bridge are reduced by 34.5% and 65% in longitudinal and transverse direction, respectively, compared to non-isolated bridge.
6. Deck acceleration are decreased in the horizontal and vertical directions due to seismic isolation.
7. The rail sections over the abutment expansion gaps are the most critical locations for the stress limit checks due to the discontinuity between the approach slab and the bridge deck.
8. The peak resultant normal stress in the rails at expansion gaps significantly increase due to seismic isolation but remain within permissible limits. The transverse bending of the rails contributes most to the normal stress.

The findings of this paper show that the deformation and stress limits in the rails are the key performance objectives that governs the design of non-isolated (integral) and isolated bridges. The isolation system should be designed considering the rail stresses as one of the performance objectives. Isolators of higher stiffness and supplementary damping devices at abutments could be used to restrict the differential displacement and reduce the normal stresses in the rail.

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