16th World Conference on Earthquake Engineering, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 4478
Registration Code: S-S-V1464760664

INVESTIGATION OF SEISMIC ISOLATION FOR CALIFORNIA HIGH-SPEED RAIL PROTOTYPE BRIDGE IN THE CONTEXT OF PROBABILISTIC PERFORMANCE-BASED OPTIMUM SEISMIC DESIGN

Y. Li[(1)], R. Astroza[(2)], J.P. Conte[(3)]
[(1)] *Postdoctoral Scholar, University of California, San Diego, yongli@ucsd.edu*
[(2)] *Assistant Professor, Universidad de los Andes, Chile, rastroza@miuandes.cl*
[(3)] *Professor, University of California, San Diego, jpconte@ucsd.edu*

Abstract

Because of the emerging transportation needs, the California High-Speed Rail (CHSR) project has been launched after significant success of high-speed rail systems in different countries around the world. In seismic prone areas, such as California, the seismic risk imposed to high-speed rail bridges is of particular concern to stakeholders and structural engineers. Seismic isolation is a promising solution to enhance the seismic performance of high-speed rail bridges. However, its effectiveness and beneficial effects need to be evaluated considering a comprehensive model of a high-speed rail bridge system (including the structure, soil, rails and their corresponding interactions) and using a probabilistic performance-based performance assessment approach. In this paper, a detailed three-dimensional (3-D) nonlinear finite element (FE) model of a CHSR prototype bridge, including soil-foundation-structure interaction and rail-structure interaction, is developed in *OpenSees*. Based on deterministic and probabilistic performance assessment, the responses (including the bridge structural and rail responses) of the prototype bridge with and without seismic isolation are compared. With a proper selection of seismic risk metrics/features used to quantify the beneficial and/or detrimental effects of seismic isolation under earthquake excitation, a parametric probabilistic seismic demand hazard analysis is performed with respect to the isolator model parameters. A well-posed problem of probabilistic performance-based optimum seismic design of seismic isolators for the CHSR prototype bridge is proposed to illustrate the suitability of seismic isolation for CHSR bridges using a probabilistic performance-based optimum seismic design framework.

Keywords: California high-speed rail bridge; Seismic isolation; Probabilistic performance-based optimum seismic design

1. Introduction

Because of the emerging transportation needs, the California High-Speed Rail (CHSR) project has been launched after significant success of high-speed rail systems in different countries around the world. High-speed rail bridges will be used as primary supporting structures to account for the land features of terrain along the CHSR alignments [1]. Considering the seismic hazard in high seismic risk regions of California and the target high-speed train service [2], seismic risk mitigation requires a proper (e.g., damage-free or low-damage) seismic design of CHSR bridges, either introducing seismic response modification devices (e.g., seismic isolators [3][4] and/or dampers) or employing self-centering schemes (e.g., precast post-tensioned dual-shell columns with rocking capabilities [5]) . Seismic isolation is a promising solution to enhance the seismic performance of high-speed rail bridges by elongating the vibration periods of the structures and adding hysteretic energy dissipation through the nonlinear deformation of the seismic isolation devices during earthquakes [6]. However, its effectiveness and beneficial effects need to be quantified by predicting the seismic response based on an advanced nonlinear finite element (FE) model of the complex bridge system, and needs to be evaluated by characterizing the structural response in probabilistic terms accounting for the existing seismic risk due to the uncertainty in the seismic input [7][8].

In order to predict the dynamic response of a CHSR bridge subjected to earthquake ground motion excitations, proper modeling of the important ingredients of such a complex real-world system is required. In this paper, a prototype bridge was designed in collaboration with engineers at Parsons Brinckerhoff, Inc.. A detailed three-dimensional (3-D) nonlinear FE model of a CHSR prototype bridge, including soil-structure interaction (SSI) and rail-structure interaction (RSI), is developed in *OpenSees* [9]. The seismic responses (including the bridge structural and rail responses) of the prototype bridge with and without seismic isolation are first compared in a single earthquake scenario.

Furthermore, the seismic responses are compared in probabilistic terms accounting for the randomness in the seismic hazard intensity measure (IM) and the record-to-record variability. The well-established probabilistic performance-based assessment framework, PBEE methodology [10][11], is used to propagate the uncertainty in the seismic input to the structural responses, leading to the probabilistic demand conditioned on a specified seismic hazard level and the probabilistic demand (unconditional) accounting for all seismic hazard levels.

The deterministic and probabilistic comparison of the seismic response of the prototype bridge with and without seismic isolation expose the beneficial and detrimental effects of seismic isolation for high-speed rail bridges during earthquakes. In order to study the feasibility and optimality of the design of seismic isolation for CHSR bridges, different design alternatives of seismic isolated CHSR bridges are allowed to be compared in probabilistic terms. Appropriate or optimum design can therefore be sought, which is essentially an inverse problem, in order to achieve the design objectives expressed in risk-based terms to balance the beneficial and detrimental effects. With a proper selection of seismic risk metrics/features used to quantify these effects, a parametric probabilistic seismic demand hazard analysis with respect to the isolator model parameters is performed in this paper. A well-posed problem of probabilistic performance-based optimum seismic design of seismic isolators for the CHSR prototype bridge is proposed to illustrate the suitability of seismic isolation for CHSR bridges. The successful illustration also shows the power of the probabilistic performance-based optimum seismic design framework [4].

2. CHSR Prototype bridge

Figure 1 shows the elevation view of the 9-span CHSR prototype bridge, which is hypothetically located in downtown San Jose with the local site categorized as soil class D [12]. The bridge superstructure, a post-tensioned single-cell box girder (42.0 ft = 12.80 m wide at the top, 17.5 ft = 5.33 m wide at the bottom, 9.5 ft = 2.90 m deep from the top to bottom surface), rests on eight single-column bents and two seat-type abutments at the bridge ends. The two interior expansion joints subdivide the bridge structure into three three-span frames. A pair of slotted hinge joint (SHJ) devices is installed across each interior expansion joint, to restrain the relative transverse displacement of adjacent bridge segments while allowing a specified amount of relative movement in the longitudinal direction [13]. Twelve pairs of seismic isolators are installed between the bridge deck and the bents or abutments, with one pair at each continuous joint and two pairs at each interior expansion joint. The pier columns are of circular cross-section (i.e., 8.0 ft = 2.44 m in diameter) and of identical height (i.e., 35.0 ft = 10.67 m from the top of the pile cap to the top surface of the pier head). The foundation system consists of two types of rigidly capped pile group foundation with vertical cast-in-place drilled shafts: 2×2 pile groups supporting the eight piers and 2×3 pile groups supporting the abutment substructures. All these piles are of circular cross-section with a length of 120.0 ft (36.58 m) and a diameter of 6.5 ft (1.98 m). Compared to the seismic isolated CHSR prototype bridge (IB) presented above, the comparable non-isolated bridge (NIB) has rigid pier-deck connections at all piers and regular bearing pads at both abutments to support the bridge deck. In addition, the expansion joint gap at the abutments is 1.0 in (25.4 mm). for the NIB compared to 4.0 in (101.6 mm) for the IB.

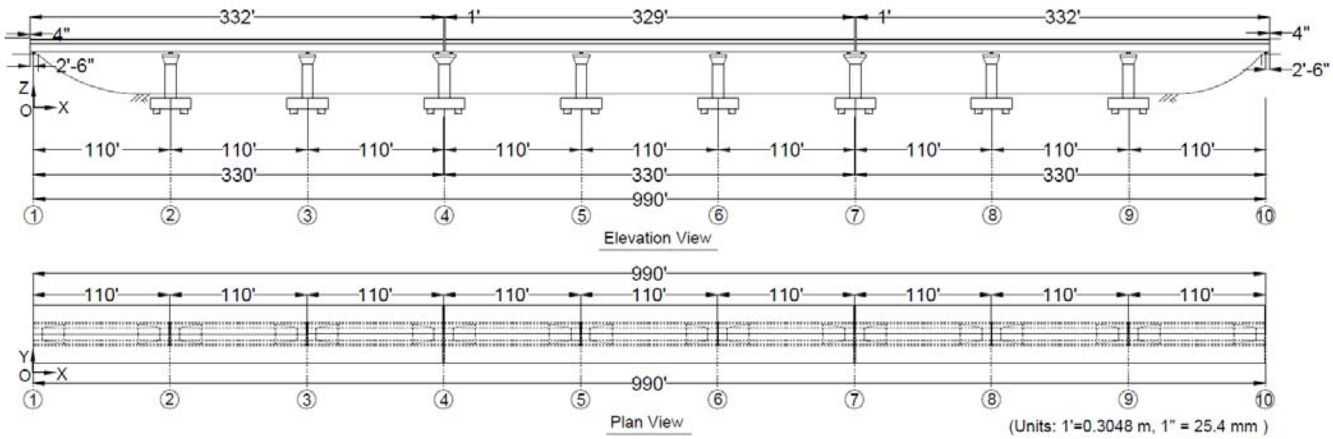

Fig. 1 – Elevation and plan view of the CHSR prototype bridge

A 3-D nonlinear FE model of the CHSR prototype bridge accounting for SSI [14] and RSI [15] was developed using state-of-the-art modeling techniques (e.g., nonlinear beam-column element with fiber sections for pier columns, dynamic *p-y* approach for soil-pile-foundation system, zero-length elements for seismic isolators and SHJ connections, etc.). Figure 2 shows the elevation view of the modeling scheme used for the nonlinear 3-D FE model of the CHSR prototype bridge. The FE model includes all the significant ingredients of the high-speed rail bridge system, e.g., the rail, the direct fixation fasteners connecting the rail to the bridge deck/roadbed subgrade, bridge deck, seismic isolators, SHJ, the abutments, the pile foundations, and a segment of track-subgrade system (i.e., rail extension) at each end of the bridge and rail boundary spring to account for the longitudinal support provided by an infinitely long rail clamped to the track base through the direct fixation fasteners.

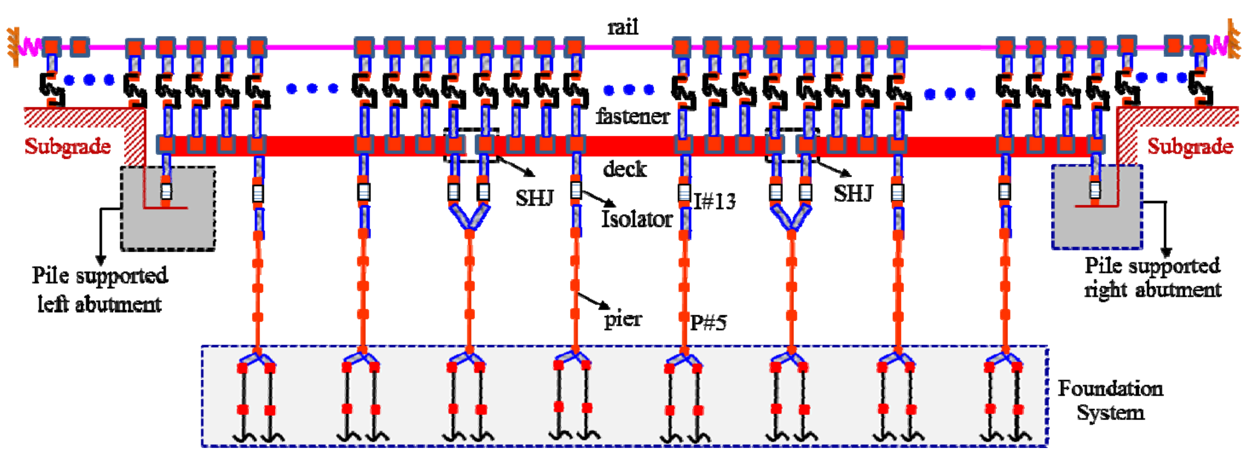

Fig. 2 – Elevation view of modeling scheme for nonlinear 3-D FE model of the CHSR prototype bridge

The mass or inertia properties of the bridge system are lumped at nodes based on the mass density and the volume of structural components. The commonly used Rayleigh damping model with specified damping ratios (2%) at two selected modes (i.e., the first transverse and the first longitudinal modes) is applied to all structural components of the bridge model (i.e., girders, piers, piles), but is not applied to the highly nonlinear zero-length elements used to model the seismic isolators, SHJ devices, shear keys, and soil springs [16], since the inherent energy dissipation capability is explicitly captured through the force-deformation hysteresis.

3. Seismic response comparison under a single earthquake scenario

Nonlinear time history analysis is employed in this study for dynamic response prediction of the bridge system subjected to an earthquake ground motion. However, for a fair comparison of the IB and NIB under a single (deterministic) earthquake scenario, the seismic response simulations must be performed subjected to an earthquake record corresponding to the same seismic hazard level for both the IB and NIB. Therefore, a scaled near-fault ground motion record (1979 Imperial Valley earthquake recorded at the El Centro station) is selected to ensure that the selected seismic input corresponds to the same seismic hazard level for the IB and the NIB. The fault parallel (FP) and fault normal (FN) components of the ground motion are applied in the longitudinal and transverse direction of the bridge, respectively. Deconvolution analysis is performed for the scaled actual ground motion record defined above to obtain the depth-varying earthquake ground motion inputs, which are applied at the far-field ends of the soil-springs. Therefore, multiple-support excitation is used for the nonlinear time history analysis.

Figure 3 shows the seismic response comparison for the 1979 Imperial Valley Earthquake scaled to the operating based earthquake (OBE) hazard level [2], with expected return period of 50 years. It is observed that the seismic isolation can signigicantly reduce the total base shear forces in the longitudinal and transverse directions of the bridge (Figure 3a). This is a beneficial effect of seismic isolation to potentially reduce the foundation cost. In contrast, the seismic isolation can possibly cause detrimental effects to the rails due to increasing displacement in the bridge deck: e.g., more axial rail stress as shown in the envelopes of the rail axial stress due to axial force *P* alone, and more normal stress due to axial force and bi-directional bending, especially at the interior expansion joints (Figure 3b). The large rail stress concentration at the bridge abutment gap is due to the existence of the shear key gaps, which exist for both the IB and NIB. Besides, with seismic isolation incorporated in the CHSR prototype bridge, the absolute deck acceleration, the pier colum drift, the pile cap translation and rotation also decrease, while the deck displacement relative to the pile cap increases.

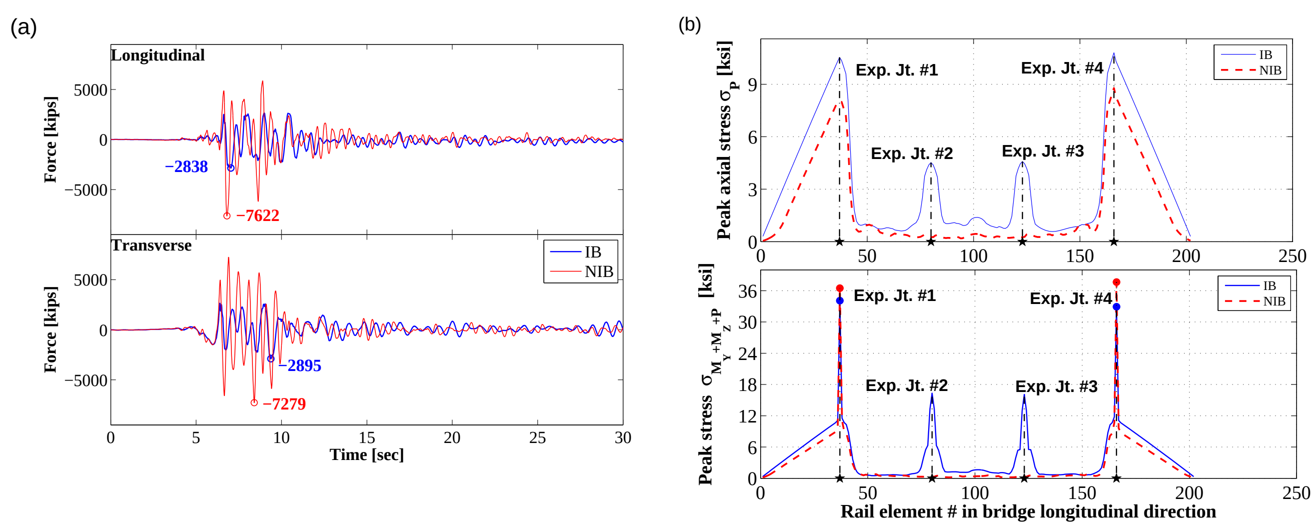

Fig. 3 – Seismic response comparison under a single earthquake scenario (1979 Imperial Valley): (a) time histories of total base shear force across all piers (piers P#1-P#8) in longitudinal and transverse directions, (b) envelopes of the rail axial stress due to *P* alone and rail maximum normal stress due to axial force and bi-directional bending in the outside-most rail line

4. Seismic response comparison in probabilistic terms

In the face of uncertainty (e.g., in the seismic input), the performance evaluation of the prototype bridge during an earthquake event requires a probabilistic assessment of structural response subjected to future earthquakes in its expected design life. This prompted the development of a well-modularized methodology, referred to as the Performance-based Earthquake Engineering (PBEE) methodology. This methodology has been promoted and developed by the Pacific Earthquake Engineering Research (PEER) Center in the past two decades [10][11]. It consists of four analytical steps: seismic hazard analysis, demand hazard analysis, damage hazard analysis, and the loss hazard analysis, to calculate the probability of exceedance of earthquake intensity measure (IM), the structural response (EDP), limit states exceedance or damage measure (DM), decision variables (DV, e.g., dollars, down time, and deaths), sequentially. These four analytical steps use the total probability theory considering a sequence of probabilistic conditioning and un-conditioning for uncertainty propagation. In this paper, only probabilistic seismic hazard and demand hazard analysis is performed for the CHSR prototype bridges with and without seismic isolation to compare the probabilistic demand (unconditional) accounting for all seismic hazard levels.

To assess the seismic hazard at a specific site, the probabilistic seismic hazard analysis aims to characterize the uncertainty in the earthquake IM (e.g., spectral acceleration at the fundamental period of the structure Sa (T_1, $\xi = 5\%$)). In this paper, the web application Probabilistic Seismic Hazard Analysis (PSHA) tool [17] developed by the U.S. Geological Survey is adopted to obtain the probabilistic seismic hazard analysis results and used for the second step of PBEE, the probabilistic demand hazard analysis. This step will provide the mean annual rate of IM exceeding a certain threshold value *im*, $\nu_{IM}(im)$.

Probabilistic demand hazard analysis consists of computing the mean annual rate of structural response parameter EDP exceeding a specified threshold value *edp,* $\nu_{EDP}(edp)$. To compute the demand hazard (unconditional by accounting for all seismic hazard levels), the unconditional demand hazard (expressed as $P\left[EDP > edp \middle| IM\right]$) needs to be calculated using time history analysis results subjected to an ensemble of earthquake ground motion records. Forty earthquake ground motion records, each with two horizontal components were selected for the bridge site and scaled by three different factors (i.e., 0.4, 0.8 and 1.4) to increase the ensemble size (i.e., 120 earthquake records in total). A statistic model is built based on linear regression in the log-space of the peak response data edp_i and the corresponding earthquake intensity im_i, where i = 1, 2, …, 120, to predict the peak structural response EDP from IM probabilistically. This is referred to as the "cloud method" for unconditional demand hazard analysis [18]. After achieving the conditional demand hazard analysis results, the demand hazard can be computed by accounting for the uncertainty in the seismic input, as indicated in the convolution integral in Eq. (1):

$$\nu_{EDP}(edp) = \int_{IM} P\left[EDP > edp \middle| IM\right] \left| d\nu_{IM}(im) \right| \tag{1}$$

Figure 4 shows the probabilistic seismic demand hazard comparison in terms of the demand hazard curves of the total base shear force across all piers (piers P#1-P#8) in the transverse direction and the rail axial stress due to P alone in the outside-most rail line. The same beneficial and detrimental effects as observed in the deterministic scenario are identified through the probabilistic demand hazard comparison after various seismic hazard levels are accounted for by un-conditioning. More importantly, the beneficial and detrimental effects are probabilistically quantified here. The introduction of seismic isolation pushed the demand hazard curve of total base shear force to the left, which means that the total base shear force demand, corresponding to the same demand hazard level (e.g., with return period of 50 years), is much smaller for IB compared with the case for NIB. In contrast to the beneficial effect of seismic isolation, more seismic demand risk is imposed on the rail stress as a detrimental effect, i.e., the rail stress level is higher for the same demand hazard level (e.g., with return period of 50 years). Similar comparison in terms of the total base shear force across all piers (piers P#1-P#8) in the longitudinal direction and the rail stress due to axial force P and bi-directional bending in the outside-most rail line and other response quantities (EDPs) are not shown here for space limitation but reported in [19].

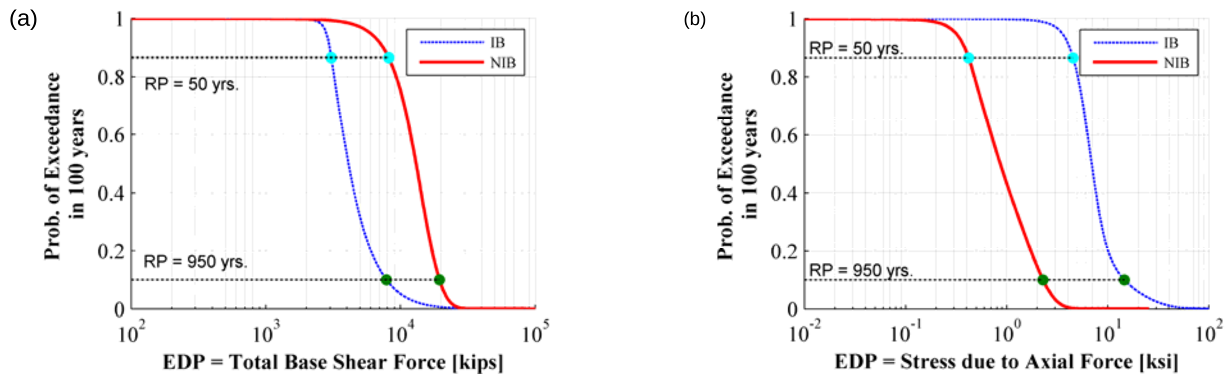

Fig. 4 – Probabilistic seismic demand hazard comparison: (a) seismic demand hazard curves of the total base shear force across all piers (piers P#1-P#8) in the transverse direction, (b) seismic demand hazard curves of the rail axial stress due to P alone in the outside-most rail line

5. Parametric probabilistic demand hazard analysis and risk-based optimization

A parametric probabilistic demand hazard analysis is performed for the CHSR prototype bridge with respect to the isolator model parameters, i.e., by varying the yield strength F_y and the initial stiffness K_1 while keeping the post-yield stiffness ratio constant (10%). In order to have a scalar measure of seismic demand as a risk index for the purpose of comparison, some risk features/indexes (e.g., statistics of the response quantities) are extracted from the conditional and unconditional demand hazard analysis results. For example, given on the OBE seismic hazard level, the conditional mean value (referred to as $E\left[\bullet|OBE\right]$), the conditional median value (referred to as $\eta\left[\bullet|OBE\right]$), or conditional 95th percentile (referred to as $Pctl.^{95th}\left[\bullet|OBE\right]$) of a certain structural response EDP can be used to measure the seismic risk imposed to the bridge. Similarly, the statistics conditioned on maximum considered earthquake (MCE) level with return period of 950 years [2] as well as the unconditional demand statistics of an EDP can be used as risk features/metrics. Various structural response parameters (EDPs) can be adopted in defining the risk features, e.g., the peak absolute deck acceleration in the transverse direction $AA_{trans.}^{deck}$, the maximum bending moment at the bottom of the pier column P#5 in the transverse direction $M_{trans.}^{P\#5}$, the maximum bending moment in all the piles under the pier column P#5 $M_{trans.}^{piles,\,P\#5}$, the rail stress due to axial force only at the abutment expansion joint $\sigma_P^{rail,\,\text{abut.}}$, rail stress due to axial force and bi-directional bending at the abutment expansion joint $\sigma_{P+M}^{rail,\,\text{abut.}}$, etc.

To explore how the probabilistic demand hazard analysis results change as a function of the isolator model parameters, Figure 5 shows the topology of two selected risk features in the design variable space (defined over the domain of the yield strength F_y and the initial stiffness K_1). When increasing the degree of seismic isolation (i.e., decreasing the yield strength and initial stiffness of seismic isolators), the risk feature value defined based on the total base shear force decreases and become much lower than the corresponding value of the NIB case denoted by the triangle in the figure (x_0 denotes the initial design). On the other side, when increasing the degree of seismic isolation, the risk feature value defined based on the rail stress increases and become much higher than the corresponding value of the NIB case. A trade-off needs to be achieved to balance the beneficial and detrimental effects through solving an optimization problem with an objective function and the constraints expressed in probabilistic terms. This practical needs call for a probabilistic performance-based optimum seismic design framework, as proposed by the authors in [4], by closing the open loop of using existing PEER PBEE methodology through optimization, i.e., extending the forward analysis of PBEE for performance assessment to practical design purposes.

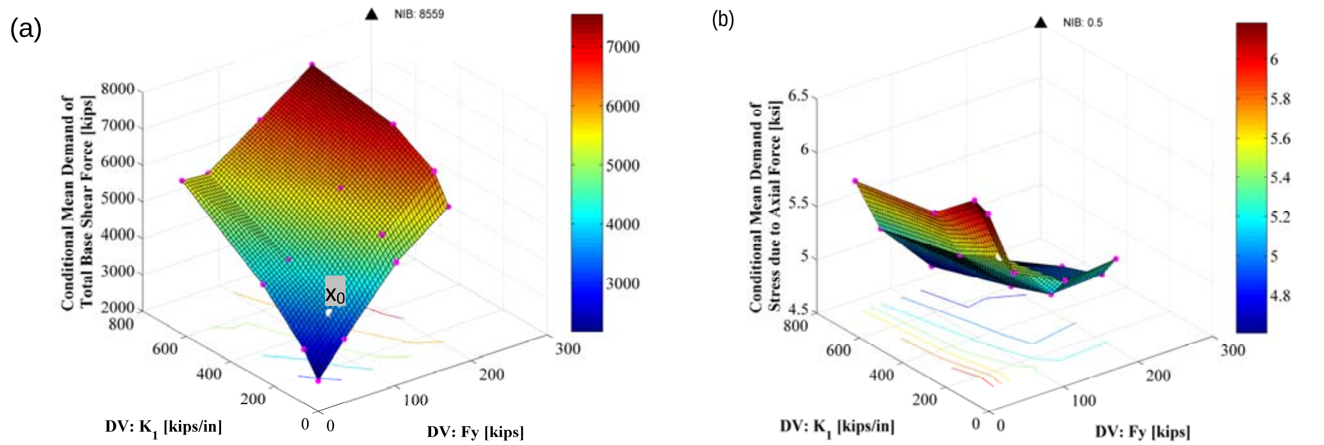

Fig. 5 – Parametric probabilistic demand hazard analysis using risk measure of: (a) conditional mean demand of the total base shear force across all pier columns on the OBE seismic hazard level, (b) conditional mean demand of axial rail stress due to the axial force on the OBE seismic hazard level

In the proposed probabilistic performance-based optimum seismic design framework, the risk features can serve as objective or constraint functions in the formulation of optimization problems [20][21][22]. The probabilistic performance-based optimum design problem can be formulated to account for the probabilistic design constraints associated with the conditional demand on both OBE and MCE seismic hazard levels, which are the two hazard levels as specified in the seismic design criteria [2]. The mathematical problem is formulated in Eq. (2), which aims to minimize the conditional mean demand of the total base shear force across all columns in transversal direction on the MCE seismic hazard level while satisfying all the constraints defined on two discretized seismic hazard levels. Here, the constraint on the maximum relative end rotation of the last element of pier column P#5, $\theta_{trans.}^{Rot.\ P\#5}$, is considered to limit the seismic damage of the pier columns when seismic isolation is adopted. The constraint on the seismic isolation component is to limit the maximum deformation level experienced by the seismic isolator component I#13 in the transverse direction (denoted as $Def._{trans.}^{Isolator\#13}$) under the MCE seismic hazard level.

$$
\begin{aligned}
&\underset{\substack{34.1kips \le F_y \le 204.7kips \\ 120kips/in \le K_1 \le 740kips/in}}{\textbf{\textit{Minimize}}} \quad \text{conditional mean: } E\left[F_{trans.}^{\text{TBS, } all\ columns} \mid OBE\right] \\
&(1)\ E\left[AA_{trans.}^{deck} \mid OBE\right] \le 0.35g \\
&(2)\ Pctl.^{95th}\left[AA_{trans.}^{deck} \mid OBE\right] \le 0.5g \\
&(3)\ Pctl.^{95th}\left[M_{trans.}^{P\#5} \mid OBE\right] \le M_{cr}^{pier}(1.5\times 10^4 kips\text{-}ft) \\
&(4)\ Pctl.^{95th}\left[M_{trans.}^{piles,\ P\#5} \mid OBE\right] \le M_{cr}^{pile}(5.3\times 10^3 kips\text{-}ft) \\
&(5)\ Pctl.^{95th}\left[\sigma_{P}^{rail,\ \text{abut.}} \mid OBE\right] \le 12.5ksi \\
&(6)\ \eta\left[\sigma_{P+M}^{rail,\ \text{abut.}} \mid OBE\right] \le 42.5ksi \\
&(7)\ Pctl.^{95th}\left[\theta_{trans.}^{Rot.\ P\#5} \mid MCE\right] \le 1.13\% \\
&(8)\ Pctl.^{95th}\left[Def._{trans.}^{Isolator\#13} \mid MCE\right] \le 20in \\
&(9)\ Pctl.^{95th}\left[M_{trans.}^{P\#5} \mid MCE\right] \le M_{e}^{pier}(4.15\times 10^4 kips\text{-}ft) \\
&(10)\ Pctl.^{95th}\left[M_{trans.}^{piles,\ P\#5} \mid MCE\right] \le M_{e}^{pile}(1.2\times 10^4 kips\text{-}ft)
\end{aligned}
\qquad (2)
$$

The solution to this well-posed optimization problem in Eq. (2) is illustrated in Figure 6. With the addition of the constraints (denoted by the constraint boundaries in the figure) imposed one by one, the feasible domain of the design variable space is shrinking more and more. The objective function is minimized at the optimum solution (shown by a star in Figure 6), namely the conditional mean demand of total base shear force across all columns in the transverse direction attained the minimum with all the other ten probabilistic constraints satisfied: six on OBE seismic hazard level and four on MCE seismic hazard level. This implies that seismic isolation can potentially be used for California High-speed rail bridges in order to take the full advantage of the beneficial effects (i.e., reducing total base shear and thus reducing foundation cost) of seismic isolation during earthquakes, given that the all the detrimental effects of seismic isolation are limited with sufficient confidence. In addition, the illustrative example also strengthened the power of the proposed probabilistic performance-based optimum seismic design framework [4] to solve risk-based design problem of large complex civil infrastructure systems.

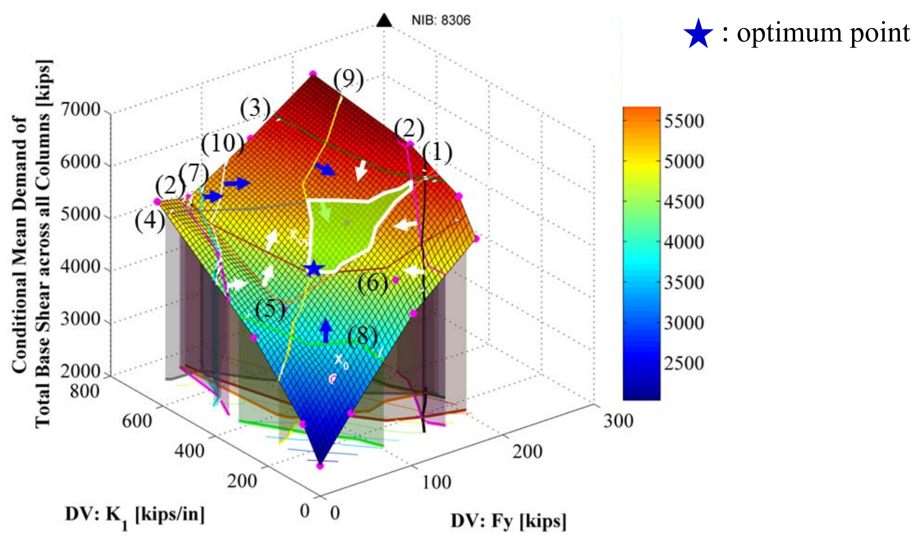

Fig. 6 – The optimization problem formulated for probabilistic performance-based design with constraints on two seismic hazard levels (i.e., OBE & MCE)

6. Conclusion

This paper investigates the beneficial and detrimental effects of seismic isolation on the performance of a 9-span California High-Speed Rail (CHSR) prototype bridge subjected to earthquake excitations. A detailed three-dimensional (3-D) nonlinear finite element (FE) model of a CHSR prototype bridge, including soil-structure interaction (SSI) and rail-structure interaction (RSI), is developed in *OpenSees* using state-of-the-art modeling techniques. The beneficial and detrimental effects of seismic isolation in CHSR bridges are illustrated and quantified in both the deterministic sense (under a single earthquake scenario) and probabilistic terms (seismic demand hazard) in the context of probabilistic performance-based earthquake engineering (PBEE) framework. The seismic isolation is potentially promising in reducing the seismic force demand (e.g., total base shear force) and thus reducing the foundation cost significantly. A well-posed optimization problem of probabilistic performance-based optimum seismic design of seismic isolators for the CHSR prototype bridge is proposed to illustrate the suitability of seismic isolation for CHSR bridges. The successful illustration also displays the power of the probabilistic performance-based optimum seismic design framework.

Acknowledgements

Support of this research by the Pacific Earthquake Engineering Research (PEER) Center's Transportation Systems Research Program under Award No. 1107-NCTRCJ is gratefully acknowledged. The authors wish to thank the following individuals for their help and insightful discussions related to the design and modeling of the CHSR prototype bridge considered in this paper: Mr. Thomas B. Jackson, Mr. Pang Yen Lin, and Dr. Kongsak Pugasap at Parsons Brinckerhoff, Inc., located in San Francisco; Dr. Roy Imbsen at Earthquake Protection System, Inc.; Prof. Steve Mahin at the University of California, Berkeley; Prof. Ross W. Boulanger at the

University of California, Davis; Prof. Scott J. Brandenberg and Prof. Ertugrul Taciroglu at the University of California, Los Angeles. Any opinions, findings, conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the sponsors.

References

[1] CHSRA (California High-Speed Rail Authority) (2015): Project update report to the California State Legislature. *www.hsr.ca.gov*, California.

[2] CHSRA (California High-speed Rail Authority) (2012): *California High-speed Train Project Design Criteria*, San Francisco, California.

[3] Li Y., Conte, JP. (2013): Investigation of Seismic Isolation for California High-speed Rail Prototype Bridge. *In Proc. 7th National Seismic Conference on Bridges and Highways*, Oakland, USA, May 20-22, 2013.

[4] Li Y, Conte JP. (2015): Probabilistic performance-based optimum seismic design of seismic isolation with applications to California High-Speed Rail prototype bridge. *Proc. of 12th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP12)*, Vancouver, Canada, July 12 -15, 2015.

[5] Guerrini G., Restrepo JI, Massari M and Vervelidis A. (2014): Seismic behavior of posttensioned self-centering precast concrete dual-shell steel columns. *Journal of Structural Engineering*. **141** (4), 04014115.

[6] Naeim F, Kelly JM. (1999): *Design of Seismic Isolated Structures: from Theory to Practice*, John Wiley & Sons. New York, NY.

[7] Li Y, Conte JP, Barbato M. (2015): Influence of the time-varying frequency content in earthquake ground motions on seismic response of linear elastic systems. *Earthquake Engineering and Structural Dynamics*, In Press, doi: 10.1002/eqe.2707.

[8] Baker JW, Cornell CA. (2003): Uncertainty specification and propagation for loss estimation using FOSM methods. *PEER Technical Report 2003/07*. Berkeley, California.

[9] Mazzoni S, McKenna F, Scott MH, Fenves GL. (2006): *Open system for earthquake engineering simulation, user command-language manual*. Pacific Earthquake Engineering Research Center, Berkeley, California.

[10] Cornell C, Krawinkler H. (2000): Progress and challenges in seismic performance. *PEER Center News*, **3**(2).

[11] Günay S, Mosalam KM. (2013): PEER performance-based earthquake engineering methodology, revisited. *Journal of Earthquake Engineering*, **17**(6), 829-858.

[12] Building Seismic Safety Council (BSSC). (2001): *2000 Edition, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA-368, Part 1 (Provisions)*: developed for the Federal Emergency Management Agency, Washington, D.C.

[13] Priestley MJN, Seible F, Calvi M. (1996): *Seismic Design and Retrofit of Bridges*, John Wiley and Sons, New York, NY.

[14] Boulanger RW, Curras CJ, Kutter BL, Wilson DW, Abghari A. (1999): Seismic soil-pile-structure interaction experiments and analysis. *Journal of Geotechnical and Geo-Environmental Engineering*, **125**(9), 750-759.

[15] UIC. (2001): *Track/bridge interaction. Recommendations for calculations*. UIC Code 774-3R. (2nd ed.), International Union of Railways.

[16] Charney, FA. (2008): Unintended consequences of modeling damping in structures. *Journal of Structural Engineering*, **134**(4), 581-592.

[17] USGS. (2008): *2008 Interactive De-aggregations*, http://earthquake.usgs.gov/hazards/

[18] Baker JW. (2005): Vector-valued ground motion intensity measures for probabilistic seismic demand analysis. *Ph.D. dissertation*, Stanford University, Stanford, California.

[19] Li Y. (2014): Probabilistic performance-based optimum seismic design of seismic isolation for California High-Speed Rail prototype bridge. *Ph.D. dissertation*, University of California, San Diego, California.

[20] Austin M, Pister K, Mahin SA. (1987a): Probabilistic design of moment-resistant frames under seismic loading. *Journal of Structural Engineering*, **113**(8), 1660-1677.

[21] Austin M, Pister K, Mahin SA. (1987b): Probabilistic design of earthquake-resistant structures. *Journal of Structural Engineering*, **113**(8), 1642-1659.

[22] Cohn M. Dinovitzer A. (1994): Application of structural optimization. *Journal of Structural Engineering*, **120**(2), 617-650.