Parametric Probabilistic Seismic Performance Assessment Framework for Ordinary Standard Bridges

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ABSTRACT: This paper focuses on the assembly and implementation of a full-fledged parametric probabilistic seismic performance assessment framework for ordinary standard bridges (OSBs) in California. The framework stems from the performance-based earthquake engineering (PBEE) assessment methodology developed under the auspices of the Pacific Earthquake Engineering Research (PEER) Center. It involves a sequential execution of analytical steps to arrive at estimates of performance measures which, for example as considered in this study, are the mean return periods (MRPs) of exceedances for a selected set of limit-states (*LS*s). Improvements from state-of-the-art literature related to various stages of the PEER PBEE assessment framework are incorporated. This includes: (1) introduction of an improved intensity measure (*IM*), i.e., average spectral acceleration over a period range, for probabilistic seismic hazard analysis (PSHA), (2) conditional mean spectrum (CMS)-based site-specific risk-consistent ground motions selection for ensemble nonlinear time-history analyses involved in probabilistic seismic demand hazard analysis (PSDemHA), (3) introduction of material strain-based engineering demand parameters (*EDP*s), (4) identification of practical damage *LS*s, and (5) development of strain-based fragility functions required in probabilistic seismic damage hazard analysis (PSDamHA) for the considered *LS*s. Four distinct testbed OSBs are selected for the study. A two-dimensional design parameter space is defined in terms of typical primary design variables involved in seismic design of OSBs, i.e., the column diameter and the column longitudinal steel reinforcement ratio. Computational models of the as-designed bridges as well as their re-designs spawned by varying the primary design variables subject to practical constraints are assessed using the implemented framework. For each testbed OSB, and for each of the considered *LS*s, a smooth surface is fitted to the MRPs computed for all the re-designs of the bridge in the primary design parameter space. Topologies of these surfaces are explored. Feasible design domains in the two-dimensional design parameter space are identified. Safety of the as-designed version and feasibility domain for the re-designs of each testbed OSB are examined and discussed.

Driven by the necessity to meet changing public expectations in the wake of natural disasters, such as earthquakes, the structural engineering community is moving towards more rational, risk-informed, and transparent approaches to structural design, amidst which probabilistic performance-based seismic design (PBSD) has emerged as the most scientific and promising one. PBSD involves designing a structure to meet more refined and non-traditional performance objectives explicitly stated in terms of the risk associated with the exceedance of critical *LS*s or certain tolerable thresholds of monetary loss, downtime, etc. (i.e., probability of *LS* or threshold exceedance in a specified exposure time). Performance objectives stated as such will not only allow an active participation of the public and stakeholders in the design and decision-making process thereby making it more rational, scientific, and transparent, but also lead to greater societal awareness of earthquake risk and consequences. Risk-targeted statements of performance objectives and designing structures to achieve such objectives inevitably calls for rigorous treatment and propagation of pertinent uncertainties involved at various stages of the performance assessment process. This has motivated and paved the way for the structural engineering community to work towards identification and filling of knowledge gaps and hence make considerable advancement in the realm of probabilistic PBEE over the last few decades. Consistently improving over time, such efforts have culminated in the fully probabilistic, rigorous and advanced assessment framework (Porter 2003) developed under the auspices of the PEER Center.

The PEER PBEE assessment framework, integrating site-specific seismic hazard analysis, structural demand analysis, damage analysis, and loss analysis, in a comprehensive and consistent probabilistic framework, has been mainly developed for analysis and assessment and not directly for design. The inherent theoretical complexity of the full-fledged PEER PBEE methodology also adds to its impeded implementation in the seismic design practice of bridges, a rather less trodden area in terms of PBEE applications as compared to building structures. This methodology, however, has been recommended as a future alternative for bridge seismic design in a recent study under the National Cooperative Highway Research Program (NCHRP 2013).

Overarchingly aiming towards a rigorous framework for PBSD of bridges, this paper presents the conceptualization and implementation of a generalized workflow for full-fledged parametric probabilistic seismic performance assessment of conventional, multiple-span, skewed reinforced concrete (RC) bridges, referred to as Ordinary Standard Bridges (OSBs), in California. Probabilistic performance-based assessments of parametrically redesigned versions of a set of testbed OSBs are carried out to investigate the effect of varying key structural design parameters on targeted structural performance measures, thus laying the groundwork for solving the design problem, which is an inverse assessment problem.

1. TESTBED OSBs AND COMPUTATIONAL MODELS

Several testbed OSBs located in regions with disparate levels of seismicity in California are selected for this study in order to cover a range of realistic design situations for OSBs and ensure that the parametric probabilistic performance-based assessment framework formulated herein is general within its scope and is applicable to a variety of potential design scenarios. Four existing California OSBs, i.e., Bridges A, B, C and MAOC (see Figure 1) previously studied in recent research projects funded by the California Department of Transportation (Caltrans) and PEER, are selected for this purpose.

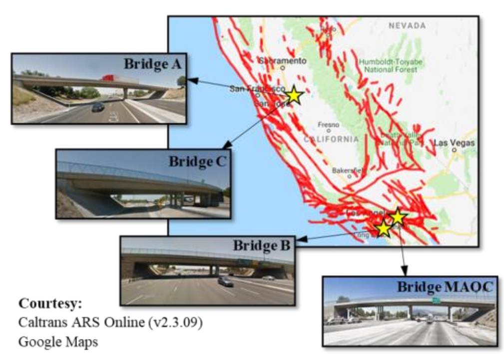

Figure 1: Location of testbed OSBs

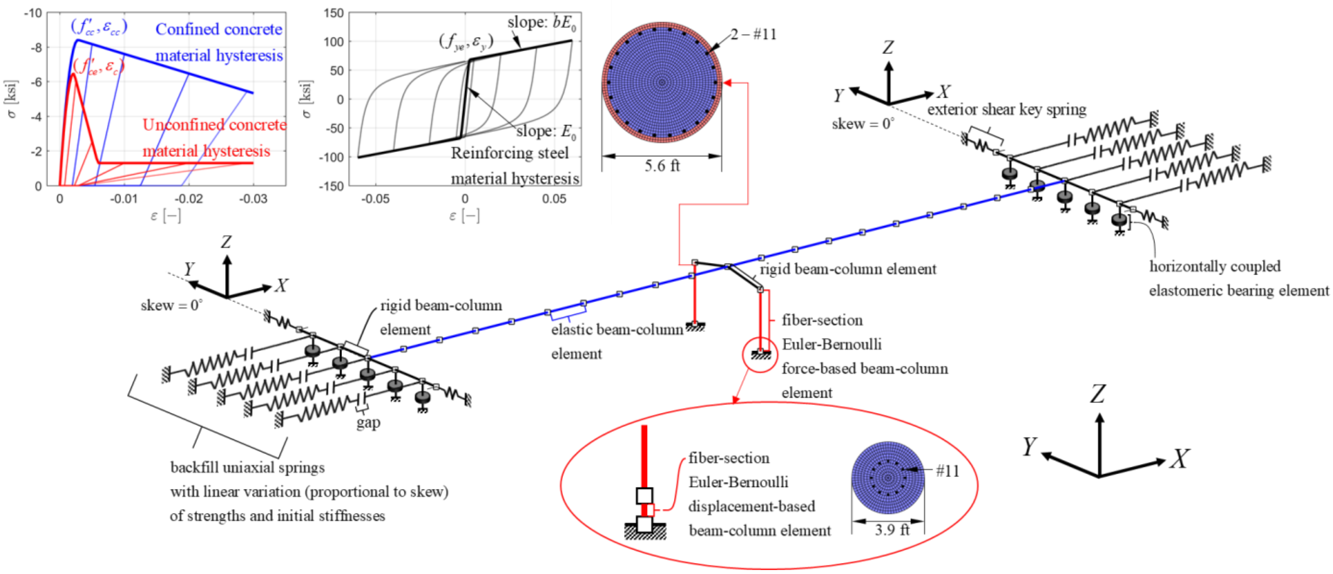

Figure 2: Schematic representation of the FE model of Bridge B in OpenSees

The selected testbed bridges conform to the definition of OSBs as described in Caltrans Seismic Design Criteria (SDC) v1.7 (Caltrans 2013). For a detailed description of the geometric and mechanical properties of the testbed OSBs, the reader is referred to the technical reports accompanying the previous studies (Beckwith et al. 2015; Kaviani et al. 2014).

Three-dimensional nonlinear finite element (FE) models (consisting of nonlinear fiber-section beam-column elements and nonlinear springs) of these bridges are constructed in OpenSees (Mazzoni et al. 2007), the open-source FE analysis software framework developed at PEER. Initially inherited Tcl input files of the OpenSees models of these bridges from previous Caltrans/PEER funded projects were revisited, parameterized, and improved (Deb et al. 2018). A schematic representation of the computational model of one of the four testbed OSBs (Bridge B) is shown in Figure 2.

2. PEER PBEE ASSESSMENT METHODOLOGY

Metrics of structural performance considered in this study are defined as the mean annual rates (MARs), or equivalently the MRPs, of exceedances for a selected set of practical *LS*s. The PEER PBEE methodology breaks down the task of predicting probabilistically the future seismic performance of a structure in terms of the MAR (ν_{LS_k}) of exceeding a *LS* (say LS_k belonging to the selected set of discrete *LS*s) into three analytical steps, i.e., (1) PSHA in terms of the ground motion *IM*; (2) PSDemHA in terms of the *EDP* associated with the considered *LS*; and (3) PSDamHA for the considered *LS*. These steps are pieced together (integrated) using the Total Probability Theorem (TPT) as shown in Eq. (1).

$$\nu_{LS_k} = \int_{IM}\int_{EDP_k} P\left[Z_k < 0 \mid EDP_k = \delta\right]\cdot f_{EDP_k|IM}\left(\delta \mid x\right)\cdot d\delta \cdot \left|d\nu_{IM}(x)\right| \tag{1}$$

where $P[Z_k < 0 \mid EDP_k = \delta]$ is the conditional probability of exceedance of LS_k (i.e., safety margin $Z_k = C_k - EDP_k < 0$ where C_k is the structural capacity associated with LS_k) given a specific value δ of the associated *EDP* (i.e., EDP_k), $f_{EDP_k|IM}(\delta \mid x)$ is the conditional probability distribution of EDP_k given a specific value x of the *IM*, and $\nu_{IM}(x)$ is the MAR of *IM* exceeding the specific value x.

Improvements of several aspects in the various stages of the state-of-the-art PEER PBEE assessment methodology are incorporated in this study. Brief accounts of such enhancements are presented next.

2.1. Improved Seismic Intensity Measure

The spectral acceleration averaged over a period range ($S_{a,\,\mathrm{avg}}$), originally proposed by Baker and Cornell (2006), is selected as the ground motion *IM* in this study. Defined as the geometric mean of spectral accelerations at different periods, $S_{a,\,\mathrm{avg}}$ is mathematically given by:

$$S_{a,\,\mathrm{avg}}\left(T_1,\ldots T_n\right)=\left[\prod_{p=1}^{n} S_a\left(T_p\right)\right]^{1/n} \qquad (2)$$

where $[T_1, T_n]$ is the averaging period range selected so as to account for the following phenomena not captured by the traditionally used *IM*, i.e., the spectral acceleration at a single predominant period of the structure: (i) uncertainty in predicting the natural period of the pre-dominant mode of vibration of RC structures such as OSBs; (ii) change in natural periods of RC structures in going from pristine conditions to cracked states under service loads; (iii) structural periods elongation due to accumulation of damage during an earthquake which leads to higher correlation of structural response with spectral accelerations at longer periods; and (iv) difference in computed periods of fundamental modes of vibration in the two orthogonal directions of the bridge.

Closed-form probabilistic characterization of $S_{a,\,\mathrm{avg}}$ given an earthquake scenario (i.e., a magnitude and source-to-site distance pair), required in PSHA calculations, is obtained using existing attenuation relationships (e.g., Boore and Atkinson 2008) for spectral accelerations at single periods. Results of seismic hazard analyses for spectral accelerations at single periods, readily available from standard PSHA software tools such as OpenSHA (Field et al. 2003), are inventively utilized to evaluate the MARs of exceedance of specific values of the chosen novel *IM*.

2.2. Risk-consistent site-specific ground motion record selection

The conditional mean spectrum (CMS) (Baker and Cornell 2006; Lin et al. 2013), representing the expected spectral shape given a specific value of the considered *IM*, is used as the target spectrum for ground motion record selection in this study. Six different seismic hazard levels corresponding to MRPs of *IM* exceedances equal to 72, 224, 475, 975, 2475, and 4975 years (numbered I through VI, respectively) are chosen and ensembles of 100 bi-axial horizontal ground motions per hazard level are selected for the seismic response assessment of each (as-designed or re-designed) testbed OSB considered.

A ground motion selection algorithm, originally developed by Jayaram et al. (2011) and recently modified by Kohrangi et al. (2017) to include $S_{a,\,\mathrm{avg}}$ as the conditioning *IM*, is implemented for the selection of site-specific risk-consistent ensembles of ground motion records representative of the six seismic hazard levels considered. Given a seismic hazard level and the corresponding value of *IM*, the conditional joint probability structure of correlated spectral accelerations at different periods is first determined. The algorithm then picks earthquake records from the NGA database that, as an ensemble, follow the complete probability structure of the target conditional spectrum defined for that hazard level. With the CMS chosen as the target spectrum over the uniform hazard spectrum (UHS), which is a more commonly used, excessively conservative, spectral envelope-based target spectrum, the natural spectral shapes of the selected ensemble of earthquake ground motion records are preserved.

2.3. Damage LSs and associated material strain-based EDPs

Flexural plastic hinge regions in RC columns of an OSB are meant to act as structural fuses in a seismic event and thereby dissipate energy through controlled inelastic material behavior.

Three damage *LS*s related to the desirable (targeted) failure mode concerning RC bridge columns (i.e., flexural hinging of columns) are considered in this study. These *LS*s are: (1) concrete cover crushing, (2) a precursor to rebar buckling, and (3) a precursor to rebar fracture. The first *LS* represents superficial damage to a bridge column and requires cosmetic repair work primarily to prevent corrosion of rebars. The other two *LS*s represent ultimate *LS*s, exceedances of which lead to significant compromise of structural integrity and imminent structural collapse.

Displacement-based *EDP*s have been found to correlate better to structural damage as compared to force-based *EDP*s (Priestley et al. 2007). Traditionally, measures of deformation such as displacements, drift ratios, curvatures, etc. have been used as *EDP*s. However, for RC flexural members, such as columns, deformations can be directly and most reliably related to structural damage through material strains (Priestley et al. 2007). Table 1 lists the strain-based *EDP*s associated with the selected set of *LS*s. Definition of these *EDP*s are based on deterministic predictive models for the chosen set of *LS*s (Goodnight et al. (2016) for LS_1 and LS_2, and Duck et al. (2018) for LS_3).

Table 1: LSs and associated strain-based EDPs

LS	***Associated strain*** (ε) ***-based EDP***
1	$\max\limits_{col}\left(\max\limits_{bar}\left(\max\limits_{t}\left\|\varepsilon_{comp}^{bar}(t)\right\|\right)\right)$
2	$\max\limits_{col}\left(\max\limits_{bar}\left(\max\limits_{t}\varepsilon_{tensile}^{bar}(t)\right)\right)$
3	$\max\limits_{col}\left(\max\limits_{bar}\left(\max\limits_{t}\varepsilon_{tensile}^{bar}(t)-\min\limits_{t'>t}\varepsilon_{comp}^{bar}(t')\right)\right)$

Ensemble nonlinear time history analyses of the testbed OSBs are carried out at discrete seismic hazard levels. Probabilistic characterizations of the considered *EDP*s, given any value of *IM*, are obtained through continuous regressions (against *IM*) of the parameters of probability distribution functions fitted to the *EDP*s at discrete *IM* levels.

2.4. Strain-based fragility functions

A fragility function expresses the probability of exceeding a system- or component-based *LS* given a specific value of a predictive demand parameter associated with this *LS*. Strain-based fragility functions developed using reliable experimental data or high-fidelity numerical data are constructed for the *LS*s considered in this study through proper identification of relevant test and research programs previously conducted. Fragility functions, typically constructed using experimental or numerical data pertaining to specimens or models with different geometric, material and mechanical characteristics, need to be normalized such that they can be used for structural components of any specified characteristics. Appropriate normalizing deterministic capacity prediction equations (Goodnight et al. (2016) for LS_1 and LS_2, and Duck et al. (2018) for LS_3) are identified and used for this purpose. Normalized fragility functions for *LS*s 1 through 3 are shown in Figure 3. These fragility functions corresponding to *LS*s 1-3 are denormalized by the respective values of the capacity predictors for the specific designs of the considered testbed OSBs and used in Eq. (1) to compute the MARs of exceedances for the selected set of *LS*s.

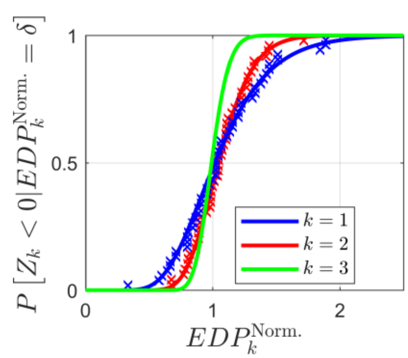

Figure 3: Normalized fragility curves (Experimental data shown as crosses)

3. PARAMETRIC PROBABILISTIC SEISMIC PERFORMANCE ASSESSMENT

3.1. Design Variables

Design variables/parameters selected for the parametric study, referred to as primary design variables, are structural parameters to which the exceedances of the selected set of *LS*s are believed to be most sensitive. These variables deemed critical to the seismic performance of an OSB are the column diameter, D_{col} , and the column longitudinal reinforcement ratio, ρ_{long} . A two-dimensional constrained (with constraints reflecting construction practice) primary design parameter space consisting of a regular grid of possible design points is defined for each testbed OSB considered.

All other bridge design parameters to be determined by meeting the requirements of capacity design, minimum ductility capacity, reinforcement ratio restrictions, etc., and/or restricted by the geometry of the bridge, available real estate, traffic requirements, etc. are referred to as secondary design variables. In the parametric study of each of the four testbed OSBs, the values of most secondary design variables are taken as per the original design of the bridge, except for the column transverse reinforcement ratio which is expressed as a practical fraction (1/2) of ρ_{long} .

3.2. Overall Workflow

A fully automated workflow (see Figure 4) incorporating an efficient utilization of available computing resources is developed for a smooth and seamless execution of the parametric full-blown probabilistic seismic performance assessment of the considered testbed OSBs. Parameterized Tcl input files of the OpenSees computational models of these bridges facilitate the automated generation of models corresponding to multiple re-designed versions of the actual bridges. The seismic performance of such re-designs generated by varying the primary design parameters, subject to practical constraints, are evaluated using the improved PEER PBEE framework described. This involves the extensive parallelization of computationally independent jobs, which is made possible through Stampede2, the flagship supercomputer at the University of Texas at Austin's Texas Advanced Computing Center (TACC). It is noteworthy to mention here that for the sizable number of nonlinear time-history analyses carried out for the performance assessment of each of the re-designs of the considered testbed OSBs, all non-collapse related numerical convergence issues encountered are resolved in an automated fashion via adaptive switching between iterative solution algorithms (e.g., Newton, modified-Newton, BFGS, Newton-Krylov), convergence criteria, etc.

For each testbed OSB, and for each of the considered *LS*s, a piecewise linear surface is fitted in the primary design parameter space to the MRPs of *LS* exceedance computed for all the re-designs of the bridge. The MRP surfaces for the considered set of *LS*s are intersected by horizontal planes corresponding to the respective specified target MRPs of *LS* exceedances which are based on discussions with and feedback from expert Caltrans engineers. A *LS*-specific feasible design domain, i.e., collection of design points in the two-dimensional design parameter space of an OSB with MRPs of exceeding the considered *LS* higher than or equal to a specified target, is identified. Such domains corresponding to each *LS* are superimposed in the primary design space of an OSB to delineate the overall feasible design domain and identify the *LS*s controlling its boundary. Overall feasible design domains obtained for the considered set of testbed OSBs are shown in Figure 5.

3.3. Discussion of results

Increasing values of the two primary design variables result in stronger, and thereby translating to safer column designs characterized by lower MARs or higher MRPs of *LS* exceedances. The MRPs of exceeding these *LS*s are found to be indeed sensitive to the chosen primary design variables thereby justifying their choice.

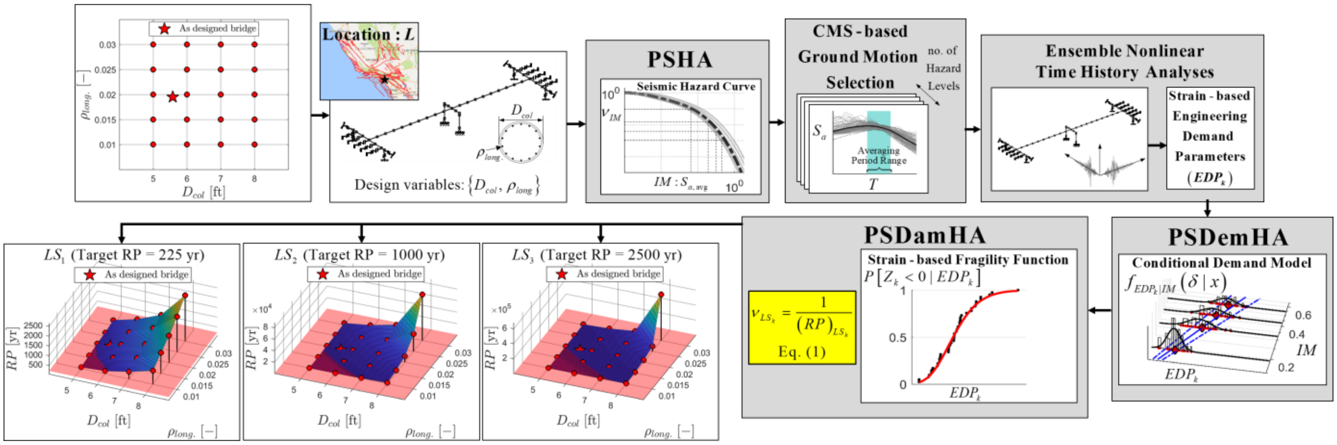

Figure 4: Overall workflow for parametric probabilistic seismic performance assessment applied to Bridge B

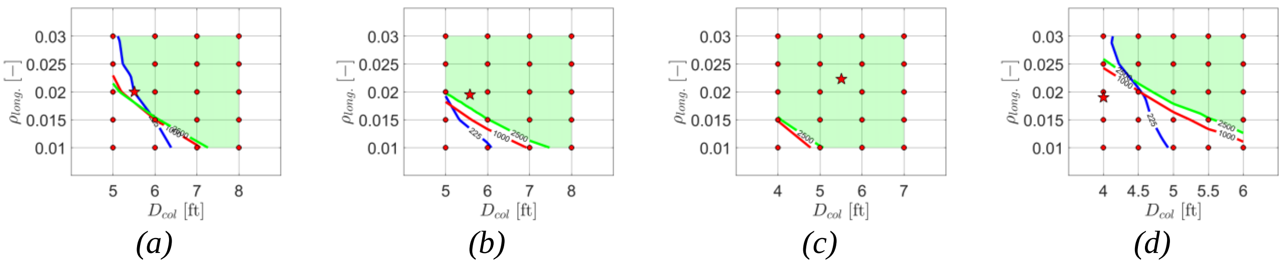

Figure 5: Overall feasible design domains (green shaded region) for (a) Bridge A, (b) Bridge B, (c) Bridge C, and (d) Bridge MAOC. Target MRP contour line for LSs 1, 2, and 3 shown in blue, red, and green respectively. As-designed testbed OSB shown as a red star.

Overall feasible design domain of an OSB is found to be controlled by different *LS*s over different regions of the primary design parameter space. The serviceability *LS* of concrete cover crushing tends to control the feasible design domain in the upper left region of the primary design parameter space (i.e., design points with small D_{col} and large ρ_{long} values). On the other hand, the ultimate *LS*s of longitudinal rebar buckling and fracture govern the feasible design domain in the lower right region of the primary design parameter space (i.e., design points with large D_{col} and small ρ_{long} values).

The seismic performance of the as-designed version of a testbed OSB is gauged by the location of the corresponding design point in the design parameter space relative to the overall feasible design domain of the bridge (i.e., does the as-designed bridge belong to the feasible design domain and how close is it from its boundary?). The seismic performance of the as-designed testbed OSBs is found to show considerable variability. These bridges originally designed following a more traditional (prescriptive) seismic design philosophy, rather than an explicitly performance-based one, are found to exhibit irregular levels of conservativeness. While some of the as-designed testbed OSBs are found to be conservative, sometimes too much, with respect to the selected *LS*s and corresponding target MRPs, others are found to lie near the borderline of safety, or clearly in the unsafe domain.

4. CONCLUSIONS

Erratic levels of conservativeness exhibited by the as-designed testbed OSBs illustrates the need for a PBSD framework for OSBs such that explicitly stated risk-targeted performance objectives are consistently satisfied by the population of OSBs in California. The parametric probabilistic seismic performance assessment framework for OSBs assembled in this study can be considered as a step forward in the direction of a rigorously implemented PBSD framework for bridges. Knowledge of the feasible design domain of an OSB in its design space emerges as an extremely valuable resource as it can be utilized to make risk-informed design decisions leading to safe and economic design of OSBs. The implementation of the framework in a practical design environment is, however, computationally overpriced. In this regard, efforts can be channeled to distill out of this comprehensive study a simplified, computationally economical, yet rigorous, and sufficiently accurate PBSD methodology for OSBs.

5. ACKNOWLEDGEMENTS

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