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Preliminary Design of Moment-Resisting Frame Buildings for Tolerable Financial Loss

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Abstract: This paper aims to transfer the current performance-based seismic assessment (PBSA) methodology to engineering practice by providing a preliminary design procedure denoted herein as the preliminary performance-based seismic design (PPBSD). PPBSD aims at guiding a conceptual design given the tolerable expected loss and target hazard level. The suggested preliminary design procedure implicitly incorporates the main sources of variability in the seismic performance assessment of building structures including the variability associated with seismic excitation. PPBSD can help stakeholders make informed decisions on how to handle potential seismic risk at the preliminary design level with minimal computational effort. The process for development of design tools required for implementation of PPBSD are described and such development is illustrated for a 4-story reinforced concrete special moment-resisting frame (RC-SMRF) building located in Los Angeles for a 475-year ground motion return period. A design example is offered to demonstrate how PPBSD can be implemented in practice. **DOI: 10.1061/(ASCE)ST.1943-541X.0002331.** *© 2019 American Society of Civil Engineers.*

Author keywords: Preliminary performance-based seismic design; Performance-based conceptual design; Seismic loss assessment; Performance-based seismic assessment.

Introduction

The main goal of current building design codes is to ensure life safety in rare earthquake events; nevertheless, these codes do aim at mitigating the likely damage and economic loss in moderate to severe earthquakes. Recent earthquakes have shown that even a moderate seismic event may expose large economic losses due to damage in buildings and other structures. Such losses are mostly unexpected for building owners and other stakeholders. Motivated by addressing performance upfront in seismic design, this research bridges the gap between current performance-based seismic assessment (PBSA) methodology for buildings (e.g., seismic financial loss assessment) and current prescriptive building design guidelines. The aim is to provide the engineering profession with tools and methods—compared with current design guidelines—that can assist a preliminary design based on performance targets; this approach is denoted as preliminary performance-based seismic design (PPBSD).

The aim of this study is to suggest a procedure for conceptual and preliminary design of buildings (denoted as PPBSD) and demonstrate how the required tools for implementation of PPBSD can be generated. PPBSD can answer questions like: What the most efficient preliminary design for a 4-story building if the tolerable expected loss is 10% of the total building's replacement cost at a 475-year return period seismic excitation? Even a code-conforming structural design may fall short of a tolerable risk of loss and unable to answer questions similar to what was proposed here.

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This paper provides a short background about performance-based engineering in research and practice. This background will help show the gaps between the research and practice of performance-based engineering. The paper will follow with the theoretical presentation of PPBSD procedure. Authors will use an example to demonstrate the application of the proposed procedure for a 4-story RC-SMRF building at the design level earthquake hazard.

Performance-Based Engineering Research and Practice

Mainstream building design and assessment guidelines [SEAOC 1996; FEMA-273 1997 (FEMA 1997); FEMA-356 (FEMA 2002); ASCE/SEI 41-13 (ASCE 2013); ASCE 7-16 (ASCE 2017)] define performance levels at the global level (i.e., operational, immediate occupancy, life safety, and collapse prevention) and correlate these performance targets with the response of structural components. The ambiguity in the definition of these performance levels and their lack of correlation with tangible performance measures (e.g., financial loss) make communication between engineers and stakeholders imperfect (Ramirez and Miranda 2009). Moreover, structural performance should be quantified in more useful terms with which stakeholders can perform risk analysis and make informed decisions. Informed by this caveat, the Pacific Earthquake Engineering Research (PEER) Center developed a PBSA methodology based on a new set of three performance measures: financial loss, down time, and casualties. Using these measures engineers can intellectually communicate with various stakeholders. Given the wealth of information available on PBSA, there is a need to develop a PBSD procedure that can assist engineers to proportion building components and explore variety of design alternatives in terms of structural materials and systems to meet target performance objectives. The goal of this research is to provide a set of preliminary PBSD tools that are in line with current building design guidelines and that can help engineers conduct a preliminary conceptual design based on target seismic financial loss. Special care is dedicated to minimizing deviation from current design methods and maximizing application by engineering profession.

Estimation of financial loss from damages in buildings due to seismic excitation and optimal design of structures for target financial loss have been two related research foci in PBSD since the 1970s (Liu and Neghabat 1972; Haug and Arora 1979; Arora 1989, 1997a, b). In this context, optimized design is accomplished through an iterative assessment process that starts with a conceptual design. This preliminary design is tuned through successive iterations of performance assessment and building redesign until the performance targets are met (Haug and Arora 1979; Arora 1989, 1997a, b). The essence of this optimization scheme is the loss assessment module where researchers have maintained their focus both in global and component scales [Ang and Lee 2001; Beck et al. 2003; Porter et al. 2004; Liu et al. 2004; Hamburger et al. 2004; Porter et al. 2006; Pei 2007; FEMA P-58-1 (FEMA 2012)]. From the design point of view, PBSD is one of the important advancements in structural engineering (Priestley 2000); however, its application to engineered buildings has remained largely unexplored until Filiatrault and Folz (2002) discussed the PBSD of wood-frame buildings through a direct-displacement methodology. A performance-based conceptual design (PBCD) procedure was introduced by Krawinkler et al. (2006) as a decision support system with which selection of one or several effective design alternatives based on performance targets was possible in line with the PEER loss estimation methodology. Van de Lindt et al. (2008) introduced the concept of loss-based seismic design for wood-frame structures based on a financial loss simulation framework which involved nonlinear time domain analysis, and Monte Carlo simulation of losses considering seismic event uncertainties. A new generation of PBSD criteria for buildings was introduced by Hamburger et al. (2004) as one of the main efforts by Applied Technology Council [ATC, ATC-58 project, aka FEMA P-58-1 (FEMA 2012)] under sponsorship of Federal Emergency Management Agency (FEMA) to quantify structural performance measures. Detailed description for buildings and damage/loss models can be found in FEMA publications [FEMA 283 (FEMA 1996); FEMA 349 (FEMA 2000); FEMA 445 (FEMA 2006); FEMA P-58-1 (FEMA 2012); ATC 72-1 (ATC 2010)].

It is clear from the preceding literature review that application of PBSD in engineering practice has been a topic of research for quite some time; however, many building owners are still under a false impression that a code-conforming structural design would lead to a building design with a tolerable risk of loss. This may have serious consequences with significant seismic damages and costs once a moderate to major earthquake hits an urban area (Bachmann 2002; Esmaili et al. 2017). Consequently, studying the efficiency and sufficiency of building code provisions in mitigating building's potential seismic loss turned to be an essential issue in the structural engineering profession and for researchers alike. Several studies have examined the effectiveness of building code provisions in providing the level of performance that the code design philosophy promises (Leil et al. 2006; Goulet et al. 2007; Haselton 2006; Haselton et al. 2007; Zareian and Krawinkler 2009; Rojas et al. 2011; Miranda and Ramirez 2012; Zareian and Krawinkler 2012). These studies focused on various PBSA components such as quantifying the collapse risk of structures (Leil et al. 2006) and ground motion selection and scaling along with structural collapse prediction (Goulet et al. 2007; Haselton 2006; Haselton et al. 2007; Zareian and Krawinkler 2009; Rojas et al. 2011; Miranda and Ramirez 2012; Zareian and Krawinkler 2012) for reinforced concrete special moment-resisting frame (RC-SMRFs) designed according to ICC 2003 (ICC 2003), ASCE 7 (ASCE 2002), and the ACI 318 (ACI 2002). In the same line, FEMA P695 (FEMA 2009) was developed as a guideline to standardize a process to estimate seismic performance factors (R, μ, Ω, which are seismic reduction factor, ductility factor, and overstrength factor, respectively) for building systems by regulating the structure's probability of collapse (Deierlein et al. 2008; Haselton et al. 2008). In this context, Haselton et al. (2008, 2011) and Zareian et al. (2010) elaborated on the application of FEMA P695 methodology for reinforced concrete special moment frames, and steel special moment frames, respectively, and investigated whether current special moment frame design procedures [ASCE 7 (ASCE 2010); AISC 341 (AISC 2005); ACI 318 (ACI 2011)] provide an acceptable margin of safety against collapse. Recently, Sinković et al. (2016) introduced a risk-based design algorithm in which pushover analysis is employed iteratively to arrive at a desired design of reinforced concrete frame buildings that comply with a tolerable collapse performance criterion.

Preliminary Performance-Based Seismic Design

Structural design of a building is an iterative process. An efficient design process consists of an iterative design and assessment sequence that starts with a conceptual design for which performance assessment is carried out, and the design is enhanced in successive iterations until the performance targets are met [FEMA 445 (FEMA 2006)]. The art of engineering, which should be practiced in this phase, is to use global information on important performance targets to arrive with a structural system that fulfills specified performance objectives (e.g., having maximum tolerable loss with a reasonable probability of exceedance) in the most effective manner. This implies exploration of design alternatives in terms of structural materials, systems, and innovative technologies.

The proposed conceptual design process for making design decisions based on acceptable performance given the ground motion hazard level is illustrated in Fig. 1. Within this setting, desired performance is specified through two parameters: (1) the tolerable expected value of loss, and (2) the probability of exceeding the tolerable expected loss due to variability in ground motion effects on the structure. With this approach, the authors have separated the variability in estimation of seismic demand on the structure (i.e., item ii above) from the variability in estimation of loss conditioned on the value of such demand (i.e., expressed as a single term, that is, the tolerable expected value of loss, item i above). Once the performance objective is defined, the information

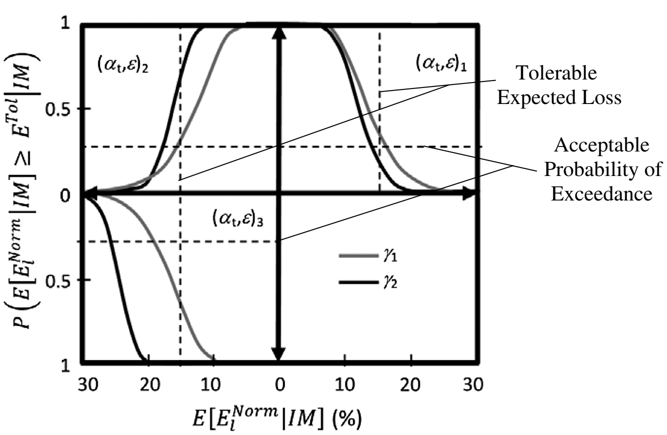

Fig. 1. Conceptual presentation of the preliminary performance-based seismic design (PPBSD) process.

provided in the form of Fig. 1 can be utilized to arrive at a conceptual design.

There are four regions in Fig. 1; the lower-right corner is used as a legend while the other three quadrants show the expected loss versus probability of exceeding the expected loss for three levels of a structural system's period coefficient α_T, where $T^* = N \cdot \alpha_T$ and N is the number of stories of the structure and T^* is the assumed fundamental period of the structure. Previous efforts in the development of performance-based design guidelines has shown that the building period coefficient (α_T) is the main parameter that not only strongly affects building performance, but also is an ideal quantity to a guide preliminary design process (Krawinkler et al. 2006; Zareian and Krawinkler 2007a, b). The value of α_T is usually between 0.15 and 0.25; values close to the lower bound are used for stiff buildings (e.g., with shear walls) and midrise buildings. However, α_T values close to the upper bound are used for low-rise and ductile (e.g., with moment-resisting frames) structures. The dependence of seismic hazard to T^* is embedded in the information illustrated in each quadrant via epsilon (ε), that is, a measure representing the difference between the logarithmic spectral acceleration (at a given period) of a ground motion record from the mean estimate of the same quantity obtained from using the information about the originating seismic event in a ground motion prediction equation (GMPE). ε is presented in a normalized form by dividing the said arithmetic difference by the standard deviation of the used GMPE (Baker and Cornell 2005). The direction of each axis shows how each quadrant should be treated to extract the required information for conceptual design. Losses may occur in the drift-sensitive and acceleration-sensitive subsystems of the building; this information is given once the occupation of the building is known as a priori (e.g., via architectural, mechanical, and electrical drawings). This paper defines γ as the ratio of total value of damageable drift-sensitive components to the total replacement cost of the building. Design tools like that illustrated in Fig. 1 should be developed for combining the target seismic hazard level at the location of the building and basic structural system properties that can readily be used by engineers for rapid PPBSD (Esmaili 2014).

The process through which PPBSD is exercised starts by marking the tolerable expected loss and the acceptable probability of exceeding of that loss in design tools similar to Fig. 1; dash lines in Fig. 1 illustrate this step. For this example, it is assumed that 15% of the total value of the building is a tolerable expected loss if the probability of exceedance of this loss is equal or less than 25%. Information for two levels of γ is illustrated: (1) γ_1 represents a building with relatively low value of drift-sensitive components, and (2) γ_2 represents an opposite scenario where the value of drift-sensitive components is high, with $\gamma_2 > \gamma_1$. Consequently, the value of acceleration-sensitive components in the building with γ_1 is higher than the building with γ_2. By examining the three quadrants in Fig. 1, it can be concluded that for a building with dominant acceleration-sensitive components, the design illustrated in the upper-right quadrant would work best. The combination of parameters in the upper-left quadrant will be an ideal preliminary design for a building in which drift-sensitive components dominate the loss. For a building with either dominant drift-sensitive or acceleration-sensitive components, the design illustrated in the lower-left quadrant would work, but the former is the better option since its probability of exceedance is relatively lower.

Successful implementation of PPBSD requires the availability of design tools similar to that shown in Fig. 1. The next section will demonstrate how such design tools can be developed by showing the development process for 4-story RC-SMF buildings. A complete PPBSD design example is presented to show the PPBSD process in detail.

Development of Tools Required for PPBSD

Implementation of PPBSD requires the design tools discussed in Fig. 1 to become available to engineers. To be able to develop such tools requires a collection of modeling and analysis efforts that are described herein. Conceptually, PPBSD design tools are extracted from a large database of information developed from loss estimation of all possible combinations of building properties and seismic hazard information. For that matter, this paper focuses on the most important building properties by utilizing generic frames (Esmaili 2014). The steps required for development of such database are described in the following. Without loss of generality, the process for developing such design aids is demonstrated with a 4-story RC-SMRF and these aids are illustrated in the Appendix.

Structural Modeling

Concepts for proper modeling of structural systems and their associated component modeling parameters have been subject of much research [ATC 72-1 (ATC 2010)]. Recent studies (e.g., Lignos et al. 2008; Gokkaya et al. 2016, 2017) have investigated the correlation among structural component parameters and their impact on performance of building structures. In this study, two-dimensional numerical models of a set of 4-story, 3-bay RC-SMRFs are created using Opensees. Each of the frames are modeled assuming that they are part of the lateral load-resisting system. It is assumed that flexural nonlinear behavior is concentrated at the ends of beams and columns and is modeled using the modified Ibarra-Krawinkler-Medina deterioration model (Ibarra and Krawinkle 2005; Lignos and Krawinkler 2009) [Fig. 2(b)]. It has been further assumed that none of the structural components are shear critical, i.e., shear failure is not modeled. Given the low-rise nature of the structure, Rayleigh damping corresponding to 5% of critical damping in the first and third modes is applied (Xiang et al. 2016). Destabilizing P-Delta effects due to gravity loads are accounted for by applying gravity loads on a leaning column in the analysis model.

For all frames in the model, story heights are 4.6 m (15 ft) in the first story (h_{base}) and 4 m (13 ft) otherwise. The bay width is variable; 6.9 m (22.5 ft), 9.2 m (30 ft), and 11.4 m (37.5 ft) are considered as common values, which are equal to $1.5h_{\text{base}}$, $2.0h_{\text{base}}$, and $2.5h_{\text{base}}$. The frames are designed according to performance-based plastic design procedure (Liao 2010) for the first mode period proportionate to the number of stories by α_T, which varies between 0.15, 0.20, and 0.25. Floor masses are assumed to be the same at all story levels, and the floor stiffness varies along the height such that a straight line deflected shape is obtained when the ASCE 7 (ASCE 2010) lateral load pattern is applied to each frame. It is assumed that the stiffness and strength of all structural elements are proportional, and the variation of beam and column strength along the height of each frame is identical to the variation of stiffness, which is tuned to the design lateral load pattern (Zareian and Krawinkler 2007a, 2009). The strong column-weak beam design philosophy is considered in the design process for all frames. The design base shear for frames is determined for two levels of performance: (1) a 1.5% maximum interstory drift ratio (IDR), (θ_u) for a ground motion hazard with 10% probability of exceedance in 50 years (design-based earthquake, DBE); (2) a 2.5% maximum IDR, (θ_u) for 2% probability of exceedance in 50 years (maximum considered earthquake, MCE).

The analytical models used for plastic hinge locations in structural components of the frames include both monotonic and

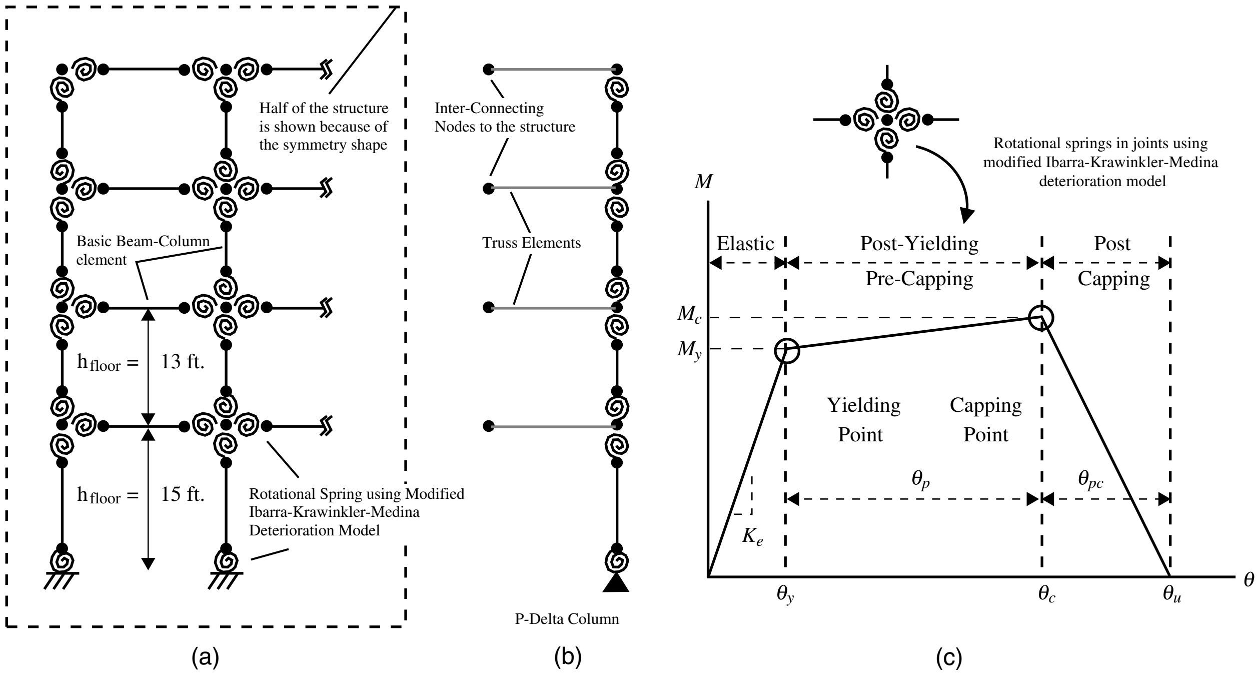

Fig. 2. (a) Schematic presentation of a concentrated plasticity model in Opensees for a 4-story RC-SMRF structural analysis model; (b) leaning P-Delta columns; and (c) backbone curve definition based on typical modified Ibarra-Krawinkler-Medina deterioration model where Ke is the initial stiffness, My is the yield moment, Mc/My is the capping moment ratio, θp is the plastic hinge rotation capacity, and $\theta pc/\theta p$ is the post capping rotation capacity ratio, θc is the capping plastic hinge rotation capacity and θu is the ultimate hinge rotation capacity.

cyclic strength and stiffness deterioration. The backbone curve for stiffness and strength of a typical frame is illustrated in Fig. 2(b). Cyclic deterioration of strength and stiffness is based on a reference hysteretic energy dissipation capacity, $E_t = \lambda M_y \theta_p$, where λ is a parameter that is estimated using experimental results (Lignos and Krawinkler 2009; Haselton et al. 2006). A trio of plastic hinge rotation capacity, θ_p, postcapping rotation capacity, θ_{pc}, and cyclic deterioration parameter, λ, of beams and columns (θ_p, θ_{pc}, λ) are set to corresponding median values and 10th and 90th percentile of the lognormal distributions fitted on the parameter values obtained based on a research done by Berry et al. (2004) and calibrated by Haselton et al. (2006): A(9%, 10%, 145), B(3%, 17%, 60), and C(1%, 26%, 25), as illustrated in Fig. 3.

Component modeling variables used herein are not necessarily the only possible set of variables that can completely represent the behavior of a structural component. Other sets of variables (e.g., cross-sectional properties) can be considered for this purpose and consequently be used in the preliminary design phase. Any complete set of such parameters is correlated and one can estimate the variables in one set using the information available from another (Lignos and Krawinkler 2009). The authors chose to use (θ_p, θ_{pc}, λ) as the set of parameters representing the plastic hinges of models used in this study for three reasons: (1) these parameters are directly used in the analytical models; (2) abundant test data are available for their characterization; and (3) they have low variability compared with other sets of parameters for representing structural component behavior.

Ground Motion Modeling

To take the variability of ground motion into account, each generic frame building is analyzed for 31 selected locations in Los Angeles as shown in Fig. 4(a). The corresponding soil type (C/CD) for each location is considered in ground motion selection and modification (GMSM). Envelopes of possible (R, M_w, ε_0) sets for Los Angeles area are found using seismic hazard deaggregation through utilizing USGS (2008) NSHMP PSHA interactive deaggregation for the DBE and MCE hazard levels [Fig. 4(b)]. Several sets of (R, M_w, ε_0) are defined for each fundamental period within the envelopes. Twenty pairs of ground motions are selected and modified for every reference points. Addressing GMSM methods is out of the scope of this research; readers can refer to wealth of literature available on this subject, some of which is summarized in NIST (2011). In this study, GMSM is performed for each reference point using the procedure described in Jayaram et al. (2011) for the DBE and MCE hazard levels. Fig. 4(c) shows the response spectra of selected ground motions for the reference point with the following characteristics: $R = 15.0$ km, $M_w = 6.8$ and $\varepsilon_0 = 0.72$ plus $V_s^{30} = 300$ m/s for DBE hazard level.

Building-Specific Loss Estimation

The PEER PBEE framing equation given in Eq. (1) is essentially the underlying framework for the building-specific loss estimation used in this study. By utilizing this equation, one can estimate the mean annual frequency of exceeding a value of loss, denoted as $\lambda_{\theta_l^{Total}}$, where IM represents the ground motion intensity measure, $\mathbf{EDP}|IM$ is a vector of engineering demand parameters that characterize the response of the structure for a given level of ground motion IM, $\mathbf{DS}|\mathbf{EDP}$ is a vector of damage states given seismic demand $\mathbf{EDP}$, and $\theta_l^{Total}|\mathbf{DS}$ is loss given damage states $\mathbf{DS}$. $G(x|y)$ is the complementary cumulative distribution function (CCDF) of X given Y; $f(x|y)$ is the probability density of X given Y; and λ_{IM} is the mean annual frequency of IM. It should be noted that Eq. (1) also makes the conditional independence assumption (e.g., that the damage state, $\mathbf{DS}$, is only a function of $\mathbf{EDP}$, and not of the IM causing $\mathbf{EDP}$), and therefore can be decoupled and solved in separate stages. Markovian dependence is assumed for all conditional distributions in Eq. (1):

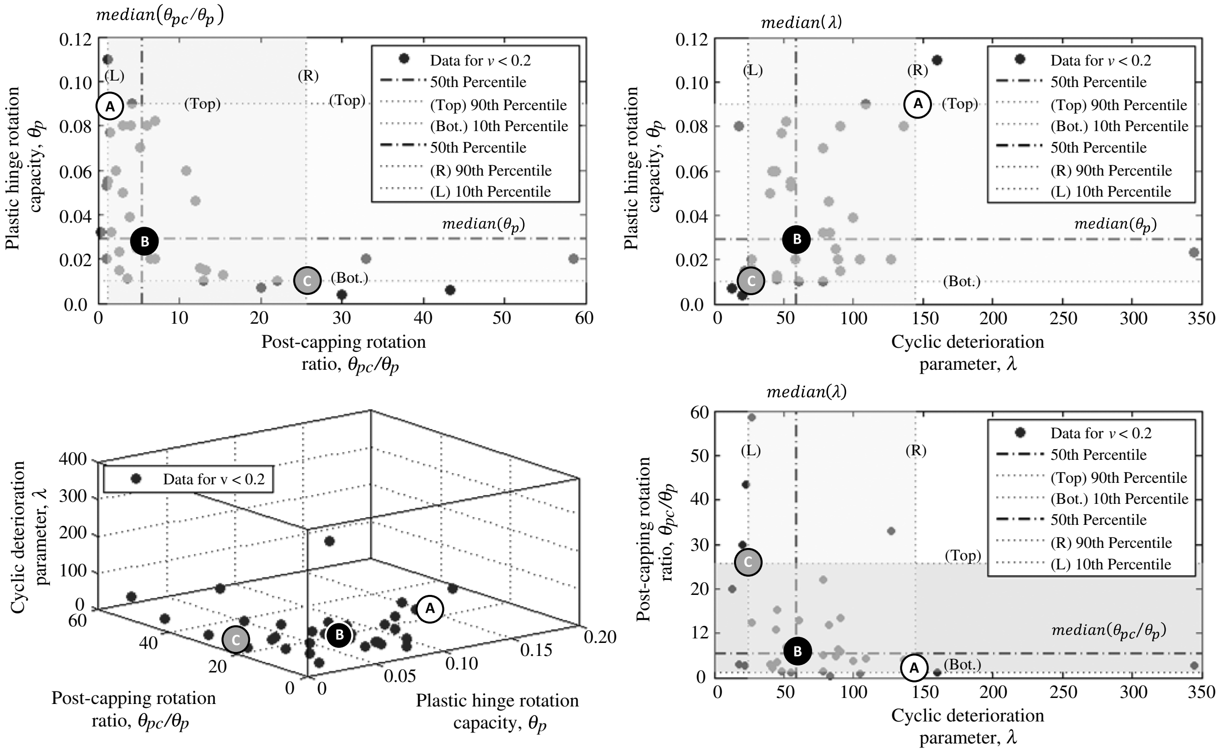

Fig. 3. Range of variation for structural component parameters θp, $\theta pc/\theta p$, and λ utilized in modified Ibarra-Krawinkler-Medina deterioration model for low axial load intensity.

$$\lambda_{\theta_l^{Total}} = \int_{\mathbf{DS}} \int_{\mathbf{EDP}} \int_{IM} G(\theta_l^{Total}|\mathbf{DS}) \cdot f(\mathbf{DS}|\mathbf{EDP}) \cdot f(\mathbf{EDP}|IM) \cdot d\lambda_{IM} \cdot d\mathbf{EDP} \cdot d\mathbf{DS} \tag{1}$$

Eq. (1) is an ideal tool for loss assessment; however, the dimensions of **EDP** and **DS** vectors within this equation makes the process of loss assessment a cumbersome task. Given this caveat, the authors aim to reformat Eq. (1) to a form that can be utilized in the preliminary design of buildings and development of design tools for PPBSD. It is hypothesized that the conceptual design can be based on an acceptable/tolerable expected value of loss at a target ground motion hazard level, and the probability of exceeding the expected loss–variability here is due to randomness in ground motion effects. The proposed framework for preliminary design utilizes (1) loss functions that relate expected loss to engineering demand parameters and (2) distribution of engineering demand parameters given the target ground motion intensity measure.

Distribution of engineering demand parameters given the target ground motion intensity measure is obtained through nonlinear response history analyses (NRHA) and by utilizing the set of ground motions that are selected for the target level of hazard. NRHA have been performed using the Opensees numerical models developed herein under the set of 20 selected ground motions. For each floor, $f \in \{1,2,3,4\}$ of a 4-story building, $\mathbf{EDP}^f|IM$ includes corresponding values of maximum interstory drift ratio, $IDR_{\max}^f$, and absolute (total) peak floor accelerations, PFA^f, for both horizontal ground acceleration components [Eq. (2)]. The collection of $\mathbf{EDP}^f|IM$ vectors for all floors is equal to $\mathbf{EDP}|IM$ as shown in Eq. (3). From this relatively small set (i.e., 20) of $\mathbf{EDP}|IM$ vectors, denoted as observations, one can generate a joint lognormal distribution—similar to the approach utilized by FEMA P58—and draw a relatively large number of realizations

$$\mathbf{EDP}^f|IM = \left[IDR^f_{\max,Dir-1}, PFA^f_{Dir-1}, IDR^f_{\max,Dir-2}, PFA^f_{Dir-2}\right] \tag{2}$$

$$\mathbf{EDP}|IM = [EDP^1|\ldots|EDP^f|\ldots|EDP^4] \tag{3}$$

Loss functions that relate expected loss to engineering demand parameters have been developed by Ramirez and Miranda (2009) in the form of $E[\theta_l^f|\mathbf{EDP}^f]$, where θ_l^f is the loss at floor level f given $\mathbf{EDP}^f$, and $E[\cdot]$ represents an expected value function. For each $\mathbf{EDP}|IM$ vector (from the substantial number of realizations described earlier), the expected value of total loss, $E[\theta_l^{Total}|IM]$, is obtained by summing the expected value of loss at each floor. The outcome will provide the required data to obtain a probability distribution function for $E[\theta_l^{Total}|IM]$ as shown in Eq. (4). Computation of the expected value of total loss conditioned on ground motion intensity, $E[\theta_l^{Total}|IM]$, as described here, may include cases in which the structural system reaches its collapse state; hence, the value of loss is equal to the total replacement cost of the building. Such cases are removed from the noncollapse observation set and used to calculate the probability of collapse. However, realizations will be drawn from both collapse and noncollapse spaces similar to the approach used in FEMA P58. Ultimately, the total loss is normalized by the total replacement cost of the building to arrive at the distribution of normalized loss given IM, $F(E[E_l^{Norm}|IM])$

$$F(E[\theta_l^{Total}|IM]) = \Phi\left[\frac{Ln(E[\theta_l^{Total}|IM]) - \overline{Ln(E[\theta_l^{Total}|IM])}}{\sigma_{Ln(E[\theta_l^{Total}|IM])}}\right] \tag{4}$$

The authors have developed a loss estimation tool (Esmaili et al. 2016) using MATLAB, called SAFER, to perform the calculations needed for development of design aids shown in

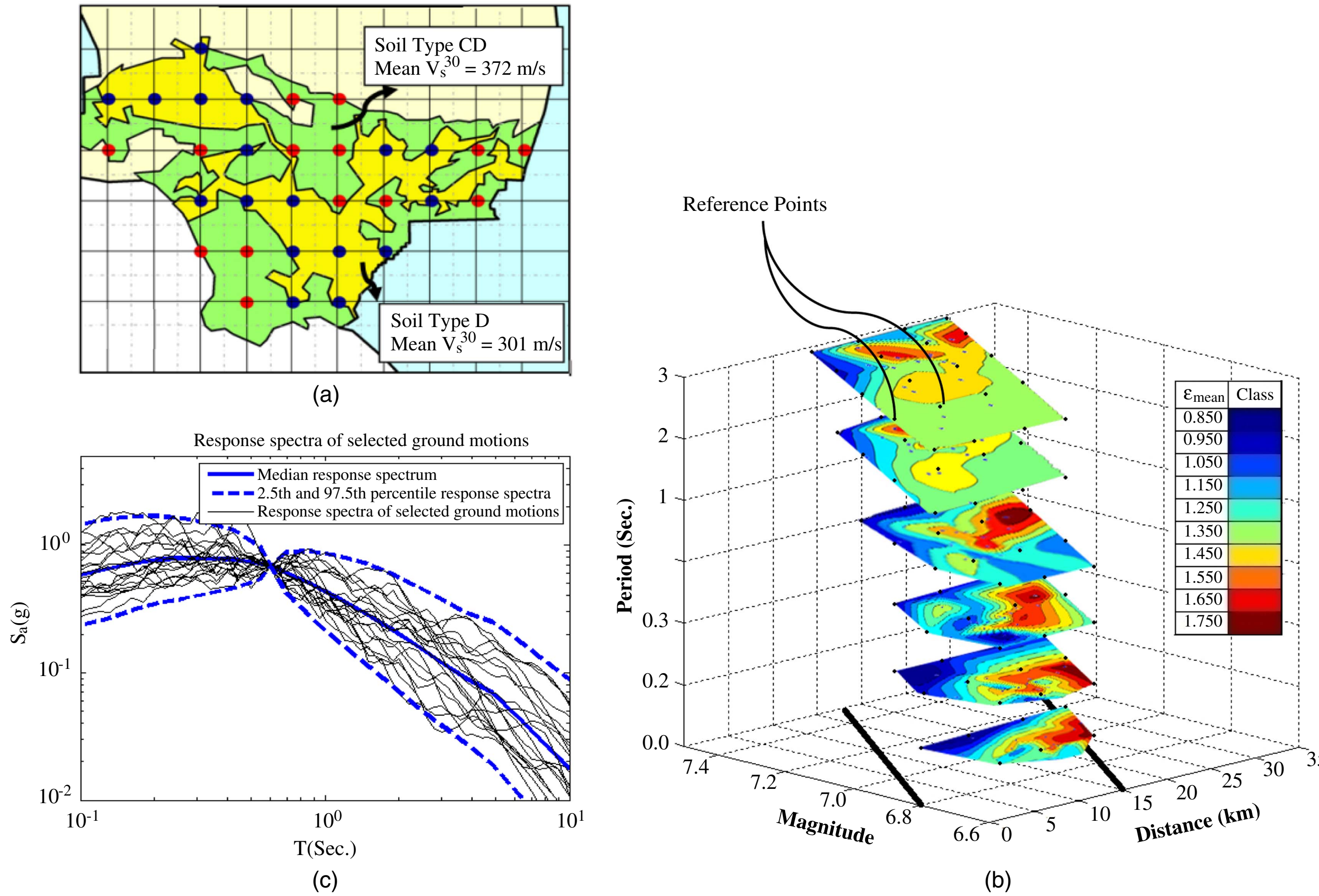

Fig. 4. (a) Representative site locations plus soil type and corresponding Vs^{30} values for those locations; (b) mean (R, M_w, ε_0) for each fundamental period (T^*) for DBE hazard level; and (c) response spectra, target, and sample exponential logarithmic means and logarithmic standard deviations of selected ground motions for a set of 20 ground motions for the triple ($R = 15.0$ km, $M_w = 6.8$, $\varepsilon_0 = 0.72$) representing DBE hazard and fundamental period of 0.6 s. [For Figs. 4(b and c), data from USGS 2008.]

the Appendix. SAFER is conceptually similar to the Performance Assessment Calculation Tool (PACT 2012) developed by ATC for performing the probabilistic computations and accumulation of losses; the two softwares estimate the expected value of total loss given IM using Monte Carlo simulation. SAFER, however, has a few key differences with PACT. For one, SAFER defines a single EDP vector for loss assessment that is comprised of EDPs in two orthogonal directions of the building. PACT, however, uses two EDP vectors for seismic loss assessment, each of which represents a single direction of seismic input. This difference provides SAFER with the capability to directly incorporate the correlation between seismic response (i.e., EDP) in two orthogonal directions into the loss estimation process. Another difference between the two software is SAFER's capability in utilizing generic *EDP-DV* functions (i.e., $E[E_l^{Norm}|IM]$) in loss estimation process in addition to PACT's component-based loss estimation approach. Readers can find a detailed description of SAFER in Esmaili (2014). CCDFs of $E[E_l^{Norm}|IM]$ are developed for RC-SMRFs as design tools for PPBSD and comparison between seismic performances of building alternatives. A set of PPBSD tools has been illustrated in Fig. 5 for $R = 7.2$ km, $M_w = 6.72$, $\varepsilon_0 = (0.487, 0.506, 0.525)$, $B_w = 22.5$ ft with structural component classes A, B, and C. PPBSD tools for other parameter combinations are provided in the Appendix. According to PPBSD design tools, α_T and γ have a prominent effect on $F(E[E_l^{Norm}|IM])$ compared to structural component classes (i.e., A, B, and C) and B_w. Consequently, PPBSD tools only the variability of α_T and γ are considered in and depicted accordingly in the Appendix.

PPBSD Design Example

The steps are presented through which the proposed PPBSD tools may be utilized in two common engineering application cases. Case 1 involves identifying the structural system properties that minimizes the probability of exceeding a tolerable expected normalized loss equal to 15% ($E^{\mathrm{Tol}} = 15\%$) at DBE hazard level (i.e., 475-year average return period) for a building with $\gamma = \gamma_{C1} = 30\%$. Case 2 involves identifying the structural system properties that brackets the probability of exceeding a tolerable range of expected normalized loss equal to 15% to a range between 10% and 25%, that is, $\left(P_{E=E^{\mathrm{Tol}}}^{Lower-limit} = 10\%, P_{E=E^{\mathrm{Tol}}}^{Upper-limit} = 25\%\right)$; at DBE hazard level for a building with $\gamma = \gamma_{C2} = 50\%$. The preliminary design process starts with performing probabilistic seismic hazard analysis (PSHA) deaggregation, which links computation of a target spectrum to the target hazard (i.e., DBE). PSHA deaggregation is performed for the location of the building and at the target hazard level to identify the causal sets of parameters R, M_w, and ε_0 for a given spectral acceleration $Sa(T^*)$, where $T^* = N\alpha_T$ (i.e., $T^* = 4 \times 0.15 = 0.60$ s, $T^* = 4 \times 0.20 = 0.80$ s, and

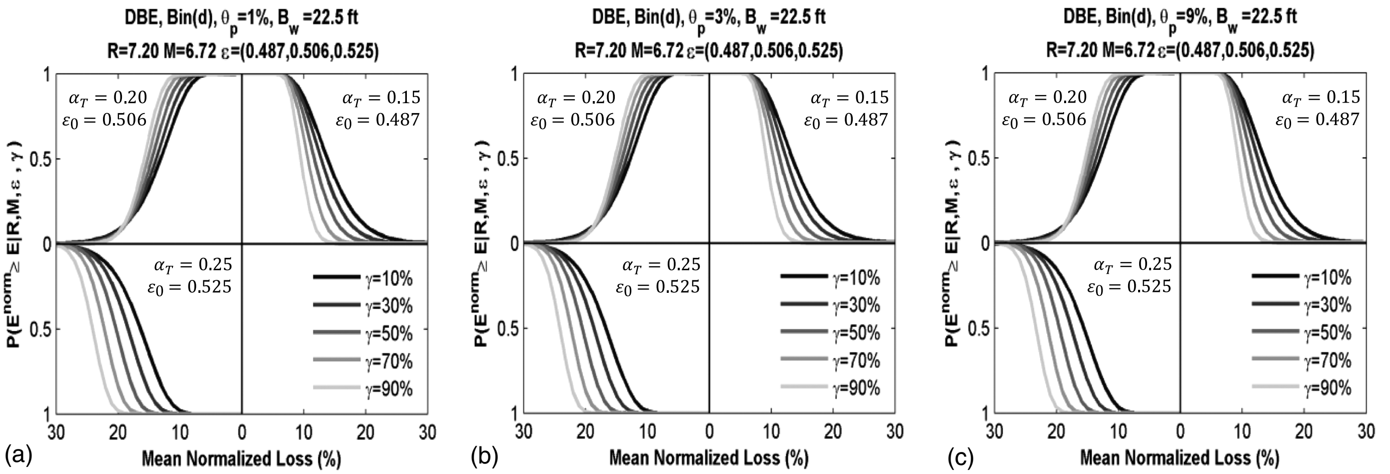

Fig. 5. PPBSD design tools for a set of 4-story commercial RC office buildings, for $R = 7.2$ km, $M_w = 6.72$, $\varepsilon_0 = 608$ (0.487, 0.506, 0.525), corresponding to $\alpha_T = (0.15, 0.2, 0.25)$ for DBE hazard level, $B_w = 6.9$ mm (22.5 ft) and structural component parameters θp, $\theta pc/\theta p$, and λ of (a) (1%, 25.6%, 24); (b) (3%, 16.3%, 59); and (c) (9%, 10.5%, 145), respectively. (Data from USGS 2008.)

$T^* = 4 \times 0.25 = 1.0$ s). Design tools are used whose parameters R, M_w and ε_0 similar to those obtained earlier, and draw the corresponding lines of $E[E_l^{Norm}|IM] = 15\%$. A procedure like that explained earlier is conducted to identify what combination of structural parameters that satisfy the design conditions.

Case 1

For this design, information for five levels of γ is illustrated in Fig. 6(a). The system selection process starts from examining the horizontal axes of the three quadrants in Fig. 6(a); $E^{\mathrm{Tol}} = 15\%$ is targeted. The probability of exceeding this loss for a building with $\gamma = \gamma_{C1} = 30\%$ can be obtained by reading the corresponding probabilities from the associated fragility curves in the three quadrants; these values are approximately 25%, 40%, and 90%, for 4-story buildings with $T^* = 0.6$, 0.8, and 1.0 s, respectively. Given that the minimum probability of exceedance is sought for this design, the stiffer building with $T^* = 0.6$ s is selected. The designer now can move forward with proportioning structural members of the 4-story building with a target period of 0.6 s.

Case 2

For Case 2, the three quadrants in Fig. 6(b) are examined. A process similar to that explained in Case 1 is followed with a minor change. In this case, the aim is to identify which quadrant (i.e., period) will result in a design in which $E^{\mathrm{Tol}} = 15\%$, $\gamma = \gamma_{C2} = 50\%$, and the tolerable likelihood of expected normalized loss is between 10% and 25%. This acceptable probability range in marked with a

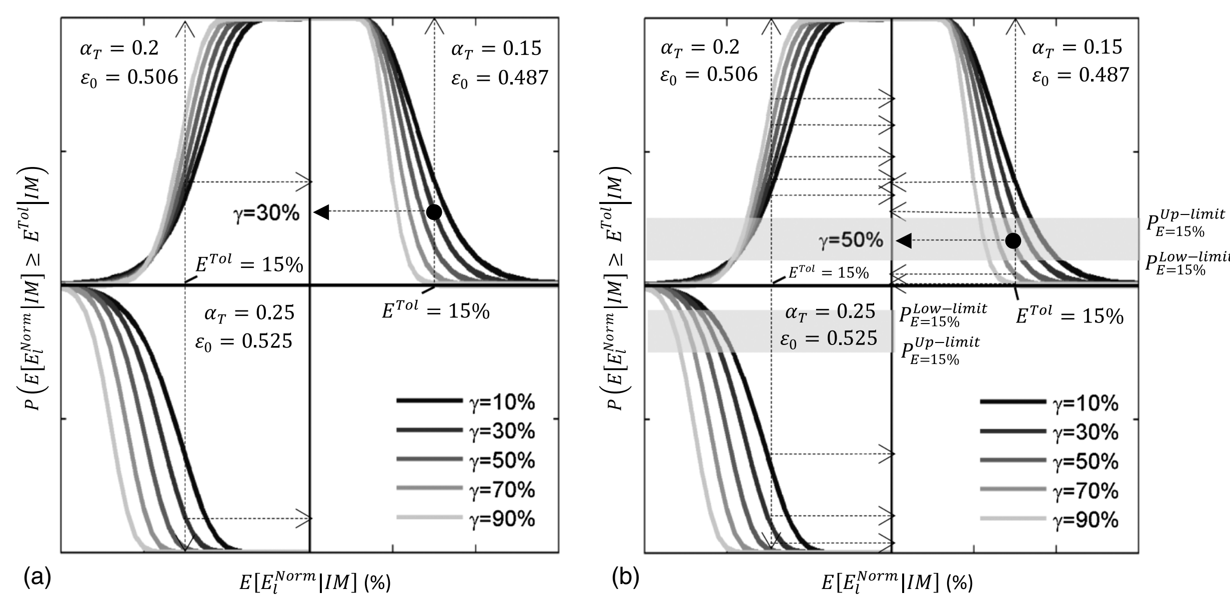

Fig. 6. PPBSD tools for a set of 4-story commercial RC office buildings, for $R = 7.2$ km, $M_w = 6.72$, $\varepsilon_0 = (0.487,\ 6150.506, 0.525)$, corresponding to $\alpha_T = (0.15, 0.2, 0.25)$ for DBE hazard level and given maximum tolerable target loss normalized by total building replacement cost, $E^{Tol} = 15\%$: (a) Case No. 1; and (b) Case No. 2. (Data from USGS 2008.)

shaded area in Fig. 6(b). As can be seen in this figure, the appropriate option is the stiffer building with $T^* = 0.6$ s. The designer will proportion the structural members of the 4-story building with a target period of 0.6 s.

Other design examples can be suggested for which one needs to identify how much the contents of a designed building can be changed with a target performance. Referring to Case 1 described earlier, one can observe that all cases with $\gamma < 30\%$ for the selected design (i.e., $T^* = 0.6$ s) are plausible as their probability of exceeding $E^{\mathrm{Tol}} = 15\%$ is less than 25%. A similar approach for Case 2 shows that all cases with $\gamma < 50\%$ are acceptable.

Conclusions

This paper has described the development and application of a conceptual design process denoted as PPBSD. The PPBSD design tools are extracted from a large database of information developed from loss estimation of all possible combinations of building properties and seismic hazard information. The proposed design approach is inthe form of comparison tools that show pros and cons of alternative building designs with respect to their seismic performance (expected loss at a target hazard level). The process for development of design tools required for implementation of

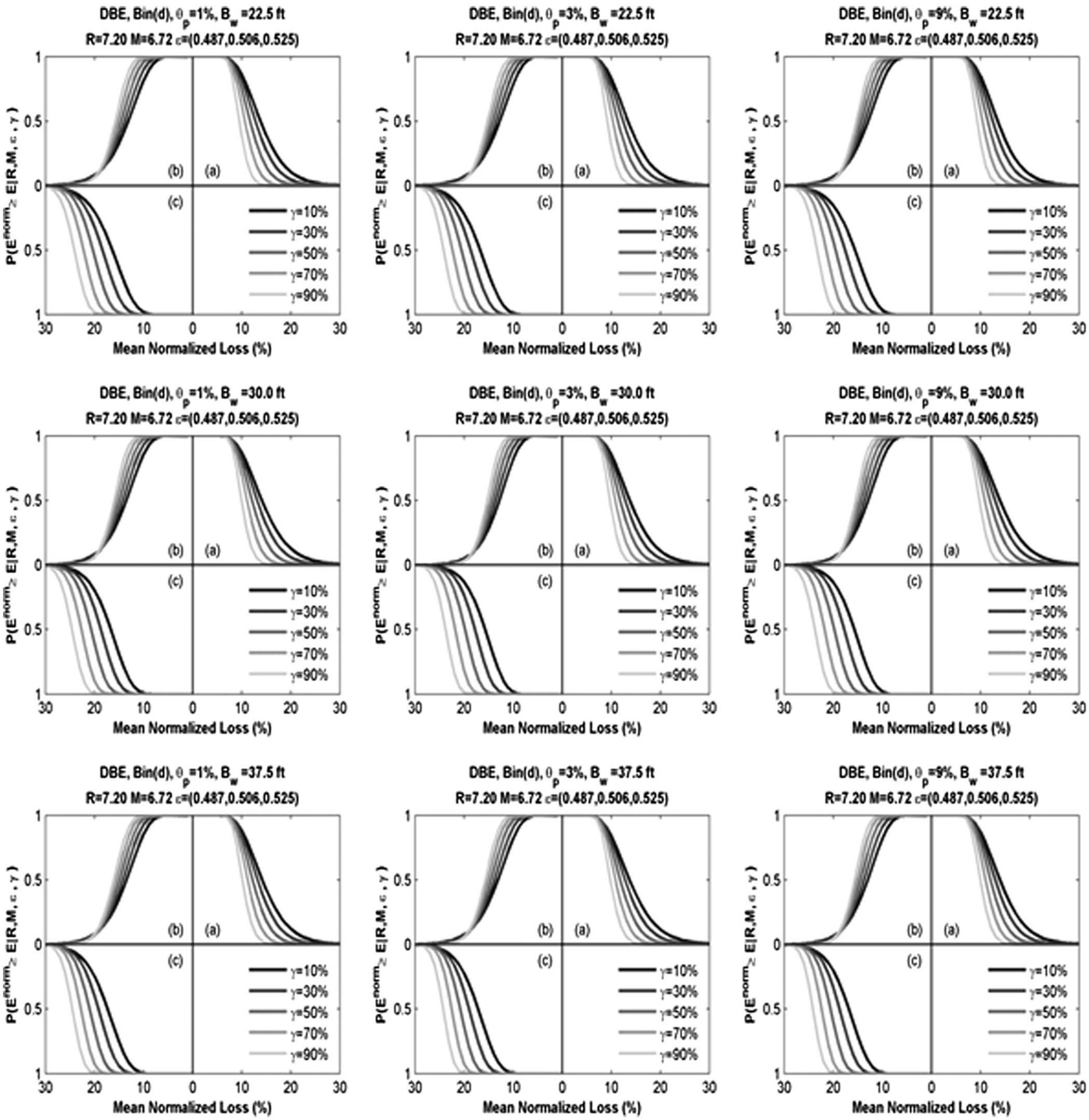

Fig. 7. PPBSD tools for a set of 4-story commercial RC office buildings, for $R = 7.2$ km, $M_w = 6.72$, $\varepsilon_0 = (0.487, 0.506, 0.525)$ corresponding to $\alpha_T = (0.15, 0.2, 0.25)$ at DBE hazard level, $B_w = 6.9$ mm (22.5 ft) and structural component parameters (θ_p, θ_{pc}/θ_p, λ) of (a) (1%, 25.6%, 24); (b) (3%, 16.3%, 59); and (c) (9%, 10.5%, 145), respectively.

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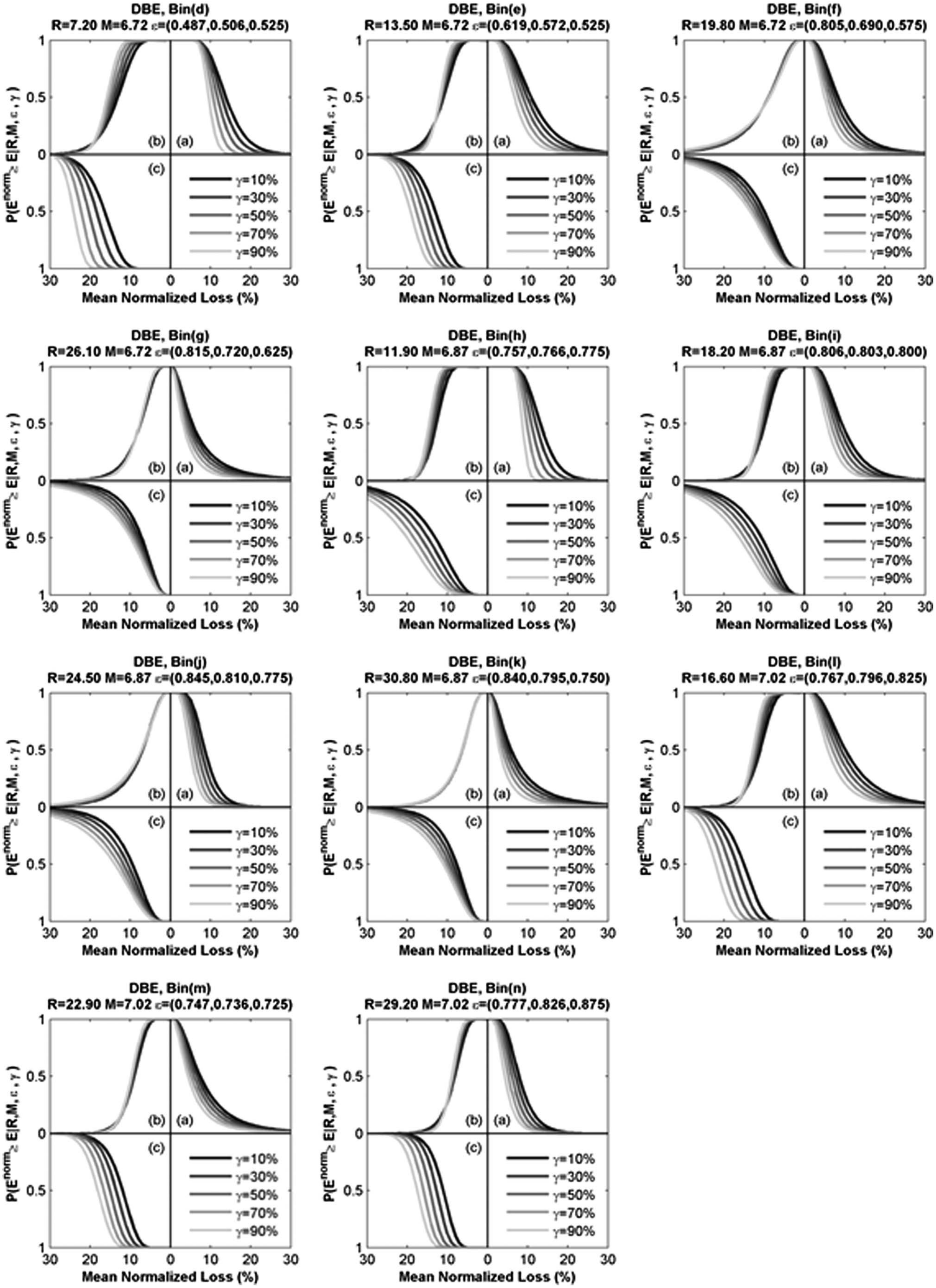

Fig. 8. PPBSD tools for a set of 4-story commercial RC office buildings at DBE hazard level. For the parts (a), (b), and (c) of each plot, the parameters are defined based on the legend.

PPBSD is described and illustrated for 4-story RC-SMRF office buildings located in Los Angeles for a 475-year ground motion return period. The PPBSD tools show that from all building parameters, period coefficient, α_T and γ have prominent effects on the probability of exceedance of building's expected normalized loss compared to structural component parameters θ_p, θ_{pc}/θ_p and λ and B_w.

Appendix. PPBSD Design Tools

The process for developing PPBSD design aids is demonstrated with a 4-story RC-SMRF as illustrated in Figs. 7 and 8 below.

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