Performance-Based Seismic Design Using REDSET – A Novel Reliability Evaluation Approach

ABSTRACT

Considering the economic impacts of recent major earthquakes all over the world, the Performance-Based Seismic Design (PBSD) concept is now under development, replacing the life safety concept, to be incorporated in the next generation of design guidelines. Although PBSD is a risk-based concept, no such acceptable concept is currently available. To fill this knowledge gap, the authors and their team recently proposed a concept known as the Reliability Evaluation of Dynamic Systems Excited in the Time Domain (REDSET). The implementation of PBSD using REDSET is demonstrated in this chapter. The structures are represented by finite elements and the seismic loading is applied in the time domain incorporating all major sources of nonlinearity and uncertainty in the design variables. The Limit Performance Functions (LPFs) are implicit for this class of problems. To retain simplicity, they are made explicit using the response surface concept and the First-Order Reliability Method (FORM) is used to extract the reliability evaluation. The basic Monte Carlo Simulation (MCS) method is used to verify REDSET. The authors then developed required serviceability LPFs correlating them with performance levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO), as suggested in PBSD. The capabilities of REDSET to implement PBSD are demonstrated with the help of examples. Uncertainties in the design earthquake time history are incorporated using multiple time histories as suggested in design guidelines. The information on the reliability estimated using REDSET correlates well with different levels of performance. The study clearly indicate that the reliability information can be obtained using only a few hundred instead of millions of deterministic nonlinear finite element analyses. The authors believe that REDSET can be used to advance the development of the PBSD philosophy further.

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Keywords: Performance-based seismic design; finite element method; first-order reliability method; response surface method; limit performance functions.

1. INTRODUCTION

The Performance-Based Seismic Design (PBSD) concept is an attractive alternative in developing the next generation design guidelines or nonprescriptive codes. It provides an option to the owner of a structure to select limiting property damage or loss of economic activities of the region, as an alternative to the current design practice of life safety. It has attracted serious attention from the professionals. In a comprehensive study funded by the Federal Emergency Management Agency (FEMA), the PBSD concept was advocated in several reports including FEMA-273 [1] and SAC [a joint venture of the Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREE)] [2-4]. The primary objective of the PBSD concept is to design structures for different performance levels satisfying some prescribed risks. Obviously, different risk levels will have different consequences, and the owner may accept such outcomes reflecting their preferences. The major challenges in developing the PBSD guidelines is the estimation of risk corresponding to different performance levels acceptable to all concerned parties. No such risk evaluation procedure is currently available. To fill this knowledge gap and with the financial support of US NSF, the authors and their team recently proposed a concept known as the Reliability Evaluation of Dynamic Systems Excited in Time Domain (REDSET) [5]. It is expected that REDSET will be essential in implementing the PBSD guidelines.

Comentado [A2]: This philosophy has already been used in many design codes.

Comentado [A3]: The introduction is too short and does not provide sufficient elements to give the background to the problem of performance-based seismic design philosophy. It is recommended to add more information on these issues.

2. LITERATURE REVIEW

The novel concept behind the PBSD guidelines is still being developed and the available literature on the topic is very limited. In developing the concept, FEMA 355F [4] identified six items that need to be addressed in developing the guidelines. They are: (1) account for uncertainty in the performance associated with unanticipated events, (2) set realistic expectations for performance, (3) assess performance variables in similar buildings located nearby, (4) develop a reliability framework, (5) set representative performance levels for various seismic hazards, and (6) quantify local and global structural behaviors leading to collapse. There is a knowledge gap specifically in addressing item (4). The SAC project [2-4] suggested a reliability framework but failed to identify any appropriate procedure acceptable to all parties. Based on a comprehensive literature review, the team concluded that the currently available reliability evaluation procedures cannot be used to implement the PBSD guidelines, and a new procedure needs to be developed as expeditiously as possible. In general, the deterministic design community is not familiar with the reliability-based concept. In developing such a novel reliability reliability-based design approach, their interests need to be addressed appropriately.

To satisfy the needs of the design community, the team decided to incorporate several features routinely used by them. Risk is always estimated with respect to a specific Limit Performance Function (LPF). In estimating the probability of failure (p_f), it is necessary to follow the same failure path used by them, i.e., structural performance should be tracked from elastic to inelastic, and to complete collapse. Before failure, the structure develops several sources of nonlinearities. To incorporate all these features, the engineering profession represents structures by finite elements (FEs). Thus, it is essential that the proposed reliability evaluation technique should also be FE based. For the most sophisticated deterministic analysis, seismic loading is applied in the time domain. Thus, in developing a reliability method, the seismic loading also needs to be applied in the time domain. To satisfy these challenging requirements or expectations REDSET is developed to fill this knowledge gap.

For nonlinear structures excited by the seismic loading in the time domain, the required LPFs become implicit. Since the calculation of derivatives of the LPFs with respect to the design variables becomes extremely tedious [6], the risk estimation by commonly used First-Order or Second-Order Reliability Method (FORM/SORM) can be very demanding. When LPFs are implicit, among several options, the basic Monte Carlo Simulation (MCS) becomes an attractive alternative but is not efficient. It requires thousands or millions of deterministic analyses for extracting reliability information, requiring several thousands of hours of computational time. To eliminate this deficiency, implicit LPFs can be made explicit using the Response Surface Method (RSM) [5]. In the context of RSM, several deterministic evaluations are conducted following a sampling scheme around a center point to generate the response information. Then, a polynomial is used to fit the response data using the regression analysis. However, the basic RSM procedure has three major deficiencies: it cannot incorporate information on the distribution of Random Variables (RVs), needs to be generated in the failure region (unknown for most problems), and the required optimal sampling scheme is an open question in generating a Response Surface (RS). To address the first two deficiencies, the team decided to integrate RSM and FORM. The iterative process of FORM will locate the Most Probable Failure Point (MPFP) incorporating the distributional information of all RVs. For the third deficiency, the authors proposed several advanced schemes. Once the explicit expression of a RS is obtained, FORM can be used to estimate the underlying risk.

Comentado [A4]: In this part we also do not have enough elements to give a complete literature review, it is recommended to add more information on this topic.

3. A NOVEL RELIABILITY EVALUATION APPROACH - REDSET

REDSET is developed addressing all the concerns discussed above. However, the process needs to be executed sequentially and systematically as discussed in the following sections.

3.1 Finite Element Evaluation

In the context of REDSET, a structure to be designed by the PBSD concept is represented by FEs. Considering its numerous advantages over the commonly used displacement-based finite element method, the Stress-Based FE Method (SB-FEM) is selected for the calculation of deterministic seismic responses [7, 8]. There are several attractive features of SB-FEM, particularly when a structure is of frame type. The tangent stiffness matrix can be expressed in an explicit form, requiring fewer FEs, and numerical integration is not necessary for the calculation of the stiffness matrix at each step of the time domain analysis. A detailed discussion of these topics can be found in [8].

Comentado [A5]: Please provide more details of the evaluation

3.2 Integration of RSM And FORM

As discussed earlier, the team decided to integrate RSM and FORM to incorporate the distributional information of RVs and to locate the failure region. However, the original RSM concept was developed in the coded variable space. The i^{th} RV is expressed as:

$$X_i = X_i^C + h x_i \sigma_{X_i} \quad where \quad i = 1, 2, \dots, k \tag{1}$$

where k is the number of RVs, X_i is the region or bound of the i^{th} RV, X_i^C is the coordinate of the center point of the i^{th} RV, h is an arbitrary factor controlling the experimental sampling region, x_i is the coded variable which has values of 0, ±1, or $\sqrt[4]{2^k}$, and σ_{X_i} is the standard deviation of the i^{th} RV. However, it does not incorporate distributional information of RVs, and selection of the simulation or failure region is subjective. The integration of RSM and FORM eliminates these deficiencies. FORM is implemented in the normal variable space. For the reliability evaluation of real structures, all RVs are not expected to be normal. In the context of FORM, all non-normal RVs need to be transformed to equivalent normal RVs at the checking point. The equivalent standard deviation ($\sigma_{X_i}^N$) and mean ($\mu_{X_i}^N$) can be calculated by equating the Cumulative Distribution Functions (CDFs) and the Probability Density Functions (PDFs) of the original non-normal RVs to those of the equivalent normal variables [6] as:

$$\sigma_{X_i}^N = \frac{\emptyset\{\Phi^{-1}[F_{X_i}(x_i^*)]\}}{f_{X_i}(x_i^*)} \tag{2}$$

and

$$\mu_{X_i}^N = x_i^* - \Phi^{-1}[F_{X_i}(x_i^*)]\sigma_{X_i}^N \tag{3}$$

where $\Phi(\ )$ and $\emptyset(\ )$ are the CDF and PDF of the standard normal variable, respectively, x_i^* is the checking point, and $F_{X_i}(x_i^*)$ and $f_{X_i}(x_i^*)$ represent the CDF and PDF of the original non-normal variables at the checking point x_i^*, respectively. Once all the non-normal RVs are transformed to equivalent normal variables, the iteration process of FORM will be initiated by substituting X_i and σ_{X_i} in Eq. (1) by $\mu_{X_i}^N$ and $\sigma_{X_i}^N$, respectively.

3.3 Explicit Representation of Response Surface

In the context of PBSD, the explicit form of a RS is expected to be nonlinear. The selection of more than second order polynomials for a RS may result in ill-condition of the system of equations [5]. The authors decided to mathematically represent a RS using a second-order polynomial without or with cross terms. They can be represented as:

$$\hat{g}(\boldsymbol{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 \tag{4}$$

and

$$\hat{g}(\boldsymbol{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 + \sum_{i=1}^{k-1}\sum_{j>1}^{k} b_{ij} X_i X_j \tag{5}$$

where X_i (*i=1,2,…,k*) is the i^{th} RV, *k* was defined earlier, b_0, b_i, b_{ii} and b_{ij} are the unknown coefficients to be determined, and $\hat{g}(\boldsymbol{X})$ is the approximate explicit expression for the RS of interest, representing the original unknown RS [$g(\boldsymbol{X})$]. The numbers of unknown coefficients to be estimated play a very important role in the efficiency and accuracy of the integrated approach. If Eq. (4) is used, the number of coefficients to be determined will be $2k + 1$. If Eq. (5) is used, it will be $(k + 1)(k + 2)/2$ [5]. Generating a RS with an optimal number of coefficients will depend on the number of RVs present in defining a LPF and the performance level as will be discussed in more detail later.

3.4 Advanced Reliability Scheme for The Selection of Experimental Sampling Points to Generate a Response Surface

The efficiency and accuracy of REDSET will depend on the selection of the experimental sampling points around a center point. This will be denoted as the Total Number of Sampling Points (TNSP) or deterministic FE analyses required to generate a RS. In the context of FORM, the iteration process will be initiated at the mean values of all RVs, and it will be the initial center point. Two commonly used schemes for selecting experimental sampling points are Saturated Design (SD) and Central Composite Design (CCD) [5, 9]. In SD, a second-order polynomial without or with cross terms can be used, and the required RS can be generated by solving a set of equations. SD requires only as many TNSP as the total number of unknown coefficients of the RS. The TNSP required to generate a RS without and with cross terms using SD can be shown to be $2k + 1$ and $(k + 1)(k + 2)/2$, respectively [5]. SD is expected to be very efficient, but its accuracy cannot be assured. CCD is expected to be very accurate but inefficient. It requires a second-order polynomial with cross terms [Eq. (5)] and a regression analysis is required to generate a RS. The TNSP required to implement CCD will be $2^k + 2k + 1$. Considering 70 RVs or *k* = 70, the TNSP required using SD without and with cross terms will be 141 and 2556, respectively. But for CCD, it will require $1.180591621 \times 10^{21}$ TNSP, indicating it cannot be used.

To retain accuracy, the authors propose to reduce the total number of RVs present in a LPF using the sensitivity analysis as suggested by Haldar and Mahadevan [6]. The sensitivity index can be defined in terms of direction cosines of RVs, readily available from the FORM analysis. RVs with low sensitivity indexes can be treated as deterministic at their mean values in subsequent iterations. The reduced number of RVs is denoted hereafter as k_r. Suppose, out of a total of 70 RVs, only 7 are found to be very sensitive, i.e., k_r = 7. TNSP required to implement CCD will be $2^7 + 2 * 7 + 1 = 143$. Based on this observation, the authors proposed numerous schemes [5] including the following Advanced Reliability Scheme (ARS). In the first iteration, a required RS will be generated using SD without cross terms. Then, using FORM, the direction cosines of all RVs will be estimated [6]. Using the information, only k_r number of RVs will be used in all subsequent iterations. Several iterations may be required to locate the MPFP. For the intermediate iterations, SD without cross terms [Eq. (4)] can be used but with k_r number of RVs. Then, in the last iteration, CCD with cross terms [Eq. (5)] will be used to extract the reliability information. Suppose the reliability of a structure needs to be estimated for a LPF with *k* = 70. Only 7 RVs are found to be the most sensitive, thus, k_r = 7. TNSPs required to implement the proposed procedure can be shown to be $(2 * k + 1) + (2 * k_r + 1) + (2^{k_r} + 2 * k_r + 1) = (2 * 70 + 1) + (2 * 7 + 1) + (2^7 + 2 * 7 + 1) = 299$. This is very reasonable as compared to thousands or millions of MCS.

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3.5 Evaluation of Performance Levels

At this time, information on RSs will be available. The information on LPFs can be generated if performance levels are known. FEMA 350 [2] defined three performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Since PBSD is implemented in terms of multiple target performance levels, FEMA-273 [1] and -350 [2] suggested allowable drift values (δ_{allow}) for the CP, LS, and IO performance levels in terms of earthquake return period and probability of exceedance as summarized in Table 1.

Table 1. Allowable drifts corresponding to CP, LS, and IO performance levels for different seismic hazards.

Performance Level	Earthquake Return Period	Probability of Exceedance	Allowable Drift (δ_{allow})
CP	2475-year	2% in 50 years	0.050*H
LS	475-year	10% in 50 years	0.025*H
IO	72-year	50% in 50 years	0.007*H

In Table 1, δ_{allow} is a function of *H*, the total height of the structure if overall top roof deflection is evaluated, or the story height if inter-story drift is considered. The information can be used to develop serviceability LPFs. Considering that a structure may fail due to excessive lateral deflection or inter-story drift due to a seismic excitation, the corresponding LPFs can be generated and the PBSD guidelines can be implemented.

3.5.1 Serviceability LPF

For the seismic loading, the serviceability LPF can be expressed as:

$$g(\mathbf{X}) = \delta_{allow} - \hat{g}(\mathbf{X}) \tag{6}$$

where δ_{allow} values can be obtained from Table 1 for a specific performance level and $\hat{g}(\mathbf{X})$ is the RS obtained from Section 3.4.

3.6 Calculation of Structural Reliability

The REDSET algorithm can be implemented with the help of 22 steps [5] and cannot be presented here. To extract reliability information, the necessary response information will be generated at the sampling points by calculating the maximum responses caused by an earthquake time history using SB-FEM. In the first iteration of FORM, an approximation of the LPF will be generated by using SD and Eq. (4) at the mean values of all RVs in the normal variable space. At the end of the first iteration, the sensitivity indexes of all RVs will be available. RVs with low sensitivity indexes will be considered as deterministic at their mean values and *k* will be reduced to k_r. The next iteration will start by using k_r number of RVs and a new LPF will be reconstructed using SD and Eq. (4). Using the updated LPF, the FORM iterations will continue until the RVs direction cosines converge to a pre-determined tolerance level [5, 6]. Then, the first estimate of β will be calculated using the standard FORM procedure and the coordinates of the new checking point (x_i^*) or center point will be recalculated as:

$$x_i^* = \mu_{X_i}^N - \alpha_i \beta \sigma_{X_i}^N \tag{7}$$

The overall updating will continue until β converges to a pre-established tolerance level [6]. In one of the ARS using CCD, in the final iteration, the information on the required RS using the regression analysis and the corresponding performance level will be used to generate a LPF will. It usually takes 3 to 4 iterations to reach the convergence of the β value. The coordinates of the last checking point $\boldsymbol{x}^*$can be used to estimate the reliability index β as:

$$\beta = \sqrt{(\boldsymbol{x}^*)^t(\boldsymbol{x}^*)} \tag{8}$$

The corresponding p_f can be estimated as:

$$p_f = \Phi(-\beta) = 1.0 - \Phi(\beta) \tag{9}$$

A flowchart of the integrated structural reliability evaluation approach is shown in Figure 1.

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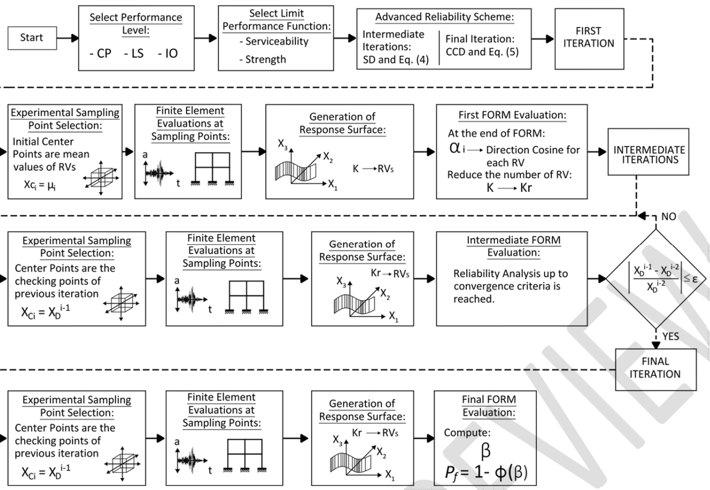

Fig. 1. Flowchart of the integrated approach for PBSD

4. NUMERICAL EXAMPLE: RELIABILITY EVALUATION

Several steel buildings were designed by experts during the SAC project. Among them, a 3-story steel building reported in FEMA-355C [3] is selected to demonstrate the application of REDSET to implement PBSD. The building was specifically designed for the Los Angeles area, satisfying the post-Northridge earthquake requirements. W-sections for columns and girders are shown in Fig. 2. Further information can be obtained in [10].

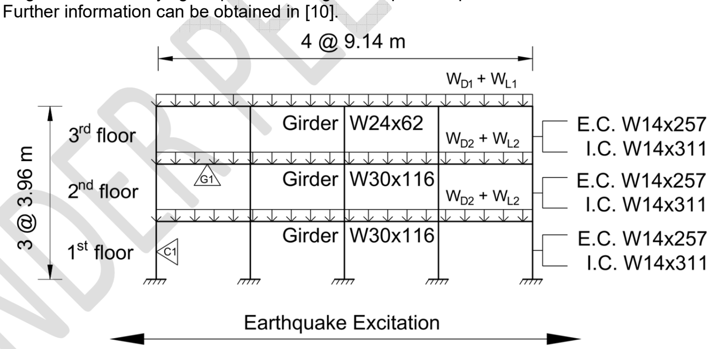

Fig. 2. A 3-story frame

4.1 Incorporation of Uncertainties in Loads and Resistance-Related Parameters

For the reliability estimation of real structures exited by the seismic loading in the time domain, uncertainty associated with all major load and resistance-related parameters must be considered, as discussed next.

4.1.1 Uncertainties In resistance-related parameters

The uncertainties associated with resistance-related parameters are widely reported in the literature [6] and the information is used in this study. The integrated approach will be demonstrated by considering the performances of a steel building. All structural elements are represented by W-sections. Young's modulus (E), yield stress of columns (F_{yc}) and girders (F_{yg}), the

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cross-sectional area (*A*), and moment of inertia (*I*) of W-sections used for structural elements are considered to be RVs with a lognormal distribution with Coefficient of Variations (COVs), as shown in Table 2.

Table 2. Uncertainties in resistance-related parameters, gravity loads, and seismic loading

Random Variable (RV)	Distribution	Mean Value	COV
E (kN/m^2)	Lognormal	1.9994X10^8	0.06
F_{yc}* (kN/m^2)	Lognormal	3.4474X10^5	0.10
F_{yg}** (kN/m^2)	Lognormal	2.4822X10^5	0.10
A (m^2)	Lognormal	***	0.05
I_x (m^4)	Lognormal	***	0.05
W_{D1} (kN/m)	Normal	32.9457	0.10
W_{D2} (kN/m)	Normal	32.9457	0.10
W_{L1} (kN/m)	Type 1	2.9188	0.25
W_{L2} (kN/m)	Type 1	2.9188	0.25
g_e	Type 1	1.00	0.20

4.1.2 Uncertainties In gravity loads

In most design guidelines [12, 13], the gravity loads are classified as Dead Load (DL) and Live Load (LL). The uncertainties associated with them are available in [5] and the authors used similar information in this paper. DL and LL are represented by a normal and Type 1 distributions with COV of 0.10 and 0.25, respectively, as shown in Table 2. W_{D1} and W_{D2} represent DL at the roof and floor levels, respectively. W_{L1} and W_{L2} are the LL for roof and floor levels, respectively.

4.1.3 Uncertainties in the seismic loading

Consideration of uncertainties in seismic loading is very challenging, and it is still evolving. Uncertainty associated with the intensity and the frequency contents needs to be considered. A factor (g_e) is considered to incorporate the uncertainty in the intensity with a Type 1 distribution and with a COV of 0.2. To incorporate uncertainty in the frequency contents, several recent design guidelines [12, 13] suggested consideration of at least seven time histories expected for the location. For PBSD, multiple performance levels have to be considered, and the corresponding risks need to be estimated, as suggested in [1-5]. Somerville et al. [14] developed three sets of ground motion time histories related to 2%, 10%, and 50% PE in 50 years for the Los Angeles (LA) area and correlated them with the performance levels of CP, LS, and IO, respectively. For every performance level, ten ground motions with two horizontal orthogonal components were proposed, providing twenty time histories per set. They applied Scale Factors (SFs) to match specific target response spectral values, on average, for periods at 0.3, 1.0, 2.0, and 4.0 seconds for site category S_D (firm soil), as suggested by the US Geological Survey (USGS). Specific information on these earthquake (EQ) sets is summarized in Tables 3-5. The authors used these three sets of ground motions for the reliability evaluation of the steel frame.

Comentado [A6]: It is not mentioned which were the main points with which the non-linear time history analyses were carried out. Furthermore, it is not clearly and precisely indicated whether spectra or records are used, as it is indicated that the spectra are scaled, but in order to carry out non-linear analyses it is necessary to carry out a time history analysis.

Table 3. Set 1: information and results for ground motions associated with 2% PE in 50 years and CP

EQ	Record Name	Scale Factor	LPF1		LPF2	
			β	TNSP	β	TNSP
1	1995 Kobe	1.15	7.50	211	4.98	211
2	1995 Kobe	1.15	5.44	196	5.35	211
3	1989 Loma Prieta	0.82	4.84	226	5.93	211
4	1989 Loma Prieta	0.82	5.44	211	5.35	211
5	1994 Northridge	1.29	7.75	211	7.08	211
6	1994 Northridge	1.29	10.12	196	5.83	226
7	1994 Northridge	1.61	3.82	196	3.49	196
8	1994 Northridge	1.61	5.08	211	4.43	196
9	1974 Tabas	1.08	6.09	196	9.78	211
10	1974 Tabas	1.08	4.20	211	7.26	211
11	Elysian Park (simulated)	1.43	6.60	211	6.01	226
12	Elysian Park (simulated)	1.43	5.54	211	5.10	226
13	Elysian Park (simulated)	0.97	6.59	211	8.52	211
14	Elysian Park (simulated)	0.97	4.12	226	4.83	211

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EQ	Record Name	Scale Factor	LPF1		LPF2	
			β	TNSP	β	TNSP
15	Elysian Park (simulated)	1.1	10.68	196	9.69	211
16	Elysian Park (simulated)	1.1	4.34	196	4.36	211
17	Palos Verdes (simulated)	0.9	10.21	211	10.13	226
18	Palos Verdes (simulated)	0.9	6.33	211	6.22	196
19	Palos Verdes (simulated)	0.88	7.75	211	10.62	196
20	Palos Verdes (simulated)	0.88	6.64	226	9.13	211

Table 4 – Set 2: information and results for ground motions associated with 10% PE in 50 years and LS

EQ	Record Name	Scale Factor	LPF1		LPF2	
			β	TNSP	β	TNSP
21	Imperial Valley, 1940	2.01	4.80	211	4.47	196
22	Imperial Valley, 1940	2.01	4.47	211	4.24	196
23	Imperial Valley, 1979	1.01	5.18	211	4.87	211
24	Imperial Valley, 1979	1.01	8.45	196	7.72	211
25	Imperial Valley, 1979	0.84	10.70	196	9.69	226
26	Imperial Valley, 1979	0.84	5.63	226	10.72	196
27	Landers, 1992	3.2	7.27	211	7.61	211
28	Landers, 1992	3.2	6.77	211	6.95	226
29	Landers, 1992	2.17	5.88	211	5.40	211
30	Landers, 1992	2.17	5.62	196	5.46	226
31	Loma Prieta, 1989	1.79	5.64	211	5.01	196
32	Loma Prieta, 1989	1.79	4.70	226	4.57	211
33	Northridge, 1994, Newhall	1.03	5.77	196	5.52	226
34	Northridge, 1994, Newhall	1.03	4.36	196	4.05	211
35	Northridge, 1994, Rinaldi	0.79	6.74	196	5.83	211
36	Northridge, 1994, Rinaldi	0.79	5.58	226	5.13	211
37	Northridge, 1994, Sylmar	0.99	5.42	211	6.16	211
38	Northridge, 1994, Sylmar	0.99	4.18	211	6.63	196
39	North Palm Springs, 1986	2.97	5.58	211	4.63	226
40	North Palm Springs, 1986	2.97	7.77	211	6.79	226

Table 5. Set 3: information and results for ground motions associated with 50% PE in 50 years and IO

EQ	Record Name	Scale Factor	LPF1		LPF2	
			β	TNSP	β	TNSP
41	Coyote Lake, 1979	2.28	7.59	226	8.89	196
42	Coyote Lake, 1979	2.28	5.21	211	4.85	196
43	Imperial Valley, 1979	0.4	8.93	211	8.12	211
44	Imperial Valley, 1979	0.4	9.68	196	8.90	211
45	Kern, 1952	2.92	9.34	196	8.17	211
46	Kern, 1952	2.92	4.24	211	3.91	211
47	Landers, 1992	2.63	3.57	211	6.08	211
48	Landers, 1992	2.63	4.68	211	4.26	196
49	Morgan Hill, 1984	2.35	3.95	226	3.58	226
50	Morgan Hill, 1984	2.35	4.33	211	3.83	226
51	Parkfield, 1966, Cholame	1.81	10.39	211	7.75	226
52	Parkfield, 1966, Cholame	1.81	9.12	211	5.48	211
53	Parkfield, 1966, Cholame	2.92	3.82	211	3.28	211
54	Parkfield, 1966, Cholame	2.92	7.69	226	6.16	211

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EQ	Record Name	Scale Factor	LPF1		LPF2	
			β	TNSP	β	TNSP
55	North Palm Springs, 1986	2.75	4.30	211	5.75	196
56	North Palm Springs, 1986	2.75	4.10	211	4.62	196
57	San Fernando, 1971	1.3	4.66	196	4.27	211
58	San Fernando, 1971	1.3	7.08	211	6.33	211
59	Whittier, 1987	1.27	6.50	226	8.12	211
60	Whittier, 1987	1.27	7.91	211	9.40	211

5. IMPLEMENTATION OF REDSET FOR PBSD OF A 3-STORY BENCHMARK STEEL BUILDING

Structural reliability of the frame according to the requirements in Table 1 and excited by the ground motion given in Tables 3 to 5, and its structural reliability are calculated in terms of β considering two LPFs: overall top roof deflection (LPF1) and inter-story drift at the 2nd floor (LPF2). δ_{allow} for LPF1 are 59.4, 29.7, and 8.3 cm for CP, LS, and IO performance levels, respectively. δ_{allow} for LPF2 are 19.8, 9.9, and 2.8 cm, respectively. For the two LPFs, the total number of RVs is 26, i.e. $k = 26$. Only 7 of them are found to be the most sensitive or $k_r = 7$. The structural reliability values estimated by REDSET are summarized in Tables 3-5 in terms of β and TNSP for each LPF. It is observed that even for two components of the same earthquake, the β values are different indicating that the design of a structure using only one earthquake time history is not adequate; several ground motions must be used, as recommended in recent building codes [12, 13].

The authors believe that the development of PBSD guidelines is a step in the right direction. The values suggested in [2], as summarized in Table 1, are reasonable. The estimated reliability indexes correlate well with different levels of performance, indicating that the proposed reliability method is viable. The inter-story drift appears to be more critical than the overall deflection. The study indicates that reliability information can be extracted using only a few hundred instead of millions of deterministic analyses. The authors strongly believe that the proposed method can be used to advance the development of PBSD.

Comentado [A7]: The results of the seismic analyses, such as displacement profiles, interstory distortions, among others, are not provided. Likewise, there is no mention of what the objective of this work was to compare with or other details that would have been of great help in understanding the proposal being made in this paper.

6. CONCLUSIONS

A risk-based design concept must be available to implement the PBSD guidelines to satisfy all the concerned parties. It must also satisfy the current design practices. Structures need to be represented by finite elements and the seismic loading must be applied in time domain to incorporate all major sources of nonlinearity and uncertainty in the formulation. However, since the required LPFs are implicit, besides the basic MCS, other procedures may not be currently available. A novel reliability evaluation procedure was proposed to fill this knowledge gap. The basic response surface method was significantly improved by removing its deficiencies and then it was integrated with FORM to locate the failure region. In this way, an implicit LPF was approximately represented. Then, FORM was used to extract reliability information. The authors developed required serviceability LPFs and correlated them with the three performance levels of CP, LS, and IO, as suggested by FEMA and SAC. To demonstrate its implementation potential, a 3-story steel building designed by experts satisfying post-Northridge design requirements is considered. The structure is excited by three sets of 20 ground motions representing three performance levels of CP, LS, and IO. The results indicated that the structure is well-designed for the serviceability requirements [2]. Seismic design of structures needs to be performed using multiple time histories, as suggested in recent design guidelines. The results correlate well with different levels of performance, indicating that the reliability method is viable. The study indicates that reliability information can be obtained using only a few hundred deterministic analyses. The authors named the novel concept REDSET.

Comentado [A8]: What results?

Comentado [A9]: In this part no conclusions are given at all but rather a short summary of the article and the results shown, however there are no results of any kind in this part.

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