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Some observations on performance-based and reliability-based seismic design of asymmetric building structures

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Abstract

In previous work by the second author and others, a framework for a reliability-based, performance-based seismic design procedure was presented. That framework was based on static pushover analyses of two-dimensional models of building structures. In this paper, some preliminary observations and results are presented from a study in which the framework is extended to be used with three-dimensional analysis models to explicitly account for the torsional behavior of asymmetric building structures. © 2001 Elsevier Science Ltd. All rights reserved.

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1. Introduction

A framework for a performance-based seismic design procedure that incorporates structural reliability concepts has been proposed by Collins et al. [1,2]. The framework focuses on predicting structural displacements and interstory drifts instead of forces as displacements are better indicators of structural damage. The basic idea behind the procedure is to develop an "equivalent" single-degree-of-freedom (SDOF) model of the structure and then to use this model with uniform hazard spectra to make probabilistic estimates of multi-degree-of-freedom (MDOF) structural response. The parameters of the equivalent SDOF model are calibrated using the results of static pushover analyses of an MDOF analytical model of the structure. Such static analyses are typically quicker and easier to carry out than multiple dynamic analyses of the same model. Deterministic design-checking equations are then used to ensure that the probabilistic estimates of the MDOF structural response satisfy the performance criteria required by the code. The performance criteria are specified in terms of the probability of exceeding critical values of structural response.

One of the shortcomings of the procedure proposed by Collins et al. [1,2] is that it is based on static pushover analyses of a two-dimensional analysis model of a building. Such analyses are acceptable if buildings are regular and symmetric or if torsional effects can be otherwise accounted for. However, many typical building designs are asymmetric and hence two-dimensional analyses may not be adequate. The purpose of this paper is to present some observations from a study in which the above-mentioned procedure was extended to situations in which the overall building configuration is asymmetric. For simplicity, a two-story asymmetric building configuration is considered.

Owing to space limitations, this paper does not provide a detailed discussion of the analytical modeling of the two-story structure or the mathematical basis for the design equations. For additional details, the interested reader is referred to the reports by Chen and Collins [3] and Collins et al. [1].

2. Brief overview of design equations

In the spirit of both performance-based design and reliability-based design, the target measures of structural

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performance must be linked with acceptable target reliability values for each limit state to be considered. Then, the performance criteria can be expressed in probabilistic terms, and design-checking equations can be developed using reliability theory. For example, if R is some measure of structural response and R_{target} is the limiting (or target) value specified by a code, then a probabilistic performance criterion could be formulated as

$$P(R>R_{\mathrm{target}})\leq p_{\mathrm{target}} \tag{1}$$

where p_{target} is the maximum acceptable probability of exceedance corresponding to the limit state in question. It is important to note that the probability statement in Eq. (1) above applies directly to structural response and not to some measure of ground motion severity. Associating a probability with a ground motion level does not provide direct risk information on the performance of a structure designed for that earthquake.

In the original design procedure proposed by Collins et al., the response measures of interest are drift and global ductility. For example, for the serviceability limit state, the probabilistic criterion shown in Eq. (1) is stated as

$$P(\Delta>\Delta^{s}_{target})\leq p^{\mathrm{s}}_{\mathrm{target}} \tag{2}$$

where Δ represents interstory drift ratio and the superscript "s" denotes the serviceability limit state for which elastic behavior is assumed. Based on this criterion, the following deterministic design-checking equation can be derived [1,3]:

$$S_{\mathrm{d}}\leq\frac{H\Delta^{S}_{CODE}}{\mu_{\mathrm{N}}\Omega^{*}FP^{*}\beta_{\mathrm{LG}}} \tag{3}$$

In Eq. (3), S_{d} is the elastic spectral displacement obtained from the uniform hazard spectrum for probability $p^{\mathrm{s}}_{\mathrm{target}}$, H is the total height of the building, P^* and β_{LG} are equivalent system parameters derived from the results of a static pushover analysis, F is the site soil factor proposed by Borcherdt [4,5], μ_{N} is the mean value of the bias factor for drift related to the equivalent system methodology, and Ω^* is a design factor which accounts for three sources of uncertainty: the seismic hazard at the site, the uncertainty in the site soil factor, and the approximate nature of the equivalent system methodology. Similar, but more complex, design equations can be derived for the ultimate limit states.

As discussed in Chen and Collins [3], the same design equations can be used for asymmetric building structures. To incorporate three-dimensional torsional effects, the equivalent SDOF model must be derived from the results of pushover analyses of a three-dimensional MDOF structure. Intuitively, however, one might expect that the equivalent SDOF model will give less reliable predictions of response in this case. The focus of this paper is to see how this may or may not affect the values of the design factors.

3. Description of static and dynamic analyses

3.1. Building model

A series of static and dynamic analyses was carried out considering the two-story moment frame structure shown in Fig. 1. This two-story building is not intended to represent a real structure. It was chosen to keep the calculations relatively simple, and it was created in an asymmetrical way in order to have more tendencies for torsional response. The first three natural frequencies of the structure are f_x=2.53 Hz, f_y=3.26 Hz, and f_θ=4.55 Hz.

3.2. Static pushover analyses

Static pushover analyses were used to formulate equivalent SDOF models for the two-story MDOF frame. Analyses were performed in the global X and Y directions separately, and an equivalent SDOF model was created for each direction. The static analyses were carried out using ABAQUS/Standard [6], a general-purpose finite element program. The lateral force distribution was chosen to be the UBC [7] lateral force distribution.

To develop the equivalent SDOF model for the 3D asymmetric model, the vector of the generalized floor displacements was redefined to capture the torsional behavior as well as the horizontal translation of the structure in the X and Y directions. The generalized displacement vector $\{u\}$ for this two-story steel frame was formed as follows:

$$\{u\}=\begin{Bmatrix} u_{x2}\\ u_{y2}\\ \theta_2\\ u_{x1}\\ u_{y1}\\ \theta_1 \end{Bmatrix} \tag{4}$$

All displacements and rotations in $\{u\}$ are at the center of mass. Drift calculations were based on displacements at the corners of the building.

For each direction, two types of pushover analyses were performed. A linear elastic pushover analysis was performed to calibrate the parameters for a "linear" equivalent system model. A nonlinear pushover analysis was performed to calibrate the parameters for a "nonlinear" equivalent system model.

3.3. Dynamic analyses

The equivalent SDOF models provide a way to relate the probabilistic information provided in uniform hazard spectra to the response of an MDOF system. Obviously, using an SDOF model is an approximation, and it is

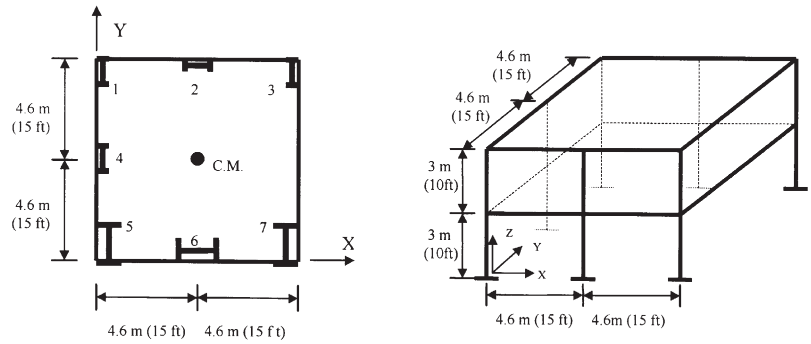

Fig. 1. Plan view and dimensions for the two-story asymmetric structure.

important to quantify the degree of approximation in developing a reliability-based design procedure. To this end, dynamic analyses of both the 3D MDOF model and the equivalent SDOF models were performed. There were 64 earthquake records used in the dynamic analyses consisting of both real and simulated earthquake records. Thirty-two of the records were classified as "small to moderate" earthquake records for which the MDOF response remained essentially linear elastic. The remaining 32 records were classified as "large" records as they tended to cause inelastic response behavior in the MDOF model. Analyses were performed in the global X and Y directions separately.

Rayleigh damping was used for both linear and nonlinear dynamic analyses. The damping ratio used in the equivalent SDOF models was 5% for both analyses. The damping for the MDOF model in the linear case was defined to have 5% damping in the first 16 natural modes of the frame. However, in the nonlinear case, the damping was set at 5% only for the first two natural modes of the frame.

3.4. Results

Fig. 2 shows some typical scatter plots that compare the maximum interstory drift ratio predicted by the "linear" SDOF models and the MDOF models. (The reference to "Virtual Work" on the vertical axis of each plot refers to the virtual work formulation of the equivalent SDOF model as discussed in Collins et al. [1,2] and Chen and Collins [3].) Fig. 3 presents similar information, but the results are for nonlinear inelastic responses of the MDOF frame. The degree to which the dots are close to the diagonal line provides an indication of the accuracy of the equivalent SDOF model in predicting MDOF response. (If the equivalent SDOF model were perfect, then all dots would lie on the diagonal line.) These plots clearly suggest that the equivalent SDOF model provides reasonable results for this structure.

The comparison of the dynamic results of the MDOF model and the SDOF model is facilitated by defining a variable called the "bias factor". The bias factor for a particular response quantity (displacement or drift) is defined as the ratio of the response calculated using the MDOF model to the response calculated using the equivalent system model, i.e.

$$\text{bias factor} = N = \frac{\text{MDOF response}}{\text{Equivalent SDOF response}} \tag{5}$$

This ratio was calculated for each earthquake record in each input direction (X and Y). Tables 1 and 2 summarize the statistics of the bias factors for roof displacement and interstory drift ratio. These statistics are the mean and standard deviation of the bias factors calculated for each earthquake record. These numbers provide a way

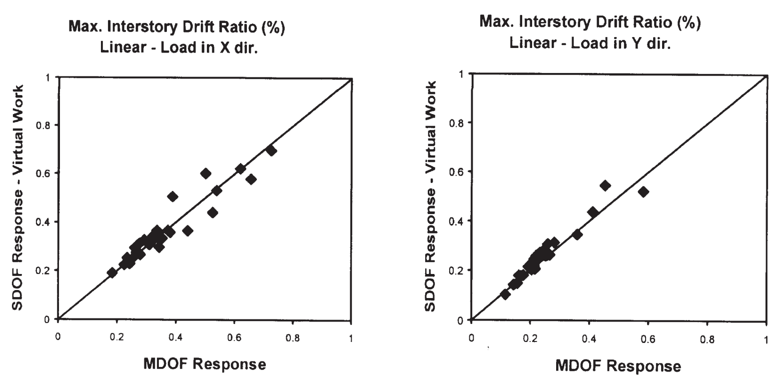

Fig. 2. Scatter plot comparing maximum responses (interstory drift ratio). Linear elastic case.

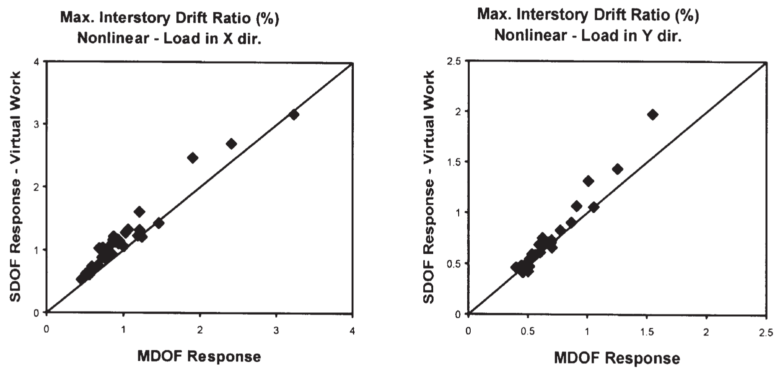

Fig. 3. Scatter plot comparing maximum responses (interstory drift ratio). Nonlinear inelastic case.

Table 1
Statistics for the bias factor for estimates of maximum interstory drift ratio for linear elastic response

	Maximum interstory drift ratio			
	Base shear		Virtual work	
	μ_N	σ_N	μ_N	σ_N
Load in X direction	0.88	0.11	0.99	0.098
Load in Y direction	0.82	0.045	0.95	0.078
Collins et al. [1,2]	0.99	0.095	1.10	0.12

"Base shear" and "virtual work" refer to different formulations of the equivalent SDOF model. See Chen and Collins [3] for details.

Table 2
Statistics for the bias factor for estimates of maximum interstory drift ratio for nonlinear inelastic response

	Maximum interstory drift ratio			
	Base shear		Virtual work	
	μ_N	σ_N	μ_N	σ_N
Load in X direction	0.76	0.093	0.84	0.091
Load in Y direction	0.76	0.071	0.86	0.090
Collins et al. [1,2]	0.96	0.14	1.03	0.13

"Base shear" and "virtual work" refer to different formulations of the equivalent SDOF model. See Chen and Collins [3] for details.

to quantify the results presented in Figs. 2 and 3. For comparison, Tables 1 and 2 show some bias statistics for a two-story, two-dimensional model of a steel moment-resisting frame considered by Collins et al. [1]. Observe that the mean values and standard deviations for both linear and nonlinear cases are comparable to those found by Collins et al. This suggests that torsional effects can be considered without significantly increasing the uncertainty resulting from the approximate SDOF model. More importantly, the uncertainty due to the equivalent system methodology is significantly less than the uncertainty in the ground motion and the soil factor. The significance of this will be discussed next.

4. Analysis of uncertainty

4.1. Calibration of design factor used in Eq. (3)

The random variables involved are the site soil factor (F), the bias factor (N) for interstory drift ratio, and the seismic hazard (S_d). In the present analysis, the probability distributions for the soil factor and the seismic hazard are the same as those used in Collins et al. Specifically, the soil factor is assumed to be lognormally distributed, and the seismic hazard is represented by a modified Type II extreme value distribution. For the drift bias factor, a Type II extreme value distribution was found to fit the data reasonably well compared to the normal or lognormal distribution. The distribution parameters were chosen so that the mean value and standard deviation for the resulting Type II distribution were close to the statistical estimates.

In general, the distribution parameters of the three random variables may vary depending on the period of the structure. However, it was assumed in Collins et al. that the distribution parameters for the bias factor for interstory drift were independent of period to simplify the calculations. The same assumption is used here. This allows us to calculate design factors for a wide range of periods. The distribution parameters for the site soil factor depend on two periods (the short-period range, $T<0.4$, and the mid-period range, $T>0.4$). The distribution parameters for spectral displacement, S_d, are different for each period, and this is reflected in the probability distribution parameters presented in Collins et al.

The calculation of the design factors followed the procedure outlined in Collins et al. [1]. Table 3 shows some typical values of the design factor to be used in Eq. (3) for the asymmetric building structure considered here. The values are presented for five different shear wave velocities, and the dependence of the soil factor on level

Table 3
Values of the design factor Ω^* for predicting linear elastic drift response at one target probability of exceedance. The dependence of the soil factor on level of input motion is considered

Period (s)	Design factor for various values of site shear wave velocity, v*				
	v=1620	v=1050	v=540	v=450	v=290
	Target exceedance probability (p_t)=50% in 50 years				
0.1	1.33	1.33	1.32	1.32	1.32
0.3	1.35	1.35	1.34	1.34	1.34
0.5	1.29	1.29	1.29	1.29	1.28
0.7	1.26	1.26	1.24	1.24	1.24
1.0	1.22	1.21	1.21	1.21	1.20
2.0	1.23	1.22	1.22	1.21	1.21
3.0	1.21	1.21	1.20	1.20	1.18

*The soil factor is a function of shear wave velocity v. Units for v are meters per second. Equivalent SDOF model and bias statistics for drift are based on load in the Y direction and base shear formulation.

of input motion (as discussed in Collins et al. [1]) was considered.

These results are very similar to those obtained by Collins et al. This is to be expected since the uncertainty in the bias factor in the present analysis is comparable to that in the original study by Collins, and the parameters for all other random variables are identical to the previous study. It is especially important to note that the uncertainty in the bias factor is relatively small compared to the uncertainty in the soil factor and the seismic hazard. According to Collins et al. [1], the coefficient of variation in the soil factor is 40–50% and the coefficient of variation in the seismic hazard is even larger. The coefficient of variation in the bias factor is less than 15% for all cases considered here.

4.2. Calibration of design factor—nonlinear cases

The design factors for the nonlinear cases were not explicitly calculated because, as noted by Collins et al. [1], the uniform hazard spectra data for nonlinear response were not adequate to make reliable calculations. However, it is useful to compare the raw bias statistics for the nonlinear cases considered in this study with those presented in Collins et al. Table 4 provides a comparison of the bias factor statistics for the maximum interstory drift ratio of the two-story asymmetric building to the values for the two-story moment-resisting steel frame presented by Collins et al. As can been seen, the bias factors are very similar to those found by Collins. In fact, the amount of scatter, as measured by the standard deviation, is actually smaller for the present study. Since all other random variables are the same as those used in Collins, the values of the calibrated design factors for

Table 4
Comparison of the bias factor statistics for estimates of the maximum interstory drift ratio for nonlinear inelastic response

	Collins et al. [1,2] two-story MRF		Two-story asymmetric building			
			Load in X direction		Load in Y direction	
	μ_N	σ_N	μ_N	σ_N	μ_N	σ_N
Virtual work	1.03	0.13	0.84	0.091	0.86	0.090
Base shear	0.96	0.14	0.76	0.093	0.76	0.071

the 3D asymmetric structure, if calculated, should be comparable to those for the 2D structure.

4.3. Sensitivity analysis

The results presented above suggest that torsional effects can be considered in the design procedure without significantly affecting the procedure itself or the design factors used in the design equations. However, the simple two-story model considered here is just one building, and there is no guarantee that other building configurations will give similar results. Intuitively, one might argue that more complex building configurations will be more difficult to represent using an SDOF model, and hence the bias statistics might be significantly different and exhibit more scatter. Furthermore, issues such as flexible diaphragms and lateral–torsional coupling might lead to more scatter. One way to address this issue is to do more analyses of complex buildings. An alternative way is to do a sensitivity study which looks at the fluctuations of the design factor, Ω^*, with bias factor statistics. The sensitivity study option was chosen for this investigation, and the results are summarized below.

For the sensitivity analysis, values of the design factor Ω^* used in Eq. (3) were calculated for various mean and standard deviation values for the bias factor for drift. Specifically, five mean values (0.7, 0.8, 0.9, 1.0, 1.1) and six standard deviations (0.05, 0.1, 0.15, 0.2, 0.3, 0.4) were considered. The dependence of the soil factor on level of input motion was neglected in the calculations to eliminate a possible source of design factor variation. Fig. 4 shows the results for three different periods of the equivalent SDOF model (T^*=0.3, 1.0, 3.0). In these figures, the design factor Ω^* is multiplied by the mean value being considered in order to distinguish the curves. If the design factors alone were plotted, all of the curves would plot on top of each other, indicating that the design factor is essentially the same for all of the mean values considered. As expected, there is a slight increase in the design factor as the standard deviation increases. These curves clearly show that the design factors are not

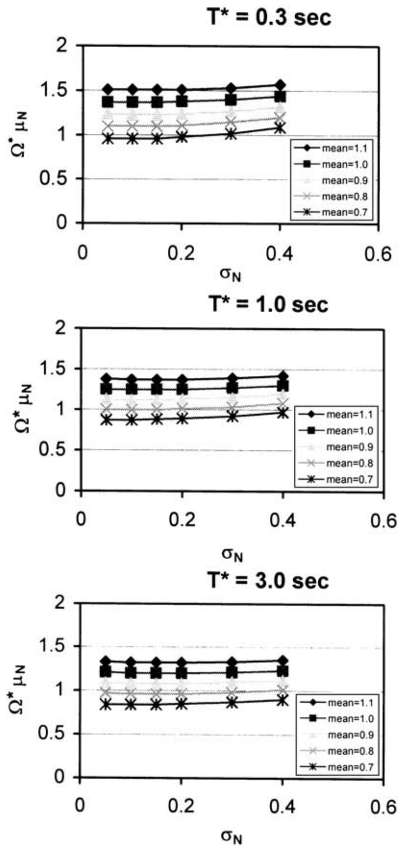

Fig. 4. Plot of design factor Ω^* showing its variation for various values of mean value and standard deviation of the bias factor for drift.

sensitive to the mean values and standard deviation of the equivalent SDOF model over a wide range of μ and σ values. Therefore, even if more complex configurations were considered and the bias factor statistics changed significantly, the end effect on the design factors might be very small or negligible.

5. Summary and conclusions

In this paper, the reliability-based, performance-based seismic design procedure proposed by Collins et al. [1,2] has been expanded to explicitly account for the torsional behavior of asymmetric building structures. The only required change is to use three-dimensional static push-over analyses to calibrate the parameters of the equivalent SDOF model. The format of the design-checking equations does not change. The results suggest that considering torsional effects in this manner does not significantly increase the uncertainty in the equivalent SDOF approach. Therefore, the design factors are comparable to those determined previously. Furthermore, it appears that the design factors are not sensitive to changes in the bias factor statistics used to quantify the approximate nature of the equivalent SDOF method. This is because the other two sources of uncertainty considered (seismic hazard and site soil effects) dominate the uncertainty analysis and control the values of the design factors. However, it must be kept in mind that these conclusions are preliminary as they are based on a very simple analytical model using some restrictive assumptions. More work is needed to consider more realistic building configurations with other torsional-to-lateral frequency ratios and the possible impact of flexible floor diaphragms.

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