11 WCEE

Copyright © 1996 Elsevier Science Ltd
Paper No. 612. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

AN ALTERNATIVE SEISMIC DESIGN PROCEDURE FOR STANDARD BUILDINGS

K.R. COLLINS[1], Y.K. WEN[2], AND D.A. FOUTCH[2]

[1] Department of Civil and Environmental Engineering, University of Michigan, 2374 G.G. Brown Building, 2350 Hayward, Ann Arbor, Michigan (USA) 48109-2125

[2] Department of Civil Engineering, University of Illinois at Urbana-Champaign, 3129 Newmark Civil Engineering Laboratory, 205 North Mathews Avenue, Urbana, Illinois (USA) 61801-2397

ABSTRACT

The seismic design provisions of many building codes specify ground motion parameters and provide simple formulas to determine the distribution of lateral forces for which a structure should be designed. Although such provisions are very simple to use, they oversimplify a complex problem and are based on many implicit assumptions which many designers may not appreciate. Furthermore, the reliability of the final design is not easily determined when these provisions are used. In this paper, a reliability-based seismic design procedure is described which attempts to address some of the shortcomings in current seismic design procedures. The key features of the proposed procedure include the following: (1) Two levels of design corresponding to two structural limit states are used; (2) Uniform hazard spectra for both linear elastic and nonlinear inelastic response are used to provide probabilistic information on seismic demand; (3) Single-degree-of-freedom models are used to evaluate the performance of the structure in terms of the probability of exceeding displacement-based limit state criteria; (4) Linear and nonlinear static push-over analyses are used; (5) A recently-proposed methodology for quantifying site soil effects is incorporated into the procedure; and (6) Deterministic design-checking equations are used to determine compliance with code requirements for global ductility and interstory drift.

KEYWORDS

Reliability-Based Design; Performance-Based Design; Seismic Design; Uniform Hazard Spectra; Seismic Design Codes; Probability; Simulation

INTRODUCTION

The seismic design provisions of most building codes in the United States specify ground motion parameters for various regions of the country and provide simple formulas to determine the distribution of lateral forces to be considered in design. The ground motion parameters are often intended to be representative of a "design earthquake" with some return period that reflects the relative likelihood of the occurrence of that level of ground motion. The simple formulas used to determine the design forces incorporate various "factors" to account for inelastic behavior of the structure, the relative importance of the structure, site soil effects on the free-field ground motion, etc. These formulas, and the seismic design procedures in which they are used, are relatively easy to apply and are based on years of experience and observations of structural performance in past earthquakes (ATC-34, 1994).

Despite their simplicity and ease of use, current seismic design procedures oversimplify a complex problem (ATC-34, 1994). There are many implicit assumptions which may not be understood or appreciated by designers. Also, it is difficult to quantify the reliability of the final design. Although the design earthquake is typically defined in probabilistic terms, the probabilistic definition of the design earthquake provides little risk information on the response (i.e., performance) of the structure designed for that earthquake (Wen, 1995).

Recently, many studies have been carried out to investigate possible improvements to future seismic design provisions in the United States. Suggested modifications include considering two or more levels of earthquake excitation (Bertero *et al.*, 1991; Bertero and Bertero, 1992; Osteraas and Krawinkler, 1990), incorporating structural reliability concepts (ATC-34, 1994), using nonlinear static push-over analyses (ATC-34, 1994; Osteraas and Krawinkler, 1990; ATC-33.03, 1995), establishing performance-based design criteria (SEAOC, 1995), and improving the techniques used to account for site soil effects on free-field ground motion (Borcherdt, 1994).

The seismic design procedure described herein is a reliability-based design procedure which incorporates some of the suggestions mentioned in the references cited above. The procedure uses uniform hazard spectra and "equivalent" single-degree-of-freedom (SDOF) models to evaluate the performance of a multi-degree-of-freedom (MDOF) structure. Performance is measured in terms of the probability of exceeding displacement-based limit state criteria. The parameters for the equivalent SDOF models are derived from the results of linear and nonlinear static push-over analyses of the structure. These parameters are then used in deterministic design-checking equations to determine if the structure satisfies the target performance criteria. Design factors (analogous to load and resistance factors) are used in the deterministic design-checking equations to account for the inherent uncertainty in the ground motion at a site, the uncertainty in predicting site soil effects, and the approximate nature of the simplified models of the structure. The sections to follow provide a basic overview of the proposed procedure. Additional details are provided in the report by Collins *et al.* (1995).

UNIFORM HAZARD SPECTRA

Figures 1 and 2 show examples of uniform hazard spectra calculated for a site near Los Angeles, California. These curves were generated by simulating earthquakes based on the seismicity of the region within 150 km of the site, computing response spectra for each simulated earthquake, and evaluating the statistics of the computed responses at each of seven periods between 0.1 second and 3.0 seconds. The uniform hazard curves show the variation of a force coefficient with period. The elastic force coefficient, C_e, is the maximum spring force developed in a linear elastic SDOF oscillator normalized by the weight of the attached mass. The yield force coefficient, C_y, is the normalized spring force at "yield", i.e., when the yield displacement is reached in a spring characterized by a bilinear inelastic force-displacement relationship. In Figure 1, the probability associated with each curve refers to the probability of exceeding the indicated value of C_e. In Figure 2, the probability for each curve refers to the probability of exceeding the target displacement ductility of 6 at the indicated value of C_y. For all curves in Figures 1 and 2, damping is assumed to be 5% of critical damping. Also, the probabilities are intended to reflect both the likelihood of earthquake occurrence as well as the potential severity of the earthquake ground motion.

EQUIVALENT SYSTEM METHODOLOGY

The equivalent system methodology provides a means of relating the response of a SDOF oscillator (obtained from the uniform hazard spectra) to the response of a MDOF structure. The starting point is the equation of motion of a two-dimensional MDOF cantilever-type structure subjected to horizontal base motion. The equation can be written as

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+\{R\}=-[M]\{1\}\ddot{u}_g \tag{1}$$

where [M] is the mass matrix (assumed to be diagonal), {u}={u(t)} is the vector of lateral displacements (relative to the ground) at each floor (one displacement per floor), [C] is the damping matrix, {R}={R(t)} is the restoring force vector, {1} is the vector with all components equal to unity, and $u_g=u_g(t)$ is the ground displacement. Dots denote time derivatives. Using information obtained from static push-over analyses, the above equation is reduced to a single equation describing the displacement response of the roof, D=D(t), relative to the base. The resulting equation is of the form (Collins *et al.*, 1995)

$$\ddot{D}+2\zeta\omega^*\dot{D}+(\omega^*)^2G(D)=-P^*\ddot{u}_g \tag{2}$$

where P^* is analogous to a participation factor, ω^* is the effective frequency of the "equivalent" system, ζ is the equivalent damping ratio, and G(D) is a function describing the relationship between the base shear and roof displacement. Figure 3 provides an illustration of how the function G(D) is determined from a nonlinear push-over analysis.

Equation (2) provides information on roof displacement. However, for design purposes, interstory drift (or interstory drift ratio) is often used as a measure of structural deformation. An estimate of the maximum interstory drift ratio can be obtained using

$$(\Delta_L)_{max}=\beta_{LG}\frac{D_{max}}{H} \tag{3}$$

where $(\Delta_L)_{max}$ is the maximum interstory drift ratio (i.e., interstory drift divided by story height), H is the total height of the structure, and β_{LG} is a conversion factor which relates the global drift ratio (D_{max}/H) to the maximum interstory drift ratio. This factor is derived using a displacement profile obtained from the static push-over analysis of the structure.

EVALUATION OF STRUCTURAL PERFORMANCE

The proposed seismic design procedure uses the equivalent system methodology and uniform hazard spectra to evaluate structural performance for two levels of earthquake excitation The pertinent performance criteria are expressed in general probabilistic terms, and deterministic design-checking equations are derived based on these probabilistic performance criteria. In the deterministic equations, the effect of site soils is represented by the soil factor, F, which is calculated using the following formula suggested by Borcherdt (1994):

$$F=\left(\frac{v_{ref}}{v_{site}}\right)^{m(T,A_a)} \tag{4}$$

In Equation (4), v_{ref} and v_{site} are shear wave velocities and the exponent m is a function of the structural period T and the ground motion intensity defined by the parameter A_a.

Serviceability Limit State

The first level of design is associated with a serviceability limit state, and the performance goal is to ensure elastic (or nearly elastic) behavior of the structure during "small to moderate" earthquakes. It is assumed that the seismic design code specifies a limiting interstory drift ratio Δ^S_{CODE} and a target probability of exceedance p_t. If the maximum interstory drift ratio in the structure is Δ, then the probabilistic performance criterion for interstory drift can be expressed as

$$P\left(\Delta>\Delta^S_{CODE}\right)\le p_t \tag{5}$$

The deterministic design-checking equation derived from this criterion is of the form (Collins *et al.*, 1995)

$$S_d \le \frac{H\Delta^S_{CODE}}{n\Omega_t FP^* \beta_{LG}} \tag{6}$$

where S_d is the elastic spectral displacement obtained from the uniform hazard curve corresponding to the target probability p_t, F is the site soil factor, n is a bias factor which reflects the discrepancy between the response of the equivalent system model and a MDOF analysis model of the structure, H is the total structure height, β_{LG} and P* are equivalent system parameters, and Ω_t is a design factor which accounts for the fact that the soil factor, the bias factor, and the elastic spectral displacement are all random variables.

Table 1 presents values of the design factor Ω_t for a target probability of exceedance of 10% in 50 years. The dependence of the soil factor F on A_a is neglected. For this assumption, the value of the design factor is independent of site soil conditions. However, the values do depend on the period of the structure. This is a consequence of the relative dispersion in the seismic hazard at each period and the differences in the level of uncertainty in the soil factor for different period ranges.

Ultimate Limit State

The second level of design is associated with an ultimate limit state, and the performance objective is to control the level of nonlinear response behavior of the structure. It is assumed that the performance criteria for this limit state are expressed in terms of the probability of exceeding threshold values of interstory drift ratio and global ductility.

Global ductility, as used herein, is defined as

$$\mu_{MDOF} \equiv \frac{D_{max}}{D_y} \tag{7}$$

where D_{max} is the maximum lateral roof displacement of the MDOF structure and D_y is the global yield displacement of the structure obtained from a static pushover analysis. (See Figure 3.) The performance criterion for global ductility is assumed to be

$$P(\mu_{MDOF} @ D_y > \mu_{CODE}) \le p_t \tag{8}$$

where μ_{CODE} is a code-specified limiting ductility and the notation "@ D_y" emphasizes the dependence of the nonlinear response on the global yield displacement. The deterministic design equation corresponding to this requirement is (Collins *et al.*, 1995)

$$D_y \ge \Omega_t^F P^* F g \left(\frac{T^*}{2\pi}\right)^2 C_y(\mu_t) \tag{9a}$$

$$\mu_t = \frac{\mu_{CODE}}{\Omega_t^{\mu} n_U^{DISP}} \tag{9b}$$

where Ω_t^F is the design factor accounting for the variability in the soil factor, Ω_t^{μ} and n_U^{DISP} are the design factor and bias factor, respectively, accounting for the approximate nature of the equivalent system model of the structure, P* is the ground motion scale factor for the equivalent system model, F is the soil factor, g is the acceleration of gravity, T* is the period of the equivalent system model for nonlinear response, and $C_y(\mu_t)$ is the yield force coefficient at T=T* obtained from the uniform hazard spectrum curve for nonlinear response corresponding to μ_t and p_t.

To control the maximum interstory drift, the performance criterion can be stated as

$$P(\Delta @ D_y > \Delta^U_{CODE}) \le p_t \tag{10}$$

where Δ is the maximum interstory drift ratio among all stories in the actual structure and Δ^{U}_{CODE} is the limiting maximum interstory drift ratio defined by the code for severe earthquake conditions. The deterministic design-checking equation corresponding to Equation (10) is (Collins *et al.*, 1995)

$$\Delta^{U}_{CODE} \geq \mu' \frac{\beta_{LG}}{H} \Omega^{\Delta}_{t} n^{DRIFT}_{U} D_{y} \tag{11}$$

where Ω_t^{Δ} and n_U^{DRIFT} are the design factor and bias factor, respectively, accounting for the approximate nature of the equivalent system model, β_{LG} is the factor used to convert the global drift ratio to interstory drift ratio in the equivalent system methodology, H is the total structure height, and μ' is a ductility corresponding to D_y.

Table 2 presents values of the design factor Ω_t^{F} used in Equation (9a) for a target probability of 10% in 250 years. This factor is period dependent, but it appears to remain relatively constant for ductilities between 2 and 6. Additional studies (Collins *et al.*, 1995) also suggest that the factor is not a strong function of the target exceedance probability for probabilities between 10% in 50 years and 10% in 250 years.

Additional studies are still needed to better quantify values of the design factors Ω_t^{μ} and Ω_t^{Δ} used in Equations (9b) and (11). However, preliminary studies suggest that these factors can vary from 1.1 to 1.4.

DUAL-LEVEL SEISMIC DESIGN PROCEDURE

The dual-level design procedure uses the equations presented above to develop a preliminary structural design and verify that the design satisfies the probabilistic performance criteria established by the code-writing organizations. A detailed, step-by-step, discussion of the design procedure is presented in Collins *et al.* (1995). A brief overview of the steps at each level of design is presented below.

Serviceability Limit State

The procedure for the serviceability limit state attempts to focus simultaneously on the relationship between stiffness and strength in satisfying the serviceability limit state criteria. First, a preliminary structural design is obtained using the uniform hazard spectra for linear elastic response, pertinent equations related to the equivalent system methodology, and the code-specified limiting interstory drift ratio Δ^{S}_{CODE}. Then, once a preliminary design is obtained, Equation (6) is used to determine if the structure is stiff enough to satisfy the performance criteria for interstory drift. Also, a linear elastic push-over analysis is performed to verify that the structure remains elastic for a base shear force derived from Equation (6). Once it is shown that the structure satisfies the strength and stiffness criteria, the structural design is evaluated using the criteria for the ultimate limit state as described in the next section.

Ultimate Limit State

For the ultimate limit state evaluation, the structural design which satisfies the serviceability criterion is checked to ensure that the structure has adequate strength and stiffness to limit the nonlinear inelastic behavior which is expected to occur during a large earthquake. The first step in the evaluation process is to develop a structural model which can be used in a nonlinear inelastic static push-over analysis. The model should use best-estimate values of material yield strengths instead of nominal design values. Then, a nonlinear static push-over analysis is performed. A plot of base shear versus roof displacement is generated as shown in Figure 3. Using this plot and other information obtained from the push-over analysis, the equivalent system parameters are calculated. A target ductility, which is a function of the code-specified limiting ductility, is calculated using Equation (9b). Then, using this target ductility and the uniform hazard spectra for nonlinear response, Equation (9a) is used to verify that the global ductility performance criterion is satisfied. If it is not, then the structure must be strengthened and possibly stiffened. If Equation (9a) is satisfied, then the ductility μ' corresponding to the actual global yield displacement D_y is determined and used in Equation (11) to see if the performance criterion for interstory drift is satisfied. If it is not, then the structure must be redesigned. If both Equations (9a) and (11) are satisfied, then the structural design satisfies the performance criteria for global ductility and interstory drift.

SUMMARY AND CONCLUSIONS

The dual-level performance-based seismic design procedure described above is significantly different from current design procedures in three ways. First, since it is difficult to achieve multiple performance objectives using a single design earthquake, two levels of ground motion are considered. However, unlike most current design codes, probabilities are associated with the performance of the structure and not with the design ground motion. Second, nonlinear behavior is treated explicitly (albeit approximately) using information from static nonlinear push-over analyses. Such analyses should help designers to better appreciate the nonlinear behavior which is expected to occur during large earthquakes and to identify critical regions of the structure requiring careful detailing and design. Third, the procedure explicitly accounts for the approximate nature of the simple analysis models, the uncertainty in local seismic hazard, and the uncertainty in the methods used to estimate the effect of site soils. Such uncertainties and approximations must be accounted for in a rational manner to achieve a reliable structural design.

REFERENCES

Applied Technology Council (ATC-34, 1994), *A Critical Review of Current Approaches to Earthquake-Resistant Design*, (Draft).

Applied Technology Council (ATC-33.03, 1995) Commentary, Nonlinear Analysis Procedure, (75% Complete Draft, Volume II).

Bertero, R.D. and V.V. Bertero (1992), Tall Reinforced Concrete Buildings: Conceptual Earthquake-Resistant Design Methodology, EERC Report UCB/EERC-92/16, University of California-Berkeley.

Bertero, V.V., J.C. Anderson, H. Krawinkler, E. Miranda, and CUREe/Kajima Research Teams (1991), Design Guidelines for Ductility and Drift Limits: Review of State-of-the-Practice and State-of-the-Art in Ductility and Drift-Based Earthquake-Resistant Design of Buildings, EERC Report UCB/EERC-91/15, University of California-Berkeley.

Borcherdt, R.D. (1994), Estimates of Site-Dependent Response Spectra for Design (Methodology and Justification), Earthquake Spectra, 10, 617-653.

Collins, K.R., Y.K. Wen, and D.A. Foutch (1995), Investigation of Alternative Seismic Design Procedures for Standard Buildings, Structural Research Series No. 600, University of Illinois at Urbana-Champaign.

Osteraas, J.D. and H. Krawinkler (1990), Strength and Ductility Considerations in Seismic Design, Report No. 90, Stanford University.

Structural Engineers Association of California (SEAOC, 1995), Performance Based Seismic Engineering of Buildings, Volumes I and II.

Wen, Y.K. (1995), Building Reliability and Code Calibration, *Earthquake Spectra*, 11, 269-296.

Table 1. Values of the design factor Ω_t used in design for serviceability limit state. (Target exceedance probability = 10% in 50 years.)

Period (sec)	Design Value of C_e	Design Factor Ω_t for v_{ref} = 540 m/s					
		v=1620 m/s	v=1050 m/s	v=540 m/s	v=450 m/s	v=290 m/s	v=150 m/s
0.1	0.46	1.29	1.29	1.29	1.29	1.29	1.29
0.3	0.91	1.31	1.31	1.31	1.31	1.31	1.31
0.5	0.71	1.26	1.26	1.26	1.26	1.26	1.26
0.7	0.62	1.21	1.21	1.21	1.21	1.21	1.21
1.0	0.49	1.19	1.19	1.19	1.19	1.19	1.19
2.0	0.27	1.15	1.15	1.15	1.15	1.15	1.15
3.0	0.19	1.15	1.15	1.15	1.15	1.15	1.15

Table 2. Values of the design factor Ω_t^F for various target ductilities. (Target exceedance probability = 10% in 250 years.)

Period (sec)	μ_t=2		μ_t=3		μ_t=4		μ_t=6	
	Value of $C_y(\mu_t=2)$	Ω_t^F	Value of $C_y(\mu_t=3)$	Ω_t^F	Value of $C_y(\mu_t=4)$	Ω_t^F	Value of $C_y(\mu_t=6)$	Ω_t^F
0.1	0.53	1.43	0.47	1.43	0.45	1.42	0.39	1.43
0.3	0.68	1.46	0.53	1.42	0.44	1.40	0.34	1.40
0.5	0.54	1.32	0.42	1.28	0.34	1.27	0.25	1.27
0.7	0.46	1.29	0.33	1.30	0.27	1.27	0.20	1.27
1.0	0.37	1.27	0.29	1.24	0.22	1.25	0.13	1.30
2.0	0.20	1.30	0.13	1.27	0.10	1.28	0.070	1.31
3.0	0.14	1.25	0.10	1.24	0.083	1.21	0.047	1.27

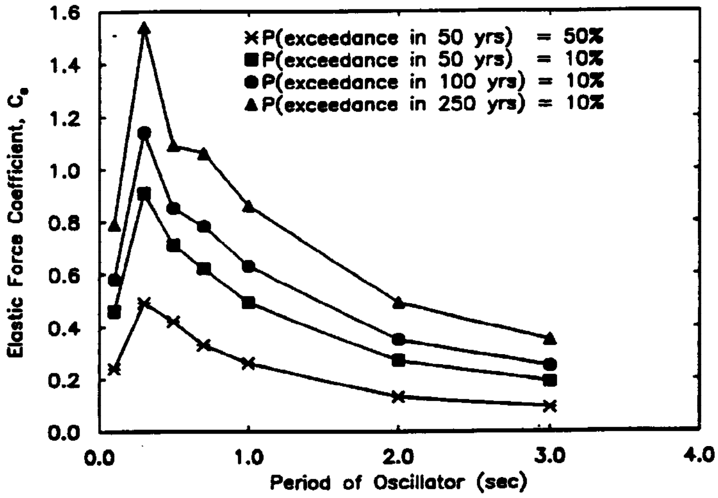

Fig. 1. Uniform hazard spectra for linear elastic response.

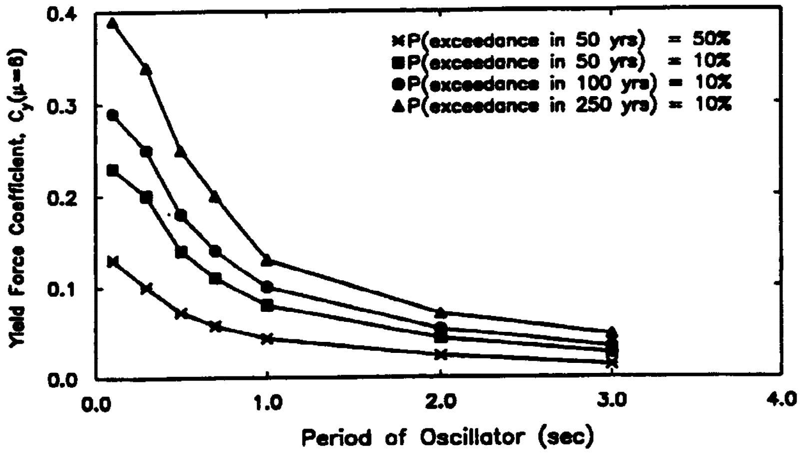

Fig. 2. Uniform hazard spectra for nonlinear inelastic response.

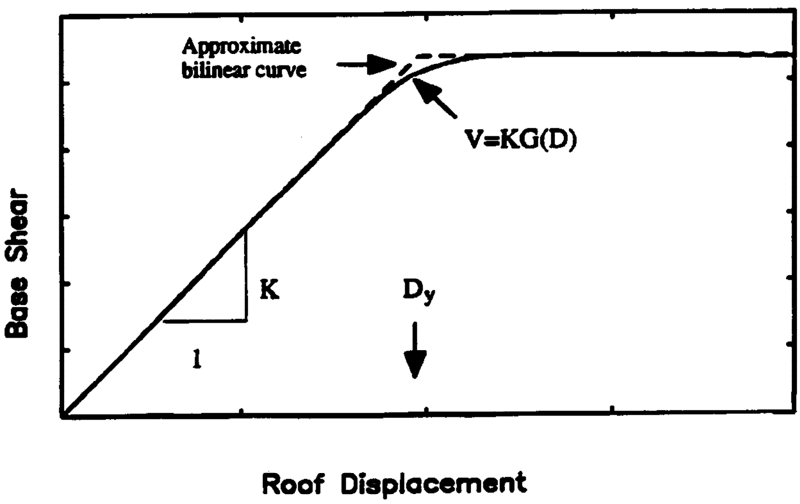

Fig. 3. Typical results from nonlinear static push-over analysis.