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Evaluation of seismic reliability of steel buildings designed according to current code procedures

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ABSTRACT: The performance and safety of buildings designed according to the recently proposed procedures in the U.S., i.e., UBC and SEAOC is evaluated. It is based on most recent analytical and experimental results of studies of structural and nonstructural components, earthquake occurrence and ground motion data and analysis, and state-of-the-art reliability methods. The study concentrates on low- to medium-rise steel buildings. Both time history and random vibration methods are used for the response analysis. Limit states considered include maximum story drift, damage to nonstructural components and content, and low cycle fatigue damage to members and connections. The risks implied in the current procedure, for example those based on various R_w factors for different structural types, are calculated and their consistency examined.

1 INTRODUCTION

The commonly accepted philosophy in design of a building under seismic loads is to ensure that it will withstand a minor or moderate earthquake without structural damage and survive a severe one without collapse. Successful implementation of this design philosophy, however, requires consideration of the large uncertainties normally associated with the seismic excitation and the considerable variabilities in structural resistance because of differences in structural type and design and effect of nonstructural components. This has not yet been fully accounted for in current practice in building design, although the need for consideration of the uncertainty involved has long been recognized.

The objective of this study, therefore, is to evaluate the reliability of buildings designed according to the recently proposed and adopted procedures; namely, the provisions recommended by the Structural Engineering Association of California (SEAOC) and Uniform Building Code (UBC). The emphasis is on the realistic modeling of building systems, uncertainty in the excitation, and nonlinear response. This paper concentrates on seismic risk analysis, modeling of ground motion, random vibration analysis and limit state risk evaluation while a companion paper (Foutch, et al., 1992) addresses in more detail building design, time history response and damage analysis.

2 SELECTION OF SITE AND RISK ANALYSIS

Two sites are considered for the study of building response, both in Southern California.One of these is

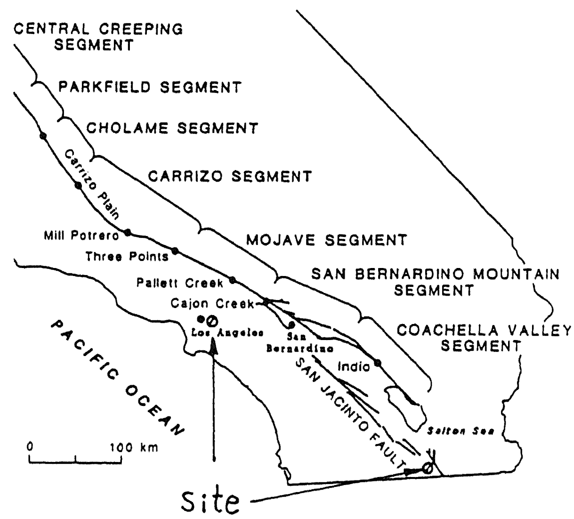

Figure 1. Segments of Central and Southern Andreas fault.

at Imperial Valley 5km from the Imperial Fault and the other one is at Santa Monica Boulevard, 60km from the Mojave Segment of the Southern San Andreas fault (Fig. 1). The potential future earthquake that present a threat to the sites are characterized as either characteristic or non-characteristic. The former are major seismic events which occur along the major fault and with relatively better understood magnitude and recurrence time behavior (U.S.G.S. Working Group Report, 1988), therefore, treated as a renewal process. The latter are local events that their occurrences collectively can

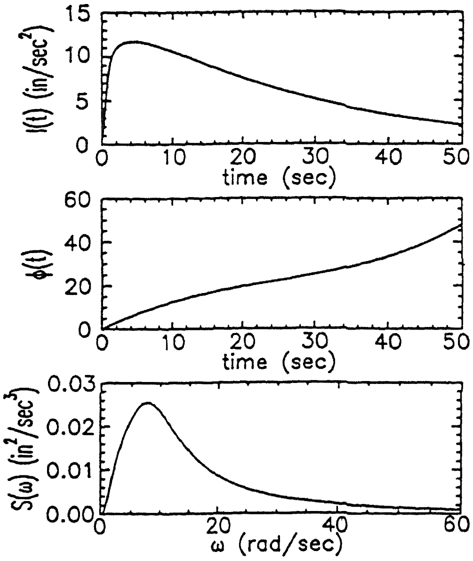

Figure 2 Ground motion model functions for the Los Angeles site due to a characteristic earthquake (M=7.5, R=60 km)

be treated as a Poisson process (Algermissen, et al., 1982; Cornell and Winterstein, 1988). Besides the recurrence time, the major parameters of the characteristic earthquake for risk analysis are magnitude (M), epicenter distance to the site (R) and attenuation, whereas those for non-characteristic earthquakes are local (MMI) intensity, I, and duration. The duration t_D is defined as the significant duration associated with the strong phase of the ground motion (Trifunac and Brady, 1975). It depends on M and R for characteristic earthquakes and on I for non-characteristic earthquakes.

3 MODELING OF GROUND MOTION

The ground motion model is based on that of a nonstationary random process whose intensity and frequency content vary with time (Yeh and Wen, 1990).

$$a(t)=I(t)\zeta[\phi(t)] \qquad [1]$$

in which $\zeta(\phi)$ is a zero mean, unit variance, stationary white noise filtered through a Clough-Penzien type linear filter. I(t) is the intensity envelope function and ϕ(t) is a frequency modulation function. While I(t) and ϕ(t) control the nonstationarity, the filter parameters determine the frequency content of the ground motion. It has been shown that the time dependent (instantaneous) power spectral density of ζ(t) at time t is

$$S_{aa}(t,\omega)=\frac{1}{\phi'(t)}S_{CP}[\frac{\omega}{\phi'(t)}] \qquad [2]$$

in which S_{CP} = Clough and Penzien (CP) spectral density. This model allows straightforward identification of parameters from actual ground accelerograms, computer simulation of the ground motion for time history response analysis, and analytical solution of inelastic structure response by the method of random vibration (Wen, 1989). For the site where actual earthquake ground motion records are available (i.e., Imperial Valley site), model parameters are estimated directly from the record and used to predict structural response to future earthquakes. For the site where no such records are available, a procedure (Eliopoulos and Wen, 1991) has been established to determine the model parameters as functions of those of the source, i.e., magnitude, epicentral distance, etc. based on empirical results given in Trifunac and Lee (1984). For characteristic earthquakes this model gives the Fourier amplitude spectrum as a function of the source parameters in which a frequency (or period) dependent attenuation law and local geology (site) condition are considered. The Arias intensity can be evaluated and used as a scaling factor for the intensity and the frequency content (parameters in the C-P filter) can be determined from the Fourier amplitude spectrum. For non-characteristic earthquakes, a similar procedure is used to determine the ground motion parameters as a function of MMI and the site condition. Also, for sites close to the fault, the important directivity effect of the rupture surface is considered in the ground motion model which is known to affect significantly the frequency content and duration of the ground motion. Figure 2, for example, shows identified intensity, frequency modulation, and spectral density functions of the ground motion model for the Los Angeles site due to characteristic earthquake. Note that I(t) varies according to the random duration and attenuation. The energy and zero crossing functions calculated from the San Fernando earthquake of 1971 and Whittier Narrow earthuake of 1987 at the site are used to identify the shape of I(t) and ϕ(t) for the noncharacteristic earthquakes. Figure 3 shows sample ground motion time histories of characteristic earthquakes at the two sites and the noncharacteristic earthquake at the Los Angeles site generated by the model. Again, these are sample histories, there are large variations in intensity and duration from sample to sample. Note that for the Imperial Valley site, the rupture propagation toward the site is assumed giving rise to long duration pulses which are known to be most damaging when the structure becomes inelastic.

4 BUILDING DESIGN

Six low-rise steel building types are designed according UBC; namely, (i) ordinary moment-resisting space frame (OMRSF), (ii) special moment-resisting space frame (SMRSF), (iii) concentric braced frame (CBF), (iv) eccentric braced frame

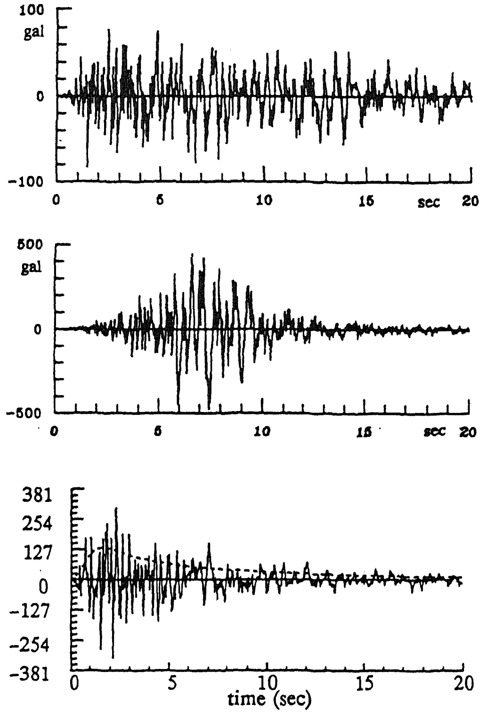

Figure 3 Simulated ground motion time histories: characteristic earthquake at the L. A. site (top), characteristic earthquake at the Imperial Valley site (middle), and non-characteristic earthquake at the L. A. site (bottom).

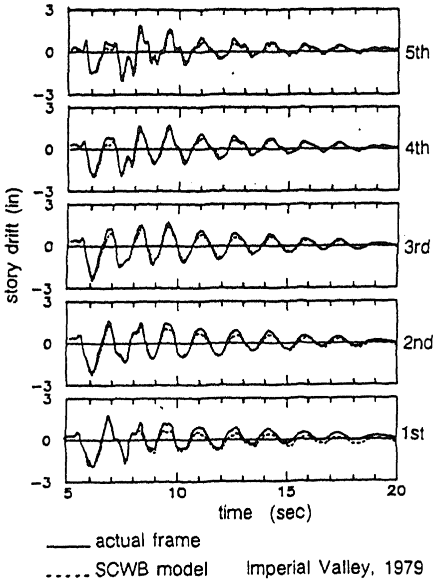

Figure 4 Comparison of story drifts between actual five story, three bay frame (analyzed using DRAIN-2DX) and SCWB model.

(EBF), (v) dual system with CBF, D/CBF, and (vi) dual system with EBF, D/EBF. Some details of the design are given in the companion paper. The R_w value varies from 6 (OMRSF) to 12 (SMRSF).

5 RESPONSE AND DAMAGE ANALYSIS

Given the occurrence of an earthquake, the response of the building is calculated by both time history and random vibration methods. The foregoing ground motion model provides the ground excitation either in the form of time histories or nonstationary random processes in which the effect of the source parameters and their uncertainties have been properly accounted for. The responses of interests are: (1) story drifts, (2) damage to nonstructural elements, (3) energy dissipation demand, and (4) damage index. In the time history method, the well known finite element program DRAIN 2DX is modified and used and the statistics of the above responses are obtained for a large number of ground motions (see Foutch, et al., 1982, for more details).

In the random vibration analysis, the time domain approach for an inelastic system (Wen, 1989) is used. It gives response statistics of interest such as maximum interstory displacement and hysteretic energy dissipation. For the SMRSF frame a strong-column weak-beam (SCWB) model is developed which localizes inelastic behavior at the base and the floor level at the beams. It allows lateral displacement and floor rotation. The hysteretic restoring moments are described by the smooth differential equation model which allows solution by the equivalent linearization method. Comparisons between the results for different acceleration records by the SCWB and DRAIN 2DX indicate that the former reproduces well the inelastic response behavior of the SMR frame (Fig. 4). Also, the accuracy of the random vibration results are verified by comparison with simulation (Fig. 5). Details are available in Eliopoulos and Wen (1991).

6 LIMIT STATE RISK EVALUATION

The random vibration analyses of the structural response provide the conditional statistics and probability of limit states being reached given the occurrence of the earthquake and that the seismic event and ground motion parameters are known. These parameters (e.g., attenuation coefficient, duration, etc.), however, are known to have large variabilities and, therefore, are treated as random variables. They may be correlated or functionally dependent (e.g., on magnitude and epicentral distance) and often play a significant, sometimes dominant, role in the overall risk evaluation. An extensive literature survey of information on these parameters and their uncertainties (e.g., USGS Report 1982, 1988; Joyner and Boore, 1988) has been

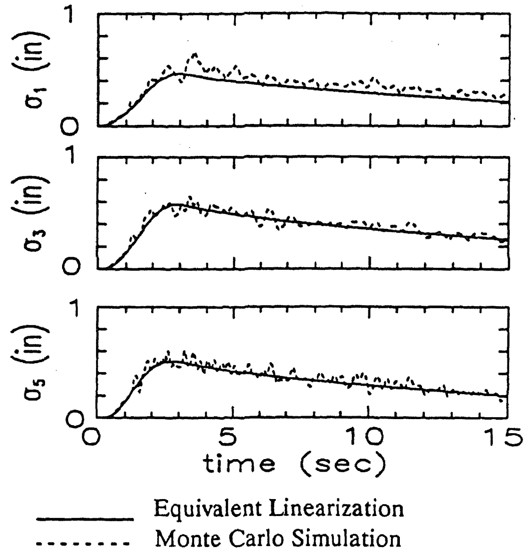

Figure 5 Root-mean-square interstory drifts of five story-three bay special moment resisting space frame

carried out, and model is developed based on the survey results. To include these parameters uncertainties into the risk analysis, a fast integration technique (Wen and Chen, 1986) based on the first order reliability method is used. In the time history/simulation approach, these parameters have been randomized according to their distributions and incorporated into the simulated ground motion time histories. A method based on a response surface technique is also being developed which is most efficient in conjunction with the time history/simulation method and when the number of parameters considered is large.

Table 1 shows the interstory drift statistics for OMRSF, SMRSF, CBF and D/CBF by the simulation method at both sites given the occurrence of an earthquake. At the Imperial Valley site, the major threat is characteristic earthquakes of magnitude 6.5. A 50% probability of fault rupture propagation toward or away from the site is assumed. The results are found to be not particularly sensitive to this assumption. At the Los Angeles site, both types of earthquakes contribute. The characteristic earthquakes, though of a larger magnitude of 7.5, contribute much less than the noncharacteristic earthquakes primarily due to the distance (60km) from the site to the Mojave segment. The random vibration/fast integration method gives comparable results. The relatively large coefficient of variation of the drift at the Los Angeles site is due to the large uncertainty in the intensity and duration of the local events. These response statistics in combination with the earthquake occurrence probability at the site are used to evaluate the risk of limit states being reached as a function of the length of the time window considered and the dormant period since the last characteristic earthquake. The last event occurred in 1979 at the Imperial Valley fault and in 1867 at the Mojave segment.

Table 2 shows the drift level (% of story height) being exceeded corresponding to various probability levels for the next 50 years at the two sites for the four buildings. Note that although both sites are in Zone 4, at equal probability levels, response (hence damage) are much higher at the Imperial Valley site. As expected, the brace frame and dual system give lower responses. The differences, however, are not as large as the different R_w values used in the design would suggest.

7 CONCLUSION AND FUTURE WORK

The reliabilities of four different types of steel building designed according to UBC in the U.S. are evaluated. The results indicate that the risks of different drift limit states show variation among the four buildings which are consistent with the design philosophy and appear to be reasonable. There is, however, a greater discrepancy in the risks and response levels for the two sites considered, both in Zone 4 of the UBC designation. The results presented and the conclusions are still tentative since the emphasis so far has been on uncertainty due to excitation. Additional uncertainties currently being investigated are those in the seismic risk and ground motion modeling as well as in the structural resistance, in particular, those due to nonstructural components (partition walls and cladding). Recent evidence indicates that cladding, etc., may increase the stiffness of the bare frame by a large factor and significantly increase its strength (e.g., Foutch, et al., 1986). Another area being investigated is calibration for more risk-consistent design. The R_w factors for different building types are being carefully examined to develop design procedures which provide a given amount of drift and damage control.

8 ACKNOWLEDGMENT

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REFERENCES

Algermissen, S. T., O. M. Perkins, P. C. Theuhaus, S. L. Hanson, & B. L. Benden 1982. Probabilistic estimates of maximum acceleration and velocity in rock in the contiguous United States. USGS open-file report 82-1033, 1982.

Cornell, C. A. & S. R. Winterstein 1988. Temporal and magnitude dependence in earthquake recurrence models. Bull. Seis. Soc. of Am., 78: 1522-1537.

Eliopoulos D. & Y. K. Wen 1991. Method of seismic reliability evaluation for moment resisting steel frames. Structural Research Series No. 562, Univ. of IL.

Foutch, D. A., S. C. Goel, & C. W. Roeder 1987. Seismic testing of a full-scale steel building - Part 1. J. of Struc. Engr., ASCE, 113, 11: 2111-2129.

Foutch, D. A., C. Y. Yu, & Y. K. Wen 1992. Reliability of steel frame buildings under seismic loads. Proc. 10WCEE.

Joyner, W. B. & D. M. Boore 1988. Measurement, characteristics, and prediction of strong ground motion. Proc. Erth. Engr. and Soil Dyn., Geotech. Div., ASCE, Utah.

Trifunac, M.O. & A. G. Brady 1975. A study of strong earthquake ground motion. Bull. Seis. Soc. of Am., 65, 3: 581-626.

Trifunac, M. O. & V. W. Lee 1989. Empirical models for scaling Fourier amplitudes spectra of strong earthquake accelerations in terms of magnitude, source to station distance, site intensity and recording site conditions. Soil Dyn. and Erth. Engr., 1, 3.

United States Department of the Interior, U.S. Geological Survey 1988. Probabilities of large earthquake occurring in California, on the San Andreas fault. Open-file report 88-398.

Wen, Y. K. 1989. Method of random vibration for inelastic structures. Appl. Mech. Rev., 42, 2.

Wen, Y. K. & H. C. Chen. On fast integration for time variant structural reliability. J. of Prob. Engr. Mech., 2, 3: 156-162.

Yeh, C-H. & Y. K. Wen 1990. Modeling of nonstationary ground motion and analysis of inelastic structural response. Struc. Safety, 8: 281-298.

Table 1. Interstory drift statistics at the Los Angeles and Imperial Valley sites given the occurrence of an earthquake.

Site	Story	OMRSF		SMRSF		CBF		D/CBF	
		μ	σ	μ	σ	μ	σ	μ	σ
Los Angeles	1	.13	.11	.15	.13	.12	.08	.10	.07
	2	.21	.17	.24	.18	.15	.10	.17	.12
	3	.23	.18	.27	.22	.18	.12	.18	.12
	4	.22	.18	.27	.22	.20	.15	.19	.14
	5	.24	.19	.28	.22	.21	.16	.17	.13
Imperial Valley	1	.74	.32	.86	.37	.73	.41	.59	.32
	2	1.00	.33	1.08	.36	.76	.34	.87	.36
	3	1.06	.34	1.19	.39	.86	.31	.92	.35
	4	1.12	.31	1.22	.37	1.08	.37	.96	.28
	5	1.28	.28	1.51	.39	1.08	.29	.88	.21

Table 2. Interstory drift (% of story height) level corresponding to an exceedance probability in 50 years.

Site	Frame	Prob (%)	Story 1	Story 2	Story 3	Story 4	Story 5
Los Angeles	OMRSF	50	.18	.29	.31	.31	.34
		25	.28	.44	.48	.47	.52
		15	.36	.56	.61	.62	.69
		10	.45	.69	.75	.77	.87
		5	.66	1.01	1.11	1.18	1.34
	SMRSF	50	.22	.34	.39	.39	.39
		25	.35	.52	.60	.59	.59
		15	.48	.70	.80	.79	.78
		10	.63	.89	1.02	1.01	1.00
		5	1.00	1.38	1.60	1.59	1.54
	CBF	50	.17	.21	.25	.29	.30
		25	.24	.30	.36	.43	.45
		15	.31	.38	.46	.56	.61
		10	.39	.47	.57	.70	.78
		5	.58	.69	.84	1.06	1.22
	D/CBF	50	.14	.24	.25	.26	.25
		25	.21	.34	.36	.39	.36
		15	.27	.44	.45	.50	.48
		10	.33	.54	.55	.62	.60
		5	.48	.79	.81	.93	.91
Imperial Valley	OMRSF	50	.64	.90	.96	1.03	1.20
		25	.89	1.15	1.22	1.28	1.42
		15	1.04	1.31	1.39	1.43	1.55
		10	1.16	1.43	1.51	1.54	1.65
		5	1.36	1.63	1.71	1.72	1.81
	SMRSF	50	.76	.97	1.07	1.10	1.40
		25	1.04	1.25	1.36	1.38	1.70
		15	1.21	1.43	1.55	1.56	1.89
		10	1.34	1.56	1.69	1.70	2.03
		5	1.55	1.78	1.93	1.92	2.26
	CBF	50	.59	.64	.77	.98	1.00
		25	.87	.90	1.01	1.26	1.23
		15	1.09	1.08	1.16	1.44	1.37
		10	1.26	1.21	1.27	1.57	1.47
		5	1.54	1.44	1.45	1.78	1.63
	D/CBF	50	.47	.75	.82	.88	.82
		25	.70	1.02	1.09	1.09	.98
		15	.88	1.21	1.26	1.23	1.08
		10	1.01	1.36	1.39	1.33	1.16
		5	1.23	1.59	1.60	1.49	1.28