RELIABILITY OF NONLINEAR STATIC METHODS FOR THE SEISMIC PERFORMANCE PREDICTION OF STEEL FRAME BUILDINGS

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SUMMARY

Nonlinear static "pushover" methods of analysis are often used within a performance-based framework to evaluate existing buildings. Acceptable building performance is typically defined by a family of structural performance and seismic hazard level pairs. This paper summarizes the results of current research, sponsored by the SAC Steel Project, on the reliability of nonlinear static methods for predicting the seismic performance of steel moment frame buildings.

As part of previous SAC studies, three steel moment frame buildings (3, 9, and 20-story), located in Los Angeles, were designed using Pre-Northridge earthquake connection details. Two DRAIN-2D models (M1 and M2) were created for each building. Model M1 is a centerline-to-centerline model, while model M2 explicitly accounts for the strength and stiffness of the panel zone and represents the more accurate model. Nonlinear dynamic time history analyses were performed for each building model using a total of 60 earthquake ground motions with seismic hazard levels having a 2%, 10%, and 50% probability of exceedance in 50 years.

In the current study, nonlinear static "pushover" analyses of the buildings were performed with the same models and ground motions used in the nonlinear time history analyses. The Coefficient Method, Capacity Spectrum Method, Equivalent System Method were used to calculate building performance response quantities. The maximum roof displacement and maximum interstory drift response for models M1 and M2 were compared with the model M2 results from the nonlinear dynamic time history analyses.

INTRODUCTION

Over the past decade, the trend in building design in seismic regions has been towards what has been commonly referred to as Performance-Based Seismic Design [OES, 1995]. The objective of this building design method is to accurately predict, in definable terms, the performance of the building during any intensity of earthquake ground motion that may occur at the building site over the lifetime, or design life, of the building. Definable performance can be accomplished by designing the building to meet a wide range of Performance Objectives. A single performance objective consists of a level of performance in terms of damage, coupled with a level of earthquake hazard. As an example, a building may be designed to be at the brink of collapse during an earthquake that is expected to occur once every 2,500 years. In order for structural performance to be predictable, consideration must be given to the reliability of the final design to meet the stated performance objectives. Since the building design and construction must be carried out in a world of uncertainties, the reliability of the final design can only be stated in probabilistic terms.

Recently, the performance-based design methodology has been applied to the evaluation, retrofit, and rehabilitation of existing buildings. In 1996, the Applied Technology Council (ATC) published the report entitled *Seismic Evaluation and Retrofit of Concrete Buildings* [ATC, 1996], also referred to as the ATC-40

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report. This report is focused primarily at the number of pre-1970's California government buildings that are of nonductile concrete construction. In 1997, the Federal Emergency Management Agency (FEMA) published the *NEHRP Guidelines for the Seismic Rehabilitation of Existing Buildings* [ATC, 1997], and is referred to as the FEMA 273/274 report. Unlike ATC-40, the FEMA 273/274 report covers a broad range of building materials, including steel, concrete, masonry, and wood construction. The methodology presented in the FEMA 273/274 and ATC-40 reports is cast in a framework that offers the building owner and design engineer the flexibility to select multiple performance objectives for the evaluation of buildings.

As part of any seismic building design or evaluation procedure, the design engineer must perform an analysis of the building, incorporating the seismic hazard at the building site, to obtain building response quantities. Building performance is deemed acceptable if these quantities are within the limits of acceptable building response. In general, building analysis involves the application of lateral earthquake forces, in addition to gravity forces, to a mathematical model of the building. Building analysis methods can be differentiated based on whether the mathematical building model is *linear* or *nonlinear*, and whether the earthquake forces are applied in a *static* or *dynamic* manner. The basic assumption in a linear building model is that the building components, i.e. beams and columns, have infinite strength and constant stiffness during the analysis. Alternately, a nonlinear building model attempts to capture the strength and stiffness decay of the building components as they are damaged. In a static analysis, a presumed pattern of lateral earthquake forces is applied to the building model. Therefore, this type of analysis does not account for the time-varying building response to earthquake ground motion, captured in a dynamic analysis. Building analysis methods can be categorized as follows (ATC, 1997): Linear Static (LS), Linear Dynamic (LD), Nonlinear Static (NS), Nonlinear Dynamic (ND). Because of the fundamental assumptions involved with the methods utilizing linear building models and the static application of lateral forces, the ND method is considered in this paper to be an "exact" analysis method. Alternately, the LS, LD, and NS methods are called "approximate" analysis methods.

Until recently, guidelines for the design of new buildings have focused primarily on linear analysis methods. The reason for this is that nonlinear analysis methods were often viewed as overly complicated for application in new building design. In addition, the use of these methods was precluded by limitations in computer capabilities and uncertainties in modeling the strength and stiffness of building components. Nonlinear analysis methods have been found more useful for the evaluation of existing buildings. The ATC-40 document emphasizes the use of NS analysis methods for predicting seismic demands. The FEMA 273/274 document describes procedures covering both the NS and ND analysis methods. These methods provide for a more realistic estimate of the demands on the building system and its components, and help identify critical regions with large deformation demands [Krawinkler, 1996].

In recent years, the NS analysis method has received more attention compared to the ND method because of its ability to address the aforementioned issues in an approximate manner without the computational and modeling effort of a ND analysis. However, an assessment of the uncertainty in the NS analysis methods must be made in order to incorporate these methods into the reliability framework of performance-based design. Part of the research by the SAC Joint Venture has focused on NS analysis method uncertainties and forms the basis for the research in this paper.

NONLINEAR STATIC ANALYSIS METHODS

A nonlinear static or "pushover" analysis requires that a nonlinear mathematical model of the building be subjected to monotonically increasing lateral forces until reaching a predetermined *target displacement*. The target displacement is an estimate of the maximum roof displacement that will be experienced by the building during a given earthquake. The lateral loads are applied incrementally and the strength and stiffness properties of each building component are updated after each load increment. The base shear versus roof displacement relationship, referred to as the *capacity curve*, is the fundamental product of a pushover analysis because it characterizes the overall performance of the building. From the pushover analysis, force and deformation demands on the building are calculated at the target displacement and evaluated against acceptable force and deformation capacities.

The lateral loads applied to each building floor during a pushover analysis can have a significant effect on the distribution of nonlinear behavior in the building and the prediction of system and component force and deformation demands. This pattern of lateral loads is intended to represent the distribution of lateral inertia forces that act on a building during an earthquake. Lateral load patterns are generally classified as either fixed or variable. In the case of a fixed lateral load pattern, the lateral inertia forces are assumed to remain unchanged during the building's response to the earthquake ground motion. This type of lateral load pattern is unable to account for the change in distribution of lateral inertia forces that occurs when the building stiffness changes due to nonlinear behavior.

Since the distribution of nonlinear behavior may vary according to the lateral load pattern, it has been suggested [Krawinkler, 1996; ATC, 1997] that multiple patterns be investigated when performing a pushover analysis. This paper addresses two types of fixed lateral load patterns, referred to as *Uniform* and *Modal*. For the uniform load pattern, forces are applied to each floor in proportion to the floor mass. This load pattern emphasizes demands in lower stories compared to demands in upper stories and magnifies the relative importance of story shear forces compared to overturning moments [Krawinkler, 1996]. The modal load pattern is a set of lateral forces applied to the building in proportion to the elastic fundamental mode shape and floor masses. The modal load pattern reflects the distribution of lateral inertia forces expected as the building responds elastically to the earthquake ground motion.

The force and deformation demands on a building are calculated from the pushover analysis when the roof displacement reaches the target displacement. Three methods for calculating the target displacement have been investigated in this research: 1) Coefficient Method, 2) Capacity Spectrum Method, and 3) Equivalent System Method. As the name implies, a target displacement calculated using the Coefficient Method [Krawinkler, 1996; ATC, 1997] is the product of the elastic spectral displacement at the fundamental natural period of the building and a series of coefficients that: 1) relate spectral displacement to roof displacement, 2) relate elastic displacement to inelastic displacement, 3) account for the effect of hysteresis shape on the displacement response, and 4) account for the effect of dynamic P-delta on the displacement response. The Capacity Spectrum Method was originally developed for the evaluation of existing buildings [Freeman et al., 1975] and is recommended for the evaluation of existing reinforced concrete buildings in ATC-40 [ATC, 1996]. Using this method, the capacity curve of the multi-degree-of-freedom (MDOF) structure is first converted to an equivalent single-degree-of-freedom (SDOF) capacity spectrum by assuming that the deflected shape of the structure can be represented by the building's fundamental mode shape. To account for energy dissipated by hysteresis during the earthquake, the capacity spectrum is used to estimate an effective damping of the building. The target displacement is calculated by finding the intersection of the capacity spectrum and demand spectrum, which is a plot of spectral acceleration versus spectral displacement at the effective damping. The Equivalent System Method is similar to the Capacity Spectrum Method in that the capacity curve obtained from the pushover analysis is converted to a force-deformation relation corresponding to an equivalent SDOF system. The target displacement is found by subjecting the equivalent SDOF system to an earthquake ground motion and converting the maximum displacement response to the roof displacement of the MDOF building. Different forms of the equivalent system method have been used by researchers to calculate the target displacement for buildings [Saiidi and Sozen, 1981; Miranda, 1991; Collins, 1995; Reinhorn, 1996].

MODEL STEEL FRAME BUILDINGS

The uncertainty of the nonlinear static analysis method was quantified by investigating a 3-story, 9-story, and 20-story WSMF building. The buildings were designed as part of the SAC research program according to the 1994 UBC requirements for buildings located in Los Angeles, California. Two models (M1 and M2) of each building were created [Krawinkler and Gupta, 1998] representing different levels of sophistication in modeling the behavior of beam-column joint panel zone. The strength and stiffness of the beam-column joint panel zones is ignored in the M1 model. The M2 model explicitly accounts for the strength and stiffness of the beam-column joint panel zone. The M2 model is considered in this paper to be the more accurate computer model since the strength and stiffness of the beam-column joint panel-zone is explicitly considered.

EARTHQUAKE GROUND MOTION

A total of sixty earthquake ground motion records, twenty in each of three seismic hazard levels having a 50%, 10%, and 2% probability of exceedance in 50 years, were used to quantify the uncertainty in the NS analysis method. Pairs of earthquake ground motion records were selected and modified as part of the SAC research project [Somerville et al., 1997]. The earthquake records were selected to represent the range of earthquake magnitudes and distances expected to contribute to the seismic hazard at a building site located in Los Angeles, California. For each seismic hazard level, a target response spectrum was developed and the ground motion records uniformly scaled to minimize the error between the average response spectrum for each pair of time histories and the target response spectrum.

ANALYSIS METHOD UNCERTAINTY

Nonlinear time history analyses were performed for the 3-story, 9-story, and 20-story buildings using the sixty earthquake ground motion records and the M2 computer models [Krawinkler and Gupta, 1998]. These results were used as the benchmark to evaluate the bias and uncertainty of the results obtained using the NS analysis methods. Maximum roof displacement and maximum interstory drift angle are the building response quantities that are addressed in this paper. For each earthquake ground motion record, the value of a particular response quantity obtained from the NS analysis method was compared to the result obtained from the nonlinear time history analysis. The ratios of the nonlinear time history result to the NS result were used to quantify the bias and uncertainty of the NS analysis methods. Tables 1 and 2 present the medians and logarithmic standard deviations (standard deviation of the natural logarithms) of the ratios for maximum roof displacement and maximum interstory drift angle, respectively. The logarithmic standard deviation is the natural indicator of dispersion when the ratios are assumed to have a lognormal distribution. Figures 2 through 7 show the median and 84th percentile values of the comparison ratios for the 3-story, 9-story, and 20-story model buildings using the M2 model and the Modal load pattern. The ratios are shown for the comparison of maximum roof displacement and maximum interstory drift illustrating the sensitivity of the results to the method of target displacement calculation.

SUMMARY AND CONCLUSIONS

In recent years, the nonlinear pushover analysis method has been viewed as an attractive alternative to the nonlinear time history analysis. This is primarily because of the ability of the nonlinear pushover analysis to provide component and system deformation demands in an approximate manner without the computational and modeling effort of a nonlinear time history analysis. However, an assessment of the uncertainty in the nonlinear pushover analysis methods must be made in order to incorporate this method in the reliability framework of performance-based design. This paper has addressed the uncertainty of nonlinear pushover analysis methods to predict maximum roof displacement and interstory drift with regards to welded steel moment frame buildings subjected to various levels of earthquake ground motion. The results of this research can be summarized by the following observations.

- On the average, the Coefficient and Equivalent System methods provide estimates of the maximum roof displacement and maximum interstory drift within 20% of the nonlinear time history analysis results for the 10% in 50 year and 2% in 50 year ground motions. There is slightly more dispersion in the demand predictions using the Coefficient method. The FEMA 273/274 document suggests scaling the target displacement calculated using the Coefficient method by 1.5 to account for the uncertainty in the analysis method. The results in this study show that in most cases, a factor of 1.2 can be used to obtain an upper bound of the 84th percentile target displacements.

- In general, the Capacity Spectrum method tends to underestimate demands compared to the Coefficient and Equivalent System methods. The comparisons also show that this method results in the least amount of dispersion in the demand predictions.

- The demand comparisons are relatively insensitive to the analytical model and load pattern compared to the method used to calculate the target displacement. It is observed that maximum interstory drift demands are load pattern dependent, with the Uniform load pattern providing slightly more conservative results for the 9-story and 20-story buildings.

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REFERENCES

ATC. (1996). *Seismic Evaluation and Retrofit of Concrete Buildings*. Prepared by the Applied Technology Council, (Report No. ATC-40), Redwood City, California, for the California Seismic Safety Commission (Report No. SSC 96-01).

ATC. (1997). *Guidelines for the Seismic Rehabilitation of Buildings, Volume I, Guidelines, and Volume II, Commentary*. Prepared by the Applied Technology Council for the Building Seismic Safety Council. Published by the Federal Emergency Management Agency (Report Nos. FEMA 273 & 274), Washington, D.C.

Collins, K.R., Wen, Y.K., and Foutch, D.A. (1995). “Investigation of Alternative Seismic Design Procedures for Standard Buildings.” *University of Illinois at Urbana-Chapaign, Report No. UILU-ENG-95-2003*. 187 pages.

Freeman, S.A., Nicoletti, J.P., and Tyrell, J.V. (1975). “Evaluation of Existing Buildings for Seismic Risk—A Case Study of Puget Sound Naval Shipyard, Bremerton, Washington.” *Proceedings of the First U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California.

Krawinkler, H. (1996). “Pushover Analysis: Why, How, When, and When Not to Use It.” *Proceedings of the 65th Annual Structural Engineers Association of California Convention*, Maui, Hawaii, October 1-6.

Krawinkler, H. and Gupta, A. (1998). “Global Displacement Demands for Steel Moment Frame Structures in Different Seismic Regions.” 5*th US National Conference on Earthquake Engineering*.

Miranda, E. (1991). “Seismic Evaluation and Upgrading of Existing Buildings.” *PhD Dissertation*, University of California at Berkeley.

OES. (1995). California Office of Emergency Services, *Vision 2000: Performance Based Seismic Engineering of Buildings*. Prepared by Structural Engineers Association of California, Sacramento, CA.

Saiidi, M. and Sozen, M.A. (1981). “Simple Nonlinear Seismic Analysis of R/C Structures.” *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers*, Vol. 107, No. ST5, pp. 937-952.

Somerville, P., Smith, N., Punyamurthula, S., and Sun, J. (1997). “Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project.” *Report No. SAC/BD-97/04*, SAC Joint Venture.

Table 1 Nonlinear Static/Nonlinear Dynamic ratio sample statistics for maximum roof displacement

EQ Hazard	Load Pattern		3-Story						9-Story						20-Story					
			M1			M2			M1			M2			M1			M2		
			CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM
50/50	Uniform	Median	0.87	1.16	0.87	0.86	1.17	0.90	1.08	1.22	1.05	1.09	1.28	1.07	1.07	1.22	1.06	1.11	1.12	1.08
		$\sigma_{ln\varepsilon}$	0.17	0.21	0.12	0.16	0.21	0.12	0.15	0.15	0.15	0.11	0.21	0.11	0.23	0.16	0.22	0.18	0.20	0.17
	Modal	Median	0.85	1.05	0.93	0.85	1.08	0.98	1.04	1.05	1.07	1.06	1.08	1.08	1.03	1.13	1.06	1.08	1.16	1.08
		$\sigma_{ln\varepsilon}$	0.17	0.14	0.09	0.16	0.13	0.10	0.15	0.13	0.13	0.11	0.10	0.10	0.23	0.18	0.22	0.19	0.12	0.16
10/50	Uniform	Median	0.81	1.28	0.91	0.79	1.30	0.91	0.90	1.21	0.90	0.94	1.26	0.96	0.99	1.24	0.98	0.98	1.27	0.95
		$\sigma_{ln\varepsilon}$	0.21	0.17	0.12	0.22	0.17	0.13	0.21	0.11	0.17	0.19	0.13	0.17	0.24	0.13	0.17	0.23	0.13	0.14
	Modal	Median	0.78	1.14	0.92	0.78	1.17	0.92	0.86	1.13	0.91	0.89	1.14	0.97	0.95	1.17	1.02	0.95	1.15	0.95
		$\sigma_{ln\varepsilon}$	0.21	0.17	0.20	0.22	0.17	0.19	0.21	0.13	0.19	0.20	0.12	0.18	0.25	0.15	0.26	0.25	0.13	0.23
2/50	Uniform	Median	0.89	1.04	0.96	0.90	1.06	0.99	0.93	1.19	0.84	0.90	1.20	0.92	0.90	1.34	0.89	0.90	1.35	0.91
		$\sigma_{ln\varepsilon}$	0.50	0.38	0.29	0.51	0.38	0.32	0.39	0.22	0.23	0.35	0.21	0.28	0.28	0.33	0.26	0.30	0.32	0.22
	Modal	Median	0.87	1.02	0.92	0.89	1.01	0.96	0.89	1.12	0.84	0.86	1.14	0.86	0.86	1.27	0.90	0.87	1.26	0.90
		$\sigma_{ln\varepsilon}$	0.50	0.39	0.26	0.51	0.38	0.24	0.39	0.22	0.23	0.35	0.21	0.23	0.28	0.34	0.17	0.30	0.33	0.23

Table 2 Nonlinear Static/Nonlinear Dynamic ratio sample statistics for maximum interstory drift

EQ Hazard	Load Pattern		3-Story						9-Story						20-Story					
			M1			M2			M1			M2			M1			M2		
			CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM	CM	CSM	ESM
50/50	Uniform	Median	0.93	1.22	0.95	0.94	1.29	0.98	1.39	1.39	1.35	1.64	1.70	1.58	1.66	1.49	1.66	1.66	1.66	1.62
		$\sigma_{ln\varepsilon}$	0.25	0.25	0.20	0.24	0.24	0.20	0.54	0.47	0.54	0.52	0.43	0.53	0.73	0.49	0.73	0.67	0.45	0.67
	Modal	Median	0.91	1.14	0.97	0.95	1.21	1.03	1.66	1.71	1.66	1.87	1.81	1.88	1.96	1.98	2.01	1.91	1.99	1.91
		$\sigma_{ln\varepsilon}$	0.25	0.19	0.19	0.24	0.18	0.17	0.54	0.51	0.52	0.51	0.49	0.49	0.71	0.56	0.66	0.65	0.49	0.61
10/50	Uniform	Median	0.81	1.26	0.92	0.86	1.33	0.93	0.74	1.18	0.77	0.83	1.31	0.84	0.74	1.21	0.71	0.75	1.23	0.71
		$\sigma_{ln\varepsilon}$	0.20	0.15	0.13	0.21	0.15	0.13	0.50	0.31	0.45	0.48	0.34	0.43	0.56	0.32	0.46	0.56	0.29	0.46
	Modal	Median	0.80	1.12	0.92	0.85	1.18	0.96	0.89	1.28	1.01	0.99	1.44	1.09	0.86	1.49	0.98	0.89	1.50	0.91
		$\sigma_{ln\varepsilon}$	0.19	0.15	0.17	0.20	0.16	0.18	0.49	0.36	0.47	0.46	0.34	0.43	0.62	0.37	0.62	0.62	0.36	0.58
2/50	Uniform	Median	0.90	1.03	0.97	0.92	1.06	1.03	0.63	1.00	0.59	0.81	1.12	0.79	0.64	1.30	0.65	0.75	1.29	0.75
		$\sigma_{ln\varepsilon}$	0.46	0.37	0.27	0.47	0.37	0.31	0.59	0.29	0.44	0.38	0.21	0.36	0.30	0.32	0.29	0.35	0.31	0.27
	Modal	Median	0.90	1.04	0.96	0.94	1.06	1.02	0.80	1.25	0.84	0.97	1.33	1.00	0.71	1.53	0.76	0.86	1.57	0.87
		$\sigma_{ln\varepsilon}$	0.46	0.38	0.24	0.47	0.37	0.22	0.59	0.26	0.49	0.37	0.23	0.33	0.33	0.32	0.27	0.37	0.31	0.29

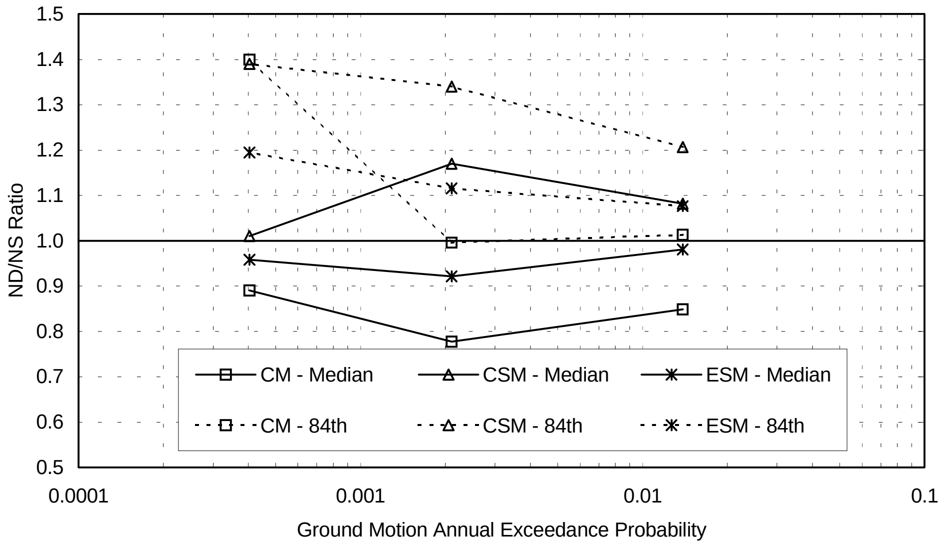

Figure 1 Maximum roof displacement ND/NS ratios for 3-story building/M2 model/Modal load pattern

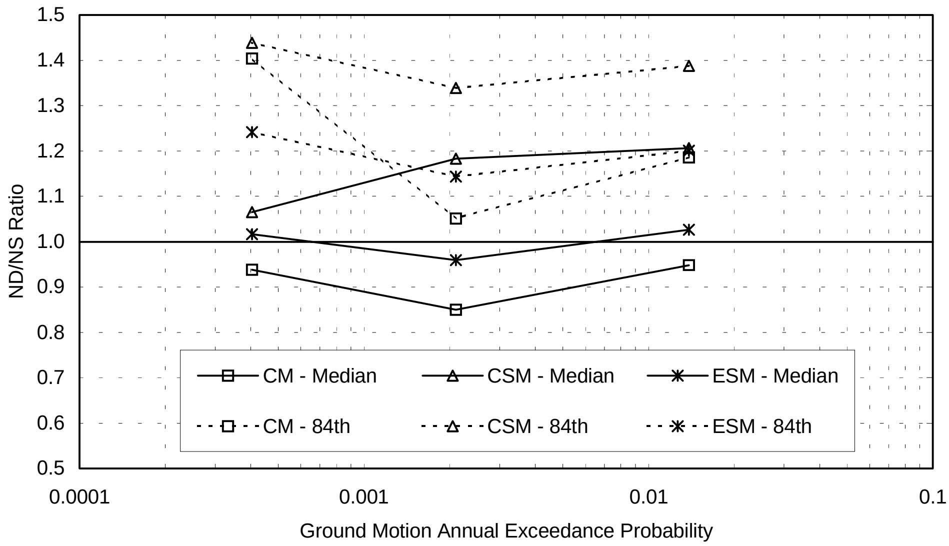

Figure 2 Maximum interstory drift ND/NS ratios for 3-story building/M2 model/Modal load pattern

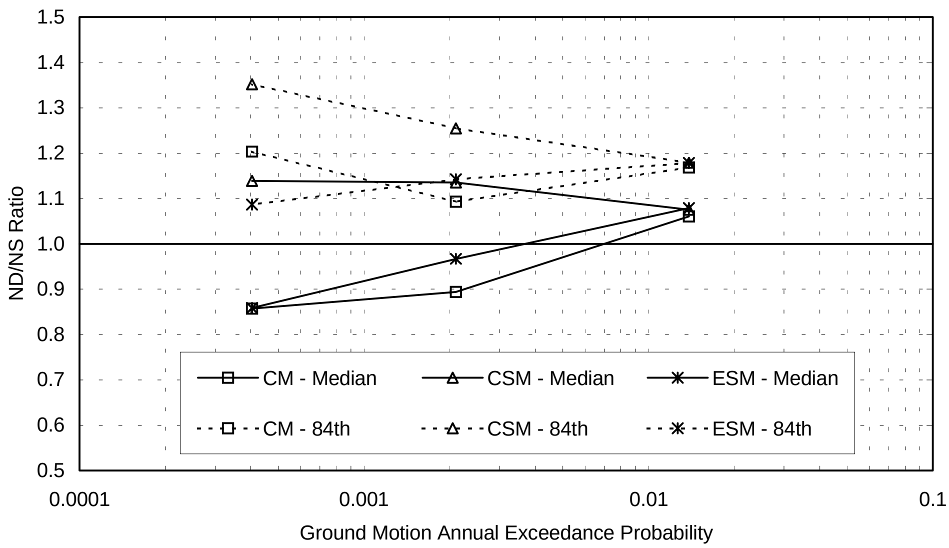

Figure 3 Maximum roof displacement ND/NS ratios for 9-story building/M2 model/Modal load pattern

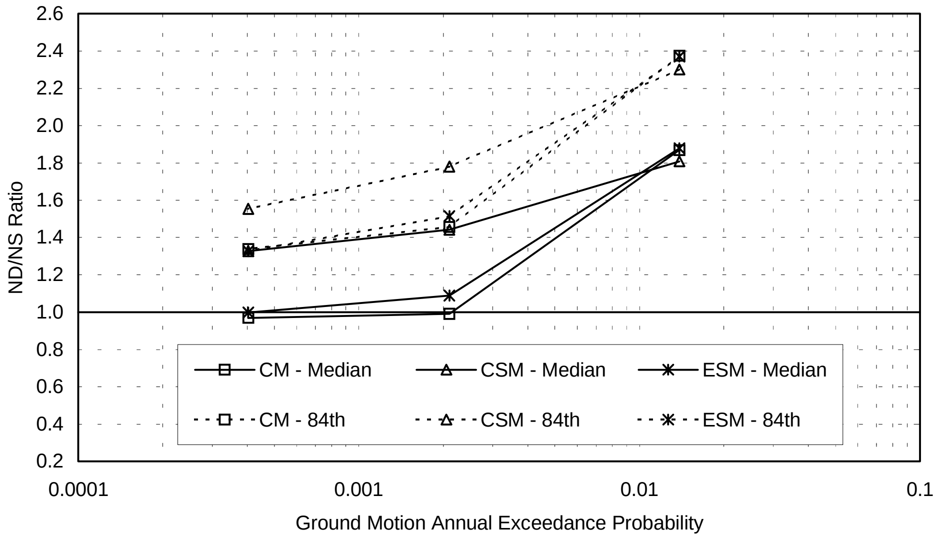

Figure 4 Maximum interstory drift ND/NS ratios for 9-story building/M2 model/Modal load pattern

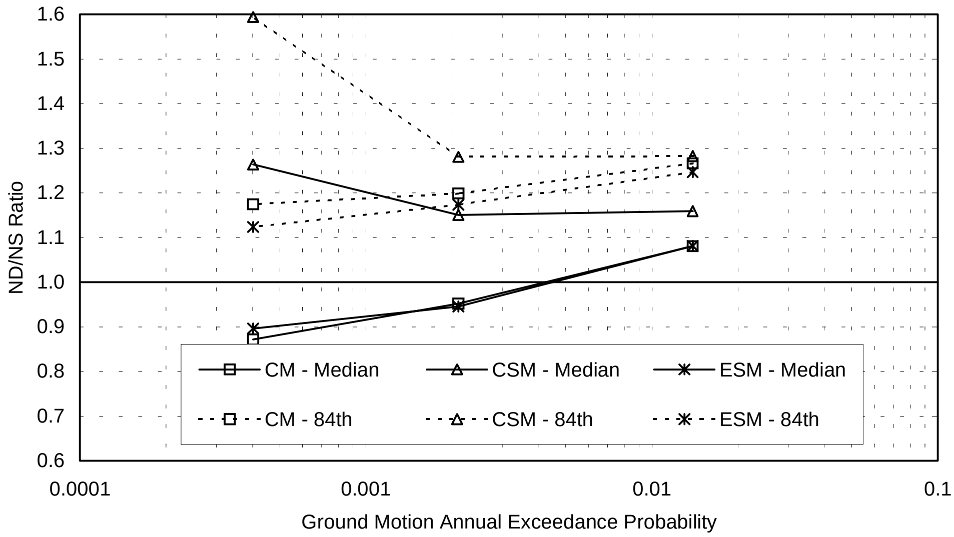

Figure 5 Maximum roof displacement ND/NS ratios for 20-story building/M2 model/Modal load pattern

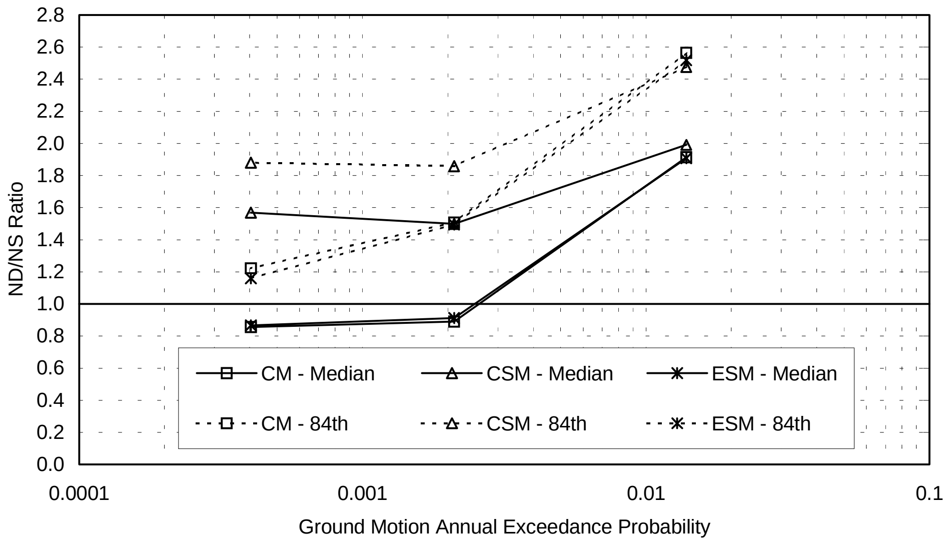

Figure 6 Maximum interstory drift ND/NS ratios for 20-story building/M2 model/Modal load pattern