Ninth Canadian Conference on Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

PERFORMANCE OF SHEAR WALL STRUCTURES DESIGNED BY A DISPLACEMENT-BASED DESIGN METHOD

J. M. Humar[1], F. E. Pina[2] and M. G. Asl[3]

ABSTRACT

A recently proposed method of displacement-based seismic design method (DBSD) is evaluated by comparing the structural response predicted by this method with the similar response obtained from nonlinear response history analyses of the structure for a suite of spectrum compatible ground motions. The structures, in which the lateral resistance is provided by reinforced concrete shear walls, are assumed to be located in Vancouver, and the seismic hazard is represented by the Uniform Hazard Spectrum (UHS) for the city, as specified in the National Building Code of Canada. A series of 20 ground motions that are compatible with the UHS is developed. The ground motions are selected from those recorded in similar seismic regions, taking into account the deaggregation of seismic hazard for Vancouver and the related magnitude distance characteristics. The selected motions are scaled according to a method suggested by Somerville to match the UHS. Response results of shear wall structures of 6, 12, 15, and 20 storeys, designed according to the DBSD, are used in a statistical evaluation. The response parameters used in the evaluation are displacements, interstorey drifts, and shear forces. It is observed that the estimates of displacements and interstorey drifts provided by DBSD are always higher than the median values obtained from nonlinear response history analysis (RHA). However, considerable variability exists in these responses. The shear forces obtained from DBSD are near the median of nonlinear RHA values.

Introduction

Prevention of structural collapse and preservation of life safety during a severe earthquake that has a fairly low probability of occurrence during the life of the building, are the primary goals of earthquake-resistant design. However, recent experience has shown that earthquakes may cause very large losses through nonstructural damage, business interruption, and downtime even when life safety is preserved. This has spurred an interest in developing methods of earthquake-resistant design that could be used to satisfy multiple performance objectives including life safety and loss minimization. A performance objective is, in fact, a combination of the level of hazard and the desired performance under such hazard. The process of seismic design that aims to meet one or more performance objectives is referred to as performance based seismic design (PBSD). A simple methodology of PBSD has been proposed in the SEAOC Vision 2000 report (1995). That report specifies a set of discrete performance levels, ranging from fully operational to near collapse, which the structure may be required to meet under specified levels of

[1] Distinguished Research Professor, Dept. of Civil and Env. Eng., Carleton University, Ottawa, On. K1S 5B6
[2] Graduate Student, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC.
[3] Structural specialist, AMEC, America, Calgary, Alberta

earthquake hazard. The earthquake hazard is determined from a probabilistic seismic hazard analysis (PSHA) and expressed in terms of the annual frequency of exceedance or the return period.

For the purpose of design, performance levels should be expressed in quantitative terms. The quantitative performance levels are usually defined through limiting values of measurable response parameters, such as storey drifts, element deformations, and ductility demands. Since storey drifts control damage to structural and nonstructural elements as well as instability induced by P-Δ effects, drift limits are commonly used to define performance levels. The method of design that ensures that drifts and displacements remain within or meet the specified limits is known as displacement-based method of seismic design (DBSD); when combined with probabilistic seismic hazard analysis it offers an efficient means of designing to meet the performance objectives.

In a recent paper Humar et al. (2006) have presented a practical method of DBSD that can be used for the design of structures of regular shape. The present work is primarily devoted to the evaluation of the proposed method of design, with particular reference to shear wall structures of reinforced concrete. For the purpose of such evaluation several shear wall structures of different heights located in Vancouver and designed by the DBSD method are analyzed to obtain their response to a suite of representative design ground motions in order to verify whether the structures perform as expected. In each case a nonlinear time history analysis (RHA) is performed.

Since the proposed DBSD method uses uniform hazard spectra (UHS) given in the 2005 National Building Code of Canada (NBCC 2005) (Canadian Commission 2005) to define the seismic demand, the ground motions used in the nonlinear time history analyses, which must represent the seismicity of the city, should be compatible with the UHS for the city of Vancouver. Records are selected from database of records with magnitude and distance characteristics similar to those of Vancouver and scaled to match the UHS. Several different procedures exist for such matching. In the present work, a unique scaling factor is used, both to ensure simplicity and to prevent the distortion of frequency content in the motion.

Three different response parameters are obtained through nonlinear RHA: interstorey drift ratios, storey displacements, and storey shear forces. The results obtained from the analyses are processed to obtain the mean and the 84th percentile values, which are then compared with the reference results obtained from DBSD. The bias and the dispersion in the results obtained from the DBSD and RHA are presented along with a discussion on the results.

It may be noted that the proposed DBSD requires the combination of modal values of certain response parameters. Such combination is carried out by using two different rules, namely, square root of the sum of squares (SRSS) and absolute sum (ABSSUM). This provides the opportunity to evaluate the accuracy of the solutions obtained with the two combination rules.

Selection of Seismic Ground Motion Time Histories

Seismicity of Vancouver

Vancouver is located near a subduction zone called Cascadia. However, the city is 150 km east of this zone and its major hazard comes not from the subduction earthquakes themselves but from the crustal and subcrustal earthquakes that originate in the subduction zone (Halchuk and Adams 2004). Nevertheless, the presence of a subduction zone plays an important role in the selection of ground motions for Vancouver, and one must search for the crustal and subcrustal activity in other similar subduction regions.

The earthquake hazard for Vancouver can be assessed from the deaggregation analysis carried out by Halchuk and Adams (2004). The distribution of earthquakes contributing to the spectral acceleration at several different periods exhibits a bimodal pattern. The contribution to the seismic hazard comes from both crustal and subcrustal activity. The crustal earthquakes contributing to the hazard have hypocenter distances less than 40 km from the site and a moment magnitudes greater than 7.0. The subcrustal earthquakes have hypocenter distances of between 40 and 80 km and magnitudes between 6.5 and 7.0.

Another important aspect that must be considered in the selection of ground motions is the definition of the target response spectrum. The NBCC 2005 provides the uniform hazard spectra corresponding to a 2% probability of exceedance in 50 years for different Canadian cities. The code provides spectral accelerations for selected values of the period; however, values of spectral acceleration for periods longer than 4 s are not specified. In the present study, spectral accelerations for periods longer than 4 seconds are assumed to be inversely proportional to the period, so that

$$S(T) = \frac{4S(4)}{T} \quad \text{for } T > 4\text{ s} \tag{1}$$

Selection of seismic regions and data

The preferred source of ground motion records should be the inventory of records produced during the local seismic events that made a major contribution to the hazard. However, for Vancouver the number of records obtained during the local seismic activity is not many and records from other seismic regions with similar tectonic settings must be included in the selection.

Three major earthquakes have occurred in the seismic zone of Vancouver and in similar zones away from the city and having a high density of instruments. These earthquakes are: the Nisqually earthquake (2001) in Washington State, USA and the Geiyo (2001) and Tokachi Oki (2003) earthquakes in Japan. Other important recorded earthquakes in similar regions include the Michoacan earthquakes (1994, 1997 and 2000) in Mexico, El Salvador earthquakes (2001) and several others in the North-west of USA. A number of records from these regions were considered in this study.

A factor that must be taken into account in selecting the records is the site condition at the site of the recording instrument. In this study, whenever possible, records measured by instruments located on sites of class C (NBCC 2005) are selected. Site class C, which is used as the reference site class in NBCC 2005, is defined as having average shear wave velocity between 360 and 760 m/sec in the top 30 meter depth.

Scaling of ground motion records

The selected ground motions must be scaled to match the UHS for Vancouver. As stated earlier such scaling of records is best achieved by using a unique scaling factor, both to ensure simplicity and to prevent the distortion of frequency content in the motion. The method of scaling proposed by Somerville et al. (1997) has been used in this work. This approach minimizes the sum of weighted squared errors between the spectral accelerations of the scaled motion and the target spectrum at several different periods, assuming lognormal distribution of the amplitudes. The ground motion spectrum is taken as the average of the spectra of motions in the two orthogonal lateral directions. Four periods are considered: T_1 = 0.3, T_2 = 1, T_3 = 2, and T_4 = 4 seconds. Weighting factors of W_1 = 0.1, W_2 = 0.3, W_3 = 0.3 and W_4 = 0.3, respectively, are applied to the squared errors at the four periods. Other choices are possible for the periods and weighting factors.

An alternative approach to scaling has been presented by Lestuzzi et al. (2004) who introduced a new definition of spectral intensity, called modified spectral intensity, SI_b. The spectral intensity measures the severity of the ground motion and is equal to the area under the pseudo-velocity spectrum between the fundamental period T_0 and an extended period $T_{s,\ \text{which}}$ corresponds to the secant stiffness at maximum displacement. If equal displacement rule is assumed, the two periods are related as follows:

$$T_s = T_0\sqrt{R} \tag{2}$$

where R is the ductility related force reduction factor. The spectral intensity is given by

$$SI_b = \frac{1}{(T_s - T_0)} \int_{T_0}^{T_s} P_{SV} dT \tag{3}$$

in which P_{SV} represents the spectral velocity. Lestuzzi et al (2004) found good correlation between the spectral intensity and the ductility demand. The ground motion may therefore be scaled such that its spectral intensity matches the spectral intensity of target spectrum.

In the present study, T_0 = 1 s. and R = 4 are considered as being representative values for ductile mid-rise structures and are used in Eqs. 2 and 3. Since this definition is arbitrary, the results obtained from spectral intensity match are used only as a reference on the selection of the suite of ground motions, and only the Somerville scaling factor is used to modify a record.

Table 1. Selected records for the city of Vancouver.

GM #	Earthquake name	File Name	New Name	Factor α_{sh}	SI_b m/sec	Diff. SI_b %	Factor α_{sm}
1	Tokachi Oki (2003)	HKD0109_NS	VAN01A	1.04	0.41	18.85	0.99
2		HKD0109_EW	VAN01B	1.34	0.41	18.85	0.99
3	Olympia (1949)	103l56ol_N04W	VAN03A	1.49	0.55	4.15	1.65
4		103l56ol_N86E	VAN03B	1.84	0.55	4.15	1.65
5	Tokachi Oki (2003)	HKD0107_NS	VAN04A	2.04	0.44	14.92	1.67
6		HKD0107_EW	VAN04B	1.92	0.44	14.92	1.67
7	Nisqually (2001)	CRO_01_NS	VAN09A	1.85	0.60	0.57	3.32
8		CRO_01_EW	VAN09B	3.90	0.60	0.57	3.32
9		SEU_01_NS	VAN12A	3.37	0.68	9.13	3.88
10		SEU_01_EW	VAN12B	4.05	0.68	9.13	3.88
11		EVA_01_NS	VAN15A	2.37	0.56	3.83	4.19
12		EVA_01_EW	VAN15B	3.13	0.56	3.83	4.19
13		SEA_01_EW	VAN16A	3.31	0.65	5.55	4.47
14		SEA_01_NS	VAN16B	3.11	0.65	5.55	4.47
15		CTR_01_NS	VAN19A	2.75	0.63	3.49	4.68
16		CTR_01_EW	VAN19B	5.18	0.63	3.49	4.68
17		UCACA02_S32E	VAN23A	5.22	0.35	24.22	4.75
18		UCACA02_S58W	VAN23B	2.63	0.35	24.22	4.75
19	Helena (1935)	KITP_01_EW	VAN25A	3.59	0.70	10.48	4.83
20		KITP_01_NS	VAN25B	3.39	0.70	10.48	4.83

Selection criteria

For the present study, more than 300 records were obtained for use in the selection process explained earlier. Most of these records came from three major earthquakes with a high density of instruments in the affected area: Nisqually earthquake 2001, Geiyo earthquake 2001, and Tokachi Oki earthquake 2003. The following criteria were considered in making a selection from all of the records considered:

1. Consider only those ground motions for which reliable information regarding soil conditions at the instrument site exists;
2. Include as many different earthquakes as possible, rather than selecting records from the same earthquake;
3. Include records from both horizontal directions for each instrument;
4. Define an upper limit on the scaling factor to avoid large modifications in the spectral response (e.g. increased high-frequency responses);
5. If the number of available records is large enough, consider those records for which the difference between the spectral intensity (SI_b) of the record and that of the target spectrum is the smallest.

Since, two records are selected from each instrument site, the average of the SI_b of the two may be used as the representative value.

Table 1 shows a summary of the 10 pairs of records that were finally selected. In choosing these records the scaling factor was limited to 5 and the maximum difference between the spectral intensity of the ground motion spectrum and the target spectrum (SI_b for the target spectrum is 0.594 m/s) was limited to 20%. In Table 1 the number in parenthesis following the earthquake name indicates the year of occurrence. The file name represents the original file name from the source, except for records from Geiyo, Tokachi Oki and Nisqually earthquakes where it represents the name of the station. The file names end with an indication of the direction of the instrument. The first factor, α_{sh}, is the scale factor required to match the spectral acceleration produced by the record at a period of 1 second to the corresponding value in the target spectrum. The spectral intensity, SI_b, was calculated between the periods of 1 and 2 seconds. The factor, α_{sm}, is the scale factor obtained by the Somerville's method. Figure 1 compares the target spectrum with the spectra for selected records.

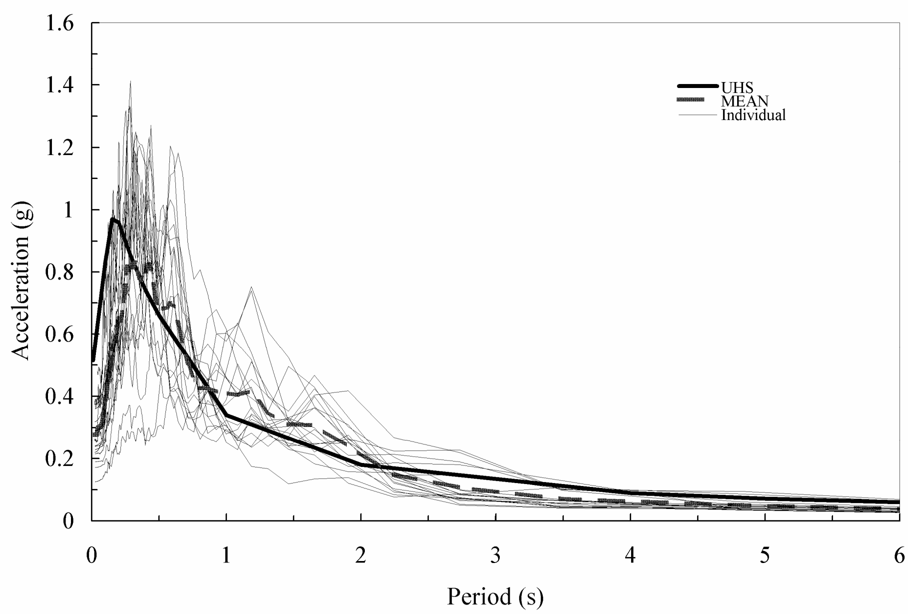


Figure 1. Acceleration response spectra of selected ground motions and the UHS for Vancouver.

Evaluation of Displacement Based Design

In the nonlinear analyses carried out for evaluating the displacement-based design the shear wall buildings are modeled by frame elements, the base element being fixed at the bottom. The plastic hinge is expected to form at the base, although the analytical formulation allows the formation of a plastic hinge at either one or both ends of any element. The properties of the plastic hinge are obtained from the moment-curvature relationship of the wall cross section. Every element is assumed to have cracked and to possess a reduced moment of inertia, which is also the assumption made in the DBSD.

The nonlinear RHA is performed with the computer program DRAIN-2DX (Prakash et al. 1998) using a plastic hinge beam column element. In this study, a constant moment capacity is specified for the wall section, ignoring any interaction with the axial load. However, the specified moment capacity is obtained from a moment-curvature analysis that takes into account the gravity induced axial load. The P-Δ effect is taken into account. The force-deformation relationship is assumed to be elasto-plastic in the absence of P-Δ effect, so that when such effect exists the post elastic branch of the relationship has a negative slope. Rayleigh damping is assumed with 5% damping in the first and the fourth mode for each of the building.

Response statistics

In the elastic range, if the response spectrum of each of the ground motions in a series matches a target spectrum, the responses of any given structure to the ground motions in the series would all be identical. However, this is not true of nonlinear responses, which may vary significantly from record to record. In addition, several simplifying assumptions are implied in DBSD. For example, the representation of a multi-degree-of-freedom system by a single-degree-of-freedom is at best an idealization. Also, the use of multi-mode analysis in which the responses in individual modes are assumed to be uncoupled, and the total response is obtained by taking the square root of the sum of squared modal responses is not strictly valid in a nonlinear analysis. These factors may lead to significant variation between the responses obtained from nonlinear RHA and that provided by DBSD. The confidence with which the results of DBSD can be used depends on the quality of statistical variation between the two sets of results. The statistical parameters or estimators used here are the geometric mean (position) and the standard deviation (dispersion) of the lognormal values. An additional benchmark used to assess the data is the response represented by the 84^{th} percentile.

Finally, the bias in the results of DBSD is deduced from the ratio between the response obtained from DBSD and the position measures of the nonlinear RHA, namely the geometric mean and the 84^{th} percentile. In either case, the DBSD is biased toward underestimating the response when this ratio is less than one and overestimating the response when the ratio exceeds one.

Presentation of results

The bias (DBSD/RHA) values are shown in Figs. 2, 3, 4, and 5 for the three response parameters: interstorey drift ratios, floor displacements, and storey shears. Results are presented for two combination rules (SRSS and ABSSUM) and the two position measures. The results were obtained by combining the contributions from the first three modes, except in the case of storey shears for the 15 and 20-storey buildings where the final results were obtained by combining the contributions from 4 and 5 modes, respectively.

Inter-storey drift ratios

The drift ratios for the 6-storey building (Fig. 2) show that when compared with the median response obtained from nonlinear RHA, DBSD results based on SRSS combination rule are fairly unbiased. On the other hand, The ABSSUM rule clearly overestimates the median response. The DBSD based on either the SRSS or the ABSSUM rule underestimates the response when compared with the 84^{th} percentile value. For the 12-storey building (Fig. 3), the SRSS rule overestimates the drift response when compared to the median value obtained from nonlinear RHA, but underestimates the response when compared to the 84^{th} percentile value. The ABSSUM rule is conservative in its predictions when compared to the median but gives almost unbiased results when compared to the 84^{th} percentile. The bias results for the 15-storey building (Fig. 4) show that the SRSS rule overestimates the drift response when compared with the median value obtained from nonlinear RHA, but underestimates the response when compared to the 84^{th} percentile value. The ABSSUM overestimates the drift with respect to both the median and 84^{th} percentile. In the 20-storey building (Fig. 5), the bias ratios show that even when the SRSS rule is used the DBSD provides drifts that are larger than both the median and the 84^{th} percentile values obtained from RHA.

Displacements

It is observed that for the 6-storey building (Fig. 2) the displacement predictions based on SRSS combinations are almost unbiased with respect to the median value obtained from nonlinear RHA. However, the response based on SRSS is lower than the 84^{th} percentile value. The response based on ABSSUM rule overestimates the median but is lower than the 84^{th} percentile value. The bias values for the 12-storey building (Fig. 3) clearly show that both the SRSS and the ABSSUM rules overestimate the displacement response when compared with the median value obtained from nonlinear RHA. On the other hand, both rules underestimate the response when compared to the 84th percentile. For the 15-storey building (Fig. 4) both the SRSS and ABSSUM rules overestimate the displacement response when compared with the median value obtained from nonlinear RHA. The SRSS rule underestimates the 84th percentile value; the ABSSUM value is close to the 84th percentile. For the 20-storey building (Fig. 5), the SRSS and ABSSUM rules overestimate both the median and 84th percentile displacement responses.

Shear Forces

The results for the 6-storey building (Fig. 2) show that the predictions of shear are almost unbiased when compared to the median value obtained from nonlinear RHA. The SRSS combination underestimates the response when compared to the 84th percentile. The responses obtained from the ABSSUM rule are larger than both the median and the 84th percentile. The results for the 12-storey building (Fig. 3) show that the shear response obtained with SRSS combination is practically unbiased with respect to the median value obtained from nonlinear RHA. The SRSS combination, however, underestimates the 84th percentile response. The ABSSUM rule overestimates the median as well as the 84th percentile responses. The results for the 15-storey building (Fig. 4) show that the SRSS results for shear are almost unbiased with respect to the median response obtained from nonlinear RHA. The SRSS rule, however, underestimates the response when compared to the 84th percentile. The ABSSUM rule overestimates both the median and the 84th percentile responses. For the 20-storey building (Fig. 5) the SRSS combination gives only slightly lower shear response, when compared to the median value obtained from nonlinear RHA, but underestimates the response when compared to the 84th percentile. The ABSSUM combination overestimates the response with respect to both, the median and 84th percentile values.

Dispersion

For the 6-storey building, dispersion in the storey drift varies between a minimum of 0.32 at the roof to a maximum of 0.57 in the first storey. The displacement has dispersion between 0.41 and 0.57. Shear force dispersion varies between 0.12 and 0.33. The base shear dispersion is 0.25. In the 12-storey building storey drift dispersion varies between a minimum of 0.34 at the roof to a maximum of 0.73 in the first storey. Displacement dispersion has a minimum value of 0.50 at the roof and a maximum of 0.73 at level 1. Shear force dispersion is found to vary between 0.14, at the 8th level, to 0.31, at the roof level. Base shear has a dispersion of 0.23. In the 15-storey building dispersion in the storey drifts varies between 0.27 at the roof to 0.74 in the first storey. The displacement dispersion for the lower floors (from 1st through 6th) is quite large, ranging between 0.5 and 0.73. An almost constant dispersion in displacements exists above level 6 with a minimum value of 0.42 at the roof. The shear force dispersion varies from a minimum of 0.16 at the 10th level to a maximum of 0.27 at the roof. Base shear has a dispersion of 0.20. In the 20-storey building, dispersion in the storey drift varies between 0.20 at the roof to 0.51 in the first storey. The displacement dispersion has a minimum value of 0.33 at the roof and a maximum of 0.51 at level 1. Maximum shear force dispersion is found at the 11th level, being 0.28, while the minimum is at the 1st level, being 0.16.

Comments and Conclusions

The results of comparison between DBSD and nonlinear RHA for each building show that the ABSSUM rule considerably overestimates the responses, the most conservative results being for the shear forces. The SRSS rule always overestimates the drift ratio and displacements responses when compared to the median response obtained from nonlinear RHA. The results obtained from the analysis of 20-storey building, which is a very flexible structure, show that structure develops very small plastic deformations under many of the ground motion records. Thus, the drifts and displacements for the 20-storey building obtained from DBSD are quite conservative when compared to those obtained from nonlinear RHA.

The shear response is always well estimated by DBSD when the SRSS combination rule is used. A plausible reason for the apparent overestimation of displacements, at the same time when the shears are predicted quite well, is the fact that the higher modes make substantial contribution to the shear response while the displacement and drift responses are governed only by the first mode (Humar et al. 2006). The DBSD is based on the assumption that for periods longer than 4 s the spectral acceleration varies inversely as the period. For the flexible buildings the first mode period is fairly long and the spectral accelerations corresponding to such long periods may be quite small for some of the records, significantly smaller than the $1/T$ variation assumed in DBSD (Eq. 1). This affects the results for the 20-storey building, which has a first mode period of 6.14 s, and to some extent for the 15-storey building whose first mode period is 4.74 s.

The higher mode effect on the shear response is clearly indicated by the nonlinear RHA. The number of modes that should be included in DBSD is related to the number of storeys in the building. According to the results from the application of DBSD and this evaluation, including 3 modes in the response gave fairly good results for the 6 and 12-storey building. For the 15 and 20-storey buildings, inclusion of 4 and 5 modes in the total response provided reasonable results.

The dispersion in responses does not vary much among the different buildings, but are fairly large. However, the dispersion is very different depending on the type of response. The minimum dispersion is seen in the shear responses and the maximum in the displacement responses.

In the present study, the responses obtained from the use of SRSS rule in association with DBSD gave fairly reasonable estimates of the median responses obtained from the nonlinear RHA. Therefore, the SRSS can be considered as being appropriate for obtaining the total response from a multi-mode pushover analysis for structures of the type studied in this work.

The proposed DBSD procedure is seen to be a reliable approach to the design of shear wall buildings. The estimates of displacements and interstorey drifts provided by DBSD are generally higher than the median values obtained from nonlinear RHA of the type of structures studied here. The shears and forces obtained from DBSD are near the median of nonlinear RHA values.

References

Canadian Commission on Building and Fire Codes. 2005. *National Building Code of Canada 2005*. National Research Council of Canada, Ottawa, ON.

Halchuk, S., and J. Adams. 2004. Deaggregation of seismic hazard for selected Canadian cities. Proceedings of the13th world conference on earthquake engineering, Vancouver, BC, Canada, paper 2470.

Humar, J., F. Pina, and M. Ghorbanie. 2006. Displacement-based seismic design of reinforced concrete shear wall buildings. Paper submitted to the *Canadian Journal of Civil Engineering*.

Lestuzzi, P., P. Schwab, M. Koller., and C. Lacave. 2004. How to choose earthquake recordings for non-linear seismic analysis of structures. Proceedings of the13th world conference on earthquake engineering, Vancouver, BC, Canada, paper 1241.

Prakash, V., G.H. Powell, and S. Campbell. 1993. *DRAIN-2DX Base program description and user guide: version 1.10.* Department of Civil Engineering, University of California, Berkeley, CA, *Report No UCB/SEMM-93/17*

SEAOC Vision 2000 Committee. 1995. Performance-based seismic engineering. *Report* prepared by Structural Engineers Association of California, Sacramento, CA.

Somerville, P., N. Smith, S. Punyamurthula, and J. Sun. 1997. Development of ground motion time histories for phase 2 of FEMA/SAC steel project. SAC Joint Venture, Sacramento, CA, *Report* SAC/BD-07/04.

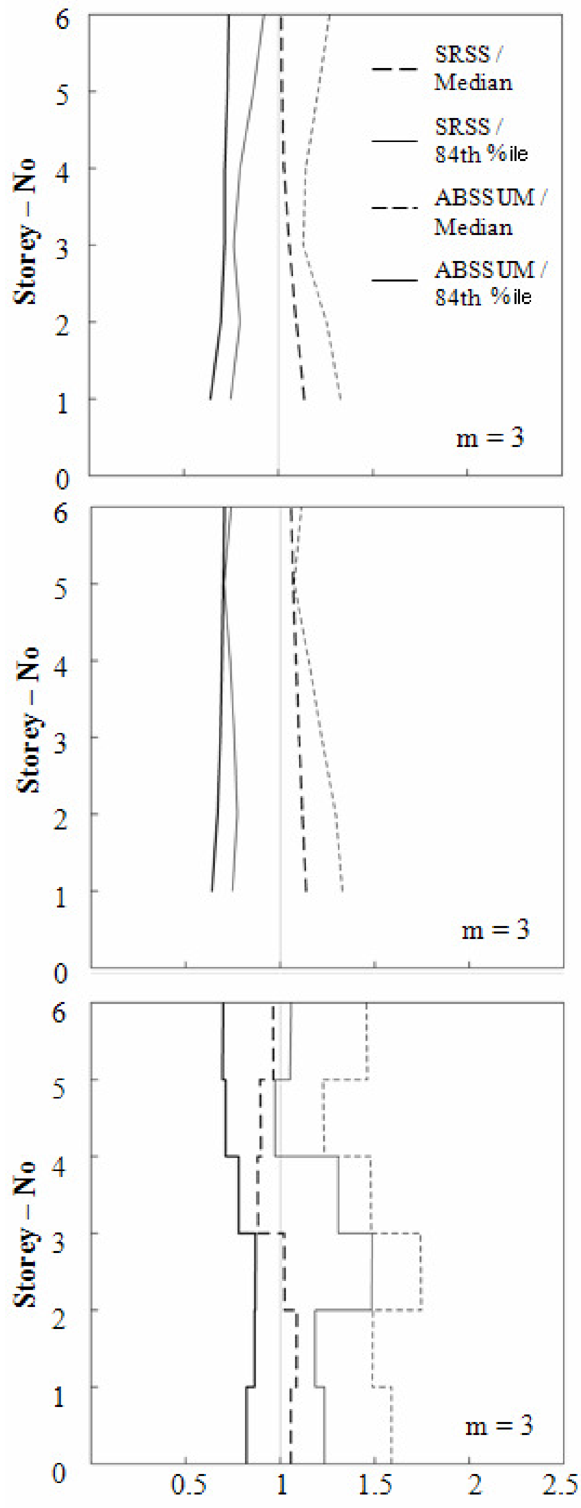


Figure 2. Bias (DBSD/RHA ratios) in the median and 84th percentile values for the 6-storey building: drifts (top), displacements (middle), shears (bottom).

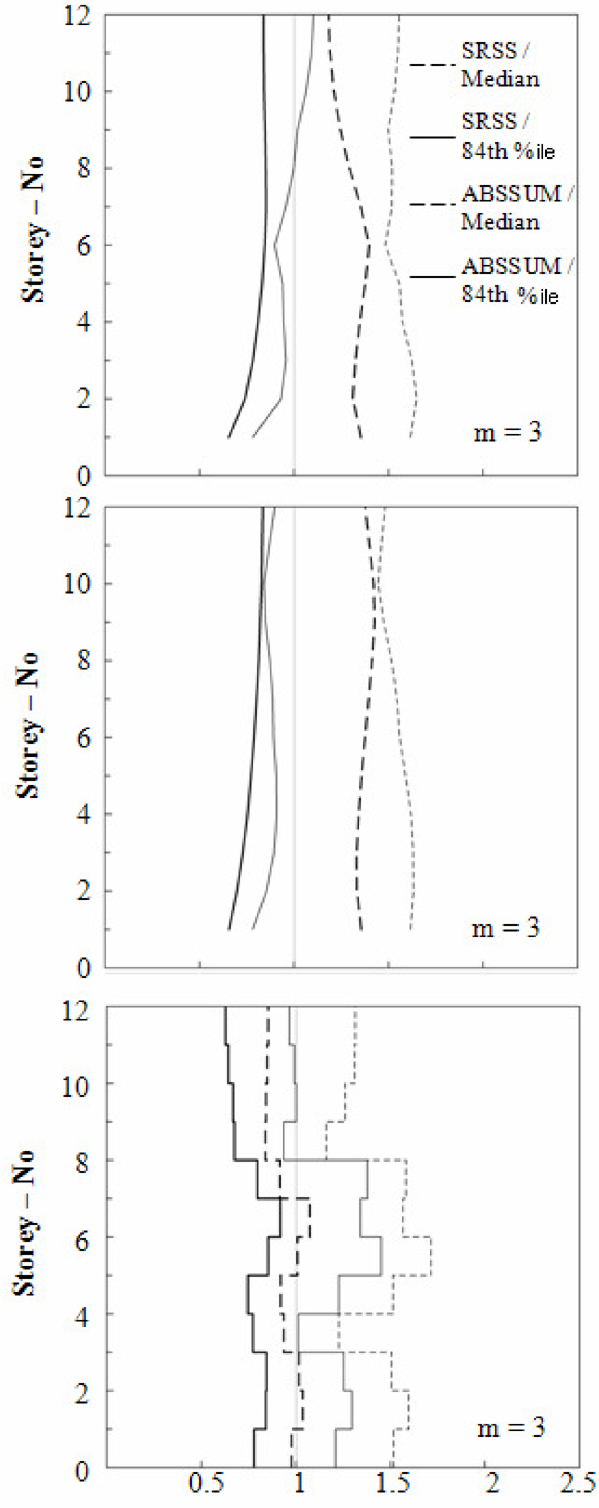
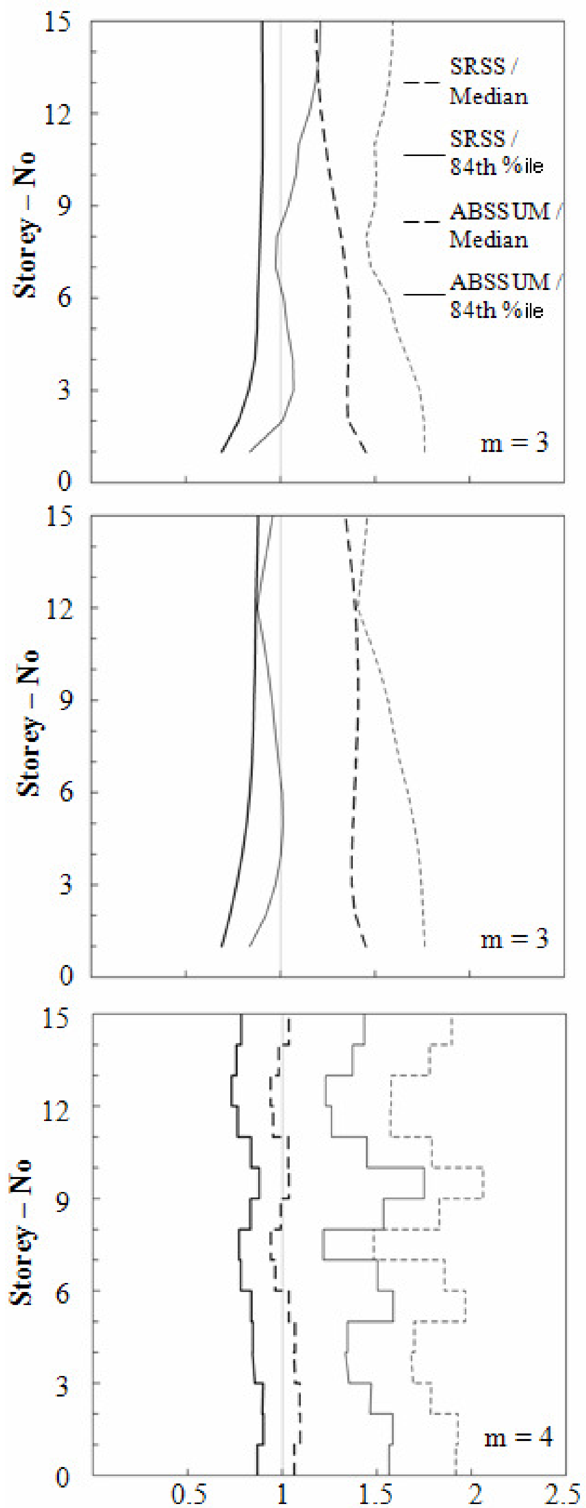


Figure 3. Bias (DBSD/RHA ratios) in the median and 84th percentile values for the 12-storey building: drifts (top), displacements (middle), shears (bottom).



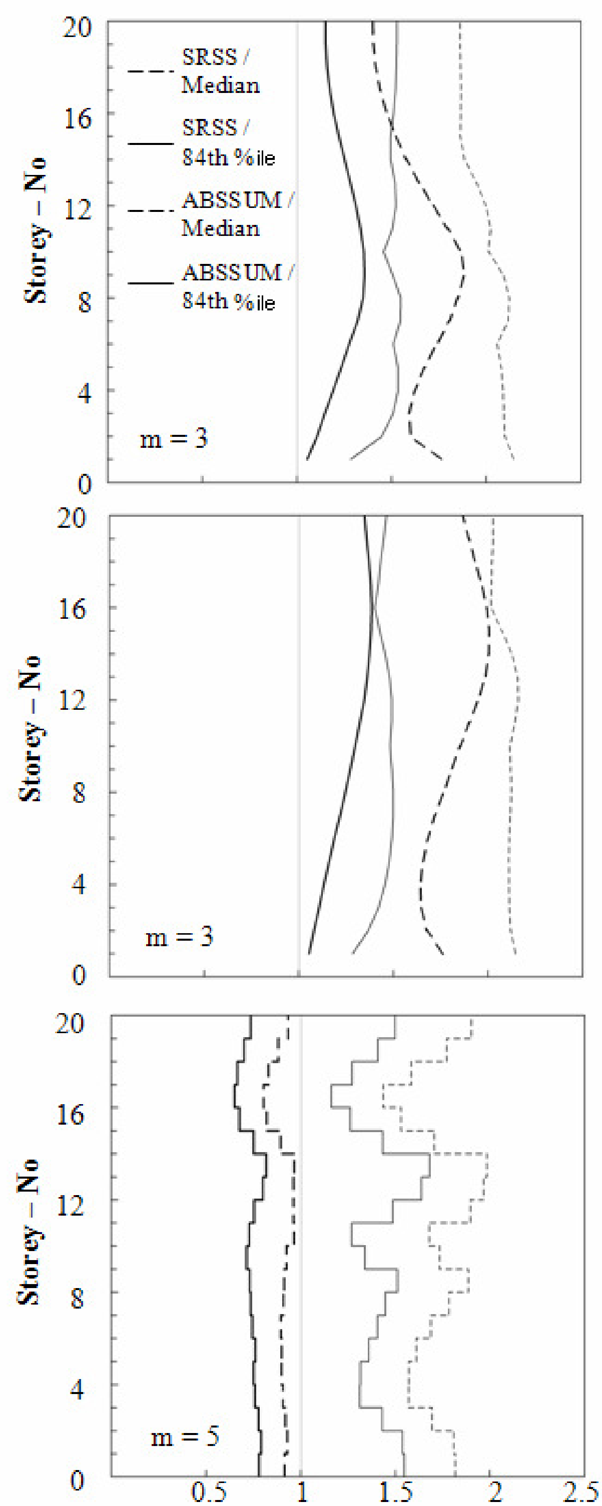


Figure 4. Bias (DBSD/RHA ratios) in the median and 84th percentile values for the 15-storey building: drifts (top), displacements (middle), shears (bottom).

Figure 5. Bias (DBSD/RHA ratios) in the median and 84th percentile values for the 20-storey building: drifts (top), displacements (middle), shears (bottom).