On the Use of Spectrum-compatible Seismic Excitations in the Design of Reinforced Concrete Buildings

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SUMMARY:
The selection of seismic motions is one of the most important input data decisions for the time-history analysis of buildings. Different methods can be used for the selection of spectrum-compatible accelerograms. This paper considers four sets of accelerograms that are compatible with the design spectrum for Vancouver, and investigates their effects on the seismic response of the studied buildings in terms of shear, ductility, and interstorey drift. The sets considered include: (i) scaled real records, (ii) modified real records in frequency domain, (iii) artificial accelerograms, and (iv) simulated accelerograms representative for Vancouver. Each set consists of 10 accelerograms. Three reinforced concrete buildings (4-storey, 10-storey, and 16-storey high) located in Vancouver were used in the study. Nonlinear time-history analyses were conducted on the buildings using the selected sets of accelerograms as excitation motions. The results showed that different types of spectrum-compatible seismic motions can lead to significant differences in the structural response. Based on this study, it was found that scaled real records are preferred when time-history analysis is used in the design of building structures.

Keywords: seismic response, accelerogram, reinforced concrete, drift, ductility.

1. INTRODUCTION

Nonlinear time-history analysis is seen as one of the most reliable methods for predicting the behavior of the buildings subjected to seismic loads. It has been widely used in research on the seismic performance of buildings, with the purpose of validating, and improving code provisions for seismic design. On the other hand, the use of nonlinear time-history analysis by design practitioners is relatively limited, and has been used only for the seismic evaluation and design of important buildings. Recent editions of modern building codes allow using nonlinear dynamic analysis for the design of buildings located in seismic regions and with heights above specified height levels (e.g., NRCC 2010; ASCE 2010; Standards New Zealand 2004). To perform nonlinear dynamic analysis, acceleration time histories (i.e., accelerograms) of seismic excitations are needed. The codes require that these accelerograms be compatible with the design spectrum. The main issues related to the use of spectrum-compatible accelerograms are: (i) the types of accelerograms (recorded or artificial) for use in the analysis, (ii) the method for selection and scaling of these accelerograms, and (iii) the number of accelerograms needed for the analysis. Since the National Building Code of Canada (NBCC) (NRCC 2010) does not provide guidance on the use of spectrum-compatible accelerograms, specifications prescribed in the U.S. Standard ASCE/SEI 7-10 (ASCE 2010) are used in this study. The use of this Standard is considered appropriate since the Canadian and the U.S. seismic design requirements are quite similar.

Different approaches have been used for the selection and scaling of spectrum-compatible accelerograms for the nonlinear time-history analysis (e.g. Tremblay and Atkinson 2001; Dincer 2003; Amiri-Hormozaki 2003). While different methods for the selection and scaling of accelerograms are in use, very few investigations on the effects of different types of accelerograms on the nonlinear response have been conducted so far. Naeim and Lew (1995) reported that accelerograms scaled in the frequency domain are not appropriate for use in seismic design since they might have unrealistic velocities, displacement, and energy content. Lew et al. (2008) suggested that in order to cover all response effects, tall buildings need to be analyzed using much more ground motion accelerograms than sets of three or seven accelerograms that are normally used in current design practice for tall buildings. Naumoski et al. (2006) investigated the nonlinear responses of two 6-storey and one 5-storey buildings, and reported significant differences in the responses from accelerograms simulated by Atkinson and Beresnev (1998) and those from scaled real accelerograms.

This paper discusses four different methods for the selection of spectrum-compatible seismic excitations for use in nonlinear time-history analysis. Based on these methods, four sets of accelerograms were selected for this study. Nonlinear time-history analyses were conducted on three reinforced concrete frame buildings designed for Vancouver, namely 4-storey, 10-storey, and 16-storey buildings, by subjecting the building models to the selected sets of accelerograms. The nonlinear responses are presented in terms of maximum interstorey drifts, curvature ductilities for beams and columns, and storey shears.

2. DESCRIPTION OF BUILDINGS AND DESIGN PARAMETERS

For the purpose of this study, three reinforced concrete frame buildings were designed. The buildings are for office use and located in Vancouver, which is in high seismic hazard zone (NRCC 2010). The buildings are identical in plan but have different heights. Figure 1 shows the plan and elevations of the buildings used in the study.

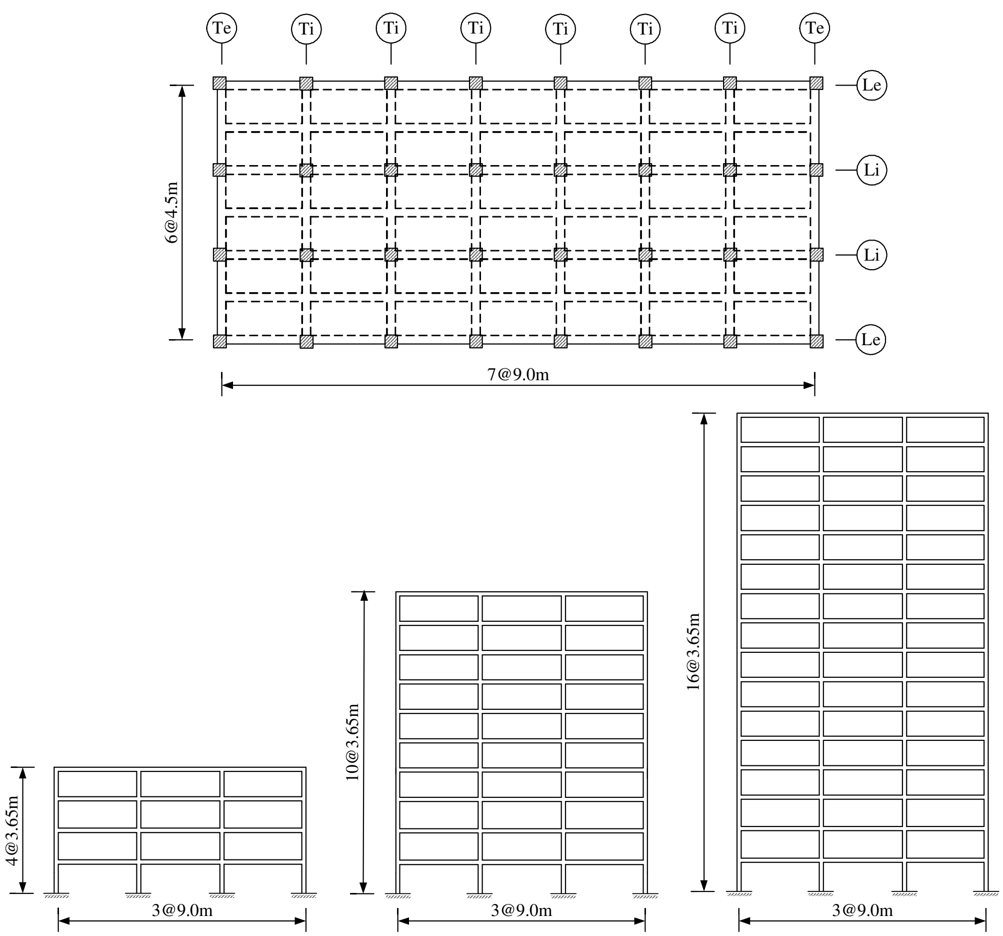

Figure 1. Plan of floors and elevations of transverse frames of the buildings.

As seen in Fig. 1 the buildings include a 4-storey, a 10-storey, and a 16-storey building. These buildings are used to represent the behavior of low-rise, medium-rise, and high-rise buildings, respectively. The plan of each building is 27.0 m × 63.0 m. The storey heights are 3.65 m. The lateral load resisting system consists of reinforced concrete moment-resisting frames in both the longitudinal and transverse directions. Secondary beams between the longitudinal frames are used at floor level to reduce the depth of the floor slabs. The floor system consists of one-way slabs spanning in the transverse direction, supported by the beams of the longitudinal frames and secondary beams. The slab is cast integrally with beams.

In this study, only the interior transverse frames (T_i) of the buildings were considered. For ease of discussion, the 4-storey, the 10-storey, and the 16-storey frames are referred to as the 4S, the 10S, and the 16S frames, respectively. The frames were designed as ductile reinforced concrete frames in accordance with the National Building Code of Canada (NBCC) (NRCC 2005). Note that the design would be the same if it were done according to the latest - 2010 edition of NBCC (NRCC 2010). The design base shears were calculated based on the seismic design spectrum for Vancouver. The foundations were assumed to be on stiff soil represented by site class C in NBCC (shear wave velocity between 360 m/s and 750 m/s). The fundamental periods of the frames were calculated according to the code formula, $T_a = 0.075h_n^{3/4}$, where h_n is the height of the frame above the base in meters. The other parameters used in the calculation of the base shears were the ductility-related force modification factor $R_d = 4$, the overstrength-related force modification factor $R_o = 1.7$, the higher mode factor $M_V = 1$, and the importance factor $I_E = 1$. Compressive strength of concrete $f_c^{'} = 30$ MPa, and yield strength of reinforcement $f_y = 400$ MPa were used in the design. The detailed design of the buildings is given in Lin (2008).

3. MODELING OF FRAMES FOR DYNAMIC ANALYSIS

Inelastic models of the frames were developed for use in the two-dimensional (2D) inelastic dynamic analysis program RUAUMOKO (Carr 2004). The beams and the columns were modeled by a beam-column element, which is represented by single component flexural spring. Inelastic deformations are assumed to occur at the ends of the element where plastic hinges can be formed. The effects of axial deformations in beams are neglected. Axial deformations are considered for columns, but no interaction between bending moment and axial load is taken into account.

For the purpose of the frame models, moment-curvature relationships for the end sections of each beam and column were computed using stress-strain model for confined concrete proposed by Mander et al. (1988) (Lin 2008). Based on the shape of the moment-curvature relationships, a trilinear hysteretic model was selected for columns, and a bilinear (modified Takeda) model was selected for beams (Fig. 2). Both models take into account the degradation of the stiffness during nonlinear responses. It is necessary to mention that the parameters of the trilinear model for each column were based on the computed moment-curvature relationships, while the values for the coefficients α and β for the bilinear model (Fig. 2(b)) were taken as 0.5 and 0.6, respectively, as suggested by Carr (2004). The natural periods of the first vibration mode of the models, obtained by RUAUMOKO, are given in Table 1. For comparison, the periods of the frames using the code formula are also included in the table. It can be seen in Table 1 that the periods obtained by RUAUMOKO are significantly larger than those given by the code formula. This was expected since it is known that the code formula provides relatively small period values that lead to conservative seismic design forces.

Table 1. Natural periods of the frame models (in seconds).

Frame model	Code formula	Ruaumoko
4S	0.56	0.94
10S	1.11	1.96
16S	1.58	2.75

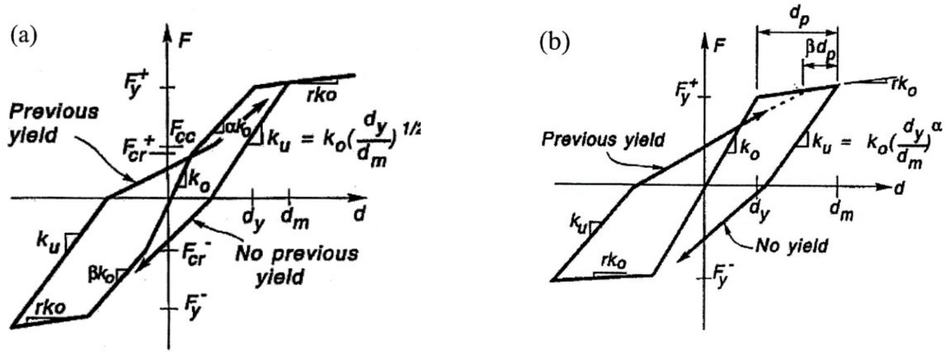

Figure 2. Hystertic model used in this study: (a) Trilinear Model for columns, and (b) Bilinear model for beams. (Adopted from Carr 2004).

4. SPECTRUM COMPATIBLE SEISMIC EXCITATIONS

In order to conduct nonlinear time-history analysis, four sets of seismic excitations were used in the study. These include,

- Set 1: Scaled real accelerograms
- Set 2: Modified real accelerograms
- Set 3: Simulated accelerograms
- Set 4: Artificial accelerograms

While Set 1 consists of actual accelerograms recorded during earthquakes, Sets 2 to 4 can all be considered as synthetic. Each set consists of 10 accelerograms. The accelerograms of each set are scaled in such a way that the mean acceleration spectrum of the set is above the design spectrum for Vancouver within the period range between 0.2 s and 4.0 s. The value of 0.2 s corresponds approximately to $0.2T_1$(where T_1 = 0.94 s is the first mode period of the 4S frame), and the value of 4.0 s is close to $1.5T_1$ (where T_1 = 2.75 s is the first mode period of the 16S frame). There are two reasons that this period range was selected. First, this range satisfies the ASCE (2010) requirement for the spectral compatibility range (i.e., between $0.2T_1$ and $1.5T_1$) for each frame individually and for all three frames together. Second, it was selected so that the same intensity of the records is used in the analysis of all three frames.

3.1 Scaled Real Accelerograms

Ground motion records from earthquakes in the Vancouver region would be the most suitable records for this study. Since such records are not available, records from earthquakes in California were selected. This is because characteristics of ground motions in California are very similar to those in Vancouver (G.M. Atkinson, personal communication 2009). Therefore, a set of 10 records was selected from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center (Table 2). The records are obtained at sites corresponding to the NBCC soil class C which was assumed in the design of the buildings. As seen in Table 2, the records are obtained from 4 earthquakes. The magnitudes of the earthquakes are between 6.9 and 7.3, and the distances are between 41 km and 88.5 km. Both the magnitude and distance ranges cover the dominant magnitudes and distances of earthquakes that have the largest contributions to the seismic hazard for Vancouver as reported in Halchuk et al. (2007).

Table 2. Selected earthquake records from the PEER database.

Record No.	Record name*	Earthquake and date	Magnitude	Distance (km)
1	BAK140	Landers, 1992/06/28	7.3	88.5
2	FTI090	Landers, 1992/06/28	7.3	64.2
3	A3E090	Loma Prieta, 1989/10/18	6.9	57.1
4	DMH090	Loma Prieta, 1989/10/18	6.9	77.0
5	B-RDL270	Trinidad, CA, 1980/11/08	7.2	71.9
6	PLC090	Landers, 1992/06/28	7.3	95.9
7	TAF021	Kern County, 1952/07/21	7.4	41.0
8	SIL090	Landers, 1992/06/28	7.3	51.7
9	A10000	Loma Prieta, 1989/10/18	6.9	47.8
10	ABY090	Landers, 1992/06/28	7.3	69.2

*Record designation in PEER database.

To achieve spectral compatibility with the design spectrum, the records were scaled in two steps. In the first step, each record was scaled such that the area of the 5% damped acceleration spectrum of the record within the period range between 0.2 s and 4.0 s is equal to the area under the design spectrum within the same period range using the method proposed by Amiri-Hormozaki (2003). In the second step, additional scaling was conducted by multiplying the accelerograms already scaled in the first step by a factor of 1.15 so that the mean spectrum of the set of records is above the design spectrum as required by ASCE (2010). The response spectra of the scaled records and the design spectrum for Vancouver are shown in Figure. 3(a).

3.2 Modified Real Accelerograms

A method described by Naumoski (2001) was used for the generation of spectrum-compatible accelerograms. In this method an initial accelerogram is modified iteratively until its spectrum matches the prescribed design spectrum. It is necessary to mention that the initial accelerogram can be any ground motion record (real or synthetic). In this study, the originally selected earthquake records for Set 1 listed in Table 2 were used as initial accelerograms. The spectrum to be matched was the design spectrum for Vancouver for soil class C. In order to raise the mean spectrum to be above the design spectrum, the accelerograms generated as described above were scaled by a factor of 1.05. Figure 3(b) shows the spectra of the accelerograms generated for use in this study.

3.3 Simulated Accelerograms

Since real records from larger earthquakes (e.g. magnitude larger than 6) in Canada are not available, comprehensive database with sets of simulated accelerograms for eastern Canada and western Canada compatible with the 2005 NBCC uniform hazard spectra was established by Atkinson (2009). A stochastic finite-fault method is used for generation of accelerograms. In this method, a large fault is divided into a number of subfaults and each sub fault is considered as a point source. Ground motions of the point sources are simulated using stochastic point-source approach. This simulation is based on a specified Fourier spectrum of ground motion as a function of magnitude and distance. According to this method, the accelerograms for the point (i.e., subfault) sources along the fault are simulated first, then they are summed in the time domain to obtain the ground motion from the entire fault. A set of 10 simulated accelerograms for the ground motions for western Canada was selected from the database. The selected accelerograms correspond to magnitude of 7.5, distances ranging from 47 km to 100 km, and soil class C. The accelerograms were scaled in such a way that the mean spectrum of the set is above the design spectrum (Fig. 3(c)).

3.4 Artificial Accelerograms

Artificial accelerograms compatible with the design spectrum for Vancouver were generated using the method proposed by Gasparini and Vanmarcke (1976), incorporated in the computer program

SIMQKE. This method is based on the well-known principle that each accelerogram can be represented as a sum of sinusoids. The frequencies of the sinusoids are within the frequency range of the design spectrum, the phase angles of the sinusoids are produced using random number generation software, and the amplitudes of the sinusoids are determined from the spectrum density function, which is derived based on the design spectrum. Then, an accelerogram is obtained by summation of the sinusoids. The accelerogram is multiplied by a specified shape function in order to simulate the shape of a real earthquake motion. The response spectrum of the resulting accelerogram is computed and compared with the design spectrum. The iterative process continues until the spectrum of the accelerogram is close to the design spectrum. Using this method, a set of 10 artificial accelerograms was generated. The spectra of the artificial accelerograms and the design spectrum are shown in Fig. 3(d).

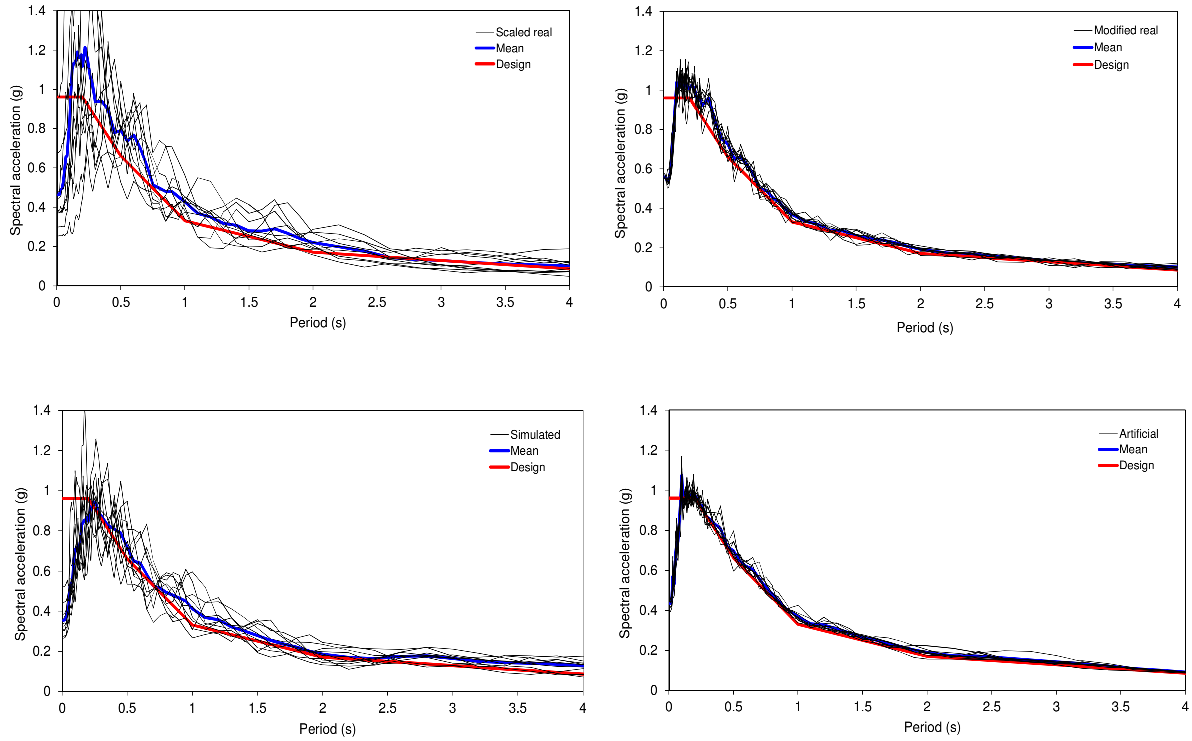

Figure 3. Response spectra (5% damping) of the sets of accelerograms used in this study:
(a) Set 1 - scaled real accelerograms, (b) Set 2 - modified real accelerograms,
(c) Set 3 - simulated accelerograms, and (d) Set 4 - artificial accelerograms.

4. DISCUSION OF RESULTS

Nonlinear time-history analyses were conducted by subjecting the 4S, 10S, and 16S frames to the selected sets of accelerograms. Among a number of response parameters resulting from the analyses, storey shears, interstorey drifts, and curvature ductilities at the end sections of beams and columns were used in this study. Given the large number of time-history analyses, and the large number of structural members in the frames, the responses resulting from the sets of excitations were statistically analysed to compute the mean (M) and the mean plus one standard deviation (M + SD) values. For each set of excitations, mean and M + SD values for shear forces and drifts were computed for each storey. The computation of the mean and M + SD curvature ductilities for the columns of each storey was done by considering only the largest ductility from all bottom and top sections of the columns of the storey, resulting from each excitation in the set. Similarly, the mean and M + SD curvature ductilities for the beams at each floor were computed by using the largest ductility from all end sections of the beams of the floor, obtained from each excitation in the set. Since the observations of the results of the three frames are very similar, only the results of the 10S frame are presented in this paper.

The results from the dynamic analysis of the 10S frame are presented in Figs. 4 to 7 in which Fig. 4 shows the results for interstorey drifts, and Figs. 5 and 6 show the curvature ductilities for beams and columns, respectively. The horizontal bars in these figures represent the mean (M) response values for each storey, and the line extensions to the bars show the M + SD response values. Figure 7 shows the mean (M) shear forces at each storey (referred to as storey shears in the further discussion). Note that the M + SD shears are not included in Fig. 7 because these are relatively close to the mean values.

It can be seen from Fig. 4 that the interstorey drifts are well below the code limit of 2.5%, which is due to the conservatism in the design. The maximum M + SD interstorey drift is 1.3% at the fifth storey. Figure 5 shows that the largest mean and M + SD ductilities for the beams are 3.35 and 3.94, respectively, and both are for the beams at the eighth storey. It is also shown in Fig. 5 that the ductilities for all the beams are larger than 1.0. This indicates that all four sets of excitations produce nonlinear deformations (i.e., yielding) in all the beams. Regarding columns, the largest mean ductility of 1.38 and M + SD of 1.60 are observed for the columns at the top storey only (Fig. 6). The column ductilities for all other storeys are smaller than 1.0 indicating elastic behaviour of the columns from all four sets of excitations. In summary, the curvature ductilities for the beams and columns show that the seismic behaviour of the 10S frame is as expected for a ductile frame designed according to the capacity method, i.e., all inelastic deformations to be in the beams, and the columns to remain in the elastic range. Certain inelastic deformations in the top storey columns as indicated by the observed column ductilities in Fig. 6 are allowed in the capacity method (Paulay and Priestley 1992).

The results for the shear forces (Fig. 7) show that all four sets of excitations produce very close storey shears. Also, it was found that the dispersions of the shears around the mean values (i.e., the standard deviations, SD) are very small. The differences between the largest and the smallest shear values (expressed as percentages of the smallest mean values) from all the sets are 2% to 8% for both the mean and the M + SD storey shears. Such results are not surprising considering the observations for the ductilities discussed above. Namely, since the beams at the floor levels experience yielding from the majority of the excitations (Fig. 5), the variations of the maximum forces in the beams (and consequently in the columns) during the nonlinear response are very small because of the small post-yield stiffness of the beams (Fig. 2).

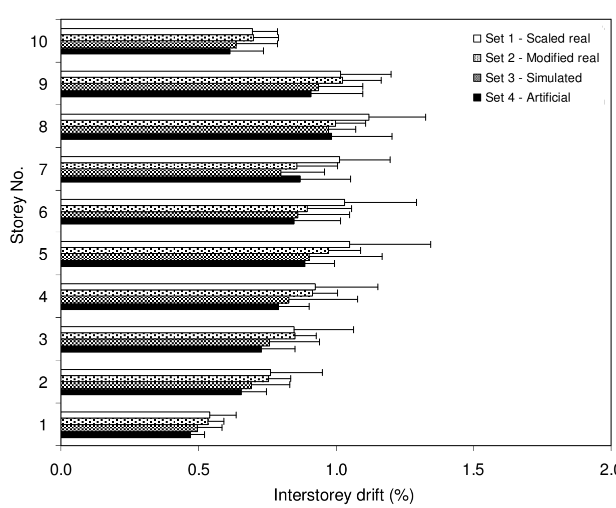

Figure 4. Interstorey drifts for the 10S frame.

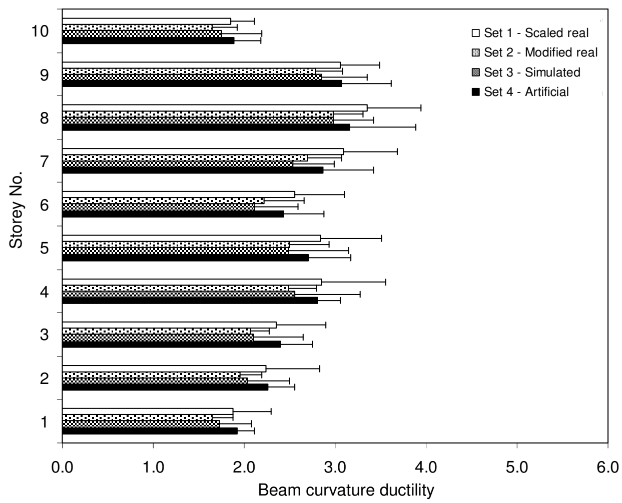

Figure 5. Beam ductilities for the 10S frame.

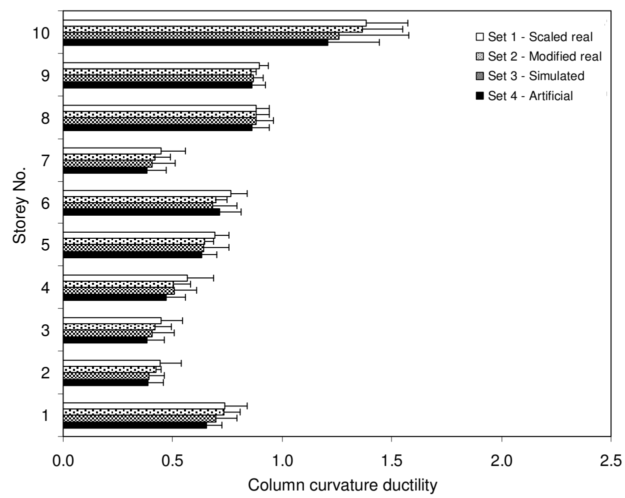

Figure 6. Column ductilities for the 10S frame.

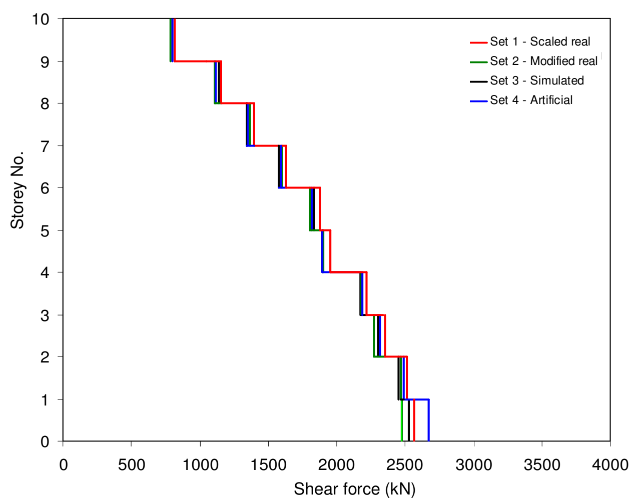

Figure 7. Shear forces for the 10S frame.

For ease of understanding the effects of sets of records on the values of responses, Table 3 shows the ranges of the differences between the largest and the smallest mean response values of the deformations (expressed as percentages of the smallest mean values) from the four sets of records considered in this study. The designation of the set of records which provided the largest responses is also shown in the table. As seen in Table 3, the maximum differences in the deformation responses resulting from the sets of records increase with the height of the building. For example, the maximum difference for the interstorey drifts is 14% for the 4S frame, 26% for the 10S frame, and 43% for the 16S frame. Based on the analysis results from all three frames, it was found that the largest responses of the frames correspond to either Set 1(scaled real records) or Set 3 (simulated accelerograms).

Table 3. Differences between largest and smallest responses from sets of records.

Frame model	Interstorey drift	Beam ductility	Column ductility	Shear force	Set No.*
4S	4% - 12%	5% - 14%	4% - 10%	2% - 8%	Set 1 or Set 3
10S	13% - 26%	10% - 22%	3% - 21%	2% - 8%	Set 1 or Set 3
16S	10% - 43%	10% - 43%	5% - 23%	1% - 8%	Set 3

*Set No. corresponds to the set of records that provides the largest responses.

5. CONCLUSIONS

The National Building Code of Canada, as well as other modern building codes around the world, allows the use of time-history analysis in the design of buildings located in regions of high seismicity and for buildings that are higher than specified height levels. The codes require the seismic excitations used in the analysis to be compatible with the design spectrum for the building location. The selection of spectrum-compatible accelerograms is one of the major issues for the time-history analysis of buildings. The objective of this study is to determine the implications of different methods for the selection of accelerograms in the estimation of the structural responses of buildings. Given this, four sets of accelerograms compatible with the design spectrum for Vancouver were selected for use in the analysis. They are, Set 1-scaled real accelerograms, Set 2-modified real accelerograms, Set 3-simulated accelerograms, and Set 4-artificial accelerograms. Each set consists of 10 accelerograms. Nonlinear time-history analyses were conducted on three reinforced concrete moment-resisting frame buildings (i.e., 4-storey, 10-storey, and 16-storey), designed for Vancouver. Each building was subjected to the selected sets of accelerograms. Interstorey drifts, curvature ductilities in beams and columns, and storey shear forces were used in the evaluation of the effects of the selected excitations.

It was found in this study that among the four sets of accelerograms, the spectra of the set with scaled real accelerograms (Set 1) and that with simulated accelerograms (Set 3) show the largest dispersion around the mean spectra of the sets. It was also found that the largest responses for interstorey drifts and curvature ductilities are either from scaled real accelerograms (Set 1) or from the simulated accelerograms (Set 3). The results in the study also show that the maximum differences between the largest and the smallest *mean* values of the deformation responses from the four sets increase with the height of the buildings. However, the results for the storey shear forces from the four sets of excitations are quite close for all three buildings. The maximum differences between the largest and the smallest mean storey shears from the sets are only about 8% which is negligible from the practical point of view.

Based on the considerations of both the spectral characteristics of the selected sets and the response results from the analysis, it can be concluded that scaled real accelerograms are preferred for use in time-history analyses of buildings. The accelerograms should be selected from earthquakes occurred in regions with similar seismological characteristics to those of the location considered, and properly scaled such that their spectra are close to the design spectrum. If such accelerograms are not available, then simulated accelerograms should be used. The advantage of using simulated accelerograms is the methods for the generation of such accelerograms take into account certain seismological characteristics of the region considered.

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