EFFECTS OF SCALING OF EARTHQUAKE EXCITATIONS ON THE DYNAMIC RESPONSE OF REINFORCED CONCRETE FRAME BUILDINGS

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SUMMARY

The effects of the scaling of earthquake excitations were investigated by analysing the nonlinear dynamic responses of three reinforced concrete frame buildings. This paper presents results for one of the buildings. It is a six-storey ductile moment-resisting frame building located in Vancouver. Artificial accelerograms compatible with the design spectrum, simulated stochastic accelerograms, and real accelerograms scaled with respect to the design spectrum according to three types of scaling were used as excitation motions. The responses of the building resulting from the artificial accelerograms compatible with the design spectrum, and from the scaled real records were found to be quite similar. The responses from the majority of the simulated stochastic accelerograms were much larger than those from the other excitations. Based on the results of this study, it is recommended that the analyses in the design of irregular reinforced concrete buildings be conducted using either artificial accelerograms compatible with the design spectrum, or real accelerograms scaled according to any method such that the 5% damped spectra of these accelerograms are close to the design spectrum over the period range of interest for the structure.

INTRODUCTION

The seismic provisions of the latest edition of the National Building Code of Canada, that should be in use from 2005 (NBCC 2005), specify seismic design spectra for use in the design of buildings. The seismic forces are directly related to the design spectra in the equivalent static method and the spectral analysis method. In addition to these two methods, the NBCC 2005 seismic provisions require the use of dynamic time-history analysis in the design of more important and irregular buildings. For the purpose of dynamic time-history analysis, acceleration time-histories of the ground motions are needed. The code explicitly requires that the design acceleration time-histories of the ground motions (referred to as design accelerograms) should be compatible with the design spectrum. However, the term “compatibility” is not defined by the code. Also, the code does not specify what type of design accelerograms should be used,

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whether recorded earthquake accelerograms (referred to as real accelerograms) or generated artificial accelerograms that satisfy the design spectrum.

A comprehensive study on the effects of scaling of earthquake excitations on the response of reinforced concrete frame buildings was conducted at the University of Ottawa. Nonlinear time-history analyses were performed on three moment-resisting frame buildings: (i) six-storey ductile frame building in Vancouver, (ii) six-storey nominally ductile frame building in Vancouver, and (iii) five-storey nominally ductile frame building in Montreal. Each building was subjected to ensembles of artificial and real accelerograms. The real accelerograms were scaled using three types of scaling with respect to the design spectrum. Because of the space limitation, this paper describes the results of only the ductile moment-resisting frame building in Vancouver. The results for the other two buildings are given in Amiri-Hormozaki [1].

**DESCRIPTION OF BUILDING AND DESIGN PARAMETERS**

The building is a six-storey moment-resisting reinforced concrete frame structure located in Vancouver, B.C. The plan of the floors and the elevation of the transverse frames are shown in Fig. 1. The storey heights are 4.0 m with the exception of the first storey which has a height of 5.2 m. For the transverse direction, the two end moment-resisting frames (marked T in the figure) provide all of the seismic resistance, with the other columns in the building carrying only gravity loads. The design of the transverse frames is dominated by lateral loads, since each frame carries one-half of the lateral load of the entire building while carrying only the gravity load of the adjacent half-bay. For the longitudinal direction, the earthquake resistance is shared among six moment-resisting frames (marked L1, L2, and L3 in the figure). With this configuration of structural systems, the design of the longitudinal frames is governed by both gravity and lateral loads. The floor system consists of a one-way slab spanning in the transverse direction, supported by the beams in the longitudinal frames; the slab is cast integrally with the beams.

![Fig. 1. Plan of floors and elevation of transverse frames of building](image)

In this study, only the transverse frames were considered. This is primarily because, as mentioned above, the design of the transverse frames is dominated by lateral seismic loads. The cross sections of all the columns of the transverse frames are 90 cm x 90 cm, and those of the beams are 50 cm x 110 cm. The reinforcement in the columns and beams of the frames is given in Naumoski [2].

The building was designed as an office building located on rock. The design was conducted according to NBCC 1995. The transverse frames were designed as ductile moment-resisting frames using force
reduction factor $R = 4$. Fundamental period of 0.6 s (i.e., 0.1 times the number of storeys, as required by the code) was used for determining the design base shear. The resulting base shear was 5.8% of the total weight of the building.

**MODELLING OF FRAME MEMBERS**

For the purpose of determining the performance of the frame when subjected to earthquake ground motions, inelastic model of the frame members were developed for use in the inelastic dynamic analysis program IDARC (Kunnath [3]). Moment-curvature relationships for the end sections of each beam and column were determined using fibre analysis of the cross sections. The concrete stress-strain relations included the effect of confinement, based on the model proposed by Mander [4]. The moment-curvature relationships were simplified into a trilinear model with the first segment corresponding to the uncracked stiffness, the second segment corresponding to the region between cracking and yielding, and the third segment to the post-yielding range. Stiffness degradation and pinching effects were taken into account in the analysis using a hysteretic model specifically developed for reinforced concrete, which closely approximates experimentally observed behaviour. A detailed description of the modelling process and the selection of the model parameters is given in Naumoski [5].

**SEISMIC EXCITATIONS**

Six different types of excitation motions were used in the analysis of the frame. This includes three ensembles of artificial accelerograms, and three ensembles of real accelerograms obtained by different types of scaling. A brief description of each of these ensembles is given hereafter.

**Artificial accelerograms compatible with the design spectrum**

An ensemble of 15 artificial accelerograms was generated using the computer program SYNTH (Naumoski [6]). Each accelerogram is compatible with the design spectrum for Vancouver as specified in NBCC 2005. To generate an artificial accelerogram, the program requires the target (i.e. design) spectrum, and an initial accelerogram. The program iteratively modifies the Fourier amplitudes of the initial accelerogram until its spectrum matches the design spectrum. Figure 2 shows the 5% damped response spectrum of one of the generated artificial accelerograms and the design spectrum for Vancouver. The excellent matching of the spectra is obvious from the figure.

**Simulated stochastic accelerograms**

The method for the simulation of stochastic accelerograms is described in Tremblay and Atkinson [7]. Using this approach, Tremblay and Atkinson [7] simulated accelerograms corresponding to the median uniform hazard spectrum (UHS) for Vancouver for probability of exceedance of 2% in 50 years. Note that the NBCC 2005 design spectrum for Vancouver is the same as the median UHS. Simulations are conducted for two magnitude (M)-distance (R) scenarios, i.e. $M=6.5$, $R=30$ km, and $M=7.2$, $R=70$ km. These two M-R scenarios are considered to dominate the seismic hazard for Vancouver. Four accelerograms are simulated for $M=6.5$, $R=30$ km, and four accelerograms for $M=7.2$, $R=70$ km. Figure 3 shows the 5% damped acceleration response spectra of the simulated accelerograms and the NBCC 2005 design spectrum for Vancouver.
Fig. 2. Response spectrum of generated artificial accelerogram and design spectrum for Vancouver

Fig. 3. Response spectra of simulated stochastic accelerograms and design spectrum for Vancouver

**Real accelerograms**

Because no records from strong earthquakes from the Vancouver region are available, an ensemble of 15 recorded accelerograms from earthquakes around the world (referred to as real accelerograms) were used in the analysis of the building. The shapes of the spectra of these accelerograms are similar to those of seismic ground motions expected in Vancouver. The spectral shape of a seismic motion is related to the peak ground acceleration to peak ground velocity ratio (A/V ratio) of the motion. Based on the seismic zoning maps of NBCC 1995, the A/V ratios of approximately 1.0 (A in g, V in m/s) are characteristic for
The selected ensemble (Naumoski [8]) has an average A/V of 1.02, with the values for individual records ranging from 0.82 to 1.21.

The selected records were scaled to represent the level of the expected seismic motions for Vancouver. The following three types of scaling were used: (i) based on spectral acceleration ordinates, (ii) based on partial area under the acceleration spectrum, and (iii) based on full area under the acceleration spectrum. In the first method, the accelerograms were scaled in such a way that the 5% damped acceleration spectra of the scaled accelerograms have the same ordinates as that of the design spectrum at the fundamental period of the structure of 1.16 s. In the second method (i.e. based on partial area), the area under the acceleration spectrum of each of the real accelerograms and that of the design spectrum between the second mode period and 1.2 times the fundamental period are the same. This period range of the excitation motions was assumed to have the largest effects on the structural response. The factor of 1.2 was intended to take into account the elongation of the fundamental structural period because of nonlinear deformations during the response. In the third method (i.e. based on full area), the area under the acceleration spectrum of each of the real accelerograms and that of the design spectrum within the entire period range (i.e., between periods of 0.02 s and 2.0 s) are the same.

Figure 4 shows the 5% damped mean spectra of the scaled ensembles of real records and the design spectrum for Vancouver. The designation of the curves in the figure is as follows: RV-SO - real records for Vancouver scaled to spectral ordinates, RV-PA – real records scaled to partial area under the spectral curves, and RV-FA – real records scaled to full area under the spectral curves. It can be seen from this figure that all three types of scaling provide relatively close mean spectra.

![Fig. 4. Mean response spectra for scaled real records and design spectrum for Vancouver](image)

**Levels of excitation**

The generated artificial accelerograms and the scaled real accelerograms as described above are assumed to represent the level of protection as required by NBCC 2005. This level is referred to as "design" level of excitation. However, Heidebrecht [9] has noted that earthquake motions as large as two to three times as those associated with the uniform hazard spectra (i.e. design spectra in NBCC 2005) can happen during
the life time of the buildings. Given this, a level of excitation that is twice the design level was also used in the analyses, and is referred to as "double" level of excitation.

ANALYSIS AND RESULTS

Analysis and response parameters
Nonlinear time-history analyses were conducted by subjecting the building to each of the accelerograms of the ensembles. The computer program IDARC (Kunnath [3]) was used for the analyses. The response parameters that were considered in this study were the interstorey drifts and curvature ductilities of the end sections of the beams and columns of the building.

For the purpose of the evaluation of the effects of the excitation ensembles on the response of the building, the response parameters resulting from the ensembles containing 15 accelerograms were statistically analyzed to compute the mean (M) and mean plus one standard deviation (M+SD) values. For each of these ensembles, mean and M+SD values of the maximum interstorey drifts were computed for each storey. The amount of data for ductility demands was much larger than that for interstorey drifts. Namely, there were 12 mean values for columns and 10 mean values for beams, and the same number of M+SD values for each storey. Therefore, for a given storey, only the maximum values of the mean and the M+SD curvature ductilities for the columns were used. Similarly, for a given floor, only the largest mean and M+SD curvature ductilities for the beams were considered.

No statistical analyses were conducted for the response parameters resulting from the simulated stochastic ensembles. This was because each of these ensembles contained only 4 accelerograms, which are not sufficient for statistical analysis. The maximum interstorey drifts and the curvature ductility demands associated with each accelerometer of the simulated stochastic ensembles were considered separately.

Discussion of results
The results from the dynamic analysis are shown in Fig. 5 to Fig. 7 for the design excitation level, and in Fig. 8 to Fig. 10 for the double excitation level. For each ensemble of 15 accelerograms, the right-hand end of the horizontal bar corresponds to the mean value for each storey. The bar is then continued further to the right by a thin line. The right-hand end of the thin line corresponds to the M+SD value. For each of the response parameters, the name of the ensemble of the excitation motions is also given in the figures. The abbreviations of the ensemble names indicate to the following: M6.5R30 – simulated stochastic accelerograms for magnitude of 6.5 and distance of 30 km, M7.2R70 – simulated stochastic accelerograms for magnitude of 7.2 and distance of 70 km, RV-FA – real accelerograms for Vancouver scaled to full area under the spectral curves, RV-PA – real accelerograms scaled to partial area under the spectral curves, RV-SO – real accelerograms scaled to spectral ordinates, and ADCV – artificial design spectrum compatible accelerograms for Vancouver.

Results for design level of excitation
The results from the analysis for the design level of excitation are shown in Fig. 5 to Fig. 7. Considering the design spectrum compatible ensemble ADCV as a reference case, the figures show that the real records RV scaled either to spectral ordinate (RV-SO) or to partial area (RV-PA) give responses that are quite close to those of the reference case, especially for the mean values. The ratios of the maximum to the minimum mean response values for the ADCV, RV-SO and RV-PA ensembles are between 1.02 and 1.14 for interstorey drifts, between 1.04 and 1.28 for beam ductilities, and between 1.06 and 1.28 for column ductilities. In some cases, the results from the ensemble of real records scaled to full area (RV-FA) are slightly larger than those from the other ensembles. This observation can be explained by considering the spectra shown in Fig. 4. This figure shows that the mean response spectrum for the scaling to full area is somewhat higher than the spectra from the scaling to spectral ordinate or partial area. On the other hand,
the spectra from the scaling to spectral ordinate and partial area are quite close to the design spectrum for Vancouver in the neighbourhood of the fundamental period of the building (i.e., 1.16 s).

The responses due to the simulated stochastic accelerograms show quite different features. For the M6.5R30 ensemble, all four response values are significantly larger than even the M+SD values of the artificial (ADCV) and the scaled real ensembles except for the two top storeys. This can be explained by considering Fig. 3. As shown in this figure, the M6.5R30 response spectra are much higher than the design spectrum, especially in the neighbourhood of the fundamental period of the building.

Given the foregoing observations, the further discussion is focussed on the results from the ADCV, RV-SO, and RV-PA ensembles, which provide similar responses. The drifts resulting from these ensembles are well below the code limit of 2.5%. The maximum M+SD interstorey drift is 1.17% at the fifth storey. The maximum M+SD beam ductility is 5.2, and the maximum mean value is 3.61, both happening at the sixth floor. Except for the fifth storey, where the maximum M+SD column ductility is 3.01, the ductilities of the columns at the other storeys are mostly below 1.0 (i.e., no yielding occurs in the columns). In general, these curvature ductilities are not significant, since members of well designed moment-resisting frames can sustain curvature ductilities of 10 to 20 (Heidebrecht [10]).

Since the frame was designed as a ductile frame, one would expect much larger ductility demands in both the columns and beams. There are two major reasons for obtaining such small ductility values. First, conservative assumptions are involved in the design of the structural members of the frame through the use of material resistance factors of 0.6 and 0.85 for concrete and reinforcing steel respectively. However, the most important reason is associated with the period used in the design of the building. As mentioned earlier, the building was designed for an estimated fundamental period of 0.6 s, i.e. 0.1 times the number of storeys, as required by the code. On the other hand, the actual fundamental period of the building is 1.16 s, for which the seismic demand is much smaller than that for the design period of 0.6 s.

**Results for double level of excitation**

The results for the double level of excitation are shown in Fig. 8 to Fig. 10. At first glance, the trend of the results from the double level of the different excitation ensembles follows the trend of the results from the design level of the ensembles. Thus, responses from the ADCV, the RV-SO, and the RV-PA ensembles are relatively comparable. The ratios of the maximum to the minimum values of the mean responses of these ensembles are between 1.02 and 1.20 for interstorey drifts, between 1.07 and 1.30 for beam curvature ductilities, and between 1.05 and 1.38 for column ductilities. The responses resulting from the RV-FA ensemble are generally slightly larger than those from the ADCV, RV-SO, and RV-PA ensembles, as was the case for the design level of excitation. Also, the majority of the responses from the simulated stochastic accelerograms are larger than those from the other four ensembles (i.e., ADCV, RV-SO, RV-PA, and RV-FA), as was observed from the design level of excitation. The reasons for obtaining larger values from the RV-FA and the simulated stochastic ensembles were discussed above in this section. Given these differences, the further discussion is associated with the results from the ADCV, RV-SO, and RV-PA ensembles.

The mean values of interstorey drifts from these three ensembles are mostly below the code limit of 2.5%, with the exception of the fifth storey where the drift is 2.97%. The maximum value of the M+SD interstorey drift is 4.41%, which is also at the fifth storey. The maximum value of the beam curvature ductilities at the mean and M+SD levels are 11.84 and 18.09 respectively. In terms of the column ductilities, the maximum values at the mean and M+SD deviation levels are 6.54 and 9.95 respectively; both of these values are for the columns of the fifth storey. The column ductilities for the other storeys are much smaller, i.e., below 2.38 at the mean level and below 4.84 at the M+SD level. These ductilities are
considered acceptable since well designed ductile members can sustain curvature ductilities as much as about 20 (Heidebrecht [10]).

**DISCUSSION AND CONCLUSIONS**

A comprehensive study was conducted to investigate the effects of the scaling of earthquake excitations on the response of reinforced concrete buildings. The results presented in this paper are for one of the three buildings analysed in the study. It is a six-storey ductile moment-resisting frame building located in Vancouver. The results for this building are representative of those of the other two buildings. Nonlinear dynamic time-history analyses were conducted by subjecting the building to ensembles of artificial and real accelerograms. The artificial accelerograms included generated accelerograms compatible with the design spectrum for Vancouver, and simulated stochastic accelerograms. The real accelerograms were scaled with respect to the design spectrum using the following three types of scaling: based on spectral ordinates at the fundamental structural period, based on the partial area under the spectra (between the period of the second mode and 1.2 times the fundamental structural period), and based on the full area under the spectra (between periods of 0.02 and 2.0 s). The evaluation of the scaling methods was conducted by analysing the interstorey drifts, and curvature ductility demands in the beams and columns of the building.

The responses of the building resulting from the artificial accelerograms compatible with the design spectrum, and from the real accelerograms scaled to spectral ordinates and partial area under the spectra were found to be quite similar. The responses from the majority of the simulated stochastic accelerograms were much larger than those from the other ensembles. This was not surprising given the significant differences between the spectra of the stochastic accelerograms and the design spectrum; the spectra of some of the stochastic accelerograms are as much as two times higher than the design spectrum in the intermediate and long period ranges.

Based on the results of this study, it is recommended that the analyses in the design of irregular reinforced concrete buildings be conducted using either artificial accelerograms compatible with the design spectrum, or real accelerograms scaled according to any method such that the 5% damped spectra of these accelerograms are close to the design spectrum over the period range of interest for the structure. The duration of the strong motion shaking of the excitations should correspond to that of the expected seismic motions at the location.

**REFERENCES**


Fig. 5. Interstorey drifts for design level of excitation
Fig. 6. Beam curvature ductilities for design level of excitation
Fig. 7. Column curvature ductilities for design level of excitation
Fig. 8. Interstorey drifts for double level of excitation
Fig. 9. Beam curvature ductilities for double level of excitation
Fig. 10. Column curvature ductilities for double level of excitation