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SEISMIC DESIGN AND PERFORMANCE EVALUATION OF A TWENTY-STOREY STEEL-FRAME BUILDING BASED ON NBCC-2005

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ABSTRACT

The new edition (2005) of the National Building Code of Canada (NBCC) uses a seismic hazard level corresponding to 2% probability of being exceeded in 50 years and the site specific uniform hazard spectra to calculate the design lateral forces, in order to provide a uniform level of seismic protection to buildings across Canada. This paper aims at determining the seismic level of protection for a twenty-story steel frame building to identify the scope for integrating the performance-based design philosophy to the code-based design process. The preliminary design of the building based on the equivalent static loads method has been modified through a number of iterations to incorporate the modal and dynamic analysis results, as recommended by the building code. The seismic performance of the building has been evaluated for spectrum compatible synthetic records as well as scaled ground motion records. The building is assumed to be located in Vancouver. The static capacity of a structure, as well as the dynamic response parameters, such as the inter-story drift, and the overall drift are used in evaluating the seismic performance. The objective of this paper is to determine if the new code specifications offer the desired level of performance under different levels of earthquake hazard and suggest modification to the design accordingly. It has been observed that the building meets the life-safety performance criteria under the 2500 year earthquake events.

Introduction

Earthquakes are associated with the shaking of the ground mass or surface, causing severe damage to infrastructure including loss of life. The casualties during an earthquake result mainly from the failure of man made infrastructure like buildings, bridges, dams, transportation infrastructure etc. Therefore, a clear need exists for providing safe structures so as to minimize the damage caused by an earthquake. Current building codes provide a force-based approach to the design of a building with a specified lateral load. The performance of the structure is indirectly controlled by limiting the inter-story drift. However, force-based design method may not be adequate to ensure a uniform performance level for structures subjected to seismic action. The damage caused by the earthquakes of Northridge, California (1994) and Kobe, Japan (1995) exposed the weakness of the code guided force-based seismic design. During these earthquakes more than 150 steel moment-resisting frame buildings collapsed although all those buildings were designed by fulfilling the code requirements, but without evaluation of the performance (Lee and Foutch 2002). The buildings were designed for equivalent static loads, but their performances under dynamic

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loading, such as ground shaking, were unknown. After these earthquakes, the necessity for the evaluation of performance of buildings became more apparent. The current version of the National Building Code of Canada (NBCC 2005) recommends dynamic analysis to ensure that the building structures achieve an acceptable level of seismic performance.

The performance of a building during an earthquake depends on many factors including the structure's configuration and proportions, its dynamic characteristics, the hysteretic behavior of the elements and joints, the type of nonstructural components employed, the quality of the materials and workmanship, adequacy of maintenance, the site conditions, and the intensity and dynamic characteristics of the earthquake ground motion experienced (Yun *et al.* 2002). In the seismic design, or in evaluation of performance of the building, all of the above mentioned factors should be considered. The definition of performance of a structure is a multi-objective concept. Performance objectives are the statements of acceptable performance of a structure (Ghobarah, 2001). Therefore, any response parameter such as the inter-story drift, peak roof displacement, lateral load capacity and residual inter-story drift, can be specified or targeted as the performance objective. In FEMA-273 (1997) both the peak and residual inter-story drifts are utilized in defining performance levels as indicators of damage. The lateral load capacity of a structure can also be defined as a performance parameter. The evaluation of performance is a reliability-based probabilistic approach (FEMA-350 2000) because of the uncertainties involved in the judgment and prediction of the characteristics of the earthquake parameters. The level of confidence, which comes from the knowledge of assessment of uncertainties, is very important for ensuring the level of performance. The level of confidence determines whether or not the structure will be able to meet the desired level of performance. The two main levels of performance as defined in FEMA-273 (1997) are Immediate Occupancy (IO) and Collapse Prevention (CP) and these performance levels are associated with the drift levels of 0.7% and 5% for IO and CP, respectively. In evaluation of the seismic performance of buildings, first it is necessary to design the structure by fulfilling code requirements followed by a set of rigorous analysis, both static and dynamic.

In Canada the western region is more sensitive to earthquakes than the eastern region because of the matrix of the rock in that region. The repetitive seismic ground shaking in the western Canada in the past made the rock formation of that region more vulnerable to earthquakes than the rest of the country. According to Foo *et al* (2001), the February 28, 2001 earthquake near Seattle, which rattled buildings and occupants in Vancouver, could be viewed as a reminder to people living in Canada's most active seismic zone, the pacific coast. It has also been reported by Foo *et al* (2001) that the earthquake that occurred at Saguenay in Quebec in 1988 was the strongest event in the eastern North America within the last 50 years. But Canada has a record of suffering from even stronger earthquake, for example the one that occurred in 1949 with a magnitude of 8.1. An average of 1500 earthquakes (NRCAN, 2006) occurs in Canada every year. Thus, designing to resist earthquakes and ensuring the required level of performance, are both important for structures located in different regions of Canada. The objective of this paper is to present the seismic design of a twenty story steel-frame building using the provisions of NBCC 2005, and evaluation of its seismic performance. Both bare and infilled frames have been considered in the performance evaluation. The infill panels have been considered to account for the effect of non-structural elements in the buildings.

Design Issues in Earthquake Engineering

The main objective of the seismic design is to construct a suitable structure that is safe against collapse due to earthquake, sustains limited damage to non-structural components and no damage to human life. In the provisions of most building codes including NBCC 2005, the design base shear induced by earthquakes is reduced from the elastic base shear to account for the ductility of the structural members i.e. the capacity to deform beyond the yield point without major structural failure (FEMA 310, 1998). The ductility of the structure is an important factor in seismic design. According to FEMA-350 (2000) the ductility of the steel-moment frames is generated through yielding and the resulting development of plastic hinges in beam-column assemblies at the beam-column connections. In FEMA-310 (1998) the fundamental requirements for all ductile moment resisting frame have been identified as follows: a) all frames should have sufficient strength to resist seismic demands, b) they have sufficient stiffness to limit inter-story drift, c) beam-column joints have the ductility to sustain the rotations they are subjected to, d) elements should be able to form plastic hinges, e) hinges should be developed in the beams before the columns at locations distributed throughout the structure.

Seismic loading provisions in most existing building codes focus on the minimum lateral seismic forces for which the building must be designed. But only specifying the lateral load is not enough to ensure that the building will perform at the desired level of performance. A structure designed to withstand the cyclic seismic force must be properly configured with accurate continuity including adequate strength and stiffness. One of the important concepts in the seismic design of buildings is the strong column/weak beam concept; it means that at a beam-column joint, the beams will yield before the columns. This is done to prevent the brittle failure of the building frame. The percentage of strong column/weak beam at the joints in each story of each line of moment resisting frames should be greater than 50% for life Safety and 75% for Immediate Occupancy (FEMA-310, 1998). In designing of the steel moment resisting frame, design of connections is very important. The seismic design of structure is mainly a capacity-based design, in which the elements of the structure are designed to dissipate energy under deformations caused by an earthquake. In the capacity-based design some zones in a member are chosen for inelastic response and the members are designed in such a manner that they are capable of developing large plastic deformation without significant loss of strength. The capacity of other members must be greater than the capacity of the members participating in the plastic deformation.

Background of NBCC 2005

Minimum requirements for earthquake resistant design are provided in NBCC 2005 to ensure an acceptable level of performance and safety of buildings in Canada. According to NBCC (2005) the minimum lateral earthquake force, *V*, is calculated by using the following equation:

$$V = \frac{S(T_a) M_V I_E W}{R_d R_0} \geq \frac{S(2.0) M_V I_E W}{R_d R_0} \tag{1}$$

where $S(T_a)$ is the spectral acceleration corresponding to the building's fundamental period T_a, M_V is the factor to account for multistory effect, I_E is the importance factor, W is the total weight of the building, R_d is the ductility related force modification factor, and R_0 is the over-strength related force modification factor. The design acceleration values $S(T_a)$ is:

$$\begin{aligned} S(T_a) &= F_a S_a(0.2) \text{ for } T \leq 0.2s \\ &= F_v S_a(0.5) \text{ or } F_a S_a(0.2) \text{ whichever is smaller for } T_a = 0.5 \\ &= F_v S_a(1.0) \text{ for } T_a = 1.0s \\ &= F_v S_a(2.0) \text{ for } T_a = 2.0s \\ &= F_v S_a(2.0)/2 \text{ for } T_a \geq 4.0s \end{aligned} \tag{2}$$

The total lateral seismic force V, is distributed in accordance with the formula

$$F = \frac{(V - F_t) W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{3}$$

where x is the story number y and n is the total number of the stories in the building. Thus, the seismic force at the top is (F_x+F_t) Additional top story force (F_t) depends on the design period of the frame. According to NBCC 2005, when T_a is equal to or exceeds 0.7s, an additional concentrated load (F_t) equal to $0.07T_aV$ but not exceeding $0.25V$ needs to be assigned to the top story, to account for higher mode effect. The shear adjustment factor M_v depends on only the modal periods and modal weights (Humar and Mahgoub, 2003).The empirical formula for the fundamental period (T_a) of the building, as a function of its total height, h_n is given by the following equation.

$$T_a = 0.085(h_n)^{3/4} \quad (4)$$

NBCC 2005 presents an objective-based format where the design is achieved through the attainment of acceptable solution, rather than just satisfying the minimum requirement (CJCE, 2003). In NBCC 2005 the site-specific spectral acceleration is used to express the seismic hazard (Humar and Mahgoub 2003), which is presented as uniform hazards spectrum (UHS).The level of hazard corresponds to a 2% probability of exceedance in 50 years (a return period of 25000 years). In comparison the NBCC 1995 provisions are based on a seismic hazard corresponding to 10% probability of exceedance in 50 years (a return period of 475 years). In a UHS the probability of exceedance is constant for all periods (Adams and Atkinson, 2003). The major changes in the seismic design provisions included in the new version of NBCC are: (a) revised formulae to calculate base shear, (b) revised formulae for estimating the fundamental period of a building for design, (c) site specific response spectra, (d) a new force reduction factor, (e) incorporation of the site coefficient to account for soil condition, and (f) a revised method to take the higher mode effect into account. The revision of the code comes from the accumulated knowledge and experience gathered from earthquakes in the last two decades. During this period the earthquakes were observed through extensive instrumentation of buildings located in moderate to high seismic zones. An updated method of analysis for the seismic forces has been adopted in the NBCC 2005. Dynamic analysis has been specified as the method to be used for the calculation of seismic design forces and deflections for buildings located in higher seismic zones, tall buildings, and buildings with structural irregularity even of lower heights. A description of structural irregularity is also provided in NBCC 2005.

Modeling and Designing the Structure

For this study a twenty-story ductile steel moment resisting frame building with regular geometric shape has been designed according to NBCC 2005. A typical floor plan and elevation of the building are shown in Figure 1. The building is assumed to be located in Vancouver, in the western part of Canada. The performance of the building along the north-south direction has been evaluated. The building consists of a series of frames in the transverse (N-S) direction to resist the lateral loads. There are three bays in each frame, where two exterior bays are of with 9 meters each and the width of the interior one is 6 meters. The center to center spacing of the frames in the E-W direction is 6 meters. The first story height of the building is 4.85 meters and others are of 3.65 meters each. The frame is symmetrical along the vertical center line. Since an interior frame is considered in the study and two-dimensional frame models are analyzed, accidental torsion has not been considered. For simplicity in the analysis and design the exterior and interior frames are kept similar. Therefore, only one interior frame with mass of its tributary area has been designed and used in the performance evaluation study. Thus the three dimensional structure has become two dimensional in design and analysis. The beam members at the same floor level are grouped in the same section type and the column sections are changed at every sixth level i.e. columns are spliced at every fifth floor level. A simplified model of the frames is developed by using 5% strain hardening. The elements of the frames are detailed to develop ductile response under cyclic inelastic deformation due to seismic action, while other elements including connections are detailed to remain elastic under the combination of gravity load and the maximum earthquake induced lateral load.

To calculate the member forces for the static design the frame has been analyzed by DRAIN-2DX (Prakash *et al.* 1993), a computer software capable of performing inelastic static and dynamic analysis of building frames. The connections between beams and columns are assumed to be rigid and chosen from FEMA-350 (2000) predefined connection types. The category of predefined connection used here is welded and fully restrained. In the capacity-based design, the column and beam sections are chosen in such a way that at any joint the sum of the capacities of the columns is greater than that of the beams. The building has been designed to satisfy the NBCC 2005 requirements and the elements are designed according to CSA S16-01 (2001). The design loads for the building considered here are gravity loads (Dead load (D), Live load (L)) and Seismic load (E). Snow load (S) has also been considered in estimating the design load. Design base shear has been calculated by using Equation 1. To calculate the member forces the design base shear has been distributed along the height of the frame in the form of an inverted triangle in analysis and the force is assigned to each story level according to the weight of the respective story level. The following combinations of load have been used to find the design forces for the members: (a) 1.25D+1.5L, and (b) 1.0D± 1.0E +(0.5L+0.25S).

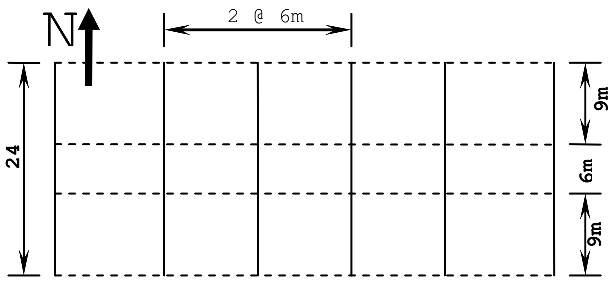

Figure 1. (a) Plan

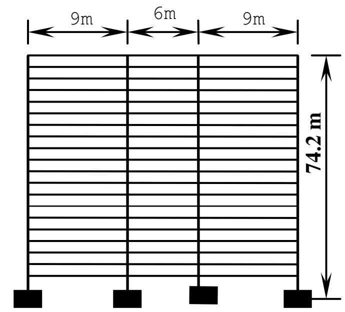

Figure 1. (b) Elevation

The design base shear is a function of the fundamental period of the building, which is initially calculated by using the empirical formula provided in NBCC 2005 (Equation 4); for the final design the fundamental period is based on the results of modal analysis. However, it is ensured that the fundamental period used in the revised calculation of the seismic force is not greater than 1.5 times of the period calculated using the empirical formula (NBCC 2005). The parameters used in the Equation 1 for calculation of the equivalent seismic force are: importance factor $I_e=1.0$, factor for higher mode effect $M_v=1.0$, ductility factor $R_d=5.0$ and the force reduction factor $R_0=1.5$. A soil type C (the reference soil), which represents very dense soil and soft rock, is considered with a site specification factor $F_v=F_a=1.0$. Type-D ductile frame is designed according to the clause 27 of CSA-S16-01 (2001), also presented in the "Handbook of Steel Construction" (CISC 2004). The steel sections used in the design of both the beams and columns are of CSA G40.21 (CISC 2004) with yield strength (f_Y) = 345 MPa, and the modulus of elasticity $E = 200x10^3$ MPa. Capacity of columns and beams has been checked for the after shake down condition. The design is then checked with detailed dynamic analysis as recommended by the code (NBCC 2005).

The moment resisting frames have also been modeled with infill panels to study the effect of the non-structural elements on the performance of the buildings. The infill panels are provided in the form of inclined strut in the mid bay at each story level of each frame. Two 100 mm thick inclined struts are provided at each story level. Clay masonry with compressive strength $f_m =8.6$ MPa is used to model the infill panels. The effective width *(w)* of the struts for different stories is calculated from the theory of beams on elastic foundation (Drysdale et al 1994). A summary of the designed sections is presented in Table 1.

Table 1: Details of the beam and column sections.

Story level↓	Exterior Column	Interior Column	Beam
Story 1-5	W310X283	W360X314	W310X129
Story 6-10	W310X253	W360X287	W310X129
Story 11-15	W310X202	W360X262	W310X129
Story 16-19	W310X179	W360X262	W310X129
Story 20	W310X179	W360X262	W310X107

Evaluation of the Seismic Performance

The seismic performance of a building can be defined in terms of its response to the ground motion imposed upon it during earthquake. In defining the level of performance of the structure the selection of performance objective is very important. The performance objectivities as define in Vision 2000 report (SEAOC 1995) are: Fully Operational, Operational, Life Safe and Near Collapse. According to Gupta and Krawinkler (2000), the evaluation of performance of the structures necessitates the ability to predict the global (e.g. roof), inter-mediate (e.g. story) and local (element) deformation demands. Though the methodologies for the evaluation of the performance of structures are still evolving, some linear and non-linear static and dynamic methods are available and widely used in evaluation of the seismic performance of structures like buildings. Modal analysis is used to calculate the mode shapes of the structure. The mode shapes of the building studied here are shown in Figure 2(b). Pushover and dynamic analysis are used to evaluate the lateral load capacity and performance levels.

Pushover Analysis

Pushover analysis represents an inelastic static analysis of a structure subjected to gravity load and monotonically increasing lateral loads. Pushover analysis is a common tool for estimating the capacity of a building subjected to lateral loads and useful for performance-based seismic design, PBSD (SEAOC 1995). PBSD is defined as the identification of hazard, and selection of the performance criteria and objectives corresponding to the desired performance level. The pushover analysis of both bare frame and infill frame has been performed using DRAIN-2DX for plane two dimensional model of the frame. The equivalent static seismic load has been applied along the frame in the shape of inverted triangle. The analysis has been carried out by considering the P-Δ effect with 5% strain hardening. The capacity of the frames has been calculated from the pushover graph by calculating the yield and ultimate displacement due to seismic load. The pushover graphs of the bare frame and the frame with infill panels are shown in Figure 2(a). The yielding of the members is also observed in the pushover analysis and the base shear coefficient corresponding to the yield displacement has been calculated. The values of base shear coefficients are 0.048 for bare frame and 0.059 for infill frame corresponding to first yielding of a beam.

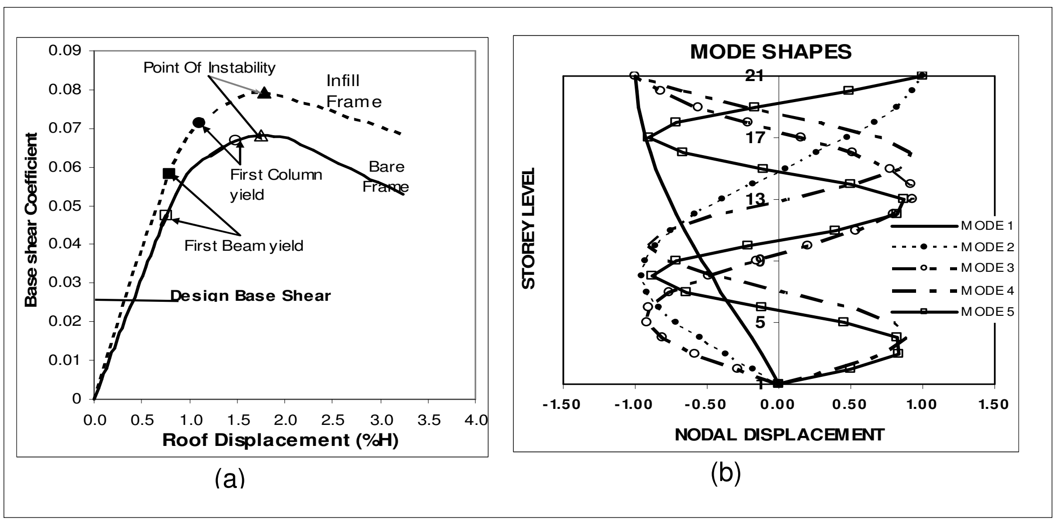

Figure 2. Analysis: (a) Pushover analysis, (b) Modal analysis.

Dynamic Analysis

Rigorous non-linear time history analysis is necessary to evaluate the performance of a building under seismic ground motion. Nonlinear analysis accounts for flexural yielding and subsequent changes in strength and stiffness (Saatcioglu and Humar, 2003). Estimation of the response/damage parameters such as, the roof displacement, and inter-story drift of a building subjected to seismic ground excitation is a major objective of dynamic analysis. The damage parameters are used for evaluating the performance of the building. To consider the effect of the gravity load on the lateral displacement, the P-Δ effect has also been considered in the dynamic analysis. Similar to pushover analysis, a two dimensional model of the frames has been used to carry out the response history analysis by using DRAIN-2DX. The analysis has been conducted for a set of thirty ground motion records. Among these, eight are synthesized and compatible to the seismic hazard spectrum for Vancouver, Canada (Trembley *et al.* 2001). The other twenty two records correspond to real ground motion, and they were collected from the database of the Pacific Earthquake Engineering Research Center (PEER, 2006) by comparing the peak acceleration to the peak velocity ratio (A/V) that is compatible with the seismicity in Vancouver. Four of the eight synthesized records are of long duration and the other four are of short duration. All these records (scaled and synthesized) correspond to the design levels of earthquake in Vancouver with a return period of 2500 years and are designated in this paper as UHS-2500 events.

Before carrying out the analysis the selected ground motion records are scaled to be compatible to the hazard spectra of Vancouver. The scaling has been done in the following two ways: *(i)* based on the acceleration ordinates, and *(ii)* based on partial area under the acceleration spectrum as described in Naumoski *et al. (*2004). In the first method the scaling factor is calculated as the ratio of the spectral value of the real motion to the ordinate of the design spectra for the fundamental period of the structure. In the second method the area under the acceleration spectra of the scaled real ground motion is the same as that under the design spectrum for the period range of second mode period and 1.2 times the fundamental period (Naumoski *et al* 2004). From the inelastic time history analysis the maximum inter-story drift corresponding to each record has been calculated. The mean drift (*M*) and mean plus standard deviation (*M+SD*) of the response due to the real ground motion has been calculated and has been checked against the code specified value. The maximum inter-story drift for each synthesized record is recorded and used for evaluation. Number of synthesized records is not enough to calculate the mean and standard deviation. Graphs of dynamic analysis are shown in Figures 3 and 4. Summaries of the ground motion (GM) records are shown in Table 2 and 3

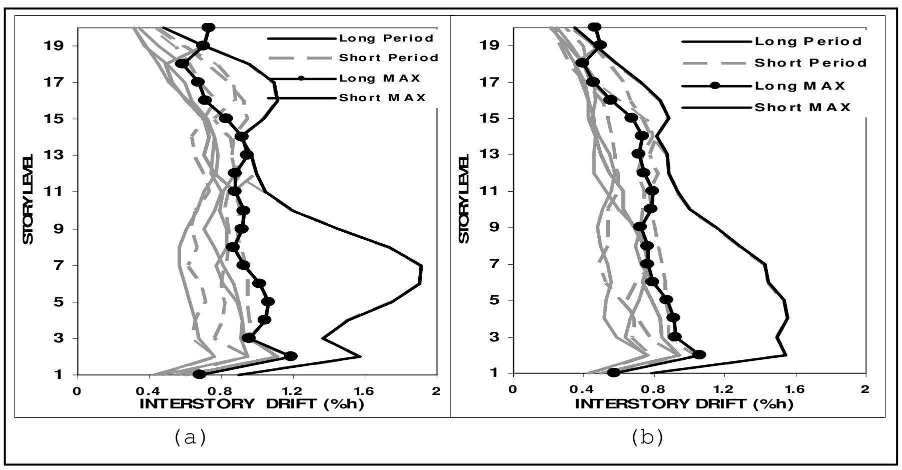

Figure 3. Dynamic Analysis for the Stochastic Ground Motion records (a) bare frame (b) infill frame.

Dynamic analysis has also been carried out for lower levels of seismic hazard for which the building is expected to sustain no or minor damage and remain fully operational or operational. The seismic hazard levels corresponding to the return periods of 475 years and 970 years have been considered. The hazard spectra corresponding to these levels of earthquakes in Vancouver were obtained from the Geological Survey of Canada and they have been designated as UHS-500 and UHS-1000 for the above-mentioned return periods, respectively. The synthesized ground motion records as developed in Atkinson and Beresnev (1998) have been adapted for these cases. For each hazard level, there are four records available, two of which have short duration (6 sec) and other two have long duration (19.66 sec). The peak ground acceleration corresponding to UHS-500 is approximately 25%g for the short duration records and 8%g for the long duration records. Those values for UHS-1000 are 30%g and 10%g, respectively. The maximum values of the inter-story drift corresponding to these levels of seismic hazard are summarized in Table 4.

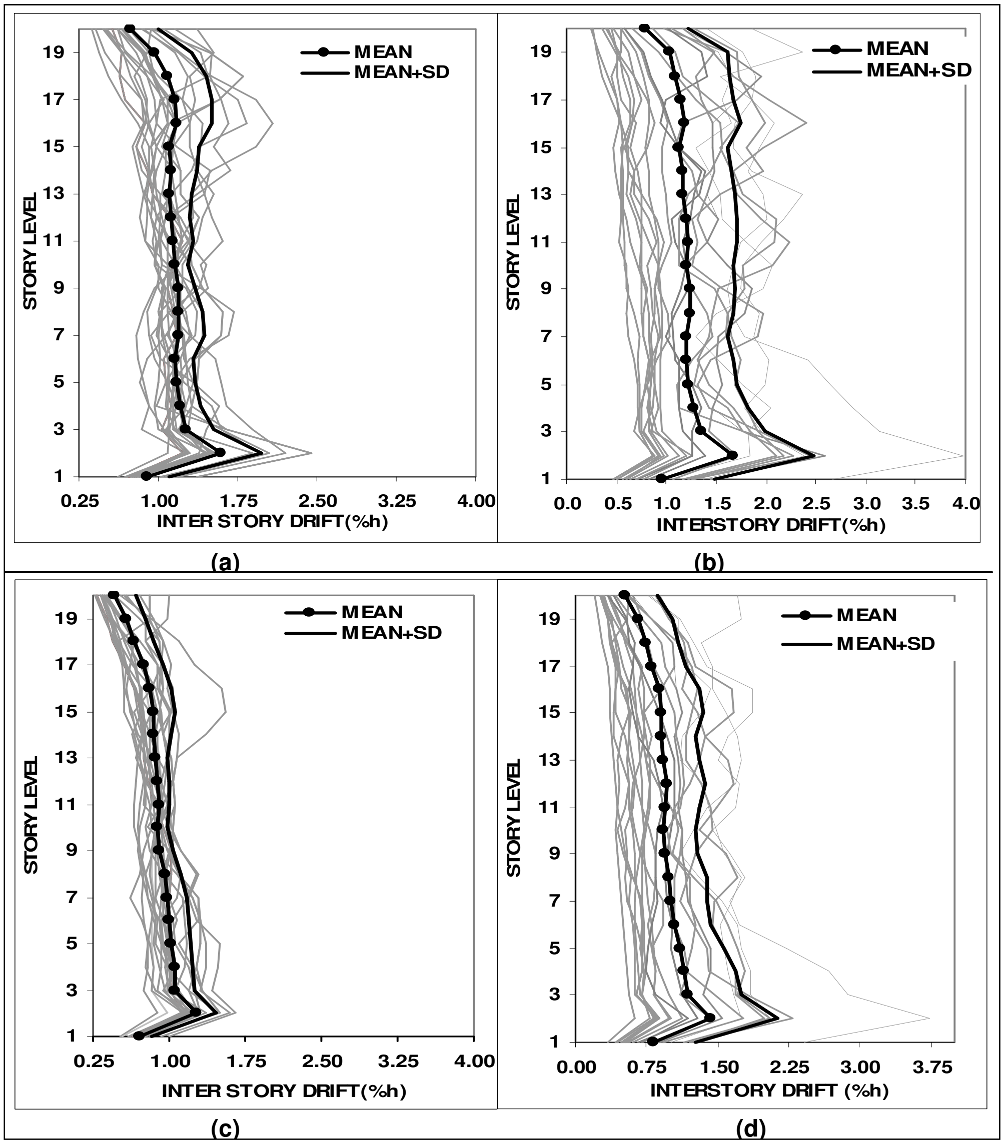

Figure 4. Dynamic analysis for the scaled real ground motion records; (a) bare frame – GM scaled by the “partial area method”, (b) bare frame – GM scaled by the “ordinate method”, (c) infill frame – GM scaled by the “partial area method”, (d) infill frame - GM scaled by the “ordinate method”.

Table 2. Summary of Stochastic Ground Motion.

Record No	LP1	LP2	LP3	LP4	SP1	SP2	SP3	SP4
Peak Acc.(cm/sec^2)	266.2	279.4	248.6	271.7	523	527	567	380
Duration (sec)	18.24	18.24	18.24	18.24	8.55	8.55	8.55	8.55

LP = Long Duration Period, SP = Short Duration Period, Acc. = Acceleration

Table 3. Summary of Real Ground Motion.

Record No.	1	2	3	4	5	6	7	8	9	10	11
Peak Acc. (g)	0.35	0.18	0.16	0.05	0.15	0.21	0.16	0.18	0.20	0.07	0.08
Record No.	12	13	14	15	16	17	18	19	20	21	22
Peak Acc.(g)	0.17	0.11	0.12	0.06	0.69	0.71	0.14	0.47	0.51	0.09	0.08

Table 4. Story Drift for UHS-500 and UHS-1000 (%*h*).

UHS-500				UHS-1000			
Infilled Frame		Bare Frame		Infilled Frame		Bare Frame	
Long	Short	Long	Short	Long	Short	Long	Short
0.361	0.366	0.423	0.433	0.721	0.571	0.799	0.748

Discussion and Conclusion

From the push over graph it has been observed that for the bare frame the first yielding occurs at a base shear coefficient (C_v) of 0.048 in beam No. 23, while the first yielding in a colulmn occurs at C_v=0.066 in column No. 41. For the infill frame the first yielding occurs at C_v = 0.059 in beam No. 23, while the first yielding in a column occurs at C_v = 0.072 in column No. 41. The hinge formation pattern is compatible with the capacity-design concept. The design base shear coefficient is 0.0264 which is almost half of the base shear coefficient required for the occurrence of the first yielding. In the frame with infill panel yielding occurs later than in the bare frame in some cases. But the ductility capacity as estimated from the pushover curve is almost the same for both frames and it is close to 2.

It is observed from the dynamic analysis that the maximum inter-story drifts for the stochastic ground motion records corresponding to UHS-2500 are 1.5% and 1.8% of story height for infill frame and bare frame, respectively. The mean (M) and mean plus standard deviation (M+SD) values of the maximum inter-story drift in the infilled frame for the real ground motion records are found to be 1.25%, 1.48% respectively when scaling by the partial area method, and 1.4%, 2.1%, respectively when scaling by the ordinate method. For the bare frame these values are 1.6%, 2.0%, 1.6%, and 2.5%, respectively. It is observed that the method of scaling effects the estimation of the response parameters and the evaluation of performance of the building. The presence of non-structural elements reduces the inter-story drift in the range of 15% to 35%. Based on the results of the analysis, it is clear that the building meets the life-safety and collapse prevention performance objectives.

The dynamic analysis of the building subjected to minor earthquakes (UHS-500 and UHS-1000) reveals that the maximum values of the inter-story drift are less than 0.5% for UHS-500 and less than 0.8% for UHS-1000. The building is expected to remain operational after such events.

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References

Adams, J. and Atkinson G., 2003. Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada, *Canadian Journal of Civil Engineering*, **30**: 255-271.

Atkinson, G.M. and Beresnev, I.A., 1998. Compatible ground motion time-histories for new National hazard maps, *Canadian Journal of Civil Engineering*, **35**: 305-318.

CISC, 2004. Handbook of Steel Construction, *Canadian Institute of Steel Construction*, Willowdale, Ontario, Canada.

CJCE, 2003. Special Issue on NBCC 2005. *Canadian Journal of Civil Engineering*. 2003 Vol.30, No.4

CSA, 2001. *CSA Standard S16-01*, Limit Sates Design of Steel Structures, *Canadian Standards Association*, Rexdale, Ontario.

Drysdale, R.G. and Ahmed, A.H., 1994. *Masonry Structures- Behavior and Design*, Prentice Hall.

FEMA-273, 1997. NEHRP Guidelines for the Seismic Rehabilitation of the Buildings, prepared for the *Building Seismic Safety Council* (BSSC), Washington, D.C., USA

FEMA- 310, 1998. Handbook for the Seismic Evaluation of Buildings – A Prestandard, prepared by the *American Society of Civil Engineers for the Federal Emergency Management Agency*, Washington, D.C. U S A

FEMA-350, 2000. Recommended Seismic Design Criteria for New Steel Moment-Frame Building, prepared by the *SAC Joint Venture for the Federal Emergency Management Agency*, Washington, D.C., U S A.

Foo S., Naumoski N., Saatcioglu M., 2001. *Seismic Hazard, Building Codes and Mitigation Options for Canadian Buildings*, Office of Critical Infrastructure Protection and Emergency Preparedness, 2nd Floor, Jackson Bldg. Ottawa, ON K1A 0W6, Canada

Ghobarah A., 2001. Performance-based design in earthquake engineering: state of development, *Engineering Structures*, **23:** 878-884.

Gupta A. and Krawinkler H., 2000. Estimation of seismic drift demands for frame structures, *Earthquake Engineering and Structural Dynamics*, **29**:1287-1305

Humar, J. and Mahgoub M. A., 2003. Determination of seismic design forces by equivalent static load method, *Canadian Journal of Civil Engineering,* **30**: 287-307.

Lee k. and Foutch D.A., 2002. Performance evaluation of new steel frame buildings for seismic loads, *Earthquake Engineering and Structural Dynamics,* **31**: 653-670

Natural Resources Canada (NRC), "Earthquakes Canada – East", http://seismo.nrcan.gc.ca/major_eq/majoreq_e.php

Naumoski N., Saatcioglu M., and Amiri-Hormozaki K., 2004. Effects of Scaling of Earthquake Excitations on the Dynamic Response of Reinforced Concrete Frame Buildings, *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, Paper No. 2917.

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

PEER, 2006. Strong Motion Database, http://peer.berkeley.edu

Prakash V., Powell, Graham H, Campbell, Scott D., 1993. Drain-2DX: Static and Dynamic analysis of Inelastic Plane Structure, *Department of Civil Engineering*, University of California, Berkeley.

Saatcioglu M. and Humar J., 2003. Dynamic analysis of buildings for earthquake-resistant design, *Canadian Journal of Civil Engineering,* **30**: 338-359.

Tremblay, R. and Atkinson, G.M, 2001. Comparative study of in elastic seismic demand of eastern and western Canadian sites, *Earthquake Spectra*, **17**(2): 333-358

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

Yun S.Y., Hamburger R.O., Cornell C. A., and Foutch D.A., 2002. Seismic Performance Evaluation for Steel Moment Frames, *Journal of Structural Engineering*, **128**(4): 534-545