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Seismic performance of concrete moment resisting frame buildings in Canada

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Abstract. The seismic provisions of the current edition (2005) of the National Building Code of Canada (NBCC) differ significantly from the earlier edition. The current seismic provisions are based on the uniform hazard spectra corresponding to 2% probability of exceedance in 50 years, as opposed to the seismic hazard level with 10% probablity of exeedance in 50 years used in the earlier edition. Moreover, the current code is presented in an objective-based format where the design is performed based on an acceptable solution. In the light of these changes, an assessment of the expected performance of the buildings designed according to the requirements of the current edition of NBCC would be very useful. In this paper, the seismic performance of a set of six, twelve, and eighteen story buildings of regular geometry and with concrete moment resisting frames, designed for Vancouver western Canada, has been evaluated. Although the effects of non-structural elements are not considered in the design, the non-structural elements connected to the lateral load resisting systems affect the seismic performance of a building. To simulate the non-structural elements, infill panels are included in some frame models. Spectrum compatible artificial ground motion records and scaled actual accelerograms have been used for evaluating the dynamic response. The performance has been evaluated for each building under various levels of seismic hazard with different probabilities of exceedance. From the study it has been observed that, although all the buildings achieved the life-safety performance as assumed in the design provisions of the building code, their performance characteristics are found to be non-uniform.

Keywords: seismic hazard; uniform hazard spectra; seismic performance; concrete moment resisting frames; pushover analysis; time history analysis.

1. Introduction

In performance-based seismic design, buildings are designed to meet sets of performance objectives related to the seismic hazards that correspond to specified probabilities of being exceeded. The Vision 2000 report (1995) of the Structural Engineers Association of California (SEAOC) has suggested a series of earthquake events with different levels of intensity for which the

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Table 1 Permissible interstory drifts (Vision 2000 1995)

Drift	Performance Level				
	Fully opera-tional (FO)	Operational (OP)	Life-safe (LS)	Near collapse (NC)	Collapse (CO)
(a) Transient	< 0.2%	< 0.5%	< 1.5%	< 2.5%	> 2.5%
(b) Permanent	Negligible	Negligible	< 0.5%	< 2.5%	> 2.5%

performance of a building may be measured. The intensity is expressed in terms of the recurrence interval or a probability of exceedance. The Vision 2000 committee report suggests qualitative performance objectives for buildings of different types and also provides some suggestions on the quantitative measures of performance based on drift levels as shown in shown in Table 1.

Seismic loading provisions in most existing building codes focus on the minimum lateral seismic forces for which the building must be designed, but do not explicitly incorporate requirements to measure the performance under earthquakes of different intensity. Implicit in these provisions is the understanding that the building will not collapse under the design earthquake but may suffer some structural and non-structural damage. The specification of the lateral forces alone is of course not enough to ensure that the desired level of performance will be achieved. It will be some time before fully performance-based seismic codes are developed. In the interim it will be useful to carry out an assessment of the performance that can be expected from the buildings designed according to the current codes.

In Canada the seismic design of buildings is performed according to the relevant provisions of the National Building Code of Canada (NBCC). The 1995 edition of NBCC adopted a two parameter seismic zoning approach to quantify the seismic hazard across the country. The level of seismic risk at any site was expressed in terms of both the peak horizontal ground acceleration and the peak horizontal ground velocity, each with a probability of exceedance of 10% in 50 years. In the new version of the code, published in 2005, the seismic hazard is represented by site dependent uniform hazard spectra corresponding to a 2% probability of being exceeded in 50 years. The 2005 NBCC seismic design provisions continue to rely on the specification of minimum lateral seismic forces for which the building must be designed and the acceptable drifts under such forces. In view of this, an assessment of the performance that could be expected from buildings designed according to the proposed requirements of 2005 NBCC is of interest. This paper focuses on an evaluation of the expected performance of buildings with reinforced concrete moment resisting frames. Performance is evaluated for buildings situated in Vancouver representing high level of seismic hazard in Canada.

The technical background to the 2005 NBCC is presented in a series of articles in the Canadian Journal of Civil Engineering (Adams and Atkinson 2003, De Vall 2003, Wightman 2003, Heidebrecht 2003, Humar and Mahgoub 2003, Saatcioglu and Humar 2003). In NBCC 2005 the seismic hazard is expressed in terms of a *uniform hazard spectrum* (UHS), which provides the maximum expected spectral acceleration S_a of a single-degree-of-freedom (SDOF) system with 5% damping. Site-specific values of the spectral acceleration $S_a(T_a)$ for the reference ground condition, defined as very firm soil or soft rock, are available from the Geological Survey of Canada (Adams and Halchuk 2003) and are specified in the table of climatic data included in the Code. The spectral values must be modified for the soil at the site. Such modification is carried out by applying the soil factors specified in the code. Two sets of soil factors are specified, F_a for the short period range and

F_v for the long period range. The design spectral acceleration, $S(T_a)$ obtained by modifying the site specific hazard spectral acceleration, $S_a(T_a)$ is used for calculating the design base shear as follows.

$$V = \frac{S(T_a)M_vI_eW}{R_0R_d} \geq \frac{S(2.0)M_vI_eW}{R_0R_d} \tag{1}$$

where M_v accounts for higher mode effect, I_e is the importance factor, and R_d and R_0 account for ductility and over-strength, respectively. The lower level cut-off in Eq. (1) is specified on account of the uncertainty associated with the determination of $S_a(T_a)$ values for periods greater than 2 s. The base shear is distributed linearly as suggested by the code (NBCC 2005).

2. Methodology for performance evaluation

The evaluation of seismic performance of any structure requires the estimation of its dynamic characteristics and the prediction of its response to the ground motions to which it could be subjected during its service life. The dynamic characteristics, namely the periods and mode shapes are obtained through an eigenvalue analysis. Inelastic time history analyses provide the damage states of the building when it is subjected to various levels of ground motion. Static pushover analysis is performed to determine the lateral load resisting capacity of a structure and the maximum level of damage in the structure at the ultimate load. These steps require definition of damage parameters, selection of earthquake records, computer modelling, and analysis of the structure. Also probabilities of failure under a given level of hazard need to be determined (Akbas *et al.* 2008). These issues are discussed in the following paragraphs.

2.1 Damage parameters

Selection of appropriate damage parameters is very important for performance evaluation. Overall lateral deflection, ductility demand, and inter-story drifts are commonly used damage parameters. Damage index developed by Park and Ang (1985) is regarded as a good representation of the structural damage in a concrete element, since it accounts for the damage caused by cyclic deformations into the post-yield level. Inelastic energy dissipation by structural elements is also considered as a measure of damage. There are other forms of damage indices (e.g., IDARC 2006, Bertero 2004) which can also be used. In this study, the inter-story drift is used as the main damage parameter and the guidance provided in Vision 2000 (1995) report as shown in Table 1 has been used to determine the performance level. Park-Ang damage indices for each element and story were also calculated and used in the evaluation. (Kunnath *et al.* 1992) which considers the end section in the damage calculation instead of the whole element. The modified version of the Park-Ang damage index as suggested in (Kunnath *et al.* 1992) has been used here (Eq. (2)).

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{\theta_u M_y} E_h \tag{2}$$

where θ_m is the maximum rotation experienced by the end section of an element during the loading history, θ_u is the ultimate rotation of the section, θ_r is the recoverable rotation after the unloading, M_y is the yield flexural strength of the section, E_h is the hysteretic energy absorbed by the section. By summing up the damage indices of all elements in a storey and multiplying each index by a weighting factor that is dependant on the amount of dissipated energy by the element with respect to the whole hysteretic energy of the storey, a storey damage index is calculated. In the same manner, the weighted sum of the story-wise damage indices provides the global damage index. Park *et al.* (1987) provides an interpretation of the values of damage index for reinforced concrete structures, according to which, a structure is deemed repairable if the damage index is lower than 0.4, while that exceeding 1 indicates collapse.

2.2 Selection of the ground motion time histories

Inelastic time history analysis requires that one or more appropriate earthquake records be selected. This is an important step in performance evaluation. It is possible to develop simulated records that would match the UHS. However, because a UHS represents a composite of response produced by many different earthquakes, a simulated earthquake record will correspond to the simultaneous occurrence of a number of potentially damaging events. Such a simulated record is therefore unrealistic. A better strategy is to generate a set of simulated records, in which individual records match different parts of the UHS. Atkinson and Beresnev (1998) have produced physically realistic stochastic ground acceleration time histories of this type which not only match portions of the hazard spectrum, but also are representative of motions for specified magnitude distance scenarios in the regions of interest. The characteristics of these artificial ground motion records are summarized in Bagchi (2001), and Tremblay and Atkinson (2001).

Apart from the UHS compatible stochastic ground motion records, a set of sixteen actual records have been used in the study. Most of these records are available at the Pacific Earthquake Engineering Research Centre (PEER 2007). The details of these records are given in Table 2. These records have been selected such that their peak velocity to acceleration ratio is compatible with the

Table 2 Details of the actual ground motion records

Record	Location / Record No.	Date	Amax (g)	Vmax (m/s)	A/V (s/m)	Duration (s)
1	Imperial Valley	18/05/1940	0.35	0.33	1.04	53.74
2	Kern County	21/07/1952	0.18	0.18	1.01	54.40
3	Kern County	21/07/1952	0.16	0.16	0.99	19.16
4	Borrego Mountain	08/04/1968	0.05	0.04	1.10	45.00
5	Friuli, Italy	15/09/1976	0.11	10.2	0.01	26.39
6	San Fernando	09/02/1971	0.15	0.15	1.01	65.18
7	San Fernando	09/02/1971	0.21	0.21	1.00	79.48
8	San Fernando	09/02/1971	0.17	0.17	0.99	62.58
9	San Fernando	09/02/1971	0.18	0.20	0.88	43.00
10	San Fernando	09/02/1971	0.20	0.17	1.19	47.08
11	Gazli, USSR	17/05/1976	0.608	65.4	0.01	16.27
12	Coalinga	22/07/1983	0.217	18.1	0.01	19.50
13	Monte Negro	15/04/1979	0.17	0.19	0.88	40.40
14	SUCH850919AL.T	19/09/1985	0.11	0.11	0.94	120.00
15	VILE850919AT.T	19/09/1985	0.09	0.11	0.83	128.00
16	Coyote Lake	06/08/1979	0.271	26.3	0.01	27.19

seismicity in Vancouver. The individual records have been scaled to the design spectral acceleration corresponding to the first mode period.

2.3 Software tools used

Modelling a structure and representing it in a computer program for suitable analysis are important steps in performance evaluation. The following computer programs are used in this study: IDARC2D (Vales *et al.* 1996) and DRAIN-2DX (Prakash *et al.* 1993). The first program, IDARC2D, is a nonlinear dynamic analysis program developed at the State University of New York at Buffalo specifically designed for reinforced concrete frame structures with appropriate hysteretic behavior of reinforced concrete members. While DRAIN-2DX is a general-purpose program for dynamic analysis of building frames developed at the University of California, Berkeley. Although reinforced concrete can be modeled using the fibre beam-column element in DRAIN2DX, it is quite complex and time consuming. A simple beam-column element with elasto-plastic hysteretic behavior has been used, instead. The reason for using these two programs is to make a comparative study between two nonlinear dynamic analysis programs and to obtain a better assessment of the performance of the frames under ground acceleration and lateral loading in general. In other words, the comparison is intended to enhance the reliability of the results (El Kafrawy 2006).

3. Details of the buildings

Buildings of two different heights, six and twelve stories respectively, are considered. The buildings are assumed to be situated in Vancouver in the west of Canada. The geometric details of the buildings are shown in Fig. 1. The buildings have 8 six-meter bays in the N-S direction and 3 bays in E-W direction. The E-W bays consist of 2 nine-meter office bays and a central six-meter corridor bay. The story height is 4.85 m for the first story and 3.65 m for all other stories. The yield stress, f_y for reinforcing steel, and the 28-day concrete compressive stress, f_c' are assumed to be 400 MPa and 30 MPa, respectively.

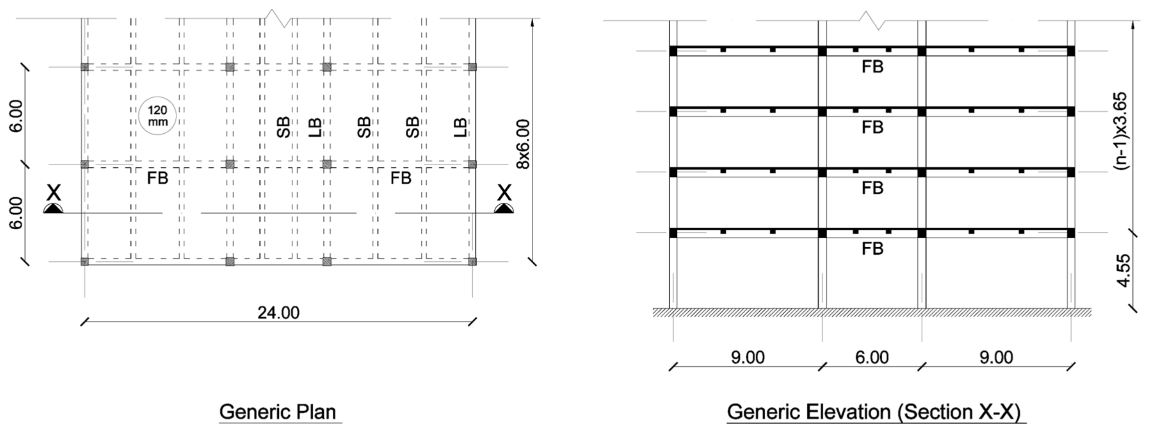

Fig. 1 Generic plan and elevation of the buildings

3.1 Modelling of the buildings

The buildings considered here can be modelled by a series of transverse frames connected by rigid links. For simplicity the exterior and interior ductile frames are kept similar. Thus a single frame along with the floor mass tributary to it can be used in the analysis, and the analysis procedure becomes two-dimensional. For consistency with this procedure, accidental torsion is not considered in obtaining the design forces.

Building structures with infill panels are also studied. The number of frames with infill panels and the arrangement of the infill panels are adjusted so that the fundamental period of the building is close to the value obtained by using the NBCC expressions.

3.2 Design of the building frames

The seismic lateral forces are obtained using the new uniform hazard spectrum (UHS) based methodology of NBCC 2005. The base shear is distributed across the height of the frame, using the procedure suggested by NBCC 2005 to obtain the floor level forces.

The building frames are designed such that the lateral load case (i.e., D+0.5L+E, where D, L and E are dead, live and seismic lateral loads respectively), not the gravity load case (i.e., 1.25D+1.5L) governs the design of the ductile lateral load resisting frames. All transverse frames are assumed to be ductile lateral load resisting. Wind load is not considered in the design, since the objective of this study is to evaluate the minimum level of seismic protection available to a building. In a case where the wind load governs the design, the structure is expected to have a higher level of seismic protection.

In general, the period of a bare frame in its fundamental mode of vibration is higher than the value obtained using the expression recommended by NBCC-2005. Non-structural elements in a frame play a significant role in stiffening the structure, significantly reducing its fundamental period of vibration. It is presumed that the code expression for the time period takes into account the stiffening effect of non-structural elements. Infill panels can be included in a frame to simulate the effect of non-structural elements. Inclusion of infill panels brings down the period of a frame structure.

In this study, both bare and infilled frames are considered. As stated earlier, the number and distribution of infill panels in a frame are chosen such that the fundamental period of the structure is close to the value recommended by 2005 NBCC. For concrete frames, the code recommends the following empirical expression

$$T = 0.075(h_n)^{3/4} \tag{6}$$

where, T is the fundamental period and h_n is the height of the building above its base. If modal analysis of the frame indicates a higher value of T, the lateral loads are revised using a higher value of the period, not exceeding 1.5T, as suggested in NBCC 2005.

The infill panels are modelled using equivalent struts (FEMA-306 1998). Clay masonry is assumed in all the cases considered here. The compressive strength, f_m of clay masonry is assumed to be 8.6 MPa and the modulus of elasticity to be 500f_m (Shoostari 1997). In DRAIN2DX, the equivalent diagonal struts have been modelled as truss elements having negligible tensile strength and the specified compressive strength. The hysteretic behaviour of the diagonal struts is assumed to

Table 3 Earthquake Design Spectra for Vancouver*

Hazard level	$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(\geq 4.0)$
UHS-2500	0.960	0.660	0.340	0.180	0.090
UHS-1000	0.614	0.357	0.197	0.108	0.054
UHS-500	0.482	0.275	0.146	0.076	0.038

*The values are in term of the acceleration due to gravity, *g*.

Table 4 Base shear calculation

Number of storeys	Period, s (NBCC), T_a	1.5*T_a, s	$S(T)$ M_v, g	V/W	W, kN	V, kN
6	1.30	1.17	0.312	0.0459	6411.12	294.33
12	2.31	1.95	0.189	0.0278	12989.7	360.70
18	3.32	2.63	0.152	0.0265	20003.6	529.51

be elasto-plastic, for simplicity. In reality, there are strength and stiffness degradation, which are not captured in the analysis.

Firm (reference) ground condition is assumed in calculating the design base shear, and the following values are assigned to different parameters: I_e = 1.0, $R_0 = 1.7$ and $R_d = 4.0$. The design spectra for Vancouver are given in Table 3, which provides values of S_a for a set of selected periods. Interpolation must be used for intermediate periods. For moment resisting frames the value of the multistory factor, M_v is 1.0 in western Canada.

The design base shears for the buildings used in this study are calculated using Eq. (3); the calculated values are shown in Table 4. It may be noted that the weight W specified in the Table is the inertial weight tributary to a lateral load resisting frame. For the purpose of design, the member forces are determined using a linear elastic analysis, where the effect of cracking in concrete is accounted for by assuming reduced moments of inertia, I for beams and for columns. The effective value of I for a beam is assumed to be $0.4I_g$ (I_g is the gross value of I), while for a bottom column it is assumed to be $0.70I_g$ and for a top column it is assumed to be $0.60I_g$. Additional shear forces and bending moments due to $P-\Delta$ effect are also taken into account in the design. The assumed values of the effective I for beams and columns as mentioned above are used for the initial design only. More accurate values of the effective I, yield moment *etc* for each element are calculated by using IDARC2D, which are validated with sectional analysis. Apart from calculating an updated value of I of a reinforced concrete section, the sectional analysis has also been used for determining the Moment-Curvature (M-Φ) relation and the Axial Force-Moment (P-M) interaction curve which are used for the DRAIN2DX analysis.

The frames are designed according to the capacity design philosophy specified in CSA Standard A23.3-2001 (2001), so that the total flexural capacity of the columns meeting at a joint exceeds the sum of the flexural capacities of the beams meeting at the same joint (El Kafrawy *et al.* 2007). Details of the reinforcement in the ductile lateral load resisting frames are given in Table 5 and Table 6.

Table 5 Section and reinforcement details for beams

Story #	External Beams			Internal Beams		
	6 Story	12 Story	18 Story	6 Story	12 Story	18 Story
1	8#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
2	8#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
3	8#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
4	8#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
5	8#20 Top 5#20 Bot.	8#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
6	8#20 Top 5#20 Bot.	8#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.	8#20 Top 3#20 Bot.	8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
7		8#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.		8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
8		8#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.		8#20 Top 4#20 Bot.	11#20 Top 3#20 Bot.
9		7#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.		6#20 Top 3#20 Bot.	11#20 Top 3#20 Bot.
10		7#20 Top 5#20 Bot.	10#20 Top 5#20 Bot.		6#20 Top 3#20 Bot.	11#20 Top 3#20 Bot.
11		7#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.		6#20 Top 3#20 Bot.	9#20 Top 3#20 Bot.
12		7#20 Top 5#20 Bot.	9#20 Top 5#20 Bot.		6#20 Top 3#20 Bot.	9#20 Top 3#20 Bot.
13			9#20 Top 5#20 Bot.			9#20 Top 3#20 Bot.
14			9#20 Top 5#20 Bot.			9#20 Top 3#20 Bot.
15			9#20 Top 5#20 Bot.			9#20 Top 3#20 Bot.
16			7#20 Top 5#20 Bot.			6#20 Top 3#20 Bot.
17			7#20 Top 5#20 Bot.			6#20 Top 3#20 Bot.
18			7#20 Top 5#20 Bot.			6#20 Top 3#20 Bot.

*All cross sections are 400×600 mm

Table 6 Section and reinforcement details for columns

Story #	External Columns			Internal Columns		
	6 Story	12 Story	18 Story	6 Story	12 Story	18 Story
1	450x450 12#20	550x550 12#20	650x650 4#25+8#20	500x500 8#20+4#25	600x600 12#25+4#20	750x750 20#25
2	450x450 12#20	550x550 12#20	650x650 4#25+8#20	500x500 8#20+4#25	600x600 8#25+4#20	750x750 16#25
3	450x450 12#20	550x550 12#20	650x650 4#25+8#20	500x500 8#20+4#25	600x600 8#25	750x750 12#25
4	450x450 12#20	550x550 12#20	650x650 4#25+8#20	500x500 8#20+4#25	600x600 8#25	750x750 12#25
5	450x450 12#20	450x450 12#20	650x650 4#25+8#20	500x500 8#20+4#25	550x550 8#25	750x750 12#25
6	450x450 12#20	450x450 12#20	550x550 4#25+4#20	500x500 12#20+4#25	550x550 8#25	650x650 12#25
7		450x450 12#20	550x550 4#25+4#20		550x550 8#25	650x650 12#25
8		450x450 12#20	550x550 4#25+4#20		550x550 8#25	650x650 12#25
9		400x400 12#20	550x550 4#25+4#20		500x500 8#25	650x650 12#25
10		400x400 12#20	550x550 4#25+4#20		500x500 8#25	650x650 12#25
11		400x400 12#20	500x500 4#25+4#20		500x500 8#25	550x550 8#25
12		400x400 4#25+8#20	500x500 4#25+4#20		500x500 8#25	550x550 8#25
13			500x500 4#25+4#20			550x550 8#25
14			500x500 4#25+4#20			550x550 8#25
15			500x500 4#25+4#20			550x550 8#25
16			450x450 4#25+4#20			500x500 8#25
17			450x450 4#25+4#20			500x500 8#25
18			450x450 4#25+8#20			500x500 8#25+4#20

Table 7 Fundamental periods of the building models

	Fundamental Period (s)		
	6 Story	12 Story	18 Story
Bare Frame	1.3	2.31	3.32
Infilled Frame	0.86	1.69	2.55
NBCC 2005, T_a	0.78	1.3	1.75
$1.5T_a$	1.17	1.95	2.63

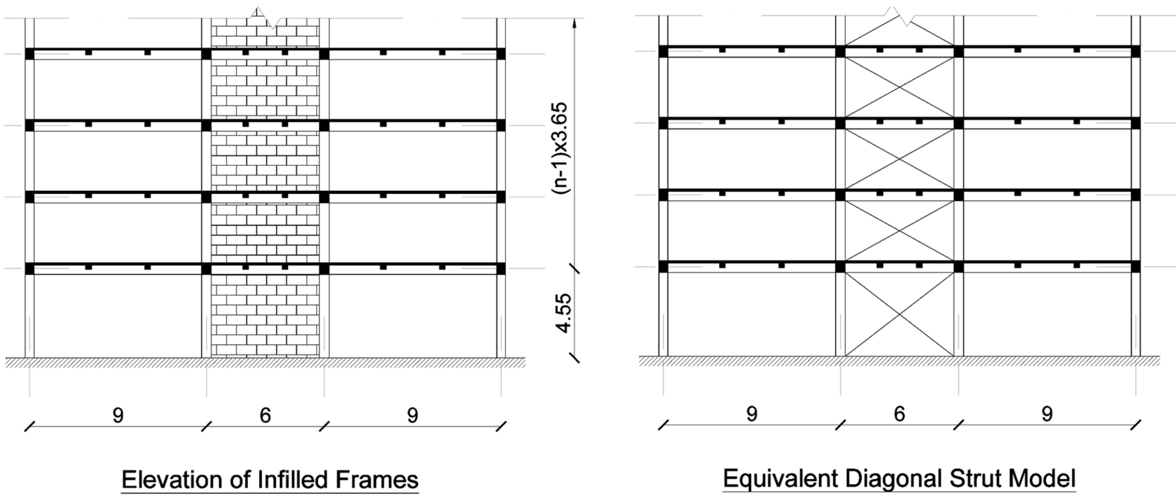

Fig. 2 Generic elevation of the infilled frame model

4. Analysis of the building frames

4.1 Modal analysis

Periods of the bare and infilled frame models of the six- and twelve-story buildings, as obtained from a modal analysis, are listed in Table 7. For the six-story building, the period of the bare frame is 1.3 s and the period of the infilled frame is 0.86 s, while the period obtained using the NBCC expression is 0.78 s. For the twelve-story building, the periods of the bare and infilled frames are 2.31 s and 1.69 s, respectively, while the corresponding NBCC value is 1.3 s. As for the eighteen-story building, the periods of the bare and infilled frames are 3.32 s and 2.55 s, respectively and the corresponding NBCC value of the fundamental period of vibration is 1.75 s.

4.2 Lateral load-resisting capacity

Pushover curves, representing the variation of base shear with the lateral roof displacement in an internal lateral load-resisting frame, are shown in Fig. 3(a) for both bare and infilled frames and are obtained using IDARC2D and DRAIN-2DX. It is observed that the inclusion of infill panels drastically improves the capacity of the frame. The effect of infill panels is generally not considered in the design. In reality they contribute a great deal of strength to the overall capacity of a frame (Fajfar *et al.* 1997), provided they are restrained against out of plane failure.

Pushover analysis provides the base shear for a given interstory drift, which is an important damage parameter and is used throughout this study. The failure point is shown on each pushover curve and it corresponds to the point of instability or to the point of 2.5% interstory drift, whichever occurs first. The point of instability is defined as the point on the push-over curve beyond which the slope of the curve becomes negative indicating that the tangent stiffness of the structure is close to zero. The point corresponding to 2.5% interstory drift is the point where the interstory drift of any of the building stories reaches the maximum limit specified by the NBCC (2005), 2.5% of the building height.

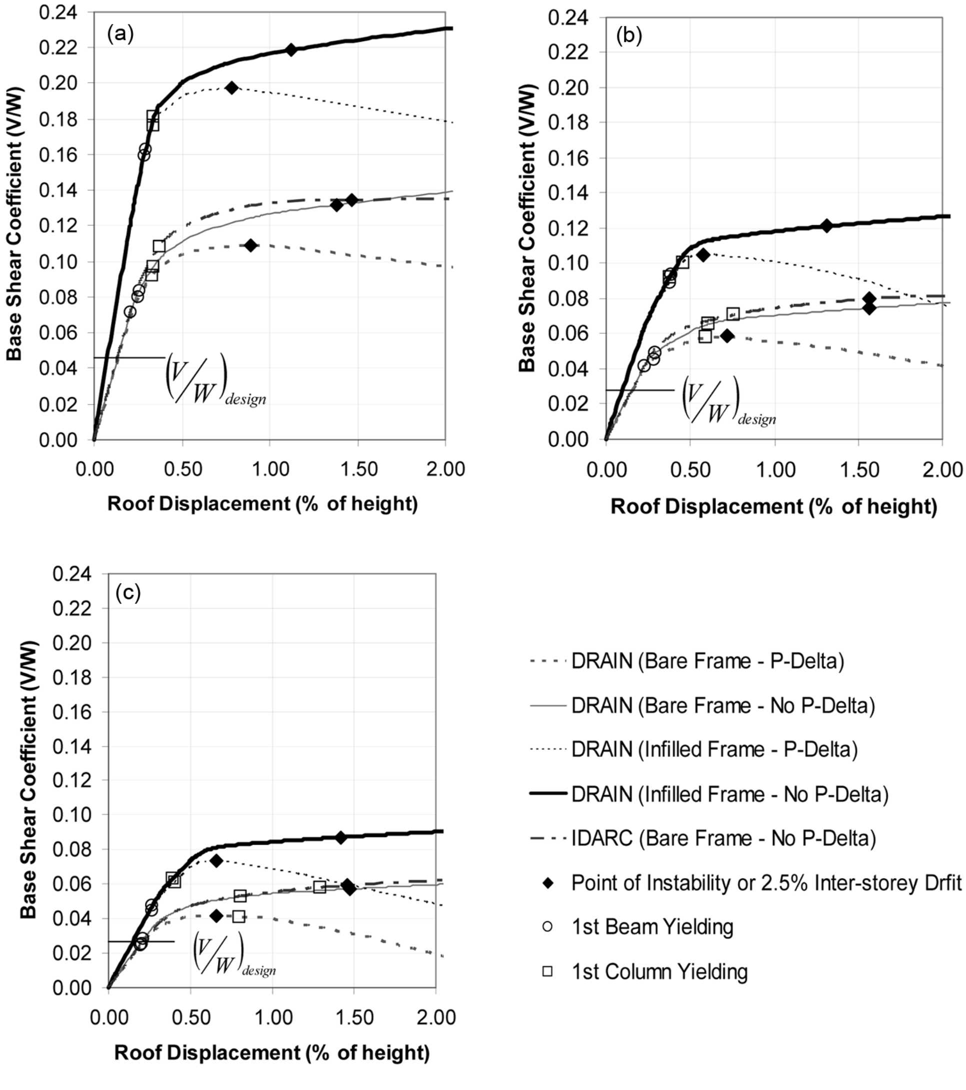

Fig. 3 Pushover curves for the buildings (a) six story, (b) twelve story, and (c) eighteen story

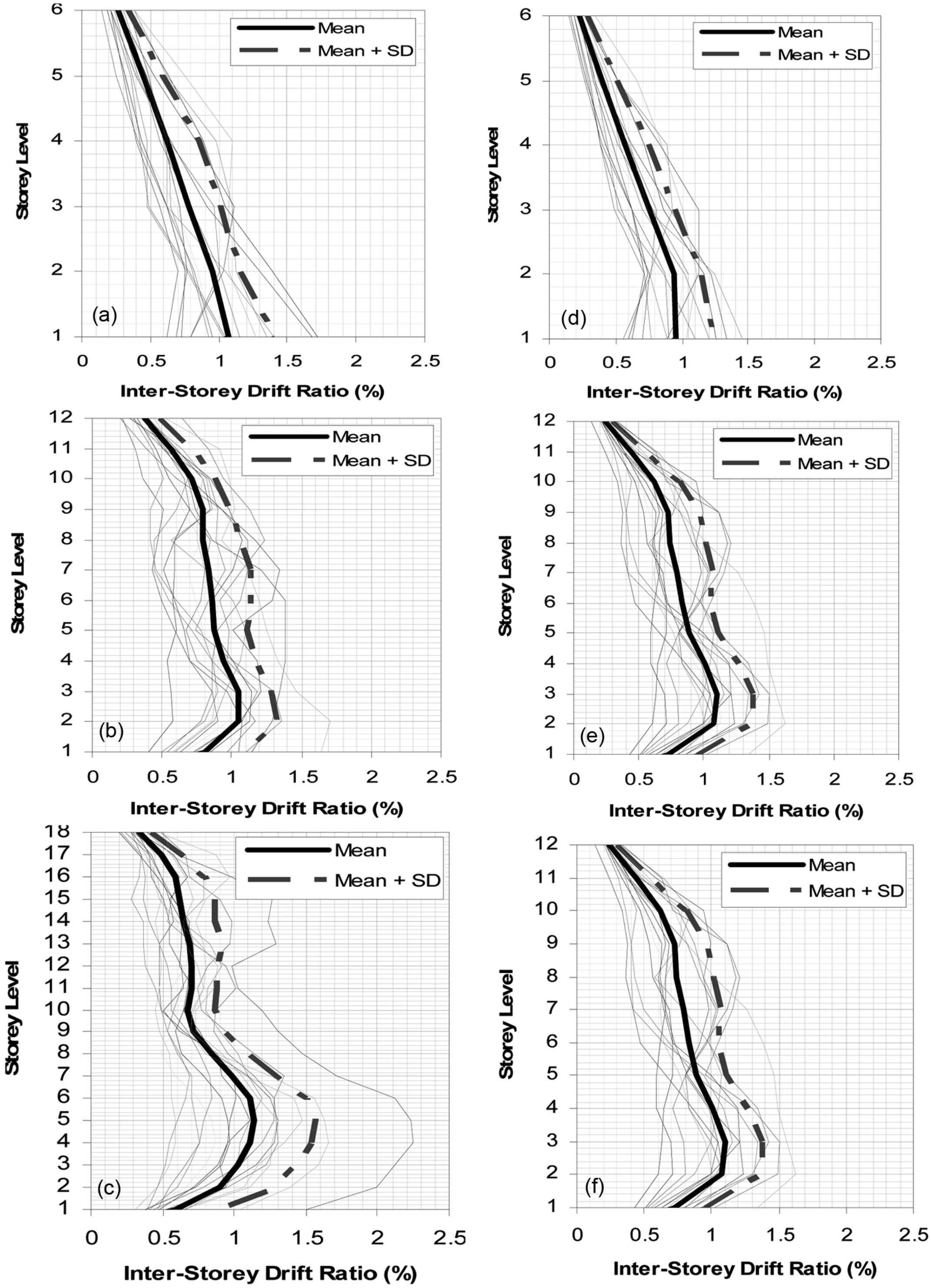

Fig. 4 Interstory drifts due to the scaled records in the bare frames – DRAIN2DX (left, a-c) and IDARC (right, d-f)

The elastic stiffness of the structure is assumed to be the average initial slope of the pushover curve. The results show that the inclusion of the infill panels with assumed configuration, increases the overall stiffness by 49%. The failure load as well increases by 81% with the inclusion of the infill panels. The sequence and patterns of hinge formation, not shown here, indicate that the bare frame behaves in accordance with the capacity design. The bare frame behaves in a ductile manner with hinges forming first in the beams meeting at a joint and then in one of the columns meeting at the same joint. The infilled frame however, shows less ductility and does not show a predictable hinging pattern like that in the bare frame. The overall ductility is calculated using the roof displacement at the collapse point (i.e., the point on the pushover curve that corresponds to instability or 2.5% interstory drift) to the yield displacement as estimated using the bilinear idealization of the pushover curve.

The pushover curves for the interior transverse frame of the twelve story building are shown in Fig. 3(b). The inclusion of the infill panels increases the overall stiffness by 68% and the failure load by 79% however it decreases the overall ductility.

Fig. 3(c) shows the pushover curves for the interior transverse frame of the eighteen story building. The inclusion of the infill panels increases the overall stiffness by 38% and the failure load by 78%. Again, the overall ductility capacity of the structure is reduced.

4.3 Dynamic response of the buildings

The dynamic response of the buildings has been determined through a set of inelastic dynamic time history analyses of the building frames under three different levels of seismic hazard, which are UHS-500, UHS-1000, and UHS-2500. Spectrum compatible artificial ground motion records corresponding to the above levels of hazard have been used in the analysis. Also a set of sixteen actual ground acceleration records scaled to represent UHS-2500 events corresponding to Vancouver have been used in the analysis. A summary of the dynamic response parameters is given in Tables 8 and 9, while the interstory drifts are shown in Fig. 4. The damage index values are found to be

Table 8 Details of the synthesized stochastic ground motion records

GMR	L1	L2	L3	L4	S1	S2	S3	S4
	UHS 2500							
Total Duration (s)	18.18	18.18	18.18	18.18	8.53	8.53	8.53	8.53
Peak Acc. (cm/s^2)	244.2	221.1	248.6	242	523	416	567	339
Peak Acc. (g)	0.25	0.23	0.25	0.25	0.53	0.42	0.58	0.35
	UHS 1000							
Total Duration (s)	17.98	17.98			6	6		
Peak Acc. (cm/s^2)	163.24	146.7			230.4	275.8		
Peak Acc. (g)	0.166	0.149			0.235	0.281		
	UHS 500							
Total Duration (s)	19.66	19.66			6	6		
Peak Acc. (cm/s^2)	68.57	65.75			206.6	224		
Peak Acc. (g)	0.07	0.067			0.211	0.228		

Table 9 Maximum interstory drift due to the synthesized records

Building	Hazard Level	Bare Frames				Infilled Frames	
		IDARC2D		DRAIN-2DX			
		Drift (%)	Performance Level	Drift (%)	Performance Level	Drift (%)	Performance Level
Six Story	UHS-500	0.54	LS	0.55	LS	0.33	OP
	UHS-1000	0.86	LS	1.07	LS	0.6	LS
	UHS-2500	2.07	NC	2.26	NC	1.37	LS
Twelve Story	UHS-500	0.76	LS	0.6	LS	0.33	OP
	UHS-1000	0.54	LS	0.63	LS	0.52	LS
	UHS-2500	2.25	NC	1.76	NC	0.98	LS
Eighteen Story	UHS-500	0.46	OP	0.44	OP	0.42	OP
	UHS-1000	0.91	LS	0.71	LS	0.4	OP
	UHS-2500	2.12	NC	1.62	NC	1.36	LS

underestimated and they are not shown in Tables 8 and 9 to save space. The response of the buildings under these three levels of seismic hazard is discussed below.

4.3.1 Performance under UHS-500 events

Using four synthesized UHS-500 records (2 long and 2 short) the performance of the bare and infilled frames has been evaluated. Because of the small number of records used in the analysis, the envelope values are used in the performance evaluation as the mean and standard deviation values are not so meaningful in this case. The data presented in Table 9 shows that the performance of the bare frame model of the six storey building could be classified as *life safe* as the maximum interstory drift is found to be 0.55% of story height using DRAIN-2DX and 0.54% using IDARC2D. As for the infilled frame, the maximum interstory drift is found to be 0.33% of story height which classifies the building as *operational*.

For the twelve story building, the maximum interstory drift of the bare frame is found to be 0.6% of story height using DRAIN-2DX and 0.76% using IDARC2D. The performance of the bare frame model could be classified as *life safe.* On the other hand, the infilled frame can be considered *operational* as the maximum interstory drift is found to be 0.33% of story height.

For the eighteen storey building, the maximum interstory drift of the bare frame is found to be 0.44% of story height using DRAIN-2DX and 0.46% using IDARC2D. As for the infilled frame, the maximum interstory drift is found to be 0.42% of story height. The performance both the bare frame model as well as the infilled frame model could be classified as *operational.* The value of the damage index is lower than 0.25 in all cases.

4.3.2 Performance under UHS-1000 events

Similar to UHS-500, four synthesized records (2 long and 2 short) are available for UHS-1000, which have been used in the analysis. The performance of the six story building under this level of earthquake can be said to be *life safe*. The maximum value of the interstory drift of the bare frame is found to be 1.07% of story height using DRAIN-2DX and 0.86% using IDARC2D. The maximum

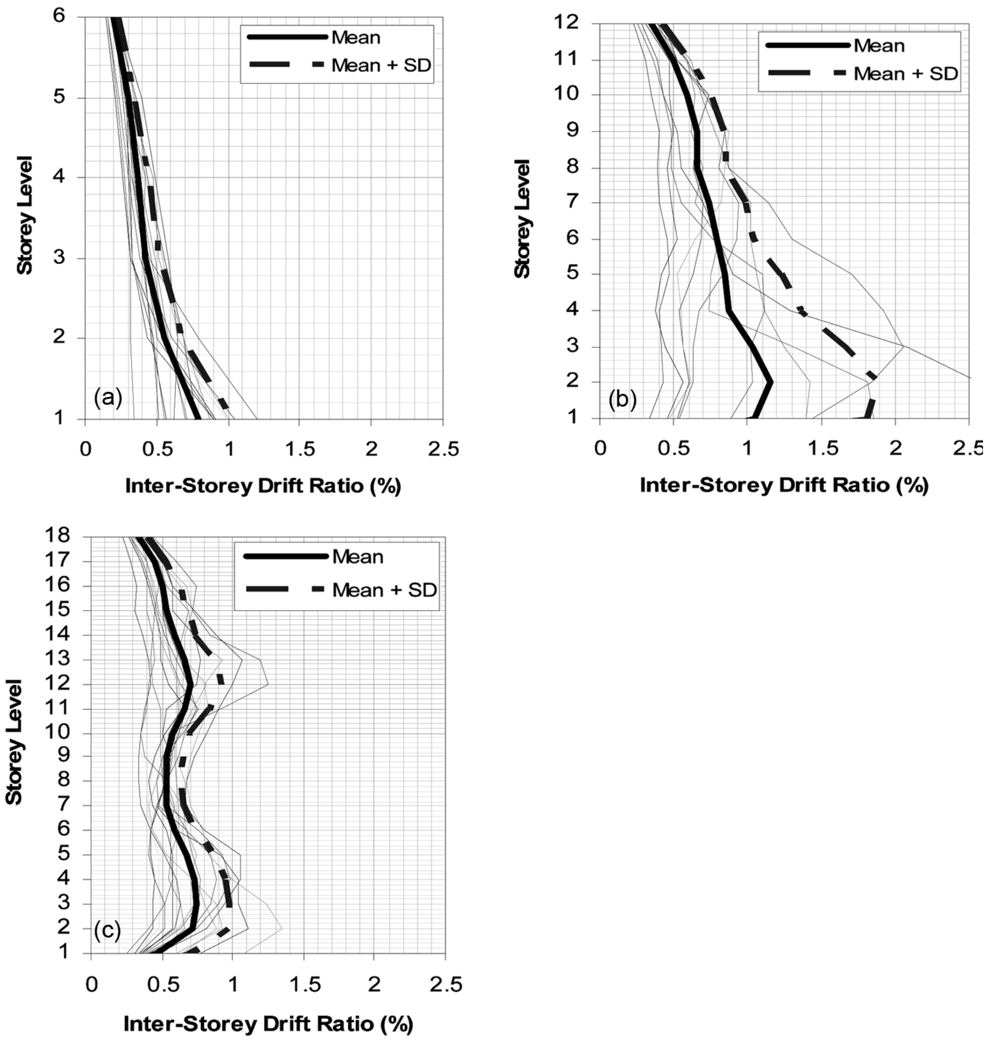

Fig. 5 Interstory drifts due to the scaled records in the infilled frames (a) six story, (b) twelve story and (c) eighteen story frames

interstory drift of the infilled frame is 0.6% which also corresponds to a *life safe* performance.

The performance of the twelve story building considering the bare frame when subjected to UHS-1000 records is estimated to be *life safe* as the maximum interstory drift is 0.63% of story height using DRAIN-2DX and 0.54% using IDARC2D. The performance of the infilled frame is also considered to be *life safe* since the maximum interstory drift of the infilled frame is found to be 0.52%.

For the eighteen storey building, the maximum interstory drift of the bare frame is 0.71% of story height using DRAIN-2DX and 0.91% using IDARC2D. The performance of the bare frame in this case can be said to be *life safe.* On the other hand, the performance of the infilled frame could be considered *operational* since the maximum interstory drift of the infilled frame is found to be 0.4%. The value of the damage index is lower than 0.3 in all cases.

Table 10 Mean plus standard deviation interstory drift values due to the scaled UHS-2500 records

Building	Bare Frames				Infill Frames	
	IDARC2D		DRAIN-2DX			
	Drift (%)	Performance Level	Drift (%)	Performance Level	Drift (%)	Performance Level
Six Story	1.23	LS	1.4	LS	1.02	LS
Twelve Story	1.37	LS	1.31	LS	N/A	Collapse
Eighteen Story	1.31	LS	1.56	NC	0.97	LS

4.3.3 Performance under UHS-2500 events

The dynamic response of the building models under UHS-2500 is examined using two different sets of ground motion records, 8 synthesized records (4 long and 4 short) and 16 actual records scaled to fit the NBCC 2005 design spectrum for Vancouver. The mean plus standard deviation value is calculated for the interstory drift values resulting from the actual records, the envelope values are used in the case of the 8 synthesized records.

For the six story building, bare frame model, the performance can said to be *near collapse* in the case of the synthesized records and *life safe* in the case of the actual records. The performance of the infilled frame model can be considered to be *life safe* in the case of both types of records. The value of the damage index is lower than 0.4 in both cases.

In the case of the twelve story building, the performance of the bare frame can be considered to be *near collapse* when synthesized records are used, *life safe* when the actual records are used. The value of the damage index is close to 0.4 in this case. As for the infilled frame, the performance was found to be *life safe* in the case of the synthesized records while in the case of the scaled records the frame collapsed under the effect of six ground motion records. The value of the damage index exceeds 0.4 in this case. This result could be attributed to the following possible reasons:

- Infilled frames attract more forces than bare frames do due to their higher stiffness and the participation of the higher modes in the seismic response. If the increase in the attracted forces does not match the increase in strength then the effect of the infill panels on the performance of the structure becomes negative.
- As shown in the results of the pushover analysis, the ductility of the infilled frame is less than that of the bare frame. Infill panels have a negative effect on the inelastic behaviour of the structure.
- As the capacity design was applied to the bare frame, its behaviour the the sequence of plastic hinge formation follow predictable patterns. The hinging pattern of the infilled frames is unpredictable and in some cases columns may yield before the beams meeting at a certain joint and eventually the building collapses faster than expected.
- Furthermore, the records are scaled using the spectral acceleration at the fundamental period of vibration while the higher modes of vibration may play a significant role in the dynamic response of the infilled frame than in that of the bare frame, which is not reflected in the scaling method. Perhaps a scaling method based on multiple modal periods would be useful.

The performance of the eighteen story building is found to be similar to that of the six story building. For the bare frame model, the performance can said to be *near collapse* as for the infilled frame model, it can be considered *life safe*. The value of the damage index is close to 0.4 in both cases.

5. Conclusions

A set of three moment resisting frame buildings, six, twelve and eighteen stories high are designed using the equivalent static load method (ESLM) of the NBCC (2005). Based on the NBCC (2005), the six and twelve-story buildings are allowed to be designed using this method. However, the eighteen-story building, because it exceeds sixty meters of height, should be designed using dynamic analysis. The ESLM in that case is used to obtain a preliminary design.

Two analysis models are used to represent the structures, a bare frame model consisting of only the reinforced concrete structural skeleton (i.e., beams and columns) and an infilled frame model that includes masonry infill panels. The infill panels are assumed to be located in the mid bays of all stories of the three buildings, the inclusion of which is to give an assessment of the contribution of non-structural elements to the seismic performance of the structures.

Based on the NBCC (2005) requirements, the performance levels achieved by all buildings are found acceptable. All structures have achieved the "Collapse Prevention" requirement based on the response inter-story drifts due to the UHS-2500 compatible records. Based on the SEAOC Vision 2000 (1995) provisions, the bare frames have over-performed and reached a *life safe* performance level in the case of the scaled records where the Mean+SD values are used in the evaluation, while they achieved a *near collapse* performance level in the case of the synthesized records where the maximum values are used in the evaluation because of the limited number of records used in the analysis.

The results of the analysis for the UHS-500 records show that the six and twelve-story bare frames have achieved a *life safe* performance level, while the eighteen-story bare frame has achieved an *operational* performance level. The results of the analysis for the UHS-1000 records show that the bare frames have achieved a *life safe* performance level for all buildings.

The non-structural elements are found to have enhanced the performance of the structures in most cases. For the UHS-500 records, the infilled frame models of all buildings studied here are found to have achieved an *operational* level of performance. The analysis of the infilled frames for the UHS-1000 records shows that the six and twelve-story frames have achieved a *life safe* performance level while the eighteen-story frame has achieved an *operational* performance level.

The performance is found to be *life safe* considering the infilled frame models of the six and eighteen-story buildings when subjected to the UHS-2500 records. In this case of the twelve-story building, the infilled frame model have been found to have reached the instability limit due to six of the scaled ground motion records, and high interstory drifts have been recorded under several other such records. For one of the scaled ground motion records, the inter-story drift has exceeded the *collapse prevention* limit of 2.5%. Considering the dynamic response of the infilled frame model of the twelve-story building under UHS-2500, the performance can be categorized as *collapse*. The result can be attributed to the fact that the increase in strength due to the inclusion of the infill panels does not match the increase in the attracted force from several ground motion records. Also, the scaling method applied in this case, the ordinate method, is based on the first period of vibration and because the infilled frame is stiffer than the bare frame, the higher modes have a greater effect on the response of the structure.

The two nonlinear dynamic analysis programs were used in the dynamic response analysis of the bare frame models produce consistent results despite the modeling difference. For infilled frames, only DRAIN2DX has been used for the analysis as IDARC2D was not found suitable. Based on the present work, the following conclusions are made.

- The ESLM of the NBCC (2005) produces a safe design that is in some cases a little conservative.
- Nonlinear dynamic analysis is only significant when using a large number of records where mean values and standard deviations could be used to give a reliable indication of the performance achieved.
- Although in most cases, infill panels enhance the seismic performance of a structure; this is not always the case. While infill panels improve the elastic behaviour of the structure, they do not enhance the inelastic deformation characteristics of the structure. As shown from the results of the push-over analysis, the stiffness and strength of the structure increase with the inclusion of infill panels, however, the deformation capacity and the ductility decrease. This increase in strength may not always overcome the increase in the dynamic force attracted by the structure.
- The "strong column – weak beam" criterion is not always satisfied in the case of infilled frames since the infill panels are not considered in the design.

The results of the dynamic analysis for real ground acceleration records depend largely on the modeling, analysis program, and the scaling method employed. However, the results of the bare frames produced using two different analysis programs show consistency in the structural response. Only ordinate method of scaling is used in the study. Further studies are necessary to include other methods of scaling, such as the scaling based on the full or partial area of the response spectra, to determine the sensitivity of the dynamic response of the structures to the scaling methods.

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