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SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS WITH MOMENT RESISTING FRAMES DESIGNED USING NBCC 2005

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ABSTRACT

Most existing building codes deal with seismic design by specifying the minimum lateral forces which the structure should resist as well as the maximum allowable drifts under these forces. However, this approach alone is not enough to ensure that the required performance is achieved by the structure. The 2005 National Building Code of Canada (NBCC) uses a seismic hazard level corresponding to 2% probability of being exceeded in 50 years. The new version of the code advises the designer to carry out a detailed dynamic analysis to evaluate the performance of a structure although it still allows the use of the equivalent static method in the design except for tall and irregular buildings. In this paper, a set of three concrete moment resisting frame buildings are designed using the new code provisions and the seismic performance of these buildings is evaluated through dynamic analysis using a set of spectrum compatible synthetic records as well as scaled ground motion records. The buildings are assumed to be located in Vancouver and they are six, twelve and eighteen storey high. The static capacity and the dynamic response parameters such as the inter-storey drifts and the damage indices of the frames are used in evaluating the performance. The objective of this paper is to determine if the new code specification offers the desired level of performance under different earthquake ground motions.

Introduction

While dynamic analysis is the best way to deal with the seismic design of structures, it is still impractical and time consuming for design office use as rigorous analysis is required. However, with the exponential rate of growth and development in computing technology nowadays, detailed dynamic analysis is expected to be commonplace in the practice. Although the NBCC 2005 recommends dynamic analysis in seismic design, it still allows the use of the equivalent static force method in designing simple and regular buildings (Humar and Mahgoub, 2003). For taller and irregular shape buildings, NBCC 2005 recommends dynamic analysis (Saatcioglu and Humar, 2003). In this study, the equivalent static force method of the NBCC 2005 is used in designing three concrete moment resisting frame buildings. Modal and dynamic analyses are used for modifying the design the structures, and the dynamic time history analysis is performed for the evaluation of their seismic performance.

Unlike the previous code (NBCC 1995) that was based on a two parameter zoning approach with a 10% probability of exceedance in 50 years or a recurrence interval of 475 years, the new 2005 version is based

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on a uniform hazard spectra (UHS) that varies from one zone to another with a 2% probability of exceedance in 50 years corresponding to a recurrence interval of 2500 years (UHS-2500). The design base shear is given by Equation 1.

$$V = \frac{S(T)M_v I}{R_o R_d} W \geq \frac{S(2)M_v I}{R_o R_d} W \quad (1)$$

where V is the design base shear or equivalent static force, $S(T)$ is the design spectral response acceleration expressed as a ratio to gravitational acceleration, M_v is the higher mode factor, I is the importance factor of the building, R_o is the overstrength factor, R_d is the ductility capacity factor and W is the dead load of the structure plus 25% of the design snow load.

The objective of this paper is to (a) evaluate the new code provisions in the design of three concrete moment resisting frame buildings, (b) by evaluating the dynamic response parameters, determine whether the desired level of seismic performance is achieved, and (c) identify the dynamic analysis, what type of fine-tuning, if any, in the design is required.

Design of the Buildings

As previously mentioned, a set of three concrete moment resisting frame buildings assumed to be located in Vancouver, British Columbia are designed using the equivalent static force method specified by the NBCC 2005, refined using the modal and dynamic analyses, and then analyzed for ground motion records using nonlinear dynamic time history analysis. The generic configuration of the buildings is shown in Figure 1 where n is the number of storeys of the building. The buildings are six, twelve and eighteen storey high, and are composed of nine moment resisting frames spaced at six meters apart. The height of the ground floor and that of the typical floor are 4.55 and 3.65 m, respectively. Moment resisting frames are used in the short direction of the buildings for lateral load resistance. Because of the symmetry in the layout of the buildings and neglecting the effect of accidental torsion, the analysis model of each of the buildings is reduced to a two-dimensional intermediate frame.

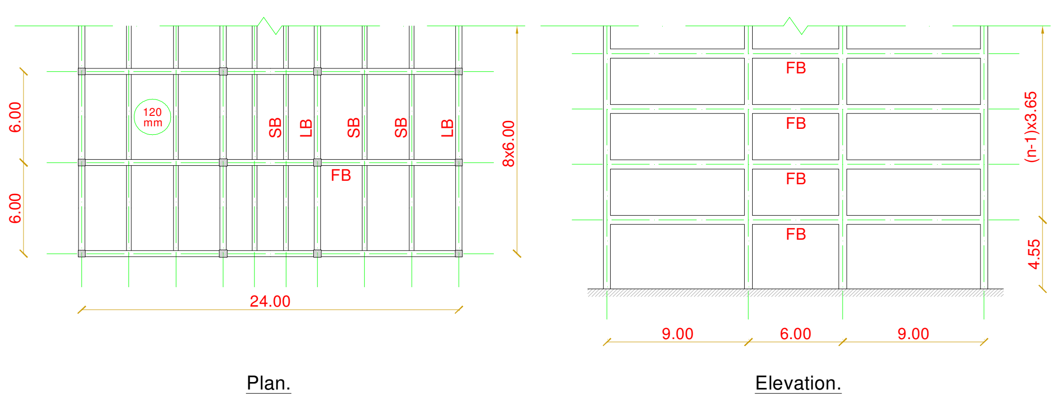

Figure 1. Generic plan and elevation of the buildings (all dimensions are in meters unless otherwise mentioned).

The frames have been designed to withstand the gravity load combination (1.25D + 1.5L) and the lateral load combination (D + 0.5L + E), where D, L and E are dead, live and seismic lateral loads, respectively. The seismic load, E is calculated based on the weight of the building including 25% of the snow load. The yield stress f_y for the reinforcing steel and the 28 day compressive strength of the used concrete are assumed to be 400 MPa and 30 MPa respectively. Live load is applied on all floors except the roof and is assumed to be 2.4 kN/m^2 on external bays and 4.8 kN/m^2 on internal bays. Snow load is applied on the roof and is assumed to be 2.2 kN/m^2. The importance factor of the structure is taken as 1. Secondary beams (SB) and longitudinal beams (LB) used in the long direction of the buildings are assumed to be

(300x350) mm^2 and (400x600) mm^2 respectively. The reinforced concrete slab used is 120 mm thick. The ductility and overstrength factors used to calculate the base shear are 4 and 1.7, respectively.

The fundamental period of vibration is calculated based on Eq. 2 which is an empirical code formula that usually tends to underestimate the period of vibration of the structure thereby increasing the design base shear which leads to an over-designed and less economic structure.

$$T = 0.075(h_n)^{3/4} \tag{2}$$

where T is the fundamental period of vibration and h_n is the height above the base to the top storey.

Table 1. Cross sections of columns.

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

Based on NBCC 2005, the designer is allowed to increase the design period using modal analysis. The period of vibration obtained from modal analysis is higher, so the code allows the increase of the design period up to the period calculated from modal analysis or 1.5 times the code period, whichever is less.

This process is done here and the updated design base shears are 294.3 kN, 360.7 kN and 529.5 kN for the six, twelve and eighteen storey frames respectively. The final cross sections obtained after a few design iterations are shown in Table 1 and 2.

Table 2. Cross sections of beams.

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

* All cross sections are 400x600 mm^2

Push-Over Analysis

Static pushover analysis is performed to simulate the response of the structure to incremental lateral loading (monotonically increasing). It gives an idea about the strength and ductility of the structure. Using IDARC2D (Valles *et al.* 1996), a nonlinear dynamic analysis computer program designed specifically for the analysis of reinforced concrete frame buildings, the pushover curves for the three frames are obtained and plotted in Figure 2. The base shear coefficient defined as the ratio of the base shear to the weight of the structure is plotted against the top storey deformation expressed as a percentage of the building height. The failure point is taken as the point where inter-storey drift reaches 2.5% in any storey level or the point of instability, whichever occurs first. In the case of these buildings the inter-storey drift level of

2.5%, not instability, governs the ultimate lateral load capacity. The base shear capacities of the six, twelve and eighteen storey frames are found to be 0.135, 0.08 and 0.059 respectively and the roof deformation capacities are found to be 1.46%, 1.56% and 1.46% respectively.

The pushover curves can be used to estimate the approximate values of the ductility factor and compare with that assumed in the design by dividing the overall deformation (i.e., roof drift) at the point of failure by the overall deformation at the end of the linear zone of the push-over curve which could be described as the yielding point of the structure. For that purpose, a bilinear indealization of the pushover curve is necessary. The point where the elastic response of the structures ends and the structure begin to behave inelastically as determined from the equivalent bilinear curve can be termed as the yield point of the system. Such yield point in the buildings studied here correspond approximately to the base shear coefficients of 0.13, 0.07 and 0.05 and the roof drift of 0.40%, 0.40% and 0.45% for the six, twelve and eighteen storey frames respectively. Estimated ductility factors obtained from the curves are 3.5, 4.1 and 3.1 for the six, twelve and eighteen storey frames, respectively, which are slightly lower than the design ductility factor R_d.

Another aspect to be considered is the yielding sequence of the elements. Based on the code provisions, plastic hinges should occur in horizontal members (*i.e.*, beams) before occurring in vertical members (*i.e.*, columns) at any joint. This is taken care of in the design by making sure that the sum of the nominal flexural resistances of the columns meeting at a specific joint is higher than the sum of the probable flexural resistances of the beams meeting at the same joint. This design concept is known as the “capacity design” or “strong column – weak beam” requirement. The yielding sequence is examined and the points where the first yielding of a beam and a column occurred are reported here. In the six storey building, the first beam yielded at the base shear coefficient, C_v of 0.072, and the first column yielded at $C_v = 0.108$. Similarly in the twelve storey frame, the first beam yielded at $C_v = 0.042$ and the first column at $C_v = 0.071$. Finally, in the eighteen storey frame, the first beam yielded at $C_v = 0.0245$ and the first column at $C_v = 0.058$. The design base shear coefficients are 0.046, 0.028 and 0.027 for the six, twelve and eighteen storey frames, respectively.

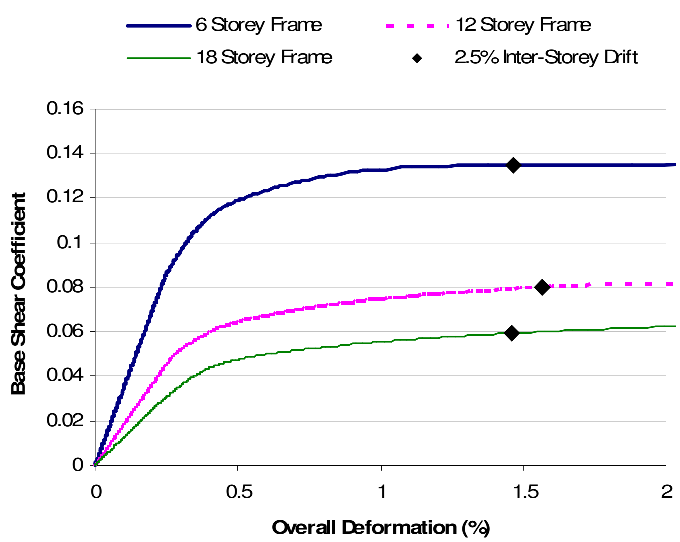

Figure 2. Push-over curves of the three frames.

Dynamic Analysis

Using IDARC2D, dynamic analyses have been carried out to evaluate the seismic performance of the three frames. Two sets of ground acceleration records corresponding to UHS-2500 (*i.e.*, the seismic events corresponding to a return period of 2500 years) are used in this evaluation; synthesized records compatible with the design spectrum of Vancouver (Tremblay and Atkinson, 2003), and actual records scaled to fit the design spectrum for Vancouver. Eight synthesized records are used in this study and they belong to the following two types; short duration records and long duration ones. The short duration records are records have relatively high magnitudes of peak acceleration as compared to the long duration records. The characteristics of these records are summarized in Table 3. In addition to the synthesized records, a suite of fifteen actual records are used in the analyses. These records were obtained from the database of the Pacific Earthquake Engineering Research Center (PEER, 2006) and they were selected by comparing the peak acceleration to the peak velocity ratio (A/V) that is compatible with the seismicity in Vancouver. These records are scaled using two different scaling methods: *(i)* the ordinate method, and *(ii)* the partial area method. The ordinate method is based on scaling the record to match the response spectral acceleration to the design spectral acceleration at the fundamental period of vibration. The partial area method is based on matching the area under the response spectral acceleration curve between 1.2 times the fundamental period and the second period of vibration to the same area under the design spectral acceleration curve (Naumoski *et al.*, 2004). The main characteristics of these ground motion records are shown in Table 4.

Table 3. Characteristics of the synthesized records.

Ground Motion	L1	L2	L3	L4	S1	S2	S3	S4
Total Duration (s)	18.18	18.18	18.18	18.18	8.53	8.53	8.53	8.53
Peak Acc. (g)	0.249	0.225	0.253	0.247	0.533	0.424	0.578	0.346

Table 4. Characteristics of the scaled actual records.

Ground Motion	N1	N2	N3	N4	N5	N6	N7	N8
Total Duration (s)	53.74	54.4	19.16	45	14.42	65.18	79.48	62.58
Peak Acc. (g)	0.35	0.18	0.16	0.05	0.04	0.15	0.21	0.17

Ground Motion	N9	N10	N11	N12	N13	N14	N15
Total Duration (s)	43	47.08	30	60	40.4	120	128
Peak Acc. (g)	0.18	0.20	0.07	0.08	0.17	0.11	0.09

Response Parameters

The Inter-storey drift is an important response parameter that can be used in the evaluation of the seismic response of a structure. Based on NBCC 2005, the maximum value of the inter-storey drift in a building should not exceed the 2.5% value when subjected to an earthquake excitation. As reported in the report of Vision 2000 (1995) report, four different types of seismic performance levels are defined and could be linked to the maximum transient inter-storey drift in a building. These levels of performance are shown in Table 5. Similar to the NBCC 2005, the 2.5% drift limit is specified in Vision 2000 (1995) as the criteria for the collapse condition. However, FEMA-273 (1997) uses an inter-storey drift of 5% as the collapse criteria (Table 6). This shows that the relation between the qualitative and quantitative measures of performance still needs refinement.

Table 5. Seismic performance levels (Vision 2000).

Performance Level	Fully Operational	Operational	Life Safe	Near Collapse
Transient Drift	<0.2%	<0.5%	<1.5%	<2.5%

Apart from the structural response parameters such as the inter-storey drift, ductility demand or dissipated energy to define the performance of a structure, a damage index can also be used for that purpose. A damage index is usually defined using the abovementioned response parameters in combination to assess the performance of a structure quantitatively. A damage index proposed by Park and Ang (1984) for reinforced concrete elements is well known. In the analysis carried out here using IDARC2D, Park and Ang damage index (Park et al., 1984) is calculated. Park and Ang damage index is defined using the amount of hysteretic energy dissipated in a structural member due to seismic excitation. Damage indices of individual elements in a structure are calculated, and then they are combined to determine the damage indices of each storey and the global damage index. Table 7 shows the interpretation of the global damage index (Park et al., 1986). The inter-storey drift and global damage index are used to estimate the performance of the buildings considered here.

Table 6. Seismic performance level (FEMA-273 1997).

Performance levels	Drift Limit	Residual Limit drift
Immediate Occupancy (IO)	0.7%	-
Collapse Prevention (CP)	5.0%	5.0%

Table 7. Interpretation of the overall damage index (Park et al. 1986).

Degree of Damage	Collapse	Severe	Moderate
Damage Index	>1	0.4 – 1	<0.4
State of Building	Loss of Building	Beyond Repair	Repairable

Performance of the Six Storey Frame

Figure 3 shows the maximum inter-storey drifts due to the synthesized records at the different storey levels of the lateral load resisting frames of the buildings considered here. Because of the limited number of synthetic records available for the analysis, calculating the statistical parameters, such as, the mean values and standard deviations is not very meaningful. Instead, the envelope values of the inter-storey drifts are considered in the evaluation of the performance. Long duration records (denoted by L) are considered separately from the short duration ones (denoted by S). The maximum inter-storey drift due to long duration records is caused by record L3 and occurs at the first storey level and is equal to 2.07% while the other three long duration records give relatively low inter-storey drifts. The maximum inter-storey drifts caused by record L1, L2 and L4 are 0.88%, 0.88% and 0.65%, respectively and all occur at the first storey level. However, since only the envelope values are to be considered, the six storey frame could be considered as “Near Collapse”. Although the short records cause higher average drift, the maximum inter-storey drift due to these records is 1.46% caused by record S1 at the first storey level. Therefore, the performance of the structure due to the short records could be considered as “Life Safe”. Figures 4 and 5 show the maximum inter-storey drifts of the three buildings at each storey level due to the actual records scaled using the ordinate method and the partial area method, respectively. In these figures, “L Max” and “S Max” correspond to the drift envelopes due to the long and short duration records, respectively. The mean values of the maximum inter-storey drifts due to the scaled records are also shown in the figures as well as the “mean plus standard deviation” value. Based on the ordinate method, the maximum “mean plus standard deviation” drift value is 1.57% occurring at the first storey level. The maximum inter-storey drift is 2.83% at the first storey level and 2.87% at the second storey level due to record N5 which is higher than the maximum allowable drift. However, since the “mean plus standard deviation” is the value considered for evaluation, based on this method of scaling the structure would be considered “Near Collapse” according to Vision 2000 (1995) report. On the other hand, based on the partial area method, the “mean plus standard deviation” drift value is 1.42% occurring at the first storey level. The performance is considered as “Life Safe”.

Based on damage index analysis, the maximum damage index caused by the long duration synthesized records is 0.21 caused by record L3 and that caused by short records is 0.17 by S3. Based on Table 5, the damage could be considered moderate and the structure repairable. The damage in the case of the scaled records is also moderate since the mean value plus standard deviation of the damage indices is 0.14 and 0.16 in the case of the ordinate method and the partial area method respectively.

Performance of the Twelve Storey Frame

The maximum inter-storey drift due to the long duration record L3 is 1.7% occurring at the fourth storey level. Similar to the six storey frame, the short duration records cause higher average drift ratio. However, in the case of the six storey frame, the maximum drift is caused by one of the long duration records namely, L3. In the case of the twelve storey frame, the maximum drift is caused by the short duration record, S3 and is equal to 2.25% and occurs at the second storey level. The performance of the structure in both cases is "Near Collapse". Considering the actual records, the maximum "mean plus standard deviation" is 1.6% and 1.26% using the ordinate method and the partial area methods, respectively, both occurring at the third storey level. The performance could be considered in the case of the ordinate method as "Near Collapse" and in the case of the partial area method as "Life Safe".

The results of the damage index analysis show that the maximum damage index caused by the synthesized records is 0.27 caused by L3 and 0.24 caused by S1 (long and short records, respectively). In the case of scaled records, the "mean plus standard deviation value" is found to be 0.21 using the ordinate method of scaling and 0.18 using the partial area method. On this basis, the damage could be classified as moderate, and the structure is considered repairable.

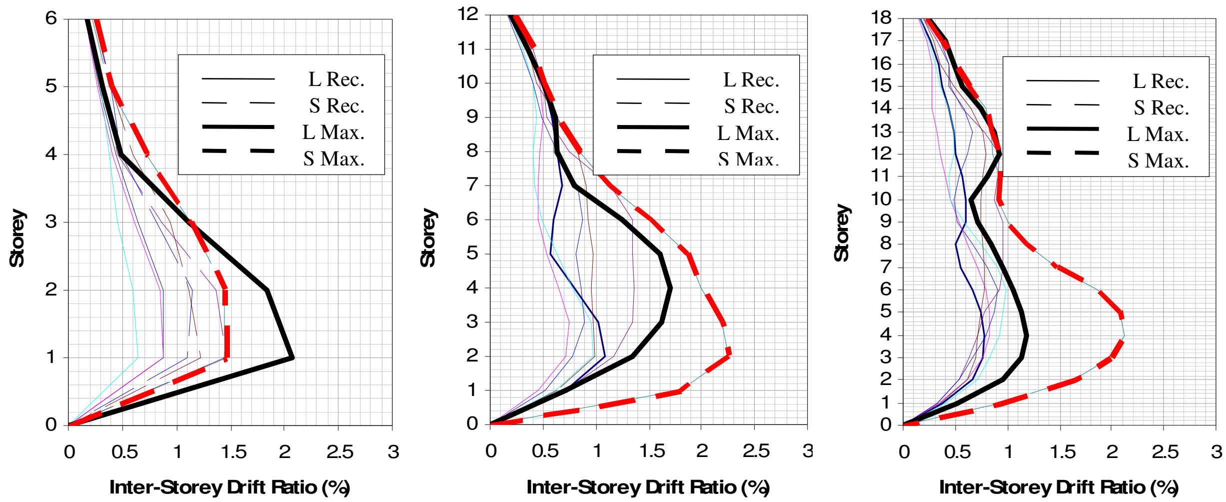

Figure 3. Maximum inter-storey drifts due to synthesized records; (a) 6 storey frame, (b) 12 storey frame and (c) 18 storey frame.

Performance of the Eighteen Storey Frame

Due to the long duration record L3, the maximum inter-storey drift is 1.18% occurring at the fourth storey level. The building is "Life Safe" in this case. The maximum inter-storey drift caused by the short duration records is 2.12% due to record S3 and also occurs at the fourth storey level. Therefore the building is "Near Collapse" in the case of short records. The results of the analysis for actual records show a maximum "mean plus standard deviation" of 1.68% and 1.43% both at the seventh floor level based on the ordinate method and the partial area method respectively. Similar to the two other buildings, the ordinate method estimates a "Near Collapse" performance while the partial area method estimates a "Life Safe" performance.

Finally, the global damage indices of the eighteen storey moment resisting frame also indicate that the structure is repairable and that the amount of damage is moderate. The maximum damage index caused by long duration records is 0.18 due to L3 while that caused by short duration records is 0.3 due to S3. The scaled records give a “mean plus standard deviation” damage index of 0.18 due to both scaling methods.

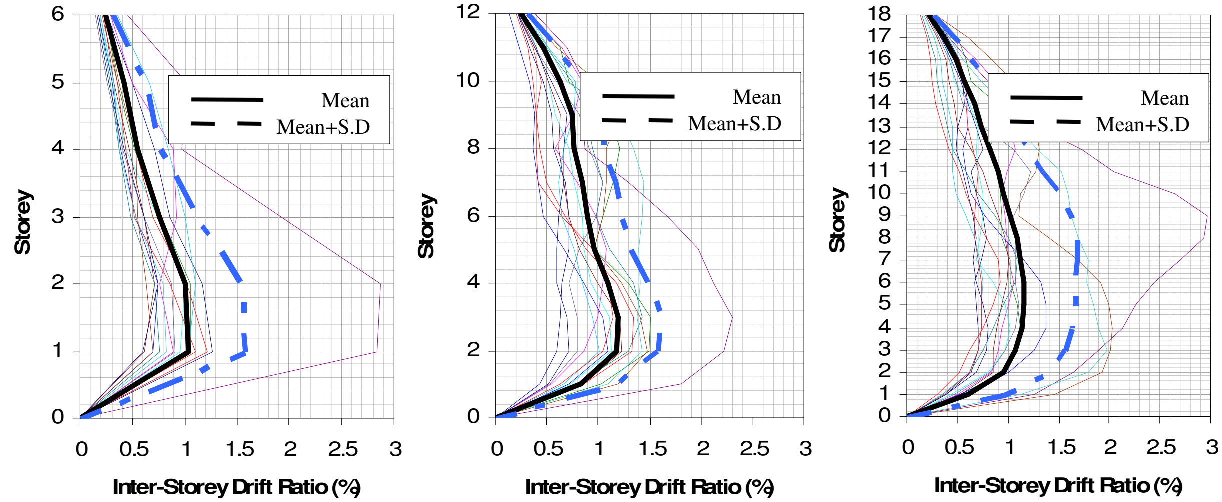

Figure 4. Maximum inter-storey drifts due to actual records scaled using the ordinate method; (a) 6 storey frame, (b) 12 storey frame and (c) 18 storey frame.

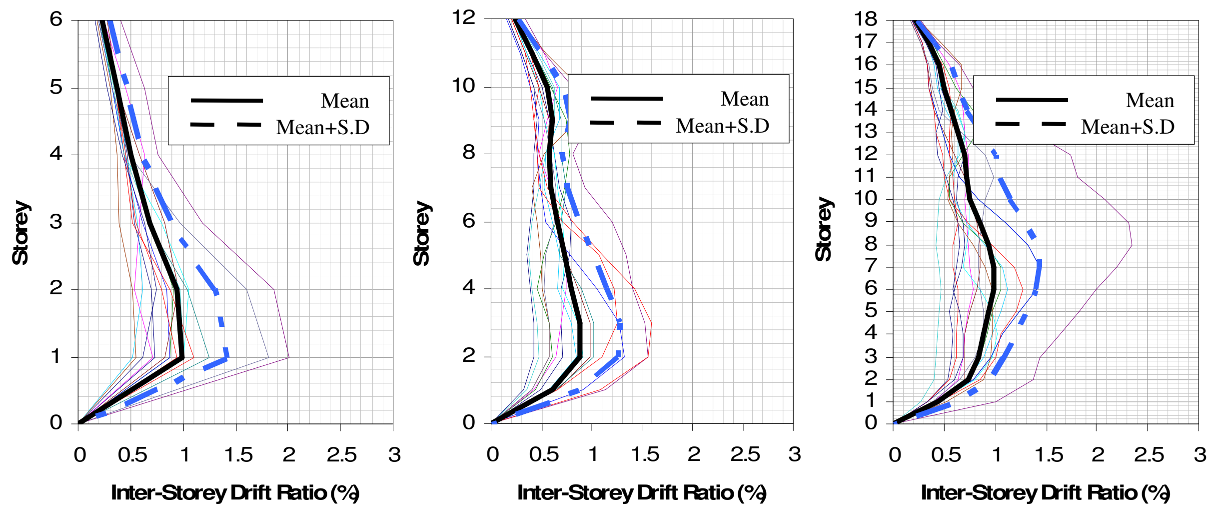

Figure 5. Maximum inter-storey drifts due to actual records scaled using the partial area method; (a) 6 storey frame, (b) 12 storey frame and (c) 18 storey frame.

Conclusions

While the push-over results might show good response to lateral loading by the three structures designed using the NBCC 2005, dynamic analysis is recommended. The evaluation of the seismic performance based on inter-storey drifts gives a good indication of the response of the structure to ground motion records. Damage indices are also an important tool in performance assessment and should be utilized in conjunction with the inter-storey drifts to obtain a better picture of the structural seismic response.

The results of the performance analysis and evaluation of the three structures under consideration in this study could be summarized as follows; all buildings based on the provisions of the NBCC 2005 have achieved the required level of performance by not exceeding the 2.5% maximum allowable inter-storey drift. However, based on Table 3 (Vision 2000, 1995), the performance was in between “Life Safe” and “Near Collapse” while based on Table 4 (FEMA-273, 1997) the performance was “Collapse Prevention”. Fine-tuning the design to reach a better performance could be done and damage indices of elements and storeys along with inter-storey drifts could be used to locate which sections which are most in need of upgrading. Furthermore, based on the damage index analysis, the designs look to be satisfactory as the amount of damage is moderate. From the pushover analysis, it is noted that the plastic hinges formation sequence is compatible with the “strong column – weak beam” concept. However, the ductility capacities determined from the idealized bilinear pushover curves are slightly lower than the value of ductility related force reduction factor, R_d as in Equation 1, used in the design.

Finally, because of the uncertainty in the dynamic analysis results in general and in how much the response parameters relate to the overall performance of the structure, relating drift values to performance levels may not be enough for seismic response evaluation. Assessing the damage indices of the elements, the storeys and the overall structure is important and if carried out along with the assessment of drifts gives better results and a better design.

Acknowledgments

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