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Performance of Six Storey Reinforced Concrete Frame Structures Subjected to Strong Seismic Ground Motions

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ABSTRACT

This paper presents the results of a detailed performance evaluation of several design variations of a generically configured six-story reinforced concrete moment-resisting frame building located in Vancouver and designed in accordance with the 1995 edition of the National Building Code of Canada. Pushover and time-history dynamic analyses were conducted using an inelastic model which incorporates tri-linear moment-curvature relationships at the end sections of each beam and column simulating nonlinear behaviour that takes into account stiffness degradation and pinching effects during hysteretic loading. The performance expectations of the various frame designs in terms of interstorey drift are evaluated and compared in order to assess both the overall level of protection and the influence of the different design parameters.

INTRODUCTION

Current building codes have benefitted considerably from experience gained during past earthquakes, including detailed studies of specific buildings as well as extensive damage surveys. However, the extensive damage and number of collapses which have occurred during recent earthquakes (e.g. Northridge in 1994 and Kobe in 1995) have heightened ongoing concerns about the level of protection afforded to buildings designed and constructed in accordance with current codes. A recent study (Heidebrecht, 1997) describes a research framework for the evaluation of the seismic level of protection. This paper is concerned with the application of that methodology to the evaluation of the seismic performance of medium height reinforced concrete moment-resisting frame structures. The context of the investigation is that the design of the frames comprising the structure has been done in accordance with the seismic provisions of the 1995 edition of the National Building Code of Canada (Associate Committee on the National Building Code, 1995), which is referred to here as NBCC 1995. Also, the detailing of the frame members and joints has been done in accordance with the companion Canadian materials standard for reinforced concrete structures (Canadian Standards Association, 1994).

DESCRIPTION OF FRAMES

Building Configuration

The generic building configuration used in this investigation comprises a six-storey office building with 7 bays in the longitudinal direction and 5 bays in the transverse direction; the building plan is shown in Figure 1. The storey heights are 4.0 m, with the exception of the bottom storey which has a height of 5.2m. The lateral load resisting structural system in each direction comprises moment-resisting reinforced concrete frames. There are six frames to resist earthquake motions in the longitudinal direction: two each of types L1, L2 and L3. In the transverse direction, earthquake motions are resisted by only the two end frames, each marked T on the plan in Figure 1. With this configuration of structural systems, the design of the longitudinal frames is governed by both gravity and lateral loads. The design of the transverse frames is dominated by lateral loads, since each frame carries one-half the lateral load of the entire building while carrying only the gravity load of the adjacent half-bay. The floor system comprises a one-way slab spanning in the transverse direction, supported by the beams of the L1, L2 and L3 type frames. The slab is cast integrally with the beams.

Seismic Design

In NBCC 1995, the minimum lateral seismic force (base shear) V is given by

$$V = (V_e / R) U \tag{1}$$

in which U = 0.6 is a calibration factor, R = a force modification factor (values range from 1 to 4), and V_e = elastic lateral seismic force, which is given by

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$$V_e = v\ S\ I\ F\ W \tag{2}$$

in which v = zonal velocity ratio (corresponding to peak ground velocity in m/s), S = seismic response factor (a function of structural period), I = importance factor (1 for buildings of normal importance), F = foundation factor (1 for buildings on rock or stiff soil), and W = dead load.

The location chosen for this design is Vancouver, B.C., a region of moderate seismic hazard with a zonal velocity ratio v = 0.20. Since the building has office occupancy, I = 1 and the site is assumed to be on rock so that F = 1. Three variations of moment-resisting frame are included: I) fully ductile (R = 4), ii) nominally ductile (R = 2), and iii) non-ductile (R = 1.5). The material properties were chosen to be the same for the slabs, beams and columns: f'_c = 30 MPa and f_y = 400 MPa, except that f_y = 300 MPa for the slabs. The total dead load W of the structure is approximately 160,000 kN. The fully ductile and nominally ductile frames were designed with two different interstorey drift limits, one being that specified in NBCC 1995 (2% of storey height) and the other being a more stringent drift limit of 1%. In all other respects, all design variations fully satisfy the requirements of both NBCC 1995 and CSA A23-3-94, including the application of capacity design principles for the determination of column moment capacities at the joints for the fully ductile frames. For these frames (R=4), base column moment capacities were designed to be in the same proportion to the capacities at the top of the first storey as the moments determined from an elastic analysis using code lateral loads.

This paper is concerned only with the performance of the transverse frames, since the design of these is dominated by seismic rather than gravity loads. The member sizes and primary design features for the five design variations are given in Table 1; details of steel reinforcement are given by Naumoski and Heidebrecht (1997). For the frames designed using the 1% drift limit, the maximum interstorey drift ratio computed from code loading (including P-Δ effects) is 0.8%, i.e. 20% below the design limit; this difference arises primarily because of the need to choose member section sizes conservatively to the nearest 10 cm. While drift controls the design of the 1% drift limit frames, the design of those with the NBCC 1995 limits is controlled by strength rather than drift, resulting in a computed drift of only 1.2%, which is 40% below the NBCC limit of 2%.

<u>Modelling for Performance Analysis</u>

For the purpose of determining the performance of the frames when subjected to earthquake ground motions, inelastic models of each frame were developed for use in an inelastic dynamic analysis program, a McMaster enhanced version of IDARC (Kunnath et al. 1992). Moment-curvature relationships for the end sections of each beam and column were determined using fibre analysis of the cross-sections. The concrete stress-strain relations included the effect of confinement, based on the model proposed by Mander et al. (1988). The moment-curvature relations were simplified into a tri-linear model with the first segment corresponding to the uncracked stiffness, the second segment corresponding to the region between cracking and yielding, and the third segment to the post-yielding range. The stiffness degradation and pinching effects were taken into account in the analyses using a hysteretic model which closely approximates experimentally observed behaviour. More detailed information on the modelling is given by Naumoski and Heidebrecht (1998).

STATIC PUSHOVER ANALYSIS

Pushover analyses of the frames were conducted by applying a monotonically increasing pattern of lateral loads which are distributed along the height of the structure in the same manner as used in the design. Figure 2 shows the results of the pushover analyses in terms of total lateral load (base shear) vs. maximum interstorey drift; the design base shear for each frame is also shown on this figure. The fully ductile frames have much lower drifts at design loads than the nominally ductile and nonductile frames. Similarly the drift at which significant yielding occurs is much less for the fully ductile frames. The frames designed with the 1% drift limit are slightly stiffer and have slightly higher load capacities than the comparable frames designed using the NBCC drift limit of 2%.

DYNAMIC ANALYSIS

<u>Seismic Excitation</u>

Each structure was subjected to an ensemble of 15 time-histories having spectral shapes similar to those of design level seismic ground motions expected in Vancouver. Spectral shapes are related to the a/v ratio, in which a is the peak ground acceleration, in units of "g", and v is the peak ground velocity, in units of m/s. The values of a and v for Vancouver, as shown in the NBCC 1995 seismic zoning maps, are both 0.20, so that a/v = 1. The selected ensemble (Naumoski et al., 1993) has an average a/v of 1.02, with values for individual records ranging from 0.82 to 1.21. Each of these time-histories was scaled in terms of its peak horizontal velocity, on the basis that the design is velocity-dependent and that the response of structures with periods ranging from 0.5 to 2.5s is related primarily to the peak ground velocity rather than to the peak ground acceleration of the earthquake motion. In order to determine the performance for a full range of excitations which could be expected during the lifetime of a structure, excitations ranged from 0.1 m/s to 0.6 m/s. While the highest excitation level corresponds to 3 times the design level, the uncertainty in estimating peak ground motions at any location is such that values ranging from 2 to 3 times the expected value can easily occur (Heidebrecht, 1995).

Performance Parameters and Criteria

The maximum transient drift in each storey was determined from the dynamic response due to each seismic excitation. The maximum values for all the time-histories at each excitation level were analysed statistically in order to determine drifts at the mean (M) and mean plus one standard deviation (M+SD) confidence levels. The M+SD values are used in the subsequent comparisons because of the requirement that there be a high level of confidence that damage will be less than some specified value; the M+SD level represents approximately an 84% level of confidence.

SEOAC (Vision 2000 Committee, 1995) has proposed a performance based approach to seismic design. Performance level is an expression of the maximum permissible extent of damage to a building when subjected to specific earthquake design ground motions (i.e. frequent, occasional, rare and very rare). Performance levels are defined as: fully operational, operational, life safe, near collapse and collapse; each has an associated damage state, ranging from negligible to complete. Design performance objectives are defined for facilities of varying significance, i.e basic, safety critical and essential/hazardous, by specifying the minimum performance level associated with each specified earthquake design level. For example, basic facilities are expected to perform at a life safe level when subjected to ground motions associated with a rare earthquake level, i.e. having a probability of exceedance of 10% in 50 years.

Of interest in the evaluation of performance is the fact that SEAOC has specified a direct link between the performance level and the maximum permissible transient interstorey drift. The maximum permissible transient drifts, expressed as a percentage of the storey height, and qualitative statements about expected damage for the three intermediate performance levels are:

operational performance: 0.5% drift, light overall building damage, negligible damage to vertical load carrying elements, original strength and stiffness retained in lateral load carrying elements with minor cracking/yielding of structural elements

life safe performance: 1.5% drift, moderate overall building damage, light to moderate damage of vertical load carrying elements with substantial remaining capacity to carry gravity loads, some reduction of residual strength and stiffness in lateral load carrying elements with lateral system remaining functional

near collapse performance: 2.5% drift, severe overall damage, moderate to heavy damage of vertical load carrying elements which continue to support gravity loads, negligible residual strength and stiffness in lateral load carrying elements

In this paper, the evaluation of performance is based on comparing the M+SD confidence level interstorey drifts with the above drift criteria.

Interstorey Drift

Figure 3 shows the distribution of overall maximum transient interstorey drifts for the ductile frame variations (A1T and A2T) at all six levels of excitation. This figure shows that, at each excitation level, there is a significant variation of maximum drift, due to variations of frequency content and duration among the time-histories. While the distributions for both frames overlap somewhat at each excitation level, it is clear that the drifts for frame A1T are consistently less than those for frame A2T. For both frames, the variation increases with the level of excitation; the coefficient of variation (COV) is typically in the range of 0.2 to 0.25 at the design excitation level (0.2 m/s), and it increases to about 0.4 at an excitation of 0.6 m/s. These values of COV confirm the importance of using M+SD confidence level results for evaluation in order to ensure a consistently high level of confidence that these values will not be exceeded.

Figure 4 shows the relationship between M+SD overall (i.e. maximum of all storey maxima) transient drifts and excitation velocity for all five frames. The M+SD overall drift for the 1% drift limit frames (A1T and B1T) at any excitation are less than that for the comparable NBCC drift limit frames; the difference is greater at the higher excitation levels. For both drift limits, the fully ductile and nominally ductile frames have very similar drifts at most excitation levels. The nonductile frame (C2T) has somewhat lower drifts than the other two frames when excitation levels are above 0.2 m/s. All frames have very similar drifts when excitations are at the design level of 0.2 m/s.

Considering the M+SD overall drift in relation to the SEAOC criteria, all frames satisfy the requirement for operational performance (0.5% transient drift) when the excitation is at the design level of 0.2 m/s. The SEAOC life safe limit of 1.5% drift is only exceeded at excitation velocities well above the design level. The life safety margin (LSM) is defined as the ratio of the velocity at which a maximum drift of 1.5% is reached (at the M+SD confidence level) to the design velocity. The LSM is a measure of the extent to which a frame can sustain high ground motions without jeopardizing life safety. The LSM values for the 1% drift limit frames are over 2.5 while the comparable NBCC drift limit frames have LSM values of just over 2. On this basis, the life safe performance of the 1% drift limit frames is superior to that of the NBCC drift limit frames.

With respect to near collapse considerations, it is assumed here that a realistic upper limit of expected excitation, which reflects the uncertainty of hazard estimates mentioned previously, is 3 times the design level, i.e. 0.6 m/s. Based on this assumption, drifts of the 1% drift limit frames at that excitation level will still be somewhat below the near collapse level, whereas drifts in the NBCC drift limit frames can be expected to reach the near collapse level. On this basis also, the performance of the 1% drift limit frames is superior to the NBCC drift limit frames.

DISCUSSION AND CONCLUSIONS

Based on the M+SD confidence level drifts, all frames meet the SEAOC Vision 2000 operational performance specification when excited at the design level. Life safety performance can be expected at excitations of 2 and 2.5 times the design excitation, for the NBCC and 1% drift limit frames respectively. Assuming that near collapse conditions should not occur below excitations of 3 times the design excitation, the 1% drift limit frames would not be near collapse at that excitation but the NBCC drift limit frames are expected to be very near collapse.

On the basis of the above, the use of a 1% design interstorey drift limit rather than the 2% used in NBCC makes very little difference in performance at the design excitation level, but improves performance considerably with reference to life safety and near collapse conditions. The slight amount of extra strength and stiffness which needs to be provided to achieve the 1% drift limit results in substantial improvement of drift performance at high excitations.

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Table 1. Description of Frame Variations

Frame variation designation	A1T	A2T	B1T	B2T	C2T
System ductility	Fully ductile		Nominally ductile		Non-ductile
Reduction factor R	4		2		1.5
Design drift limit (%)	1	2	1	2	2
Design seismic load V (kN)	5300	4850	10600	9700	12920
Elastic period (s)	0.90	1.16	0.90	1.16	1.16
Maximum interstorey drift ratio, computed from code loading (%)	0.8	1.2	0.8	1.2	1.2
Column size (cm)	100 x 100	90 x 90	100 x 100	90 x 90	90 x 90
Beam size (cm), width x overall depth	100 x 110	50 x 110	100 x 110	50 x 110	50 x 110

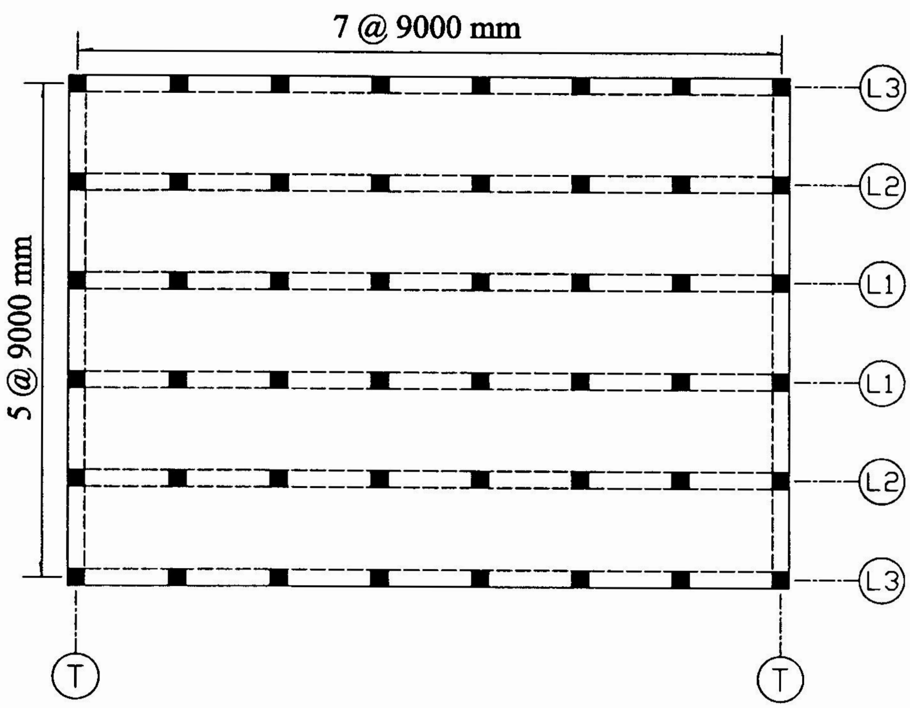

Figure 1 Plan of Six Storey Reinforced Concrete Frame Building

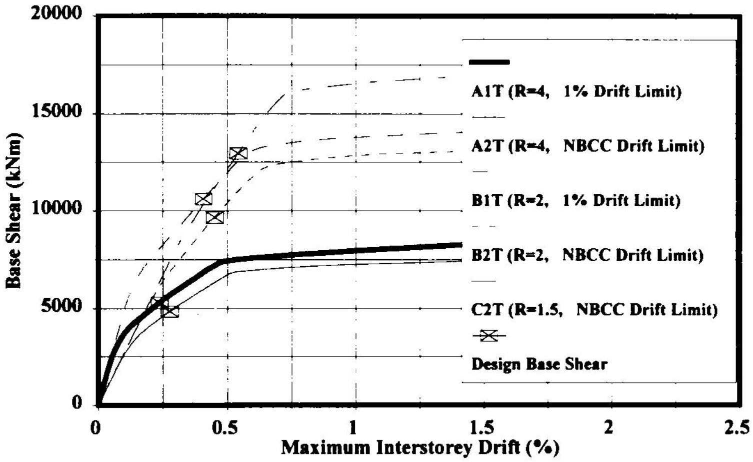

Figure 2 Pushover Analysis Results

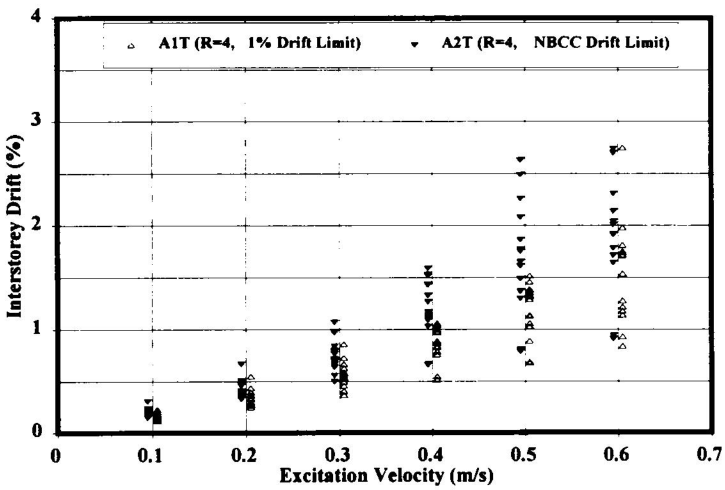

Figure 3 Maximum Transient Interstorey Drifts

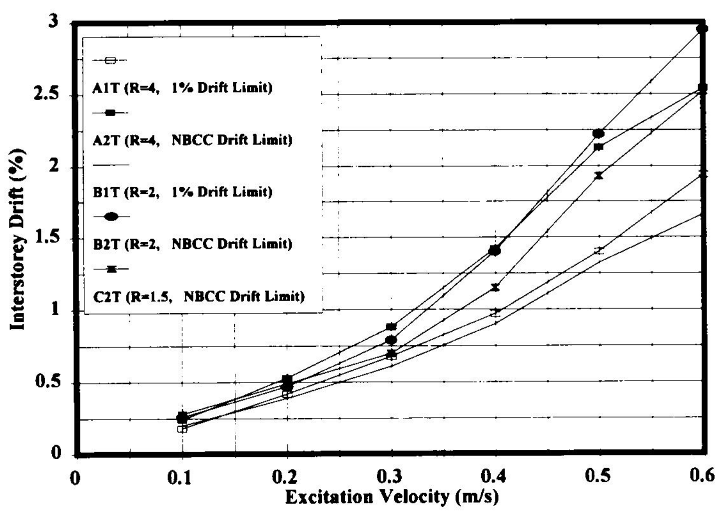

Figure 4 Mean Plus One Standard Deviation (M+SD) Interstorey Drift