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PERIODS OF REINFORCED CONCRETE FRAMES DURING NONLINEAR EARTHQUAKE RESPONSE

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SUMMARY

This paper presents the results of a detailed evaluation of the relationships between post-excitation periods of medium height reinforced concrete frame structures and both the inelastic damage of the structure and the intensity of seismic ground motion. 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 was analysed by conducting nonlinear dynamic analyses using a selected ensemble of excitation time-histories. Post-excitation periods for design level excitations are only marginally larger than the elastic periods but increase by 50% to 100% for excitations at three times the design level excitation. While there is some scatter for the different excitation time-histories, the relationships between structural response parameters at the mean plus one standard deviation level (M+SD) and the corresponding post-excitation periods are very nearly linear. These results can be used to estimate the extent of period elongation associated with specific deformation levels, i.e. interstorey drift and member curvature ductilit

INTRODUCTION

It is well known that reinforced concrete frame structures soften during nonlinear seismic response due to increased cracking and deterioration of the concrete at post-yield levels of deformation; one effect of this softening is the elongation of periods. While softening is also usually accompanied by a loss of strength, which is an undesirable effect, period elongation can be beneficial in reducing the effect of the seismic excitation because spectral accelerations generally decrease with increasing period. While there has been some evidence of the extent of period elongation from observations of structures damaged during earthquakes, there is limited information on the relationship between the extent of period elongation and the intensity of strong seismic ground motion.

A recent study of the performance of medium height (six storey) ductile reinforced concrete frames subjected to varying levels of strong seismic ground motion [Heidebrecht and Naumoski 1999] included the determination of the fundamental periods immediately after the cessation of the seismic ground motion. The objectives of this paper are to show how the periods of frames lengthen with increasing levels of seismic excitation and to present relationships between structural period and degree of inelastic deformation. The frames were designed 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].

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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 5.2 m. 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 the types marked L1, L2 and L3 on the plan in Figure 1. In the transverse direction, earthquake motions are resisted by only the two end frames, each marked T. 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 for non-ductile lateral load resisting systems to 4 for fully ductile systems), and V_e = elastic lateral seismic force, given by

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

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 $v = 0.20$, which represents the expected maximum velocity at a 10% in 50 year probability of exceedance. Since the building has office occupancy, $I = 1$ and the site is assumed to be on rock so that $F = 1$. Since the frames are designed to be fully ductile, $R = 4$. 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 150,000 kN. The design of the structure fully satisfies 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. 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. Only the L2 type longitudinal frame was designed; the other longitudinal frames are assumed to have the same reinforcing steel ratios.

The member sizes and primary design features for the transverse and L2 longitudinal frames are given in Table 1; details of steel reinforcement are given by Naumoski and Heidebrecht [1997].

Modelling of the Frames

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].

Table 1. Member sizes and percentages of longitudinal reinforcing steel

FRAME		Transverse		Longitudinal	
Elastic Period (s)		1.16		1.04	
COLUMN SIZE (cm)		90 x 90		90 x 90	70 x 70
Column location:		ext	int	ext	int
Percentage longitudinal reinforcing steel for columns entering joints at floor level indicated to right	6	1.64	1.68	1.68	2.04
	5	**0.99**	**0.99**	**0.99**	**1.22**
	4	**0.99**	1.48	1.48	**1.22**
	3	**0.99**	2.07	2.07	**1.22**
	2	1.33	2.42	2.42	**1.22**
	1	1.73	2.42	2.42	**1.22**
	base	2.96	3.46	3.46	3.27
BEAM SIZE (cm) width x overall depth		50 x 110		35 x 70	
Position of steel:		top	bott.	top	bott.
Percentage longitudinal beam reinforcing steel for beams at floor level indicated to right	6	**0.36**	**0.36**	1.02	0.49
	5	0.55	**0.36**	1.22	0.61
	4	0.82	0.55	1.43	0.82
	3	1.15	0.82	1.63	0.82
	2	1.27	1.02	1.84	1.02
	1	1.40	1.15	1.84	1.02

DYNAMIC ANALYSIS

Seismic Excitation

Each frame 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 Ag≅, 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].

Results

The maximum values of interstorey drift and curvature ductility demand in the beams and columns are determined for each time-history. The structural response for 10 seconds of free vibration after the end of each earthquake excitation is analysed to determine the post-excitation first mode periods. This is done by computing Fourier amplitude spectra for the post-excitation free vibration displacement time histories at each floor level.

Figure 2 shows the post-excitation periods of both frames for each time-history as a function of the excitation velocity; this figure also shows lines connecting the mean periods at each excitation. Note that the elastic periods for the transverse and longituidnal frame, as given in Table 1, are 1.16s and 1.04s respectively. It can be seen from this figure that there is very little period elongation in the transverse frame for excitation at the design velocity, with a mean period of 1.3s, which is about 10% greater than the elastic period. However, at the same excitation level, the longitudinal frame shows an increase of 25%. The extrapolation of the transverse frame periods to zero excitation tends towards the elastic period of 1.16s, which is to be expected. However the same extrapolation of the longitudinal frame period tends to a value somewhat larger then the elastic period of 1.04s. This deviation is because some of the beams in the longitudinal frame have already cracked due to gravity loads prior to any dynamic excitation being applied, which leads to a zero excitation period which is larger than the elastic period computed from the stiffnesses of uncracked sections.

At higher excitation levels, there is increased scatter of period, due to different amounts of inelastic deformation for the different time histories. The mean periods for both frames show extensive softening of the structure; mean period elongation for v = 0.6 m/s, i.e. 3 times the design level, is about 65% for both frames. Filiatrault et al. [1998] reports a doubling of period at excitations of twice the design level based on shaking table tests on two 1/2 scale model frames. Figure 2 shows that at twice the design excitation, the greatest elongation is approximately 60%. However, given the scatter of results and the significant effect of duration of ground motion on period elongation, these results are not inconsistent with those of Filiatrault et al.

Figure 3 shows the relationships between post-excitation periods and maximum interstorey drifts for both frames, both for each time history and mean values at each excitation level. Given the scatter of the results shown in Figure 2, there is much less scatter in the maximum drift associated with a specific period, particularly at the lower drift levels. Also, the relationship between mean values is essentially linear and is almost identical for the two frames, even though each has quite different strength and stiffness characteristics. A maximum drift of 1% is associated with 25% and 45% increases in period (from the elastic values) for the transverse and longitudinal frames respectively; the comparable increases for 2% drift are 65% and 85% respectively.

Figure 4 shows the relationships between post-excitation periods and maximum beam and column curvature ductilities, both for each time history and mean values at each excitation level. The beam curvature relationships are quite similar for the two frames but the column curvatures are markedly different. The mean value relationships are very nearly linear in all cases. Figure 4a shows that increases in beam curvature ductility are accompanied by rather modest changes in period. However, as shown in Figure 4b, the gradients of period increase with increasing column curvature ductility are much larger, particularly for the longitudinal frame. While the longitudinal frame columns remain elastic for all excitations, there is a tremendous change in post-excitation periods. This is actually not surprising because the softening is taking place due to inelastic beam deformation and not column deformation.

The scatter among these relationships is much larger than for maximum interstorey drifts, which is quite expected given that the detailed pattern of local deformations varies a great deal among frames subjected to different excitations. Beam curvature ductilities in the order of 8 in the transverse frame are associated with post-excitation periods ranging from 1.4 to 2 s. The scatter for transverse frame column curvatures is similar. As expected, the scatter for longitudinal frame column curvatures is very large because, as indicated above, elastic column curvatures have no relationship to softening in this frame

DISCUSSION AND CONCLUSIONS

Based on the results presented and discussed above, it is observed that the extent of softening of these reinforced concrete frame structures, as represented by period elongation, varies significantly after the inelastic deformation begins to take place. In these frames, and particularly in the longitudinal frame, most of that inelastic deformation takes place in the beams rather than in the columns. However, for any given level of beam curvature ductility, there is considerable scatter of post-excitation period. This indicates that the extent of softening is also affected by other factors, most likely the differences in hysteretic energy dissipation arising primarily from differences in the duration of strong motion excitation. For these frames at least, maximum beam curvature ductilities in the order of 10 are associated with period increases (from the elastic value) ranging from 50 to 100%.

There is a somewhat better correlation between post-excitation periods and maximum interstorey drift. This is no doubt due to the fact that maximum drift is in some sense the integrated effect of beam and column deformations, which tends to smooth out differences among maximum beam and column curvature ductilities. There is very little scatter for maximum drifts below 1% and increased scatter above that level, due to an increasing extent of inelastic deformation associated with the higher drift levels.

The quantification of period increases associated with deformations, both drifts and member curvatures, can be used to improve estimates of structural performance at high levels of seismic excitation. Effective spectral accelerations, for a given level of deformation, can be determined by calculating these based on elongated period rather than elastic periods. Use of the full post-excitation period is probably not realistic, and further research is needed to determine the appropriate period which should be used, i.e. one which is likely to be slightly less than the post-excitation period

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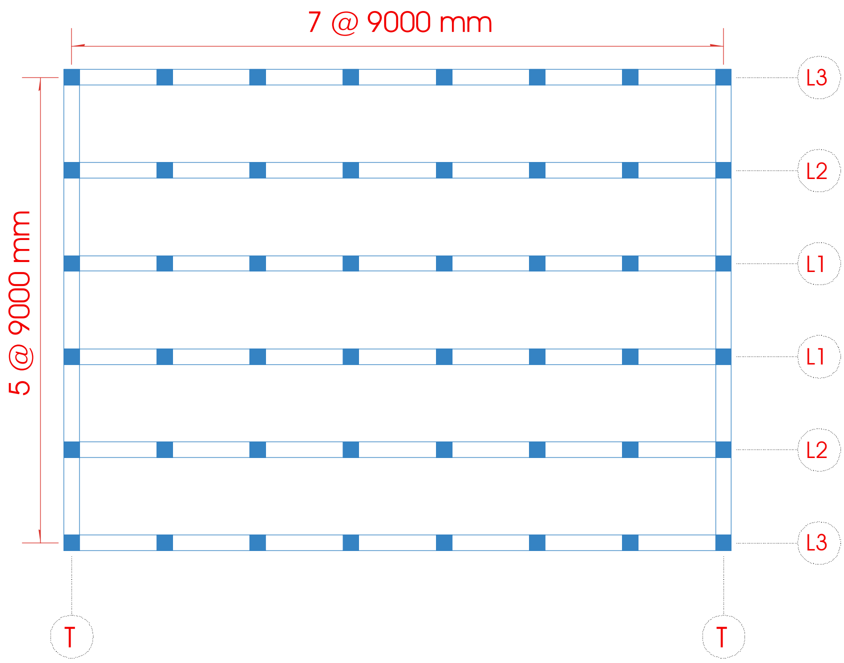


Figure 1 Plan View of Six Storey Reinforced Concrete Frame Structure

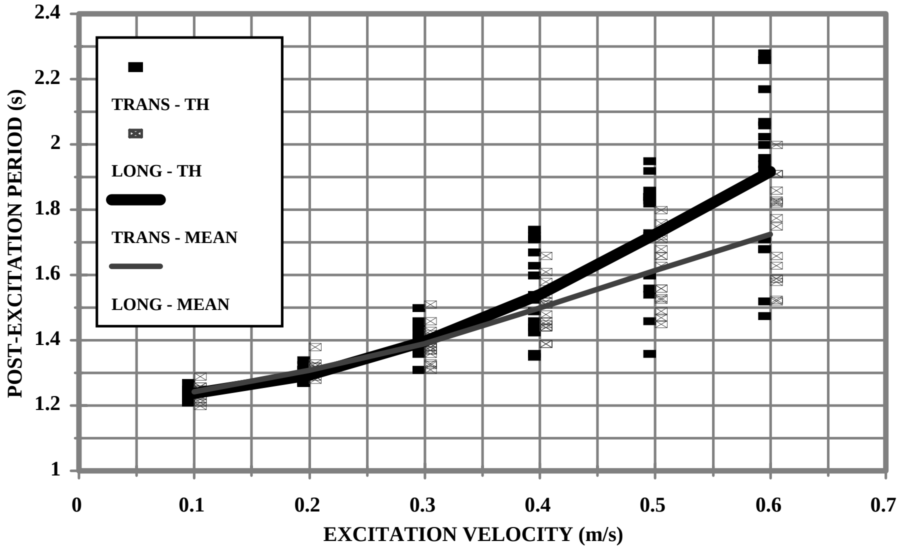


Figure 2 Individual Time-History and Mean Post-Excitation Periods Vs. Excitation Velocity

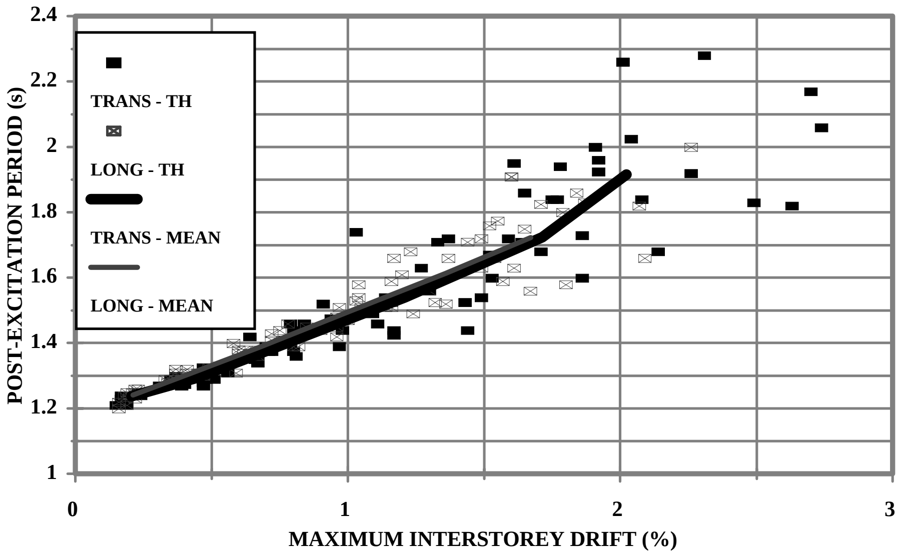
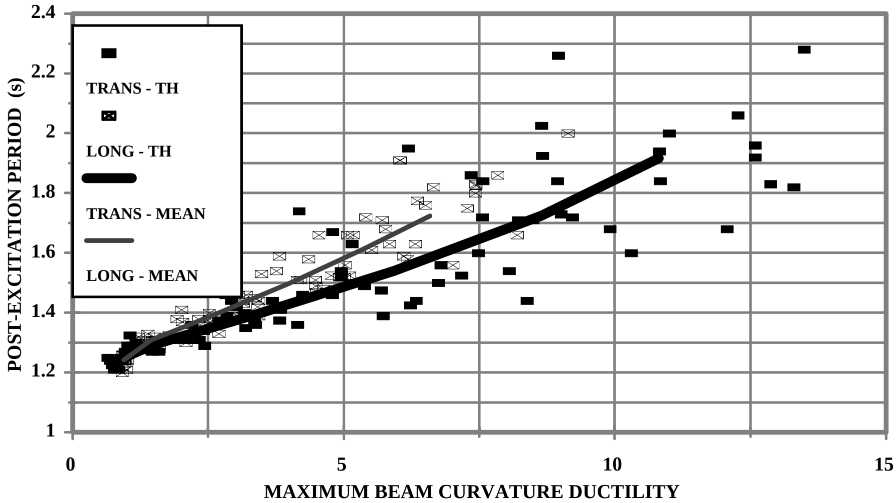
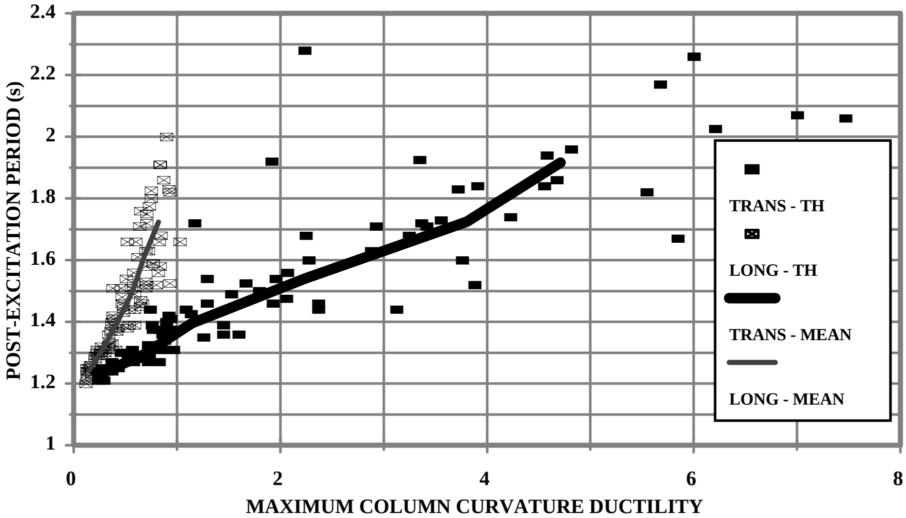


Figure 3 Individual Time-History and Mean Post-Excitation Periods Vs. Maximum Interstorey Drift



Beam Curvature Ductility



(b) Column Curvature Ductility

Figure 4 Individual Time-History and Mean Post-Excitation Periods Vs. Maximum Member Ductiliti