ELONGATION OF THE FUNDAMENTAL PERIODS OF REINFORCED CONCRETE FRAME BUILDINGS DURING NONLINEAR SEISMIC RESPONSE

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ABSTRACT:

This paper presents results from a study on the relationship between the increase (i.e., the elongation) of the fundamental periods of reinforced concrete frame buildings designed according to the National Building Code of Canada and the intensity of seismic ground motions. For the purpose of this study, three buildings were designed including a 4-storey, a 10-storey, and a 16-storey building, that can be considered typical of low, intermediate, and high rise buildings respectively. The buildings were designed for the city of Vancouver, Canada, which is in a high seismic region. A set of 40 recorded accelerograms representative of seismic motions in the Vancouver region were selected for use in the seismic analysis. Each building was subjected to a series of seismic excitations producing responses that ranged from elastic to significant inelastic responses. The free vibrations of the buildings, after the end of each excitation motion, were analyzed to determine the post-excitation fundamental (i.e., the first mode) periods. The computed periods were statistically analyzed to determine the relationship between the elongation of the fundamental period and the intensity level of the seismic excitation.

KEYWORDS: reinforced concrete, building, period, elongation, seismic, analysis.

1. 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 the structural vibration period. While softening is 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 intensity of strong seismic ground motions. The objective of this paper is to investigate such relationship for reinforced concrete frame buildings. This was done by seismic analyses of three reinforced concrete buildings, which are described hereafter.

2. DESCRIPTION OF BUILDINGS

Three reinforced concrete frame buildings were used in this study. Figure 1 shows the plan and the elevations of the buildings. The buildings are for office use and are located in Vancouver, which is in a high seismic hazard zone (NRCC 2005). The buildings are the same in plan but have different heights. As shown in the figure, the buildings include a 4-storey, a 10-storey, and a 16-storey building, which are considered representative of low-rise, medium-rise and high-rise buildings respectively.

The plan of each building is 27.0 m x 63.0 m (Fig. 1). The storey heights are 3.65 m. The lateral load resisting system consists of moment-resisting reinforced concrete frames in both the longitudinal and the transverse directions. There are four frames in the longitudinal direction (designated *Le* and *Li* in Fig. 1; *Le* – exterior frames, and *Li* – interior frames) and eight frames in the transverse direction (*Te* and *Ti*). The distance between both the longitudinal and the transverse frames is 9.0 m. Secondary beams between the longitudinal frames are used at the floor levels in order to reduce the depth of the floor slabs. The secondary beams are supported by the beams of the transverse frames. The floor system consists of a one-way slab spanning in the transverse direction, supported by the beams of the longitudinal frames and the secondary beams. The slab is cast integrally with the beams.

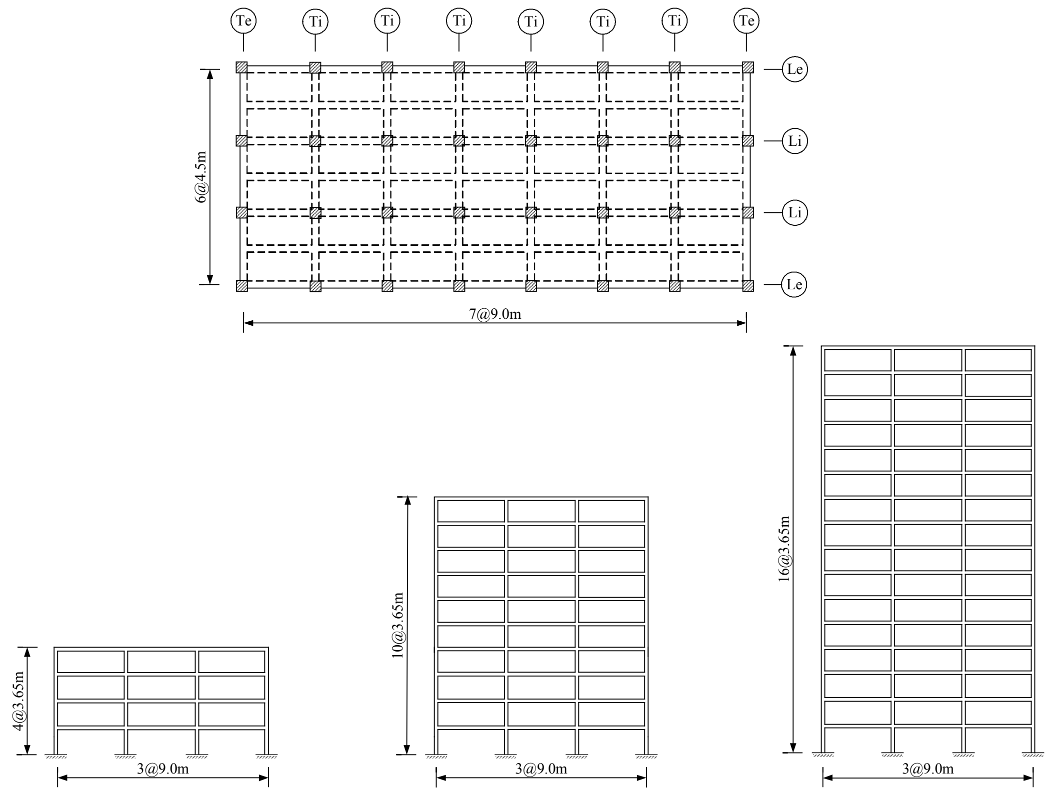

Figure 1 Plan of floors and elevations of transverse frames of the buildings.

3. DESIGN OF FRAMES

In this study, only the interior transverse frames (*Ti*) of the buildings were considered. For ease of discussing, the 4-storey, the 10-storey, and the 16-storey frames are referred to as the 4S, the 10S, and the 16S frames respectively. The frames were designed as *ductile* reinforced concrete frames. The gravity and the seismic loads were determined according to the 2005 edition of the National Building Code of Canada (NBCC) (NRCC 2005). Each frame was treated as an individual structural unit with its own gravity and seismic loads.

The lateral loads due to earthquake motions were determined in accordance with NBCC using the equivalent static force procedure. 'Reference' ground conditions, represented by site class C in NBCC, were assumed at the building locations. The seismic base shear force for each frame, *V*, was computed according to the code formula:

$$V=S(T_a)\cdot M_V\cdot I_E\cdot W/(R_dR_o) \quad (3.1)$$

where, $S(T_a)$ is the design spectral acceleration at the fundamental lateral period of the frame, M_V is the higher mode effect factor, I_E is the importance factor, W is the total weight associated with the frame, R_d is the ductility-related force modification factor, and R_o is the overstrength-related force modification factor. The fundamental periods of the frames were computed according to the code formula for reinforced concrete moment-resisting frames, $T_a = 0.075h_n^{3/4}$, where h_n is the height of the frame above the base in meters. The design spectral accelerations, $S(T_a)$, were determined from the seismic design spectrum for Vancouver. The values of the other parameters used in Equation (3.1), as specified in NBCC, are: $M_V = 1$, $I_E = 1$, $R_d = 4$, and $R_o = 1.7$. The design values for the fundamental periods of the frames, T_a, the spectral accelerations, $S(T_a)$, and the base shear coefficients, V/W, are listed in Table 1.

Table 1 Design parameters for the frames.

Design Parameter	Frame 4S	10S	16S
Period, T_a (s)	0.56	1.11	1.58
$S(T_a)$ (g)	0.603	0.312	0.237
V/W	0.089	0.046	0.035
Max. drift (%)*	1.65	1.61	1.63

*Drifts are expressed as a percentage of the storey height.

The member forces for use in the design were determined by elastic analyses of the frames subjected to the combinations of gravity and seismic loads as specified in NBCC. Load-deflection (P-Δ) effects were taken into account in the analysis. As required by NBCC, maximum inelastic interstorey drifts were calculated as R_dR_o times the drift obtained from the elastic analyses. The maximum calculated drifts for the frames are given in Table 1. It can be seen that the calculated drifts are smaller than the design drift of 2.5% allowed by NBCC.

The member forces obtained from the elastic analyses were used in the design of the frames. The design was conducted in accordance with the requirements for ductile moment-resisting frames specified in CSA standard A23.3-04 (CSA 2004). Compressive strength of concrete $f_c' = 30$ MPa, and yield strength of reinforcement $f_y = 400$ MPa were used in the design. The dimensions of the columns and beams, and the reinforcement obtained from the design are given in Lin (2008).

4. MODELLING OF FRAMES FOR DYNAMIC ANALYSIS

In this study, the computer program RUAUMOKO (Carr 2004) was used for the inelastic dynamic analysis of the frames subjected to seismic motions. It is a two-dimensional (2-D) analysis program, which provides a wide range of modelling options. The program includes different types of elements for modelling structural members and a number of hysteretic behaviour models.

For each frame, a 2-D inelastic model was developed for use in RUAUMOKO. The beams and columns were modelled by a 'beam-column' element, which is represented by a single component flexural spring. Inelastic deformations are assumed to occur at the ends of the element where plastic hinges can be formed. The effects of axial deformations in beams are neglected. Axial deformations are considered for columns, but no interaction between bending moment and axial load is taken into account. A trilinear hysteretic model was selected for the columns, and a bilinear (modified Takeda) model was selected for the beams from the models available in RUAUMOKO. Both models take into account the degradation of the stiffness during nonlinear response. RUAUMOKO provides several damping models for nonlinear analysis. In this study, Rayleigh damping of 5% of critical was assigned to the first and the second vibration modes of the models. The damping was specified to be proportional to the initial stiffness of the models.

The first mode periods obtained by RUAUMOKO for the 4S, the 10S, and the 16S frames are 0.94 s, 1.96 s, and 2.75 s respectively. These are significantly larger than those used in the design (Table 1). This was expected since it is known that the code formula provides relatively small period values that lead to conservative seismic design forces.

5. SEISMIC EXCITATIONS

Ground motion records from earthquakes in the Vancouver region would be the most suitable for the analysis of the frames considered in this study. Since such records are not available, recorded ground motions from earthquakes in California were selected. It is commonly believed that the characteristics of crustal earthquakes that might occur in the Vancouver region are similar to those of California earthquakes.

For the purpose of this study, forty earthquake records were selected from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center. All the records were obtained at sites class C (shear wave velocities between 360 m/s and 750 m/s), which was assumed in the design of the frames. The records were obtained from 22 earthquakes with magnitudes ranging from 5.8 to 7.3, and at distances ranging from 10 km to 109 km. The peak ground accelerations of the records are between 0.05 g and 0.36 g. A detailed discussion of the characteristics of the records can be found in Lin (2008).

6. INVESTIGATION OF THE PERIOD ELONGATION

When a structure is subjected to a seismic motion, the response of the structure represents a combination of the contributions of different modes. Therefore, the periods of all the modes contributing to the response are contained in the response time history. When an increase in the modal periods occurs, as is the case during nonlinear response, this increase can be seen in the response time history.

In this investigation, the displacement time histories of the roof responses of the frames were used to determine the period elongation. Each frame was subjected to the selected records scaled to five intensity levels in terms of the spectral acceleration at the fundamental period of the frame models, i.e., $Sa(T_1)$. The five intensity levels are designated as $Sa(T_1)_{ref}$ to $5Sa(T_1)_{ref}$, where $Sa(T_1)_{ref}$ is named the *reference* intensity level (i.e., the *reference* spectral acceleration). Records scaled to the reference intensity level are expected to produce responses corresponding to a 'global' ductility of about 1.0 (i.e., elastic responses), where the global ductility is represented by the ratio of the maximum roof displacement obtained from nonlinear dynamic analysis of the frame, to the 'global' yield displacement of the frame from the pushover curve. Note that the reference spectral acceleration for each frame was determined by considering an equivalent inelastic single-degree-of-freedom system of the frame that was developed according to the method proposed by Fajfar (2000). The selected intensity levels $Sa(T_1)_{ref}$ to $5Sa(T_1)_{ref}$ are intended to produce 'global' ductilities of approximately 1.0 to 5.0 respectively. Expressed in terms of spectral acceleration, $Sa(T_1)$, the intensity levels range from 0.18 g to 0.90 g for the 4S frame, from 0.14 g to 0.70 g for the 10S frame, and from 0.11 g to 0.55 g for the 16S frame. These intensities are considered to be suitable for the investigation of the period elongation since they produce responses to the frames that are well in the inelastic range.

For illustration, Fig. 2 shows the roof displacement responses of the 10S frame subjected to same motion but scaled to two intensity levels. The response in Fig. 2(a) is for the motion scaled to $Sa(T_1)_{ref}$ (i.e., 0.14 g) and that in Fig. 2(b) is for the motion scaled to $5Sa(T_1)_{ref}$ (i.e., 0.70 g). Considering the intensity levels of the excitations (discussed above), Fig. 2(a) represents elastic response, and Fig. 2(b) represents inelastic response of the frame. Note that the duration of the excitation motion is 40 seconds, and the last 10 seconds of the response time histories are free vibrations of the frame, after the end of the excitation. It can be seen that there are significant differences in the periods of the elastic and the inelastic responses. The elastic response (Fig. 2(a)) consists almost entirely of a single-period vibration, i.e., the elastic period of the first mode. The response in Fig. 2(b), however, shows much longer period (especially after the 10 s mark), which is the elongated first mode period of the frame as a result of inelastic deformations of the frame members. A residual displacement resulting from inelastic deformations is also seen at the end of the free vibration in Fig. 2(b).

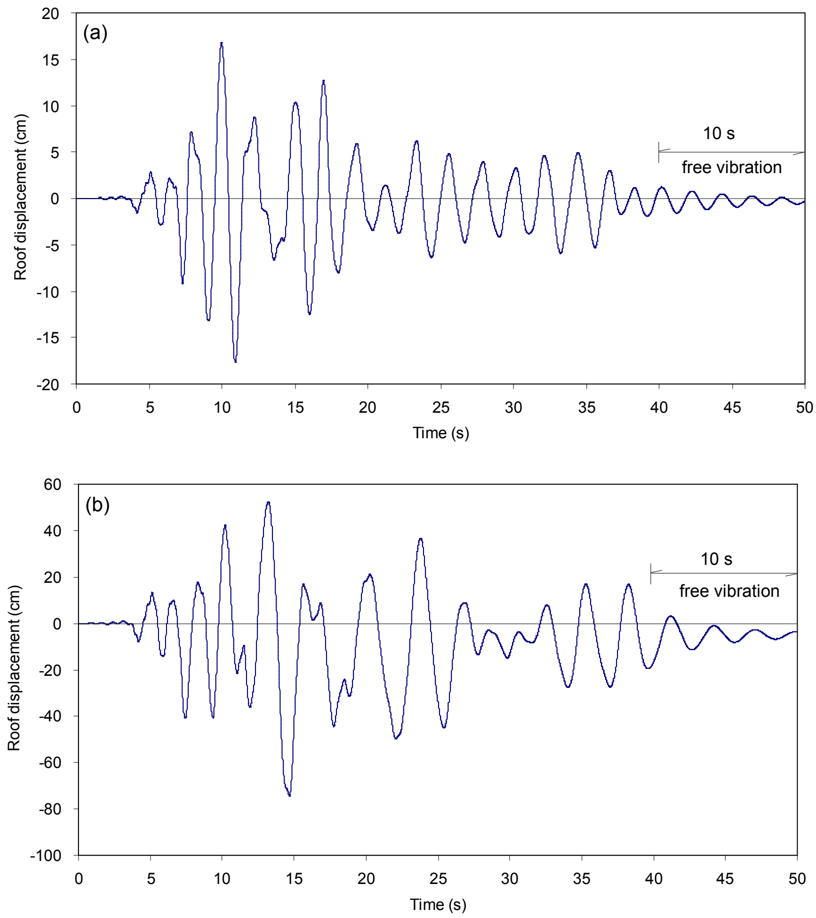

Figure 2 Displacement time histories of the roof response of the 10S frame subjected to ground motion scaled to: (a) $Sa(T_1)$=0.14 g, and (b) $Sa(T_1)$=0.70 g.

To quantify the elongation of the first mode period, Fourier amplitude spectra were computed considering the duration of the forced vibration of the responses (e.g., the first 40 seconds in Fig. 2). For some of the motions there were difficulties in identifying the elongated first mode period because several periods were present in the Fourier spectra. Given this, the elongation of the first mode period was determined by considering *only* the free vibration, after the end of the excitation motion. For each excitation motion, 10 seconds free vibration was considered. Figure 3 shows the Fourier amplitude spectra of the free vibration responses in Fig. 2. It is seen that the period of the elastic response (Fig. 2(a)) is about 2.0 s which is close to the elastic first mode period of the 10S frame (T_1=1.96 s), and the period of the inelastic response (Fig. 2(b)) is 2.9 s, i.e., about 50% larger than T_1=1.96 s. Since these periods are determined from the free vibrations, after the end of the excitation motions, they are referred to as the post-excitation periods.

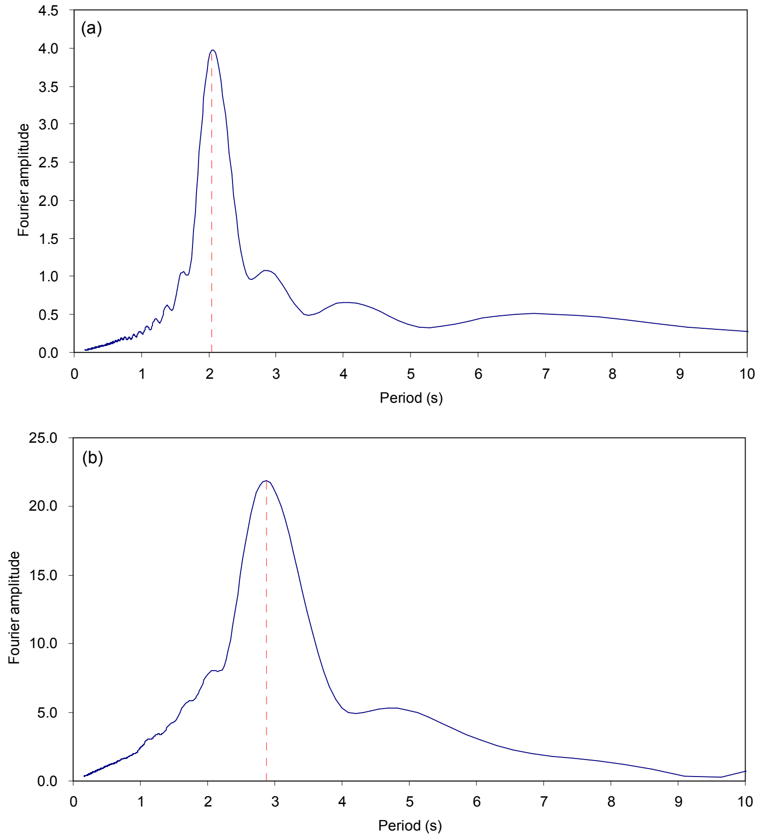

Figure 3 Fourier amplitude spectra for the free vibrations of the 10S frame (after the forced vibrations) due to ground motion scaled to: (a) $Sa(T_1)$=0.14 g, and (b) $Sa(T_1)$=0.70 g.

The post-excitation first mode periods obtained from the analyses of the 4S, the 10S, and the 16S frames are shown in Figs. 4(a), 4(b), and 4(c) respectively. The five stripes of results correspond to the excitation levels used in the analyses. Lines connecting the mean values of the periods at each excitation level are included in the figures to illustrate the general trends of the computed periods. The *elastic* first mode periods of the frames are also shown in the figures.

It can be seen in Fig. 4 that the *mean* periods increase almost linearly with the increase of the excitation. It was found from the figure that the mean period increase for the highest excitation level ($5Sa(T_1)_{ref}$) is about 55% for all three frames. This enables one to determine the period elongation for any 'global' ductility level considered. The dispersion of the periods also increases with the increase of the excitation. At the highest excitation level, the dispersion expressed by the coefficient of variation (COV) (which represents the ratio of the standard deviation to the mean period, i.e., σ/μ) is about 0.13 for all three frames.

Figure 4 also shows that the computed periods for excitations scaled to the *reference* intensity levels of the frames (i.e., the results for $Sa(T_1)$ of 0.18 g, 0.14 g, and 0.11 g for the 4S, the 10S, and the 16S frame respectively) are somewhat larger than the elastic periods of the frames. This indicates that the responses of the frames at the *reference* intensity levels are not purely elastic, but some inelastic deformations occur during the responses resulting in small lengthening of the elastic periods. This is expected because of the assumptions involved in the estimation of the reference levels, as discussed in Lin (2008). In general, Fig. 4 shows that the period elongation can be significant for higher excitation levels as a result of the softening of the frames due to inelastic deformations during the response.

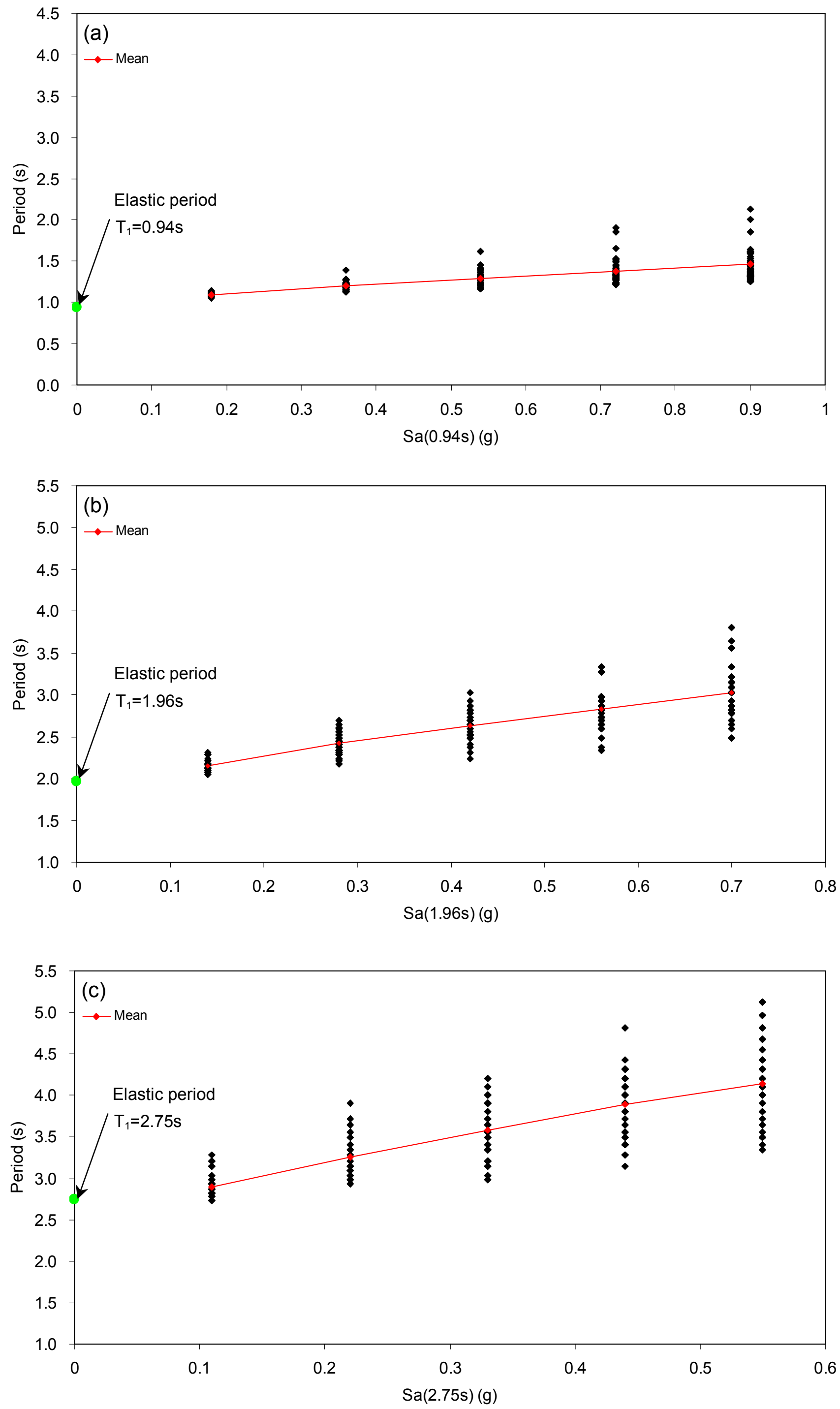

Figure 4 Post-excitation periods of the frames for ground motions scaled to five intensity levels of $Sa(T_1)$: (a) for the 4S frame, (b) for the 10S frame, and (c) for the 16S frame.

7. DISCUSSION AND CONCLUSIONS

Three reinforced concrete frames (4-, 10-, and 16-storey high) designed for Vancouver were used to investigate the elongation (i.e., the increase) of the first mode periods of the frames during nonlinear seismic response.

Forty records representative of seismic motions in the Vancouver region were used as excitation motions. Each frame was subjected to the selected records scaled to five intensity levels producing responses corresponding to global ductilities between 1.0 (i.e., elastic responses) and 5.0. Note that the 'global' ductility is represented by the ratio of the maximum roof displacement obtained from nonlinear dynamic analysis of the frame, to the 'global' yield displacement of the frame from the pushover curve. The displacement time histories of the roof responses of the frames were used to determine the period elongation.

It was found that the *mean* period elongation is almost linearly proportional to the intensity of the motions (Fig. 4). For the intensity producing a 'global' ductility of 5.0, the mean elongated period for each frame was found to be about 55% larger than the corresponding elastic first mode period. This enables one to determine the period elongation for any 'global' ductility level considered.

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