Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 810

EFFECTS OF STRONG-MOTION DURATION ON THE RESPONSE OF REINFORCED CONCRETE FRAME BUILDINGS

L. Lin[1], N. Naumoski[2], M. Saatcioglu[3] and S. Foo[4]

ABSTRACT

There are a number of studies related to the effects of the strong-motion duration on the response of building structures. However, the findings regarding these effects are very contradictory. Some studies report significant effects, and other studies report minimal or no effects. The objective of this paper is to contribute to the understanding of this issue. It describes results from seismic analyses of reinforced concrete frame buildings subjected to earthquake motions with different strong-motion durations. Three buildings (4-, 10-, and 16-storey high) designed for Vancouver were used in the study. Forty-four ground motion records were used as excitation motions. The records were scaled to different intensity levels in order to produce responses ranging from elastic to large nonlinear responses of the buildings. The scaling was conducted with respect to the spectral acceleration at the fundamental periods of the buildings. The response parameter considered was the maximum interstorey drifts of the buildings. The results showed that the strong-motion duration does not have effects on the drifts when the seismic excitations are scaled to spectral acceleration at the fundamental building period.

Introduction

The strong portion of an earthquake record at a given site, which is normally referred to as the strong-motion duration, depends on a number of parameters including the type and the magnitude of the earthquake, the distance from the earthquake source to the site where the record is obtained, the soil condition at the site, etc. Normally, the strong-motion duration during small earthquakes, at short epicentral distances, and at stiff soil sites is expected to be smaller than that during large earthquakes, at long distances, and at soft soil sites.

A number of studies have been conducted in the past for the quantification of the strong-motion duration and its effects on the response of structures. Bommer and Martinez-Pereira

[1]Postdoctoral Fellow, Geological Survey of Canada, Ottawa, Ontario, Canada, K1A 0Y3
[2]Adjunct Professor, Dept. of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada, K1N 6N5
[3]Professor, Dept. of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada, K1N 6N5
[4]Risk Specialist, Public Works and Government Services Canada, Gatineau, Quebec, Canada, K1A 0S5

(1999) reviewed about 30 different definitions of strong-motion duration. Given the differences in the assumptions involved in these definitions, one can expect significant variations in the computed strong-motion duration using different definitions. Regarding the effects of the strong-motion duration on the response of structures, the conclusions from different studies are very contradictory. Some studies report significant effects, and other studies report minimal or no effects. Hancock and Bommer (2006) observed that the differences in the conclusions of different studies are primarily associated with the response (i.e., the damage) parameter used for the quantification of the effects of strong-motion duration, and the definitions used for determining the strong-motion duration.

The objective of this paper is to investigate the effects of the strong-motion duration on the drift of reinforced concrete frame buildings designed according to the National Building Code of Canada. This was done by seismic analyses of three reinforced concrete buildings designed for Vancouver.

Description of Buildings

The plan and the elevation of the buildings used in this study are shown in Fig. 1. The buildings are for office use and are located in Vancouver, which is in a high seismic hazard zone (National Research Council of Canada, 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 can be 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 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 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.

Design of Frames

In this study, only the interior transverse frames of the buildings were considered (i.e., frames Ti in Fig. 1). 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 according to the 2005 edition of the National Building Code of Canada (NBCC) (National Research Council of Canada, 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 force procedure. 'Reference' ground conditions, represented by site class C in NBCC, were assumed at the building location. The seismic base shear for each frame, V, was computed according to the code formula:

$$V = S(T_a)\ M_v\ I_E\ W/(R_d R_o) \tag{1}$$

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 (Fig. 2). The values of the other parameters used in Equation (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 acceleration, $S(T_a)$, and the base shear coefficients, V/W, are listed in Table 1.

The seismic forces along the height of the buildings were determined as specified in the code. The member forces for use in the design were determined by elastic static analysis of the frames for the combinations of gravity and seismic loads as required by NBCC. 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. 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).

Modeling of Frames for Dynamic Analysis

The computer program RUAUMOKO (Carr 2004) was used for the inelastic dynamic analyses of the frames. For each frame, a 2-D inelastic model was developed for use in RUAUMOKO. The beams and columns were modeled 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 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. 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 periods are significantly larger than those used in the design (Table 1). This is expected since it is known that the code formula provides relatively small period values that lead to conservative seismic design forces.

Seismic Excitations

For the purpose of the selection of seismic excitations, the definition for strong-motion duration proposed by Trifunac and Brady (1975) was used in this study. According to this definition, the strong-motion duration of an earthquake record represents the portion of the record over which 90% of the total energy (i.e., 90% of the total integral of the squares of the accelerations) is accumulated, as illustrated in Fig. 3. This definition was chosen since it is one of the most widely used definitions for strong-motion duration.

Based on the strong-motion duration, two sets of earthquake records were selected from the database of the Pacific Earthquake Engineering Research (PEER) Center. One of the sets is characterized by strong-motion duration (SMD) ranging from 4.4 s to 11.7 s and is referred to as the short SMD set. The other set is characterized by strong-motion duration of 19.0 s to 36.2 s and is referred to as the long SMD set. Each set consists of 22 records. All the records were obtained from California earthquakes and were recorded at sites class C (shear wave velocities between 360 m/s and 750 m/s). The records of the short SMD set were obtained from 8 earthquakes with magnitudes ranging from 5.7 to 7.2, and at distances ranging from 10 km to 63 km, while those of the long SMD set were obtained from 6 earthquakes with magnitudes between 6.7 and 7.7, and at distances between 47 km and 110 km. These two sets are considered appropriate for the investigation of the relationship between the structural response and the strong-motion duration since they provide information on the responses within the two limit ranges, i.e., within the short and within the long strong-motion duration ranges.

Analyses for Determining the Effects of Strong-Motion Duration

Scaling of Ground Motion Records

For the purpose of the response analysis of the frames, it was necessary to scale the selected ground motion records. For each frame, the records of both sets were scaled to a series of intensity levels. The scaling was conducted to the spectral accelerations at the fundamental periods of the frames, i.e., to $Sa(T_1)$. The fundamental periods used were those obtained by RUAUMOKO, i.e., 0.94 s for the 4S frame, 1.96 s for the 10S frame, and 2.75 s for the 16S frame. Scaling to $Sa(T_1)$ was employed since it has advantages relative to the scaling to peak ground motions (Shome et al. 1998) and is widely used in research. Reference $Sa(T_1)$ levels were considered those corresponding to the spectral accelerations of the design spectrum for Vancouver (Fig. 2) at the fundamental periods of the frames. The reference $Sa(T_1)$ levels for the 4S, the 10S, and the 16S frames are 0.37 g, 0.18 g, and 0.14 g respectively. For each frame, the selected sets of records were scaled to 10 intensity levels between $0.3Sa(T_1)$ and $5Sa(T_1)$ in order to obtain responses ranging from elastic to significant nonlinear responses.

For illustration, Figure 4 shows the 5% damped acceleration spectra of the selected sets of records scaled to the reference level for the 10S frame, i.e., to $Sa(T_1)=Sa(1.96s)=0.18$ g. It is seen from the figure that the spectral values of the short SMD set are much larger than those of the long SMD set for periods below about 1.5 s (i.e., the short SMD set is characterized by much higher frequency content than that of the long SMD set). This is expected since the records of the short SMD set are obtained at smaller distances than those of the records of the long SMD set. Namely, records obtained at short distances are normally characterized by high frequency content and short strong-motion duration, while records at large distances are characterized by low frequency content and long strong-motion duration.

Response Analyses and Results

In this study, the maximum interstorey drift over the height of the frames was used as a response parameter. It is a ‘global’ response parameter and has been used in a number of studies on the seismic performance of buildings (e.g., Ruiz-Garcia and Miranda 2005, Tothong and Luco 2007, Lin 2008). The maximum interstorey drift is also used in the NBCC seismic design provisions to limit the lateral deflections in the design for seismic loads (e.g., maximum drift of 2.5% of the storey height is allowed by NBCC for buildings of normal importance). Recognizing that the structural damage correlates with the interstorey drift, SEAOC (Vision 2000 Committee 1995) and ASCE (2000) have specified structural performance and damage levels in terms of maximum interstorey drift.

Each frame was subjected to the records of the short SMD and the long SMD sets scaled to each intensity level, i.e., between $0.3Sa(T_1)$ to $5Sa(T_1)$, as discussed above. Nonlinear time history analyses were conducted using the program RUAUMOKO (Carr 2004) and the maximum interstorey drifts of the frames were determined for each excitation motion. Since the program RUAUMOKO does not identify the collapse, large interstorey drifts (of the order of 10% and above) were obtained for the higher excitation levels (e.g., higher than $3Sa(T_1)$). Certainly, such large drifts cannot be resisted by the frames, i.e., the frames would collapse when their ultimate drift capacities (i.e., collapse drift limits) are exceeded. In this study, 5% drift was used as ultimate drift capacity, and drifts above 5% were not considered. The considerations for the selection of the value of 5% as ultimate drift for the frames analyzed in this study are discussed in detail in Lin (2008).

Figure 5 shows the computed drifts vs. strong-motion duration for excitations scaled to $3Sa(T_1)$. This scaling level was selected as a typical level for the illustration of the distribution of the computed drifts. The observations from the results for the other scaling levels are similar to those from Fig. 5. The two groups of results shown in the figure correspond to the two sets of records, i.e., the short SMD set (points in black), and the long SMD set (points in red).

It is seen from the figure that there is no correlation between the interstorey drift and the strong-motion duration. For the 4S frame (Fig. 5(a)), the ranges of the drifts for the short SMD and the long SMD sets are relatively comparable. The results for the 10S and the 16S frames (Figs. 5(b) and 5(c)) show that the short SMD set provides even larger drifts than those of the long SMD set. These observations can be explained by considering the higher mode effects on the structural response, and the spectral shapes of the scaled records (Fig. 4). Based on the modal participation factors, the response of the 4S building is dominated by the first mode vibrations, and therefore, the differences in the spectra of the sets for periods below the fundamental period have negligible effects on the structural response. On the other hand, the higher modes have significant contributions to the responses of the 10S and the 16S frames. Considering the shapes of the response spectra of the scaled records, one can see that the higher mode effects for the 10S and the 16S frames are much larger for the short SMD set than those for the long SMD set, which explains the distributions of the maximum drifts in Figs. 5(b) and 5(c).

Discussion and Conclusions

The objective of this study was to determine the correlation between maximum interstorey drift and strong-motion duration for reinforced concrete frame buildings. Three buildings (4-storey, 10-storey and 16-storey) were used in the study. The buildings were assumed to be located in Vancouver (which is in a high seismic zone) and were designed according to the seismic provisions of the 2005 edition of the National Building Code of Canada. Two sets of earthquake records were used as seismic excitations. One of the sets is characterized by short strong-motion duration, and the other set is characterized by long strong-motion duration. The records were scaled to the spectral acceleration at the fundamental building period. A series of scaling levels were used in order to obtain responses ranging from elastic to large inelastic responses.

The results from this study showed that the strong-motion duration does not have effects on the maximum interstorey drifts when the seismic excitations are scaled to the spectral acceleration at the fundamental building period (i.e., no correlation between maximum interstorey drift and strong-motion duration could be seen from the results). This is an important finding since the scaling of earthquake records to spectral acceleration is currently the most widely used type of scaling in dynamic analyses of structures. However, additional research is needed by using more reinforced concrete frame buildings with different configurations, buildings with other structural systems (e.g., shear wall systems), and steel building structures in order to verify this conclusion for building structures in general.

References

American Society of Civil Engineers, 2000. *Prestandard and commentary for the seismic rehabilitation of buildings*, Report FEMA 356, Reston, Virginia.

Bommer, J.J., and A. Martinez-Pereira, 1999. The effective duration of earthquake strong motion, *Journal of Earthquake Engineering* 3 (2), 127-172.

Carr, A.J. 2004. RUAUMOKO – Inelastic dynamic analysis program. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Hancock, J., and J.J. Bommer, 2006. A state-of-knowledge review of the influence of strong-motion duration on structural damage, *Earthquake Spectra* 22 (3), 827-845.

Lin, L. 2008. Development of improved intensity measures for probabilistic seismic demand analysis, *Ph. D. Thesis*, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada.

National Research Council of Canada, 2005. *National Building Code of Canada*, Ottawa, Ontario, Canada.

Ruiz-Garcia, J., and E. Miranda, 2005. Performance-based assessment of existing structures accounting for residual displacements, *Report No. 153*, The John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanford, CA, 407 p.

Shome, N., C.A. Cornell, P. Bazzuro, and J.E. Carballo, 1998. Earthquakes, records, and nonlinear responses, *Earthquake Spectra* 14 (3), 469-500.

Structural Engineers Association of California (SEAOC), Vision 2000 Committee, 1995. *Performance based seismic engineering of buildings*, San Francisco, CA.

Tothong, P., and N. Luco, 2007. Probabilistic seismic demand analysis using advanced intensity measures, *Earthquake Engineering and Structural Dynamics* 36, 1837-1860.

Trifunac, M.D., and A.G. Brady, 1975. A study on the duration of strong earthquake ground motion. *Bulletin of the Seismological Society of America* 65, 581-626.

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)	0.613	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.

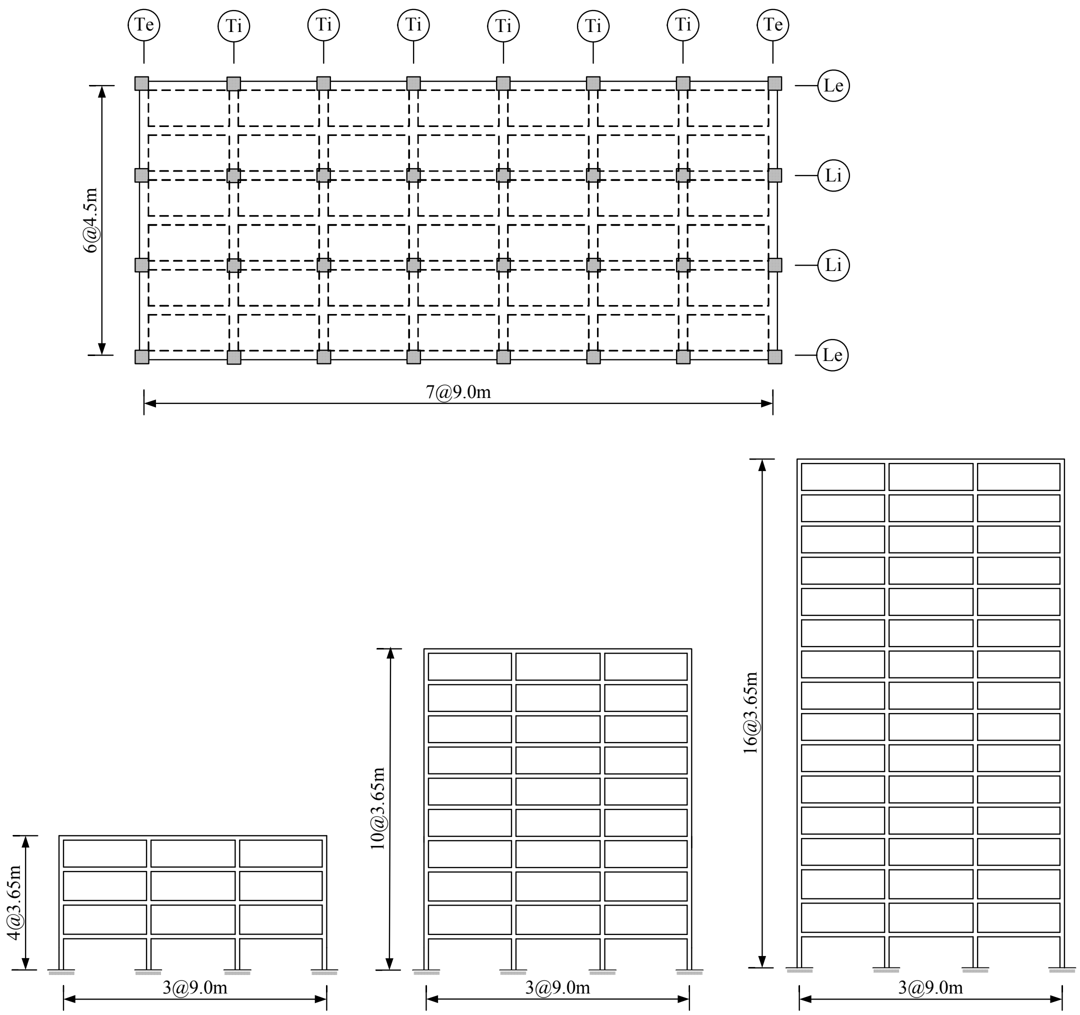

Figure 1. Plan of floors and elevations of transverse frames of the buildings.

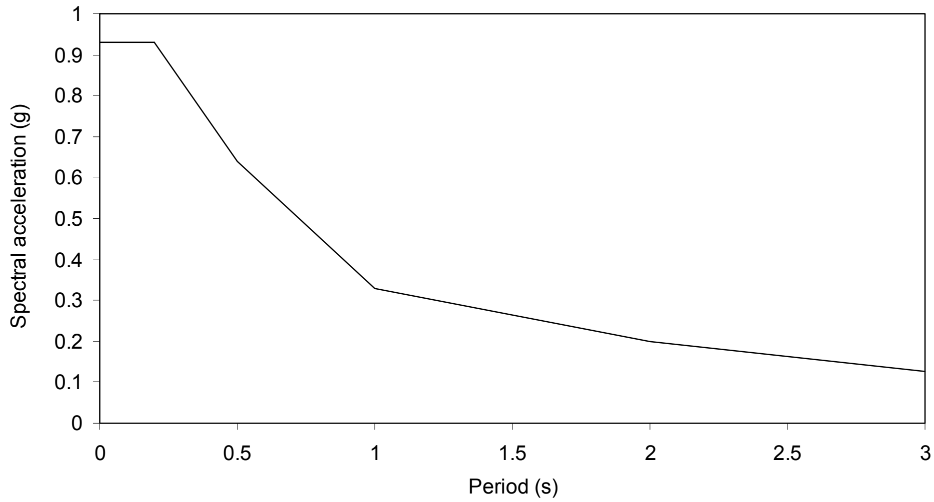

Figure 2. Seismic design spectrum for Vancouver for soil class C.

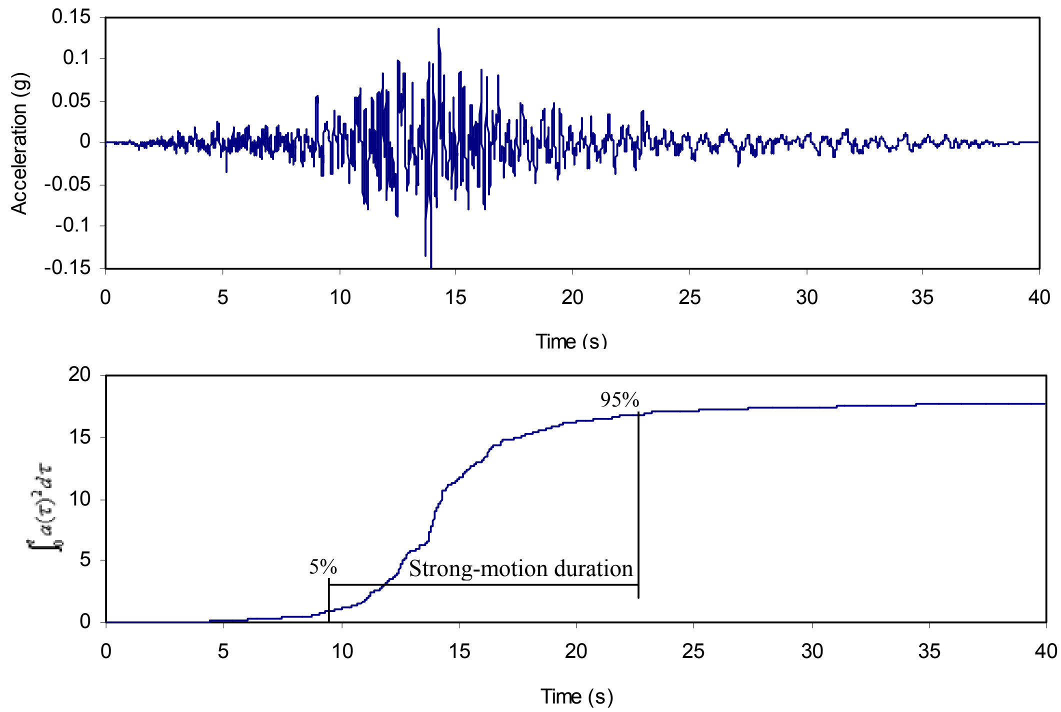

Figure 3. Strong-motion duration according to Trifunac and Brady (1975).

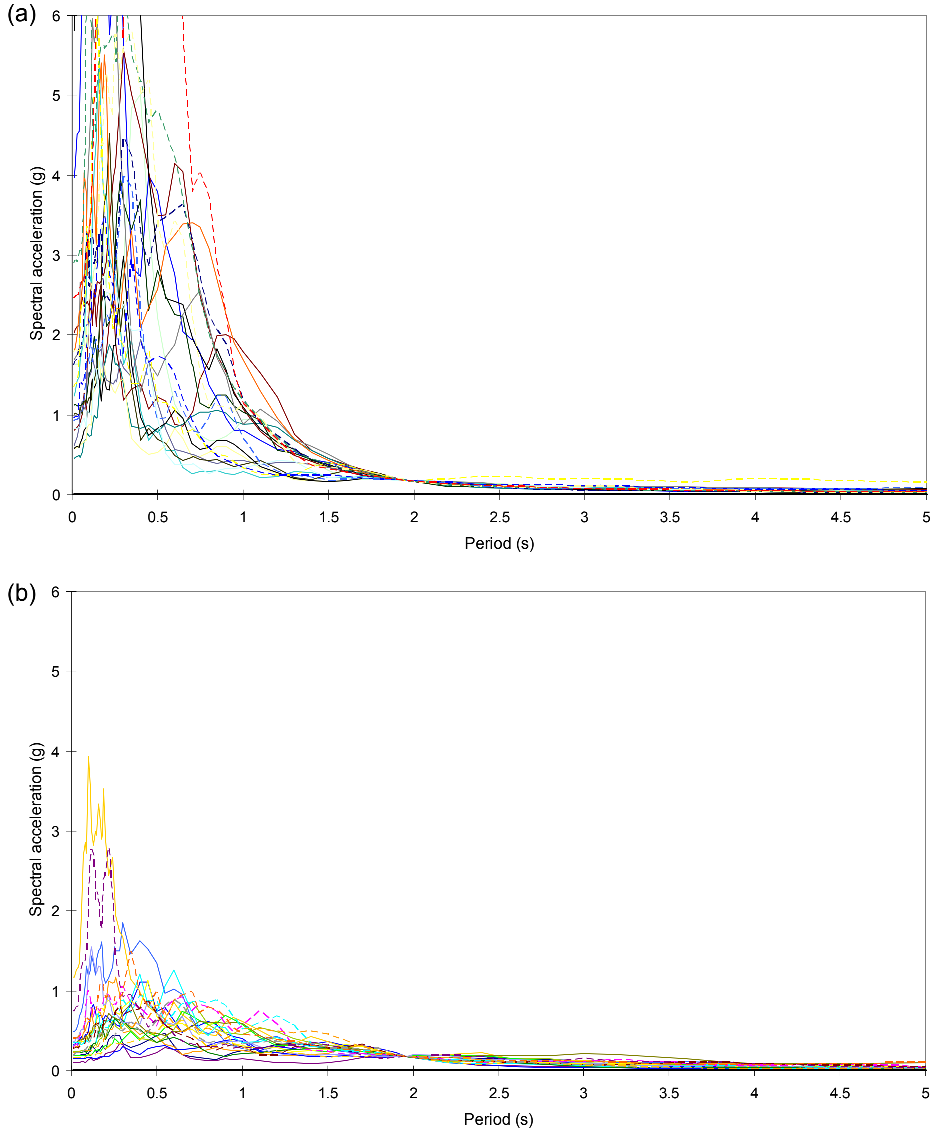

Figure 4. Scaled records to spectral acceleration of 0.18 g at the fundamental period of the 10S frame: (a) Short SMD set, and (b) Long SMD set.

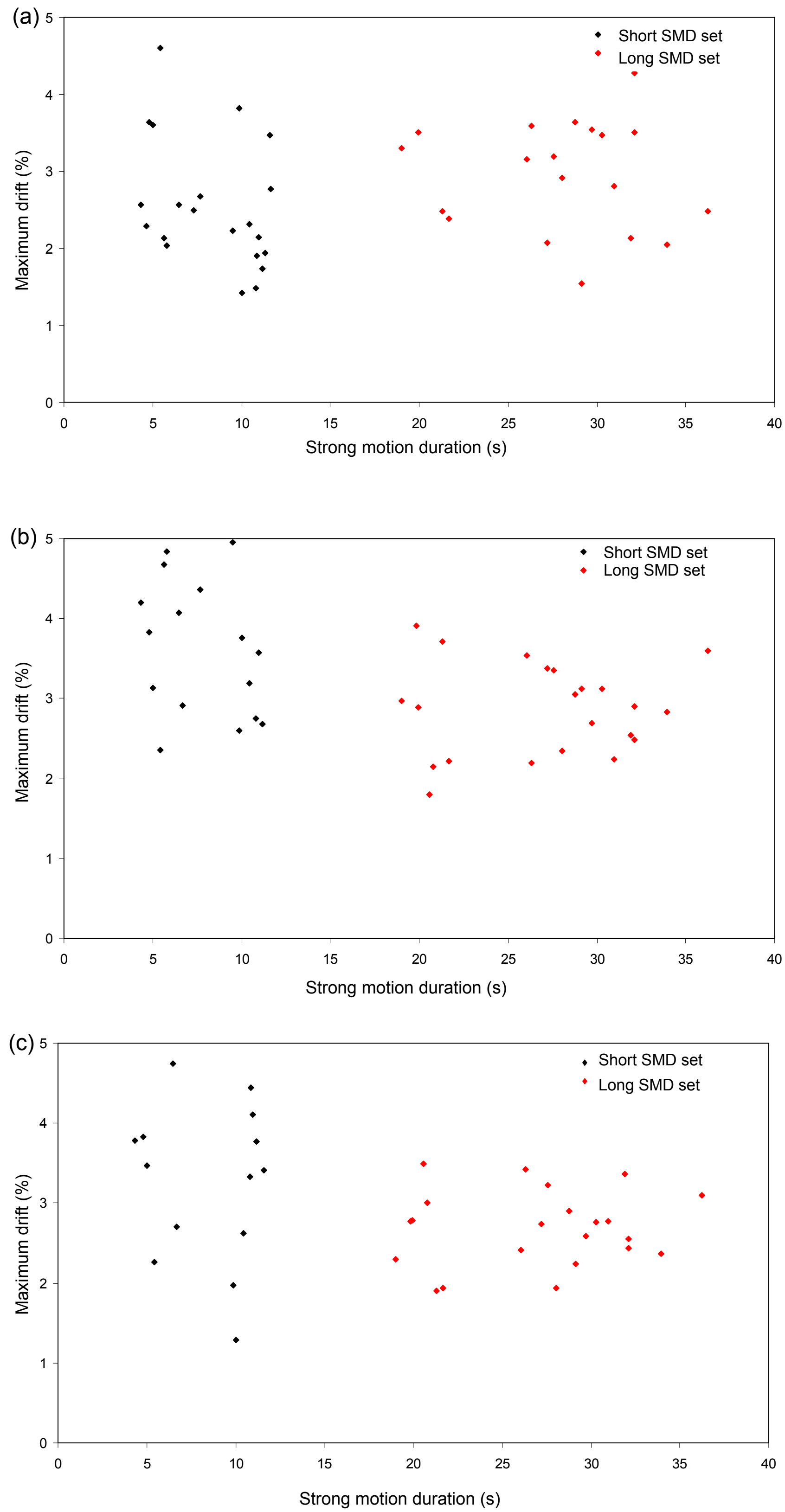

Figure 5. Maximum interstorey drift vs. strong-motion duration for: (a) the 4S frame, (b) the 10S frame, and (c) the 16S frame.