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Seismic performance of R.C. structures in the Eastern U.S.A.

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ABSTRACT

Seismic performance of a R.C. frame designed according to current design provisions for structures located in moderate seismic risk was investigated analytically. A number of actual ground motions recorded in the northeast U.S. and southeast Canada, and artificially-generated earthquakes were utilized for this purpose. The dominant frequencies of the actual earthquakes were found to be several times larger than lower-mode frequencies of the structure. The level of excitation was not significant to cause structural or nonstructural damage. When the structure was subjected to a "rare" earthquake, which was obtained by amplifying the time scale of one of the recorded motions, the structure sustained inelastic action in the beams while the columns remained mostly elastic. The computed inter-story values indicate damage to nonstructural elements. In general, the structure appears to have adequate strength, but the overall stiffness is somewhat low.

INTRODUCTION

The state of art and practice in earthquake resistant design of structures has seen significant developments in the past two decades. Most of the changes can be traced to experimental and analytical studies conducted for better understanding of the behavior of the components that provide significant lateral resistance. The emphasis of previous investigations has been mostly on mitigating seismic hazard in regions with high seismicity, with little attention to other regions. Nevertheless, historical records and recent seismic events (e.g., the earthquake of 25 November 1988 in the Saguenay, Quebec with a felt area as far as Boston) indicate the possibilities of ground motions with appreciable magnitudes in regions currently designated having low to moderate seismic risk. The seismic performance of structures in the Eastern and Central U.S. has become an important issue. Considering that only a limited number of significant earthquakes have occurred since the installation of strong motion instrumentation in these regions, a comprehensive database is still not available.

As a result, most of the available studies have focused on the response of structures under artificially-generated ground motions which are anticipated in the Eastern U.S. (Sidel et al. 1989). Furthermore, the performance of beam-column joints in buildings designed primarily for gravity loads have been investigated experimentally (Pessiki et al. 1990). Dynamic response of a 1/6 model of a lightly

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reinforced concrete frame under Taft S69E 1952 record was investigated through shake table tests (El-Attar et al. 1990). Considering the paucity of sufficient data on the seismic response of reinforced concrete structures in zones with moderate seismic risk, the research reported herein was undertaken. Using actual and artificially-generated ground motions, the behavior of a six-story reinforced concrete frame with a setback was investigated analytically.

TEST STRUCTURE

The structure was a six-story moment resisting reinforced concrete frame with a fifty percent setback at the midheight, as shown in Fig. 1. The structure was assumed to be an imaginary office building located in UBC Zone 2, and it was designed following the ACI 318-89 (ACI 1989) and UBC-88 (UBC 1988) provisions for frames located in moderate seismic risk regions. The design forces due to wind loading were found to be larger than those computed under earthquake loads specified by building codes. The members were detailed according to common practice for such structures. Inter-story drifts, estimated crack widths, and member deflections were within limits required by building codes (ACI 318-89 and UBC-88). The structure appears to be somewhat flexible; the computed first-mode vibration period was found to range between 0.7 to 1.0 sec. (obtained by using uncracked or cracked gross section properties, respectively) which is larger than the values expected for reinforced concrete frames similar to the test structure.

GROUND MOTIONS

The geological conditions of the northeast U.S. are similar to those in the adjoining regions in Canada, and hence seismic motions in these two areas are expected to have close attributes. In this study, records from Canada and the northeast U.S. with magnitudes of 5 or larger were used. These ground motions include 25 November 1988 Saguenay, Quebec; 31 March 1982 Miramichi, New Brunswick; 6 May 1982 Miramichi, New Brunswick; and 19 January 1982 Franklin Falls, New Hampshire with peak accelerations of 0.125g, 0.34g, 0.11g, and 0.031g, respectively. All of these records were measured within 100 km from the epicenter. As seen from the pseudo acceleration response spectra and Fourier amplitude spectra (Fig. 2), the earthquakes have more energy at higher frequencies, or for a narrow range of frequencies. A similar observation has been made for other Eastern North America earthquakes (Atkinson 1989), and has been attributed to site effects, crustal conditions, or source effects. For example, although the 31 March 1982 Miramichi record has a sizable peak acceleration (0.31g), its dominating frequency is approximately 25 Hz. At such high frequency, only very stiff building structures or bridges seem to be prone to damage. By comparing the first-mode frequency of the test structure (ranging between 1.0 to 1.4 Hz) and the dominant frequency of the ground motions, the level of excitation for the structure would be limited. As a result, artificially-generated ground motions with more energy closer to the frequencies of the test structure were also considered.

Based on the earthquake source model proposed by Boore and Atkinson (Boore and Atkinson 1987), three records with epicentral distances of 80 km, 160 km, and 640 km were derived using data from the East Coast. The records were generated for hard rock conditions, i.e., shear wave velocity was taken as 3.5 km/sec., and the records were assumed to have a moment magnitude of 7. The peak acceleration drops as the distance from the epicenter is increased, but the records farther from the epicenter indicate more energy at smaller frequencies (Fig. 3). Considering the peak accelerations and frequency contents, the record with epicentral distance of 80 km was selected for the analytical studies.

ANALYTICAL MODEL

Using the actual and artificially-derived ground motions, the response of the test structure parallel to the setback was investigated. The structure was modeled as two plane frames representing the interior frame and the two exterior frames. The floor slabs were assumed to act as rigid floor diaphragms. The beam-column joints were assumed to be rigid, i.e., rigid end zones equal to the column width and beam depth were used for the beams and columns, respectively. The additional rotational flexibilities due to possible reinforcement slip were not considered.

The analyses were conducted by using the computer program DRAIN-2D (Kannan and Powell 1973) which was modified to incorporate a trilinear, unsymmetrical version of Takeda's model commonly used to simulate stiffness degradation of reinforced concrete members. Hence, it was possible to define different moment strengths and stiffnesses under negative and positive bending. The contribution of the floor slab to the strength and stiffness of the supporting beams (particularly under negative bending moment) was modeled using techniques described elsewhere (Chern 1990). Moment-axial load interaction was ignored for the columns, and both the columns and beams were modeled by a yield interaction surface which neglects axial load. Member stiffnesses were computed based on flexural deformation only. Effects of gravity loads on element strength were considered by initializing the member end forces equal to those under gravity loads. Viscous damping was assumed to be proportional to the mass and original stiffness, and a damping ratio equal to 0.05 was used for the first two modes.

RESPONSE OF THE TEST STRUCTURE

The response of the test structure was gauged in reference to the value of inter-story drift, magnitude of roof lateral displacement, peak base shear, and damage pattern which may be inferred by formation of plastic hinges. The inter-story drift profiles over the height of the structure are plotted in Fig. 4. The largest value of drift occurred in the fifth floor when the structure was subjected to the 25 November 1988 Saguenay record. The maximum computed drift (0.14% of floor height) is approximately ten times smaller than the limit of 1.5 percent of inter-story height which is normally considered acceptable (Algan 1983). For the maximum inter-story drift sustained by the test structure, no damage to the structural or nonstructural elements is expected. The observed good performance is despite small stiffness of the structure (first-mode period ranging between 0.7 to 1.0 sec). The ground motions could not apparently excite the structure significantly. The peak values of base shear and roof displacement also indicate a similar observation (Table 1). For example, the largest base shear sustained by the structure was 0.027W (W=total weight) comparing to 0.04W under design wind loads. The analyses indicate that none of the elements experienced inelastic action.

It should be noted that the analyses did not account for brittle failure modes such as pullout of discontinuous bottom beam bars in the beam-column joints or column splice failure. Experimental tests on beam-column connections found in reinforced concrete structures similar to the one studied herein indicate that although failure would be eventually initiated by pullout of discontinuous bars, such connections have rather stable hysteresis loops for drifts up to 1.5 to 2 percent story height (Pessiki et al. 1990). For this range of drifts, the response of connections with continuous and discontinuous bottom beam bars were observed to be similar. Furthermore, connections with discontinuous bottom bars could sustain joint shear stresses as large as 80 percent of the value resisted by those with continuous bars through the connection. Lightly-confined column splices were also found to perform adequately. Such local failure modes would not apparently occur for the range of drifts considered in the analyses. Hence, even though the effects of poor detailing could not be simulated analytically, the good performance of the test structure concluded based on the dynamic analyses would likely remain valid.

To examine the behavior of the structure under "rare" ground motions, the 25 November 1988 Saguenay record was changed such that the frequency of the structure would be closer to the dominant frequencies of this record. For this purpose, the time scale of the actual acceleration history was arbitrarily amplified by a factor of 5. Under the altered Saguenay record, the structure experienced a maximum base shear equal to 0.16W, and the largest inter-story drift occurring at the fifth floor indicates damage to nonstructural elements. Some plastic hinges (Fig. 5) formed in the structure, but the level of inelastic action (approximately quantified by rotational ductility demands) was not significant. Despite the level of excitation (Table 1), the structure exhibited adequate strength. Nevertheless, the structure appears to be somewhat flexible with potential damage to partitions and other nonstructural elements over a number of floors.

SUMMARY AND CONCLUSIONS

The behavior of an imaginary reinforced concrete frame located in regions with moderate seismicity was investigated analytically. Actual recorded motions from earthquakes in the northeast U.S. and southeast Canada, and artificially-generated earthquakes were used for this purpose. The actual records exhibit more energy at high frequencies, considerably larger than the natural frequencies of the structure. Within the limitations of the modeling techniques, the following conclusions may be drawn.

(1) When subjected to the actual recorded motions, the structure was not excited significantly. The peak base shear was about half of the base shear under the design wind loads, and the inter-story drifts were much smaller than values considered to cause damage. Damage to structural and nonstructural elements would be unlikely. The structure exhibited adequate strength and stiffness.

(2) For the level of drifts experienced by the structure, the effects of poor details such as discontinuous beam bottom bars through beam-column connections or lightly-confined column splices appear to be minimal.

(3) Under a "rare ground motion", plastic hinges were formed mainly in the beams. The level of inelastic action was rather limited indicating that the structure has sufficient strength. Considering the likelihood of damage to nonstructural elements at several floors, the overall stiffness of the structure appears to be somewhat low.

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Table 1. Extreme values

Ground motion	Roof displacement (in.)	Base shear (kips)
25 Nov. 88 Saguenay	0.5 (0.058)*	19 (2.4)+
31 March 82 Miramichi	0.3 (0.035)	6.9 (0.86)
6 May 82 Miramichi	0.1 (0.012)	6.2 (0.77)
19 Jan. 82 Franklin Falls	0.005 (0.0006)	2.5 (0.31)
Artificial record	0.58 (0.067)	21.6 (2.7)
Modified Saguenay	7.6 (0.88)	126 (16)

* Roof displacement as a percentage of total height.
+ Base shear as a percentage of total weight.

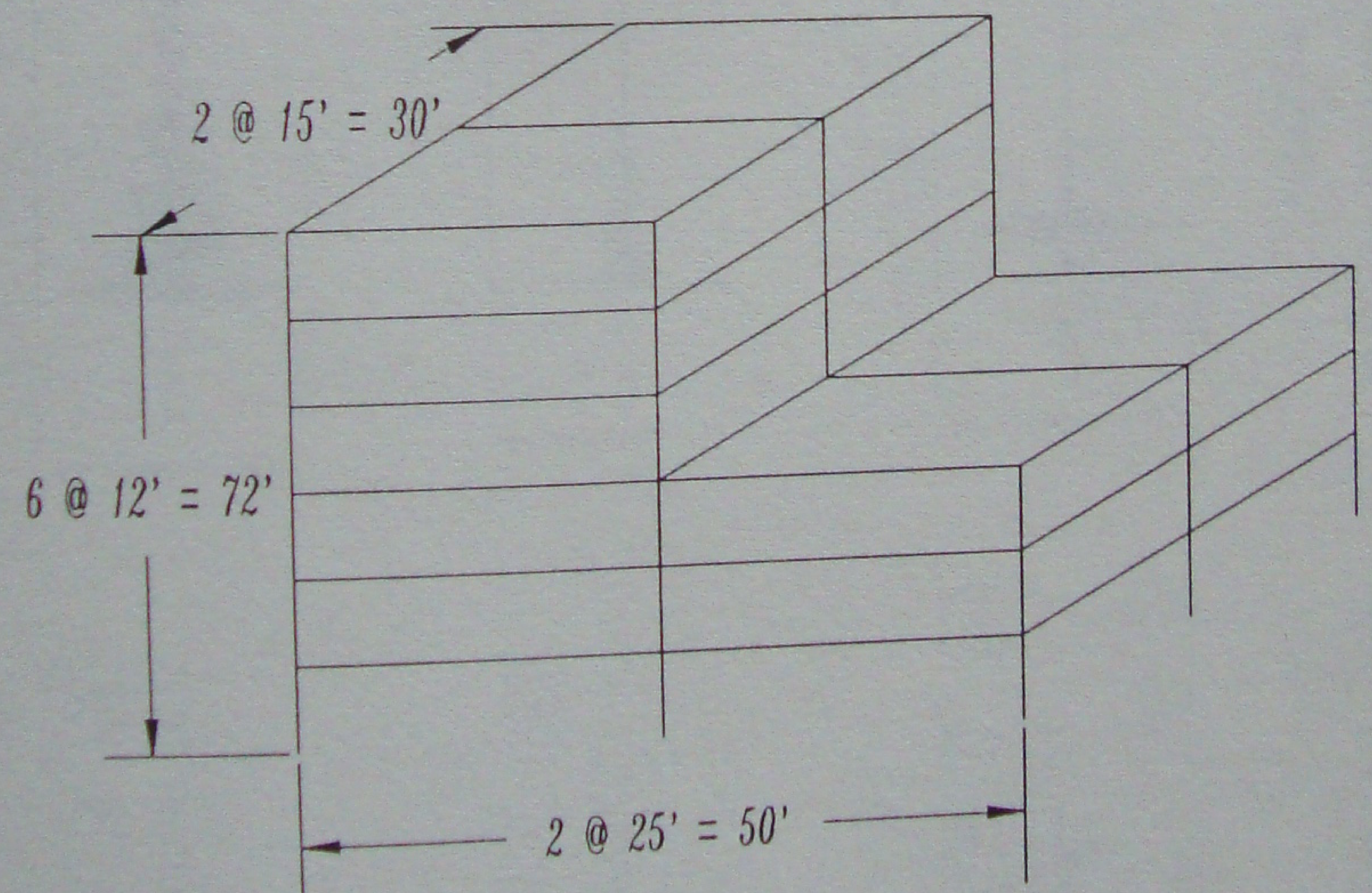

Fig. 1 Overall geometry of test structure.

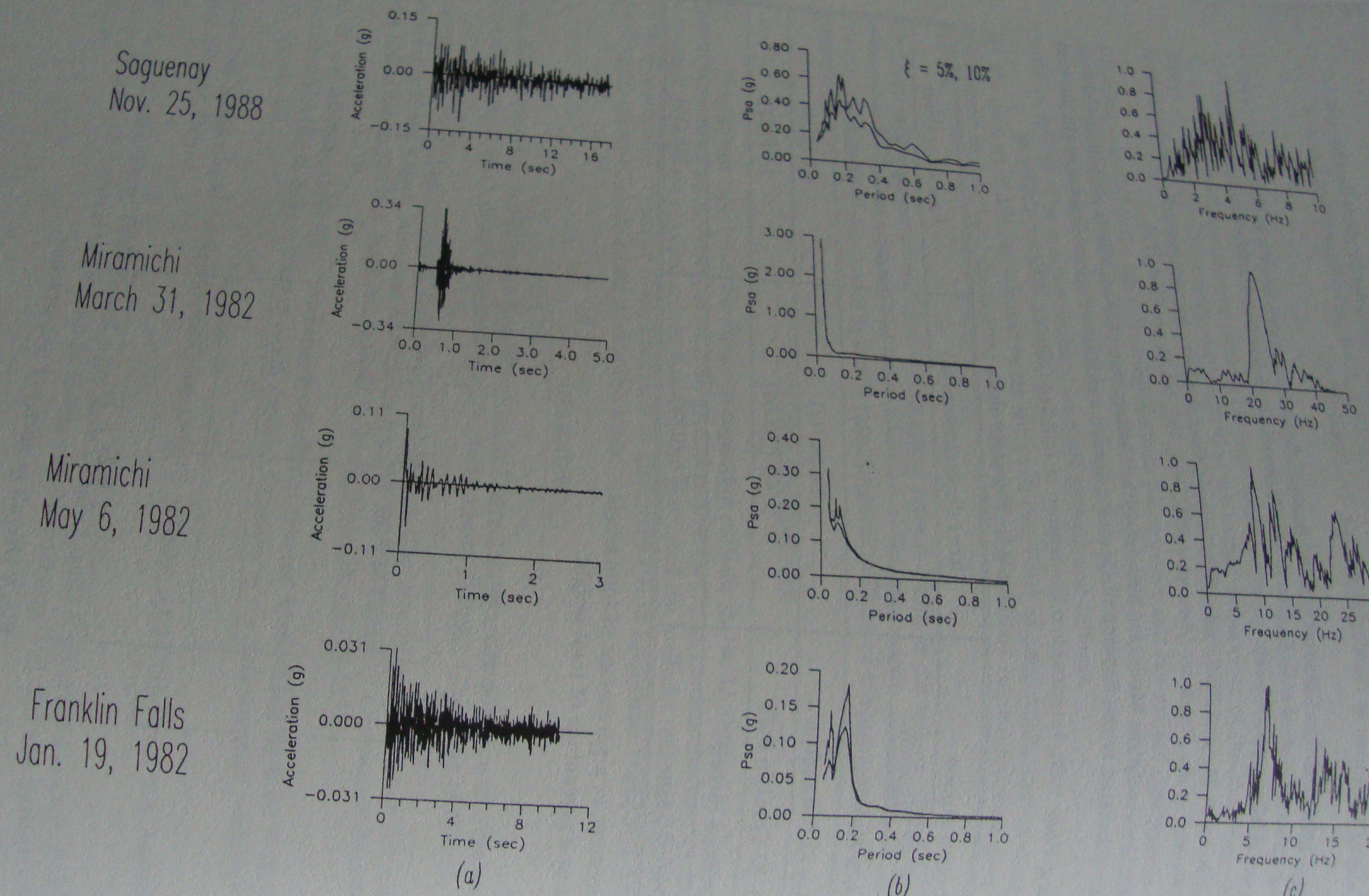

Fig. 2 Characteristics of recorded motions.

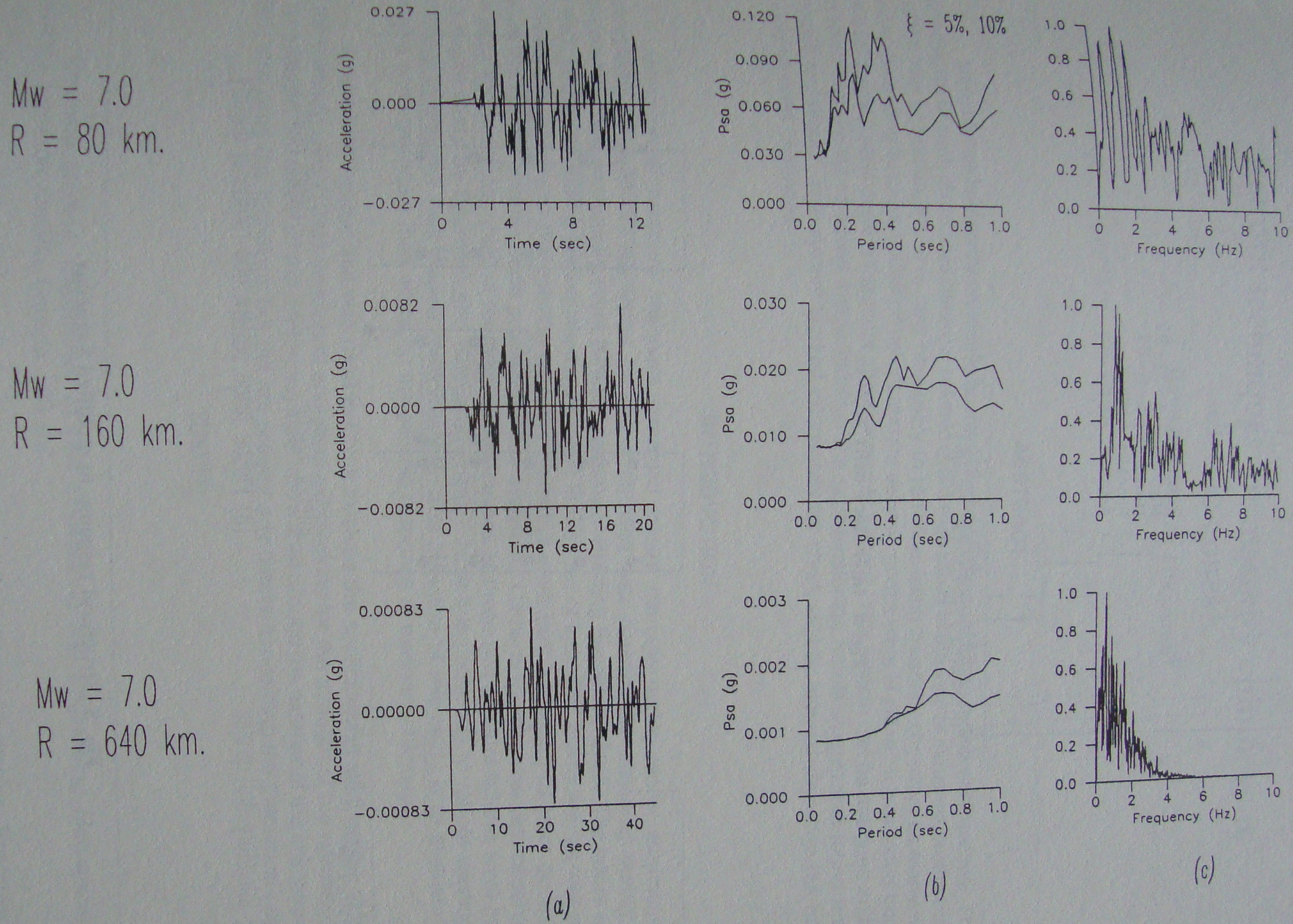

Fig. 3 Characteristics of the artificial records.

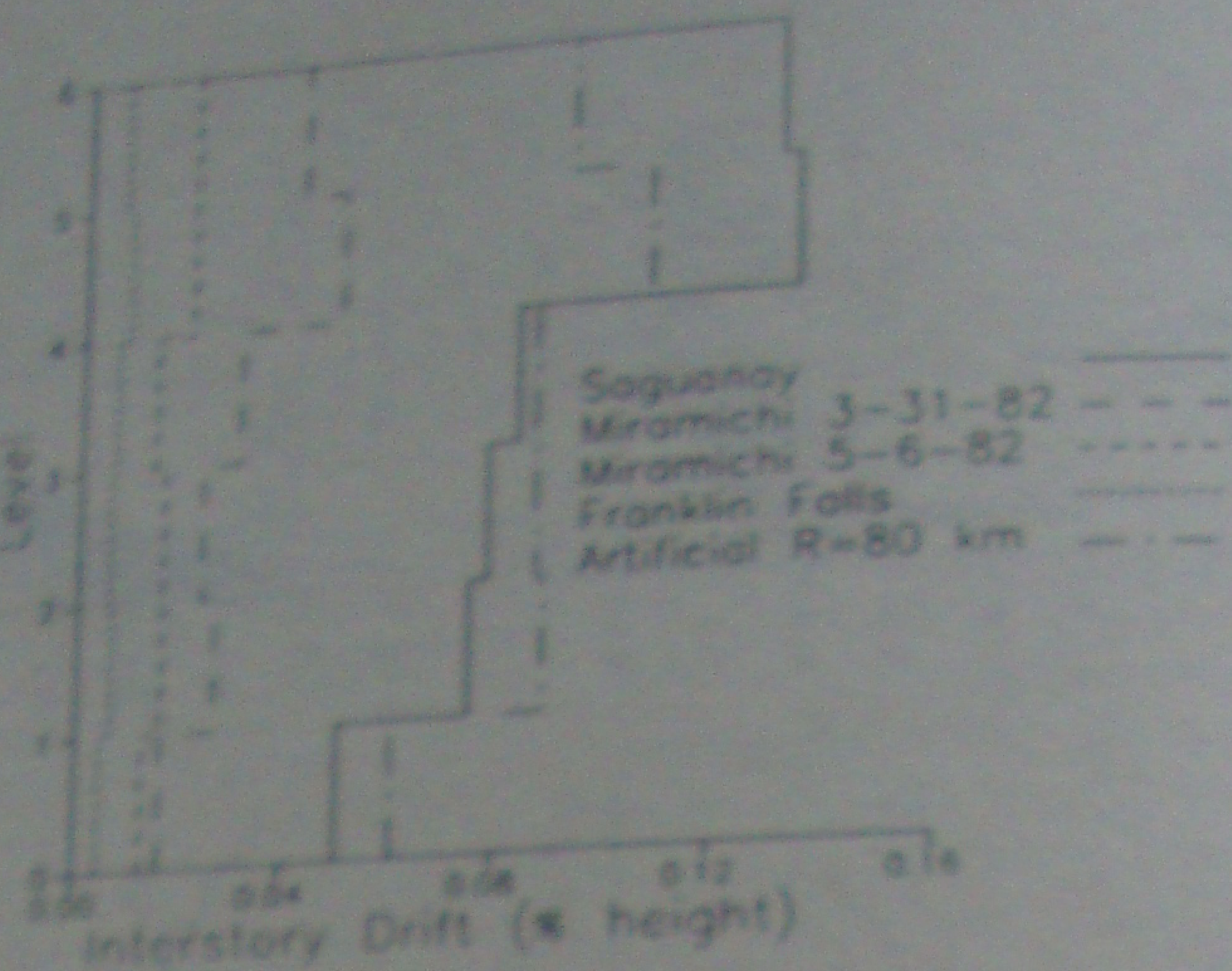

Fig. 4 Inter-story drift profiles.

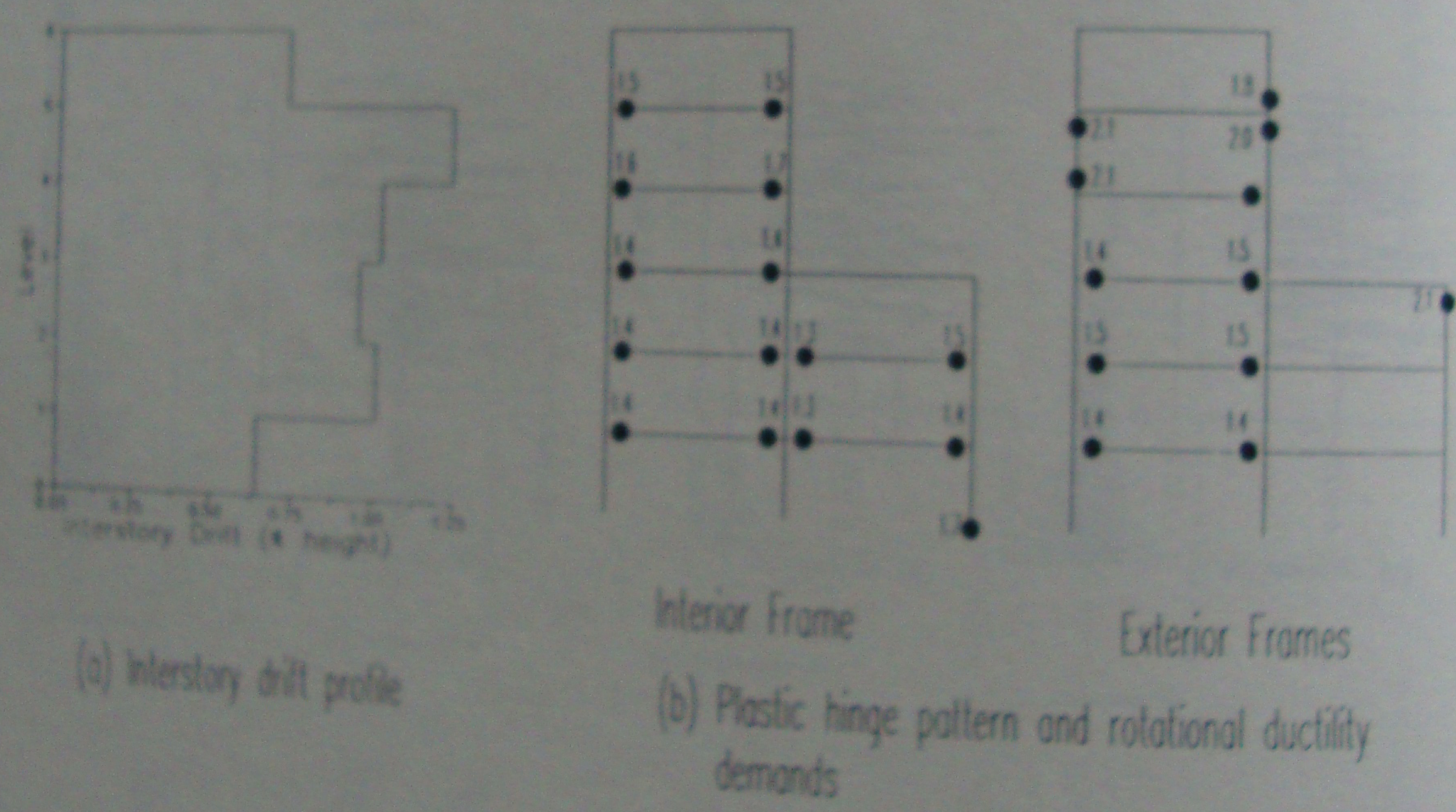

Fig. 5 Structural response to modified Saguenay.