Seismic performance of high-rise reinforced concrete frame buildings located in different seismic regions

T.J. Zhu[I], W.K. Tso[II], and A.C. Heidebrecht[III]

ABSTRACT

Seismic areas in Canada are classified into three categories for three combinations of acceleration and velocity seismic zones ($Z_a<Z_v$, $Z_a=Z_v$, and $Z_a>Z_v$). This paper evaluates the seismic performance of high-rise reinforced concrete frame buildings located in these three categories of seismic areas. Two frame buildings having 10 and 18 storeys are designed to the current Canadian seismic provisions, and their inelastic responses to three groups of ground motions are examined. The results indicate that the distribution of inelastic deformations is significantly different for high-rise frame buildings located in seismic regions with $Z_a<Z_v$, $Z_a=Z_v$, and $Z_a>Z_v$.

INTRODUCTION

Seismic regions in Canada are classified into three categories for three combinations of acceleration and velocity seismic zones ($Z_a<Z_v$, $Z_a=Z_v$, and $Z_a>Z_v$) (Heidebrecht et al. 1983). Seismic areas having $Z_a>Z_v$ are influenced mainly by small or moderate nearby earthquakes, and ground motions in these areas are expected to exhibit high frequency content and have high peak acceleration-to-velocity (A/V) ratios. Seismic regions with $Z_a<Z_v$ are affected mainly by large distant earthquakes, and ground motions in these regions are expected to have low frequency content and low A/V ratios. The specification of seismic design forces for buildings having fundamental periods longer than 0.5 sec is directly tied to zonal velocity, irrespective of seismic region category. For short-period buildings, three different levels of seismic design force are used for the three different categories of seismic areas having $Z_a<Z_v$, $Z_a=Z_v$, and $Z_a>Z_v$. The objective of this paper is to evaluate the seismic performance of high-rise reinforced concrete frame buildings located in the three different categories of seismic regions.

STRUCTURAL MODELS

Two reinforced concrete ductile moment-resisting frame (MRF) buildings having 10 and 18 storeys were considered. These two buildings are designated as 10S and 18S and have the same floor plan as shown in Fig. 1. The effect of seismic action was considered in the E-W direction. The elevations of the interior

---

[I] Post-Doctoral Fellow, Dept. of Civil Engineering, McMaster University, Hamilton, Ontario, Canada L8S 4L7

[II] Professor, Dept. of Civil Engineering, McMaster University, Hamilton, Ontario, Canada L8S 4L7

frames in this direction are shown in Fig. 1. The beam and column sizes are also depicted. The two buildings were designed for combined gravity and seismic effects in accord with the 1990 edition of the National Building Code of Canada (NBCC 1990) (Associate Committee on National Building Code 1990), and their structural members were proportioned and detailed according to the 1984 edition of the Canadian Reinforced Concrete Design Code (CAN3-A23.3-M84) (Canadian Standards Association 1984).

The design gravity loads are shown in Table 1. The seismic design base shear, V, for each frame was specified from the formula:

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

in which V_e = the seismic design force representing elastic response, R = the force modification factor, and U = 0.6. For the ductile MRF buildings considered, R was taken as 4. V_e is given by

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

in which v = zonal velocity ratio, S = seismic response factor, I = importance factor, F = foundation factor, and W = dead weight. The two frames were assumed to be located in regions with high seismicity (Z_v=6). Accordingly, v was taken as 0.4. F and I were set to 1.0. To determine S, the fundamental periods of the two frames were estimated from the formula, T = 0.1 N, where N is the number of storeys.

The design base shear for each frame was distributed over its height based on the NBCC 1990 distribution formula. The frames were designed based on the following three load combinations:

$$\begin{aligned} &1.25\,D + 1.5\,L \\ &1.25\,D + 1.0\,Q \\ &1.25\,D + 0.7\,(1.5\,L + 1.0\,Q) \end{aligned} \tag{3}$$

in which D = dead load, L = live load due to use and occupancy, and Q = seismic load.

The effects of geometric nonlinearity (P-delta and slenderness effects) were considered according to CAN3-A23.3-M84. The factored beam design moments and column design axial forces were obtained from the elastic analyses of the frames under the three load combinations. The factored column design moments were related to the beam moment capacities by considering the equilibrium of each joint, coupled with a column overstrength factor. The column overstrength factor was set to 1.2 which is slightly higher than the minimum value of 1.1 required by CAN3-A23.3-M84. The final design results are given by Zhu (1989).

GROUND MOTION DATA

A total of 45 strong motion records recorded on rock or stiff soil sites were selected from the McMaster University Seismological Executive (MUSE) Database. The 45 records were obtained from 23 different events with magnitude ranging from 5.25 to 8.1, and they were recorded at epicentral distances ranging from 4 to 379 km. The 45 records were subdivided into three groups according to their A/V ratios, with 15 records in each group. The records having A/V < 0.8 g/m/s were classified into the low A/V group whereas those having A/V > 1.2 g/m/s were categorized into the high A/V group. The records with 0.8 g/m/s ≤ A/V ≤ 1.2 g/m/s were classified into the intermediate A/V group. These three groups of records were taken as representative ground motions in the three categories of seismic regions having $Z_a<Z_v$, $Z_a=Z_v$, and $Z_a>Z_v$. Fig. 2 shows the distribution of magnitude and epicentral distance for the three groups of records. It can be seen that the ground motions with high A/V ratios were obtained in the vicinity of small or moderate earthquakes whereas those having low or intermediate A/V ratios were recorded at large distances from large or moderate earthquakes.

To indicate their frequency content, 5% damped elastic response spectra were computed for the three groups of records scaled to a peak velocity of 0.4 m/s which is the design zonal velocity for the frames. Fig. 3 shows the mean response spectra for the three groups of records. The high A/V group of records has higher frequency content than the low A/V group. The elastic design spectrum (V_e/W) is superimposed in Fig. 3. Also shown in Fig. 3 is the inelastic design spectrum (V/W). The estimated and actual fundamental periods of the 10S and 18S frames are also shown in the figure. The duration of strong shaking was estimated for each of the 45 records based on the definition by McCann and Shah (1979). A statistical summary of the computed strong-motion durations for the three groups of records is presented in Table 2. The low A/V group of records has longer duration of strong shaking than the high A/V group.

ANALYSIS PROCEDURE AND RESPONSE PARAMETERS

Many hysteretic models have been proposed for reinforced concrete structural members (Otani 1980). In this study, it is assumed that sufficient transverse reinforcement has been provided for the structural members, and stiffness and/or strength deterioration due to shear or bond loss is not significant. Accordingly, the relatively simple dual-component element (Clough et al. 1967) was used to model the beams and columns. The effect of axial force on yield moment was considered for each column by a yield moment-axial force interaction curve. A simplified solution (Wilson & Habibullah 1987) was used to account for the second-order P-delta effect for each column. The general-purpose computer program DRAIN-2D (Kanaan & Powell 1973) was used to perform dynamic analysis for the frames. The DRAIN-2D was also modified to perform inelastic static analysis for the frames subjected to monotonically increased lateral loading (Zhu 1989).

The response parameters considered are interstorey drift for overall response and curvature ductility and cumulative plastic rotation for member response. The curvature ductility is representative of the response parameters for damage caused by large inelastic deformation excursions, and the cumulative plastic rotation represents the response parameters for damage due to sustained reversals of inelastic deformations.

STATIC ANALYSIS RESULTS

The inelastic behaviour of the frames subjected to monotonically increased lateral loading was studied. The lateral loading was distributed over frame height according to the NBCC 1990 distribution formula. The base shear versus roof displacement curves for the 10S and 18S frames are depicted in Fig. 4. The points for the first beam and column hinges are shown in the same figure. To indicate the overall lateral strengths of the frames relative to their seismic design forces, the design base shear levels are also shown in the figure.

The base shear-roof displacement response is nearly linear up to the formation of the first column hinge. Thereafter, the overall stiffness of the frames decreases drastically. The actual lateral strengths of the frames are higher than their design base shears. This is particularly true for the 10S frame. The strength for an overall drift of 0.5% is about 29% and 13% higher than the design base shear for the 10S and 18S frame, respectively. The lower overstrength for the 18S frame is mainly because the P-Delta effect for this frame is very significant. The seismic overstrength of the frames can be attributed to the following factors. First, the column strengths were keyed to the beam strengths based on the weak beam-strong column criterion. Second, strain-hardening effect was considered for the beams and columns in the analysis.

The inelastic responses of the frames to the three groups of records were analyzed statistically. All the records were scaled to a peak velocity of 0.4 m/s which is the design zonal velocity for the frames. The mean and mean plus one standard deviation (mean + σ) values of the response parameters were obtained for each group of records. The mean + σ level is appropriate for design purposes. A comparison between the mean and mean + σ values indicates the dispersion characteristics of the response parameters within each group of records.

Figs. 5 to 6 show the statistical results of the response parameters for the 10S and 18S frames. It can be seen that the distributions of inelastic deformations over frame height are significantly different for the three groups of ground motions. For the high A/V group of ground motions, the interstorey drifts in the upper storeys are higher than those in the lower storeys. The high A/V group of records also produces more inelastic deformations in the upper storey beams and columns. This "whiplash" effect can be ascribed to significant effect of higher mode participation. Since the high A/V ground motions have higher frequency content, they prompt the higher modal responses of the 10S and 18S frames.

For the low A/V group of ground motions, the interstorey drifts in the lower storeys are higher than those in the upper storeys. Beam inelastic deformations are concentrated in the lower storeys. This is particularly true for the cumulative plastic rotation. In the lower storeys of the 18S frame, the curvature ductility for the low A/V group of records is about twice that for the high A/V group whereas the difference increases up to about four times for the cumulative plastic rotation. The cumulative plastic rotation depends on both the peak inelastic response and the duration of strong shaking. The analysis of the ground motion data has indicated that the low A/V ground motions have longer duration of strong shaking than the high A/V ground motions. Since the beam peak inelastic response (indicated by the curvature ductility) for the low A/V group of records is already higher than that for the high A/V group in the lower storeys, the combined effect of strong-motion duration results in much higher cumulative plastic rotation for the low A/V group of records. The low A/V ground motions also produce higher inelastic deformations at the base of the first storey columns than the high A/V ground motions. The lower storeys of a high-rise frame are more vulnerable than the upper storeys due to the high axial forces carried by the lower storey columns. The concentration of inelastic deformation in the lower storeys would increase second-order P-delta effect. Therefore, low A/V ground motions are more damaging to high-rise frames than high A/V ground motions even when the frames are designed based on peak ground velocity.

To gain insight into the effect of ground motion frequency content on the inelastic response, the sequences of plastic hinge formation are shown in Fig. 7 for the 10S frame subjected to three example earthquake records, one from each of the three A/V groups. These three records are (a) the 1985 Mexico, Mesa Vibradora, N90W component; (b) the 1952 Kern County, Taft, S69E component; and (c) the 1935 Helena, Carroll College, N00E component, and they have an A/V ratio of 0.36, 1.01, and 2.03 g/m/s, respectively. It can be seen in Fig. 7 that for the Mesa Vibradora record which has a low frequency content, plastic hinges develop in the lower storey beams first and then migrate from the bottom to the top of the frame. Column plastic hinges are located only at the base of the first storey columns. For the Carroll College record which has a high frequency content, beam plastic hinges develop in the upper storey first, and column hinges are developed only in the upper storeys. The Taft record has a broad range of significant frequency content, and consequently, it produces column plastic hinges both at the base of the first storey columns and in the upper storeys.

An examination of the curvature ductility demands for the columns reveals that the use of a column overstrength factor of 1.2 does not prevent column yielding in the upper or middle storeys. This is particularly true for the high and intermediate A/V groups of records which tend to excite the higher modes of vibration. At a joint, the distribution of the beam moments to the columns above and below the joint during dynamic response can be considerably different from that assumed in the design. The effect of higher modal responses can alter the moment distribution at the joint to such an extent that the sum of the beam moments is resisted entirely by one of the two columns. Previous studies by Paulay (1981) have indicated that a dynamic magnification factor should be used in the design of columns to prevent upper or middle storey columns from yielding.

CONCLUSIONS

Because of their different dynamic characteristics, ground motions in the three categories of seismic areas produce different distributions of inelastic deformation demands on high-rise frame buildings. For high A/V ground motions, the "whiplash" effect is very significant due to significant contribution of higher modal responses, and inelastic deformations in the upper storeys can be higher than those in the lower storeys. In particular, plastic hinges would be developed in the upper storey columns. Therefore, careful detailing should be given to the upper story columns and beams of high-rise buildings situated in high A/V seismic regions ($Z_a > Z_v$).

For low A/V ground motions, inelastic deformations are concentrated in the lower storeys. This is particularly true for the cumulative plastic rotation because of the combined effect of the duration of strong shaking. Therefore, special attention should be given to the design and detailing of the columns and beams in the lower storeys of high-rise buildings located in low A/V seismic areas ($Z_a < Z_v$). Because the lower storeys are more vulnerable than the upper storeys due to the high axial loads carried by the columns, low A/V ground motions are more damaging to high-rise buildings than high A/V ground motions even when the buildings are designed based on peak ground velocity.

ACKNOWLEDGEMENTS

The writers wish to acknowledge the support from the Natural Science and Engineering Research Council of Canada for the work presented herein.

REFERENCES

Associate Committee on National Building Code. 1990. National Building Code of Canada 1990. National Research Council, Ottawa, Ont.

Canadian Standards Association. 1984. Design of concrete structures for buildings, CAN3-A23.3-M84. Ottawa, Ont.

Clough, R.W. and Benuska, K.L. 1967. Nonlinear earthquake behaviour of tall buildings. J. Eng. Mech. Div., ASCE, 93, 129-146.

Heidebrecht, A.C., Basham, P.W., Rainer, J.H. and Berry, M.J. 1983. Engineering applications of new probabilistic seismic ground-motion maps of Canada. Can. J. Civ. Eng., 10, 670-680.

Kanaan, A. and Powell, G.H. 1973. General purpose computer program for inelastic dynamic response of plane structures. Report No. EERC 73-6, Univ. of Calif., Berkeley, Calif.

McCann, N.W. and Shah, H.C. 1979. Determining strong-motion duration of earthquakes. Bull. Seism. Soc. Am., 69, 1253-1265.

Otani, S. 1980. Nonlinear dynamic analysis of reinforced concrete building structures. Can. J. of Civ. Eng., 7, 334-344.

Paulay, T. 1981. Developments in the seismic design of reinforced concrete frames in New Zealand. Can. J. Civ. Eng., 8, 91-113.

Wilson, E.L. and Habibullah, A. 1987. Static and dynamic analysis of multi-story buildings, including P-delta effects. Earthquake Spectra, 3, 289-298.

Zhu, T.J. 1989. Inelastic response of reinforced concrete frames to seismic ground motions having different characteristics. Ph.D. thesis, McMaster University, Hamilton, Ont.

Table 1. Design gravity loads (kN/m^2)

	Dead load	Live load
Roof	6.5	1.0
Floor	7.0	2.4

Table 2. Statistical summary of strong-motion durations (sec)

	Low A/V	Inter. A/V	High A/V
Mean	12.58	7.41	2.7
COV*	0.452	0.290	0.558

* Coefficient of variation

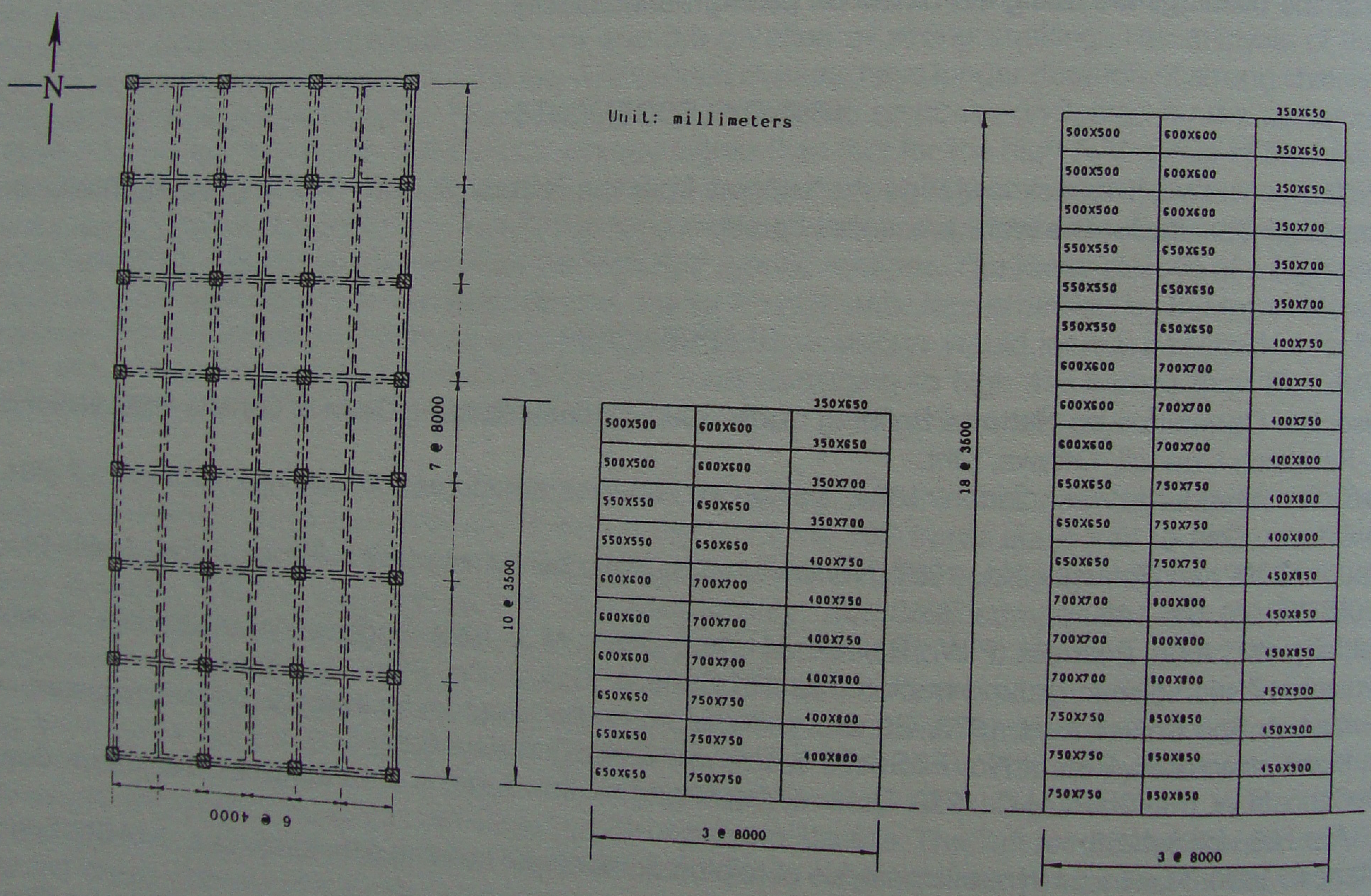

Figure 1. Floor plan and frame elevations

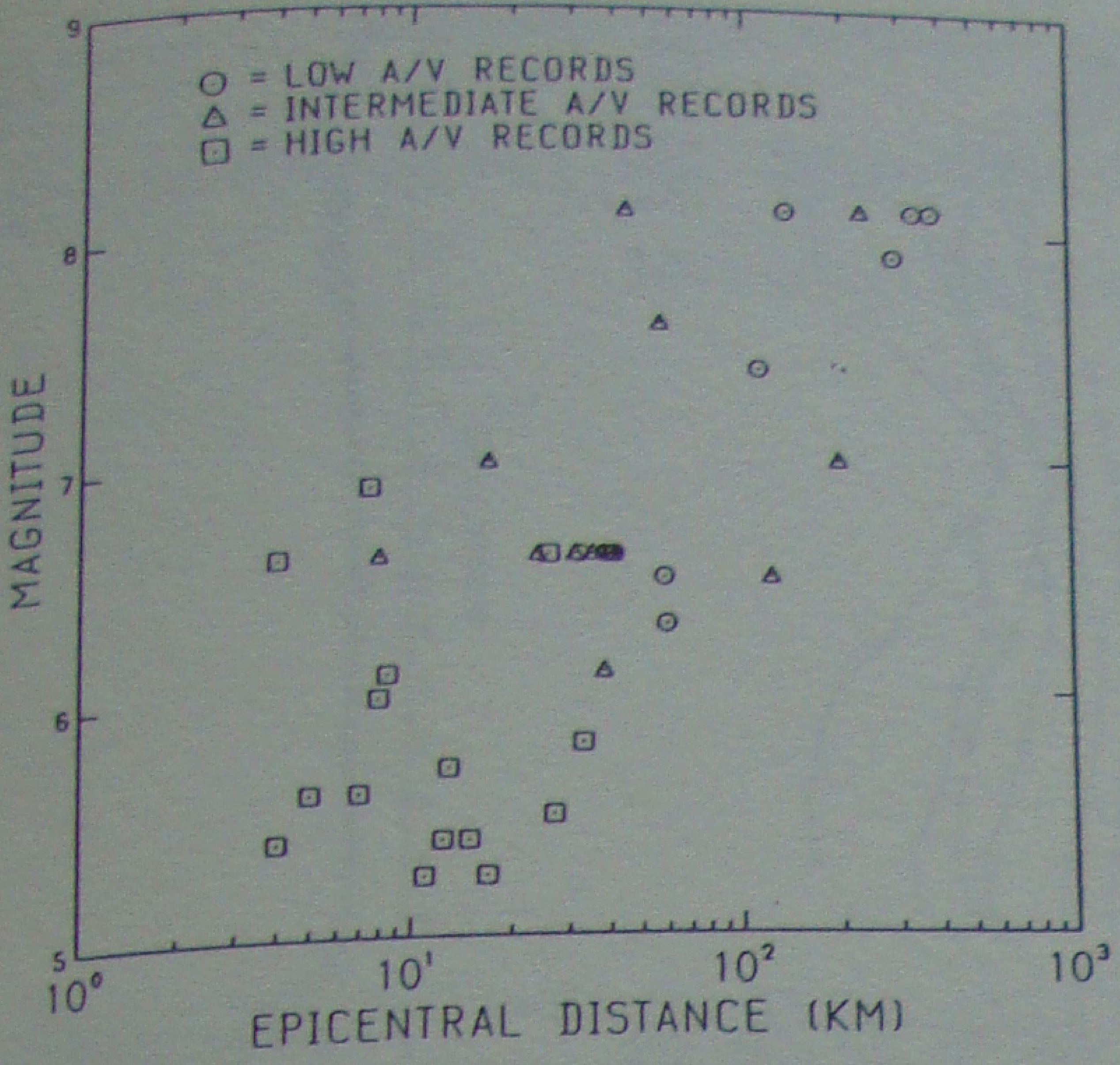

Figure 2. Distributions of magnitude and epicentral distance for three A/V groups of records

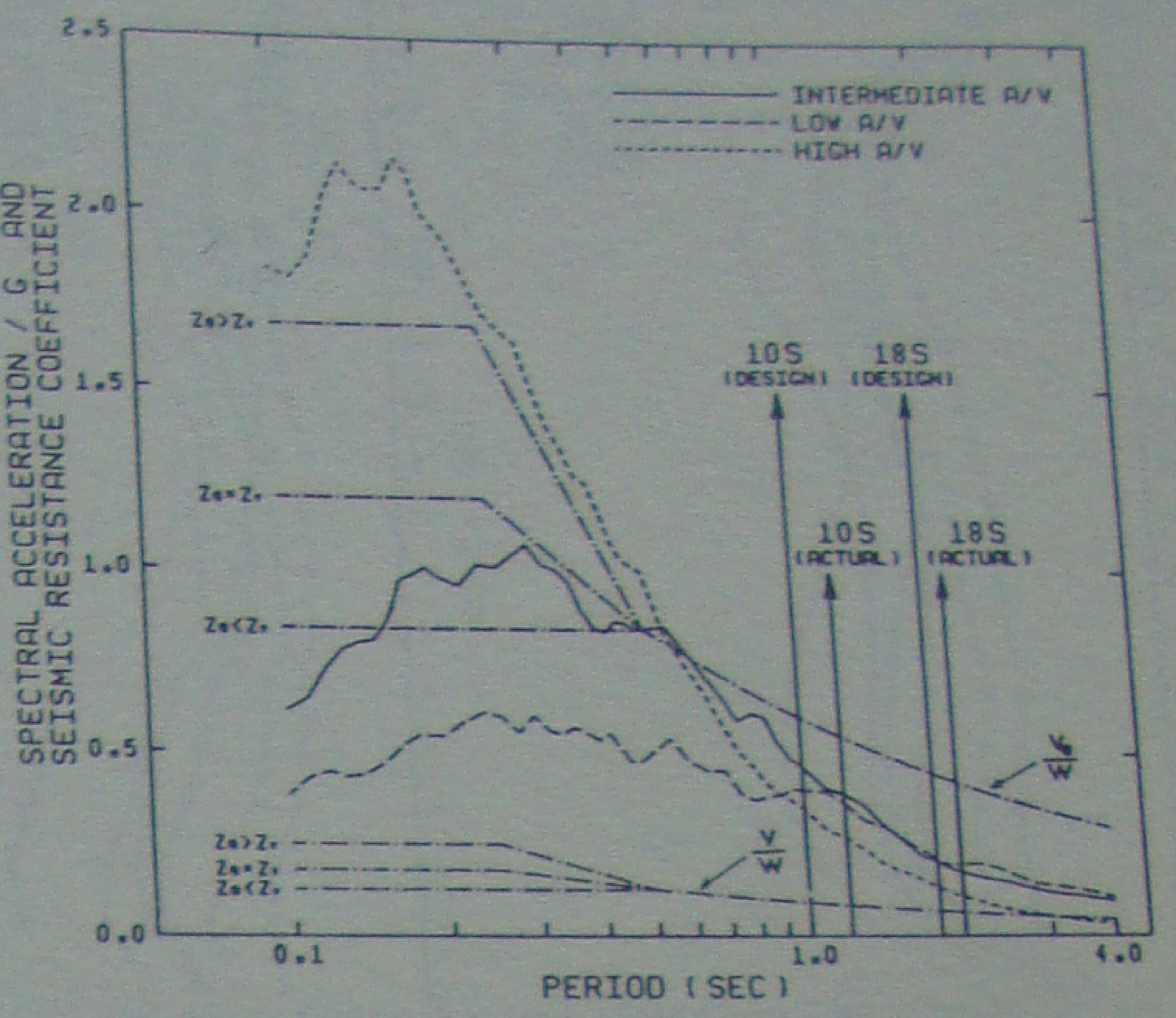

Figure 3. Mean elastic response spectra of three A/V groups of records and elastic and inelastic design spectra

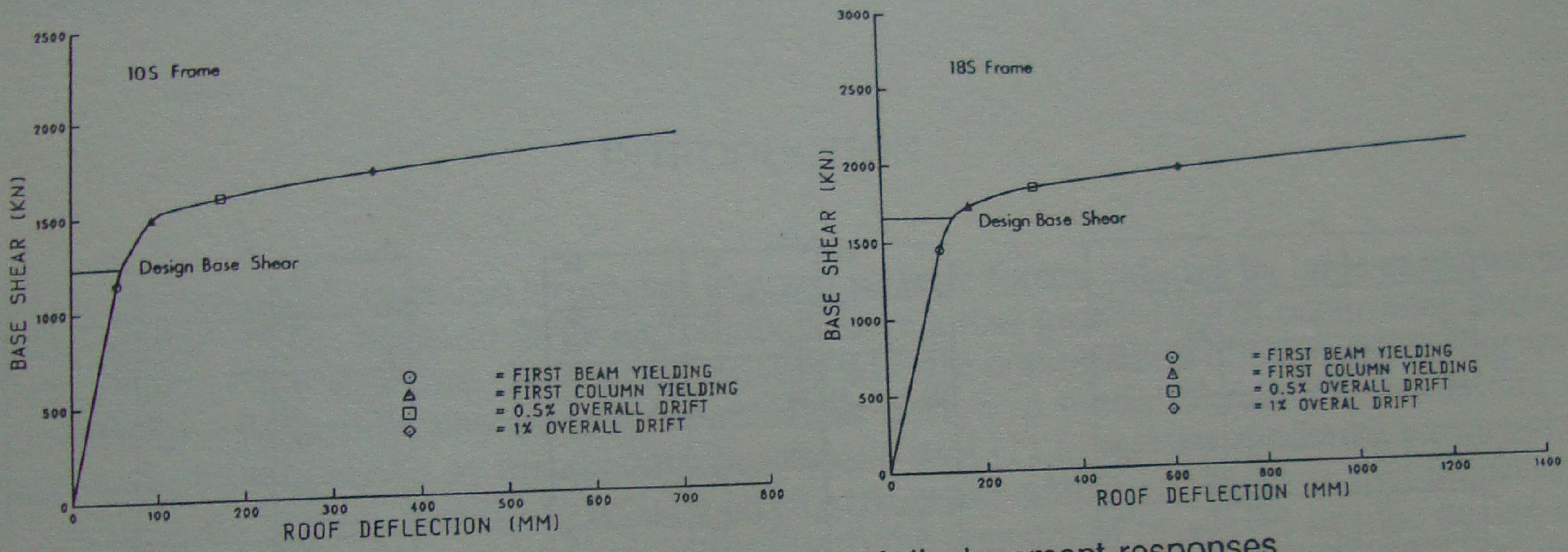

Figure 4. Base shear versus roof displacement responses for monotonically increased lateral loading

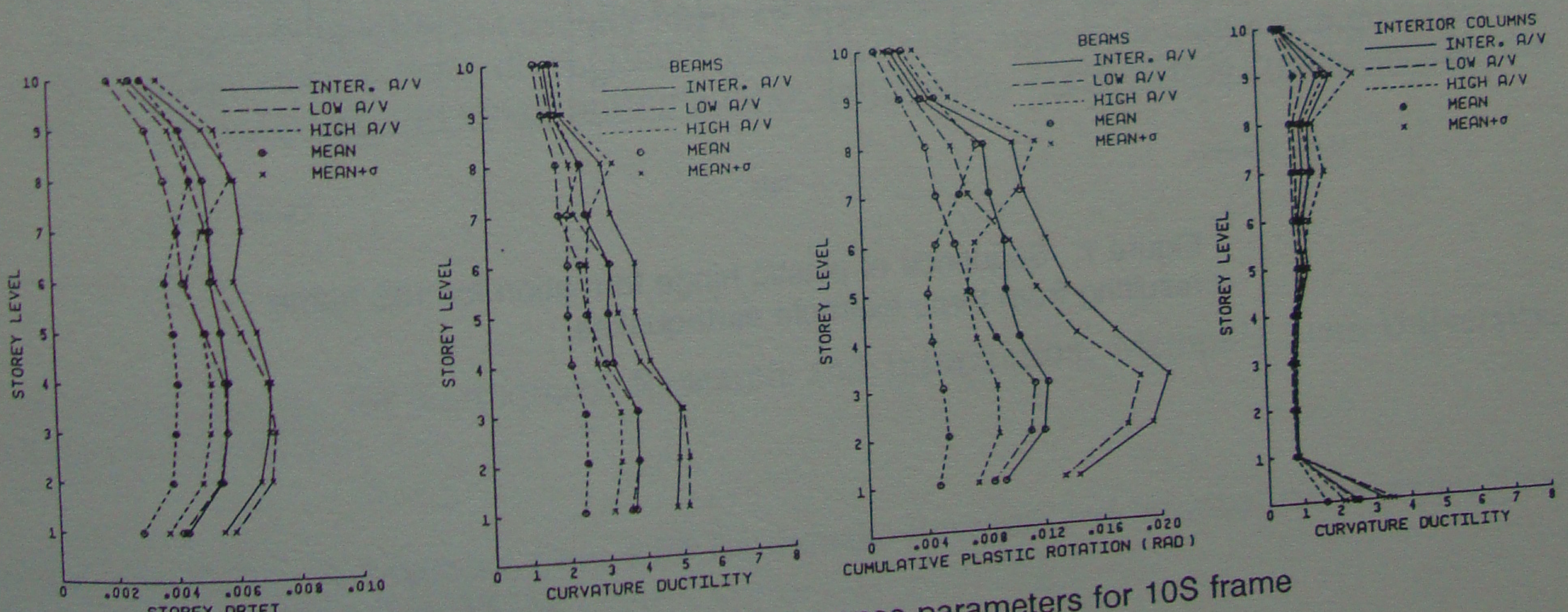

Figure 5. Statistical results of response parameters for 10S frame

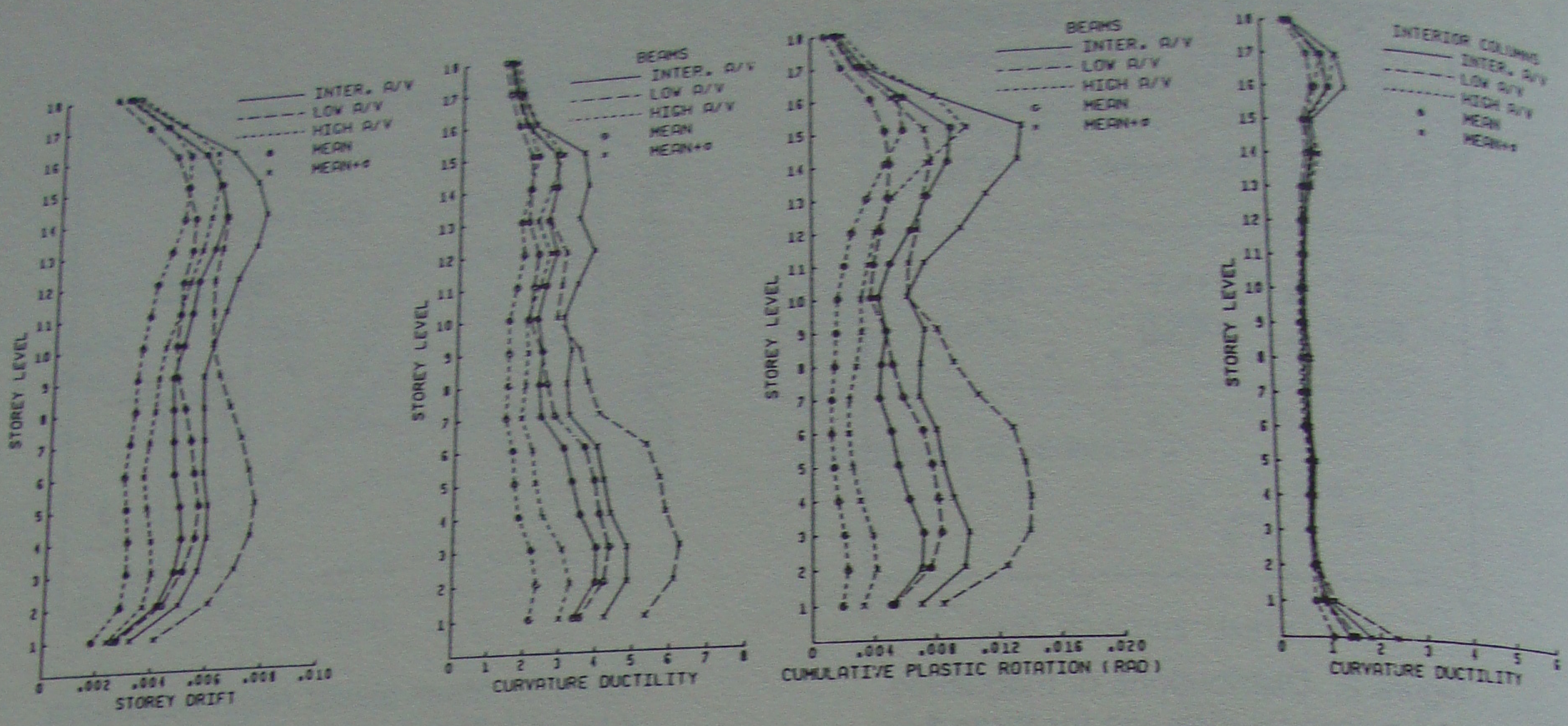

Figure 6. Statistical results of response parameters for 18S frame

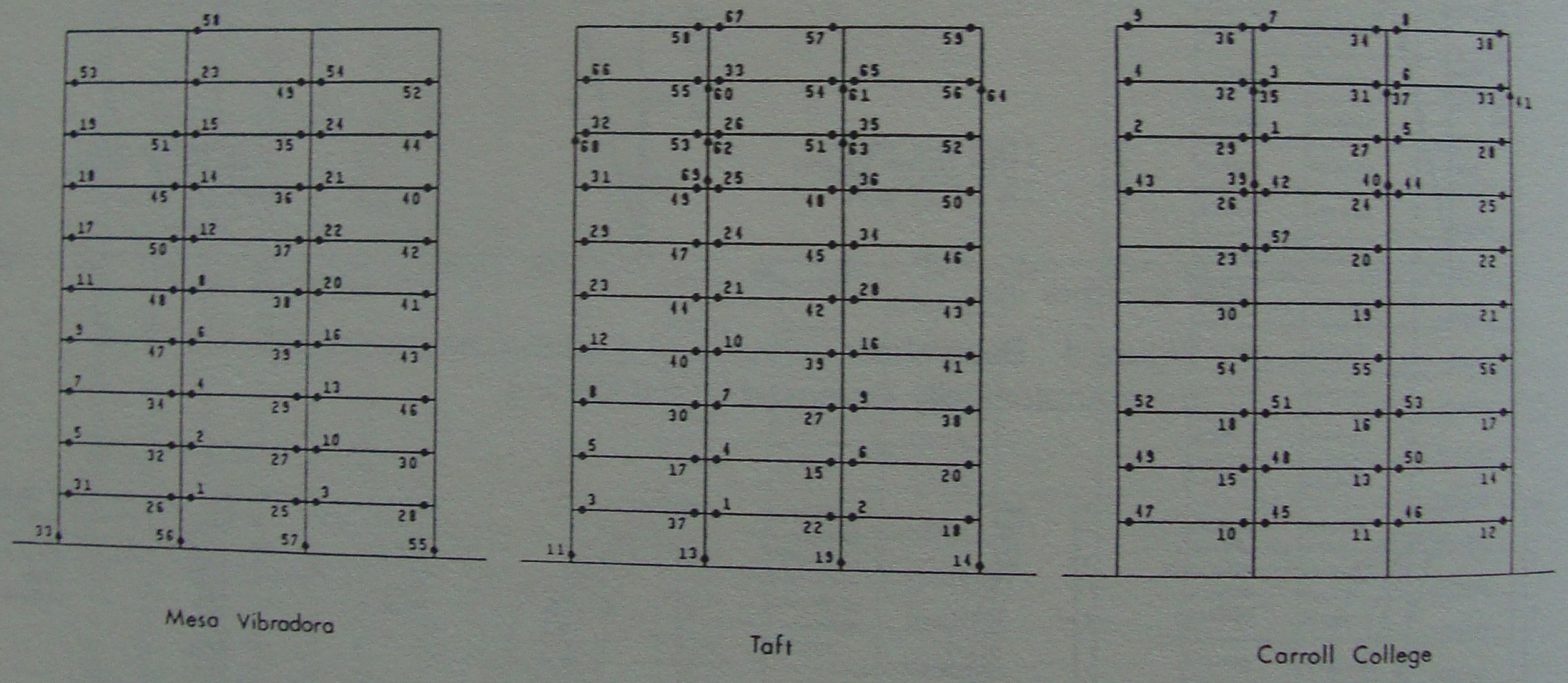

Figure 7. Sequence of plastic hinge formation for 10S frame resulting from three example earthquakes