

SOME COMMENTS ON REINFORCED CONCRETE STRUCTURES FORMING COLUMN HINGE MECHANISMS

T. E. Kelly*

ABSTRACT

Three buildings relying on column hinge mechanisms for post-elastic energy dissipation were studied using an inelastic dynamic computer program. The structures were an eight storey wall structure with ground storey columns, an eight storey frame with rigid, non-yielding beams, and a single storey frame with rigid, non-yielding beams.

Parameters varied were earthquake input, design base shear and strain hardening ratio.

All structures exhibited deformations far in excess of deflections under code static loading. The eight storey structures showed a tendency towards incremental collapse from P-delta effects when low, probably realistic, strain hardening ratios were used.

1. INTRODUCTION

The New Zealand Code of Practice for the Design of Buildings, NZS 4203:1976⁽¹⁾, Cl 3.3.3.4.1 requires that column hinge mechanisms be avoided except for one and two storey structures. Research has shown that axial loads have adverse effects on member ductility capabilities^(5,6,7), and structures in which inelastic energy dissipation is concentrated in vertical members have historically performed poorly in earthquakes. There have recently been suggestions that with modern standards of ductile detailing presently required such structures may be capable of satisfactory behaviour under seismic loading. It was further suggested that because buildings such as that modelled in structure I (Fig 1) are similar to structure III (Fig 1) in that both essentially consist of a single rigid mass above a flexible lower storey they might qualify as "two storey" buildings.

In this study three structures relying on column hinge mechanisms for post-elastic response were analysed using an inelastic computer program to determine member ductility requirements and structure deformations. The parameters varied in the analyses were (a) the earthquake input, (b) the design base shear level, and (c) the member strain hardening ratio.

2. STRUCTURES ANALYSED

The three structures analysed are shown in Figure 1, together with the models used to represent them in the computer analyses. Structures I and II are typical bents from an eight storey building, with masses consistent with such bents being spaced at 6 m centres. Structure I is a wall structure reducing to columns at the ground storey, while structure II is a frame with beams far stiffer and stronger than columns, the beams being taken as rigid and non-yielding for these analyses.

Neither of these frames would comply with the requirements or spirit of NZS 4203:1976. Structure III is a single storey structure modelled to have the same total mass as Structure I and II and the same column sizes as Structure I. As this structure is only single storeyed a column hinge mechanism is permitted by NZS 4203, and as the columns are reinforced concrete the limitations on axial stress in NZS 4203:1976 C 3.3.3.5.2 would additionally apply. Notwithstanding the fact that the structural form of structures I and II did not comply with the code, to examine the validity of the requirements all structures were designed by the equivalent static force method of NZS 4203. In the derivation of the applied horizontal force the factors were taken as $C = 0.15$, $I = 1.0$, $M = 1.0$ and $R = 1.0$. As structures I and II did not comply with NZS 4203, the structural type factor, S , was arbitrarily taken as twice the value for a "ductile frame with an adequate number of possible plastic hinge positions", i.e. $S = 0.8 \times 2 = 1.6$. To enable direct comparisons, structure III was also designed for $S = 1.6$. As the code allows a value of $S = 1.0$ for structures such as structure III, each model was also analysed with strengths based on $S = 1.0$. The total base shear for design was therefore:

$$V = W_t \quad (\text{CISMR})$$

$$V = 0.24 W_t \quad \text{for } S = 1.6, \text{ and}$$

$$V = 0.15 W_t \quad \text{for } S = 1.0$$

The column sizes were selected so that:

- (a) static load deformations were within code limitations,
- (b) maximum reinforcing ratios were not exceeded⁽³⁾, and
- (c) the fundamental frequency of each structure was similar.

This last specification was to ensure that the response of each structure to the input earthquake records was of similar magnitude. The design of structures I and III was

* Engineer, Ministry of Works and Development, Wellington.

governed by requirement (b) and that of structure III by requirement (c).

3. ANALYSIS

The structures were analysed using the DRAIN-2D⁽²⁾ computer program. This program uses step by step numerical integration techniques to produce a time history of inelastic response to a given acceleration record.

Each structure was analysed for the following conditions:

1. Earthquake record El Centro 1940 N-S
2. Earthquake record Artificial B1
3. Earthquake record Artificial A1, each with a strain hardening ratio of 6% and design strength derived from loadings using structural type factor $S = 1.6$.
4. as for (2) but with strength based on $S = 1.0$
5. as for (2) but a strain hardening ratio of 1%
6. as for (2) but zero strain hardening, i.e. elastic - perfectly plastic.

For all these analyses damping was taken as 5% of critical for each of the first two frequencies, the integration time step as $\Delta t = 0.01$ secs, and P-delta and shear deformation effects were included.

Member yield capacities were programmed to have the simple required design strengths with no overstrength factors.

The artificial earthquakes A1 and B1 are described fully in reference 8. Briefly, they are randomly generated simulated accelerograms scaled and filtered to have an average spectrum intensity of 150% and 100% of El Centro 1940 for records A1 and B1 respectively. Earthquake A1 is intended to model shaking in the vicinity of the causative fault of a Richter Magnitude 8 shock and B1 the strong shaking of a Richter Magnitude 7 earthquake.

For the El Centro earthquake records the first 10 secs. of record was used, and for the artificial earthquakes the first 20 secs. of record was used.

For structures I and III, in which axial loads varied during the analyses, the effect of axial load on the yield moment was modelled. In the model used for structure II, the column strengths and stiffnesses were summed and modelled as a single member. The joint rotations were eliminated (simulating very stiff beams) by fixing each node against rotation. As this model does not take into account earthquake induced axial load, the member is specified as a beam-type element, i.e. yield moment independent of axial load level.

4. RESULTS

The maximum displacements and associated member displacement ductilities for each structure are listed in Table 1. Figures 2 to 4 are plots showing the time history of ground storey displacements and ductilities for each analysis. Note that the vertical scales are different for Figures 2, 3 and 4.

4.1 Effect of Earthquake Record

For all three structures the largest deformations and ductility demands occurred under artificial earthquake B1. The effects of the A1 record were a little less, and the smallest response was caused by the El Centro earthquake. Figure 5 shows the response spectrum of a 5% damped single-degree-of-freedom oscillator for each of the earthquake records used in the analyses. Also marked on Fig. 5 are the fundamental periods for each structure, based on the elastic properties. Natural periods increase as members yield and so the relative magnitudes of the inelastic response to each record cannot be predicted from Fig. 5. The positions occupied on the spectra show that even a relatively small increase in period would change the relative response to each of the earthquakes.

Note that the response spectra are derived from the full earthquake records whereas the analyses utilized only a portion of each record, so it is possible that the maximum response for each record was not obtained. This may account for B1 having a greater effect than A1 in the analyses reported here.

4.2 Effect of Design Shear Level

Predictably, the deformations and ductility requirements were greater when a lower structural type factor, and therefore lower yield level, was used in design. When the yield level was reduced by 37.5% of its original value, i.e. S decreased from 1.6 to 1.0, the increases in interstorey displacements were 46%, 10% and 16% for structures I, II and III respectively. The design shear level seemed most critical for the eight storey structure with columns at ground level only. This can be accounted for by the large axial forces caused by over-turning moments on the structure, as the yield moments in this structure were affected greatly by the axial force level. Because of the lower yield displacement associated in all such cases with $S = 1.0$, the displacement ductilities increased by a correspondingly greater proportion than the interstorey drift.

4.3 Effect of Strain Hardening Ratio

In this study the strain hardening ratio is defined as the slope of the plastic portion of the bi-linear moment rotation relationship. It is expressed as a proportion of the elastic stiffness slope. Therefore, a 6% strain hardening ratio would lead to a 6% increase in moment capacity at ductility 2.

The strain hardening ratio had a far greater effect than any other parameter varied for the eight storey structures, and a less marked effect for the single storey structure. For both eight storey structures reduction of strain hardening caused very large increases in interstorey displacements and therefore in ductility demands in the columns. For the structure with rigid beams the ratio of first storey displacement to top storey displacement was 0.35, 0.52 and 0.92 for strain hardening ratios of 6%, 1% and 0% respectively. This illustrates

the concentration of inelastic deformations in the ground storey as the strain hardening ratio decreased.

The time history plots in Figs 2 to 4 show that for the eight storey structures with elastic-perfectly plastic hysteresis curves the structures are tending towards incremental collapse. With low strain hardening ratios there is little resistance after yielding, and therefore large storey drift and consequently large P-delta moments. Succeeding accelerations are unable to return the building to its original position. The P- Δ moments reduce the capacity of the hinging columns to resist earthquake shears as displacements increase, and these moments effectively reduce the additional moment capacity caused by strain hardening.

At the end of the 20 secs. of analysis the P-delta moments amounted to 38% and 25% of the yield moments for the wall structure and rigid beam structure respectively, effectively decreasing the strain hardening ratios by 2% and 1% for the respective structures. As the structures were apparently displacing increasingly in one direction, they would shortly have reached an unstable condition.

The single storey structure did not exhibit sensitivity to the strain hardening ratio to the same extent, with the elasto-plastic P-delta moment reaching a maximum of only 12% of the yield moment.

5. DISCUSSION OF RESULTS

The interstorey drifts were considerably in excess of the values calculated from code requirements for regular structures. Under code loading of $0.24 W_t$ the maximum interstorey displacement was $\Delta_i = 9.6$ mm for structures I and III and with the magnification factor of 1.25 from NZS 4203:1976, the maximum interstorey displacement was 12 mm, or 0.004h. For structure II the respective values were 3.3 mm, 4.2 mm and 0.0014h. As can be seen from Table 1 these values were exceeded by large amounts for all analyses, e.g. by factors of 6.3, 7.1 and 4.8 for structures I, II and III respectively under the action of earthquake B1 and with 6% strain hardening. The deflections were far greater when lower strain hardening ratios were used, and for all structures were far in excess of the maximum value of 0.010h permitted by the code.

The maximum displacement ductilities for all structures were generally within the likely capabilities of well detailed column sections for all analyses using $S = 1.6$ and strain hardening ratios of 6%. However, the wall structure with columns had extremely high earthquake induced axial loads, with a maximum value of $P = 0.63f_c A_g$ where P is the total of earthquake induced and gravity axial loads. Detailing for, and achieving, ductility at such stress levels is generally recognised to be extremely difficult, and in fact reference (3) restricts the maximum load to $0.6f_c A_g$ for hoop ties and to $0.7f_c A_g$ for spiral confinement.

When an S factor of 1.0 or a strain hardening ratio of less than 6% was used the ductility requirements increased considerably and the ability of confining reinforcement to

preserve strength and stiffness during such high plastic rotations would be suspect, especially in the presence of relatively high compressive loads.

The modelling of the post-elastic moment-rotation relationship for the column plastic hinges proved to be one of the most important parameters in this study, and unfortunately it is also one of the parameters where model behaviour is only a gross approximation of real behaviour. The bilinear moment rotation relationship modelled in the program takes no account of the stiffness degradation that inevitably occurs in reinforced concrete sections nor of the strength loss that will occur in a compression hinge after several cycles of high plastic rotations.

The inelastic loading cycles on the columns can be determined from Figs. 2 to 4. For example, from Figure 2(a) it can be seen that for earthquake B1, strain hardening ratio 6%, over 20 cycles of plasticity occurred with a maximum displacement ductility of 5. However, most cycles of plasticity had an amplitude of ductility of the order of 2 or 3. This loading pattern is quite different from loads imposed on test specimens, and makes it difficult to estimate from experimental evidence what strain hardening ratio would be closest to true behaviour. In tests conducted by Davey⁽⁴⁾ on elements of similar moment-shear ratio the approximate average strain hardening ratio was 3.8% for the first cycle at ductility 4 reducing to 1.7% and 1% in the subsequent cycles at similar ductility. This is based on the maximum moment attained in each cycle compared with the ultimate moment capacity of the specimen. Davey's specimens were lightly loaded axially and had circular hoop reinforcement. For the case studied here with high axial load levels and rectangular ties behaviour could be expected to be less satisfactory, and hence a reasonable strain hardening ratio would certainly be less than 6%, and would probably be less than 1%, perhaps becoming negative with a net drop in moment capacity. A single run with the eight storey rigid-beamed structure using a negative strain hardening, -6%, showed total instability and collapse after approximately 10 secs. of the B1 record.

6. CONCLUSIONS

The three structures studied, all of which relied on column deformations for energy dissipation, showed deformations far in excess of code loading deflections.

The two eight storey structures proved to be very dependent on strain hardening for overall stability. At low strain hardening ratios they showed a tendency towards continually increasing deformations in one direction, indicating progressive failure as the P-delta moments reached high proportions of the ultimate column moments. For columns with such high axial loads it is considered that even higher standards of ductile detailing would not provide increasing moment capacity at high plastic rotations so the models using low strain hardening ratios would probably be the most realistic. In a real structure stiffness degradation would have

a further detrimental effect.

The single storey structure was less sensitive to the strain hardening ratio than the eight storey structure because, even though it had the same total mass, the lever arm to the centre of mass was lower, and hence the total overturning moment was lower. Nevertheless, the deformations were considerably greater than those given by code loading for regular structures.

The results of this study indicate that multistorey reinforced concrete structures relying on column hinge mechanisms are unsatisfactory for earthquake resistance when designed to code loading, or even twice code loading, in spite of ductile detailing. There is a significant risk that large deformations would initiate incremental collapse, and in any case would certainly lead to severe non-structural damage and preclude repair.

The question arises as to whether larger columns, required by the axial stress limitations of NZS 4203 C3.3.3.5.2 applicable to 1 and 2 storey column mechanisms, would have adequately controlled the large displacements in the structures analysed. The axial load limitations relate to balanced failure conditions and are therefore a function of column area, and could be met by columns in the form of walls transverse to the direction of attack. Such walls would have a flexural capacity and stiffness similar to that of the columns used in the analyses, and the displacements which are the cause of the unacceptably high ductility demands would not be reduced. While the restrictions on axial loads in the code apply to reinforced concrete columns, the displacement effects must be equally, if not more, serious in the case of structural steel.

Restrictions on the axial load level would prevent non-ductile compressive hinging but the problem is fundamentally one of strength. Therefore, the adverse effects of column hinging in the multistorey buildings examined could only be controlled by the choice of a suitably high S factor. Present knowledge is insufficient to codify the requirements for the possible range of structures which could be designed if Clause 3.3.3.5.2 were interpreted to include cases with a high, presumed non-yielding, upper structure. The code should therefore be amended to clearly exclude structures in which the centre of gravity of a semi-rigid mass carried by lower storey flexible elements is substantially higher than in a two storey building.

7. ACKNOWLEDGEMENTS

This investigation forms part of a continuing study of seismic aspects of building design being conducted in the Structural section of the Ministry of Works and Development under the general direction of Mr. B. W. Buchanan, Research and Development Engineer. The writer wishes to thank Mr. Buchanan and also Mr. O. A. Glogau, Chief Structural Engineer, for their continued guidance and assistance. Grateful acknowledgement is made to engineering and draughting staff for their assistance, and to Mr. K. E. Williamson whose comments prompted this study.

The permission of the Commissioner of

Works to publish this paper is acknowledged.

8. REFERENCES

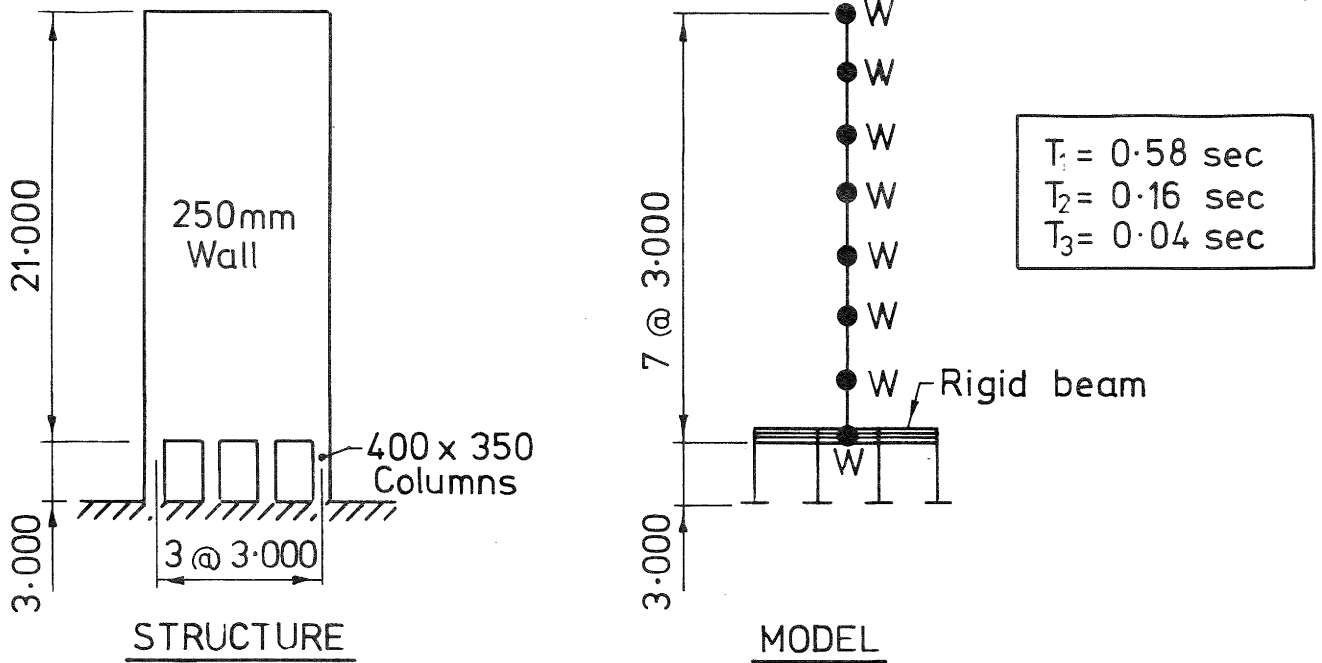
1. "General Structural Design and Design Loadings for Buildings", NZS 4203:1976 Standards Association of New Zealand.
2. Kanaan, A. E. and Powell, G. H., "Drain 2-D : A General Purpose Computer Program for Dynamic Analysis of Inelastic Frame Structures", University of California, 1973.
3. Park, R., "Columns subjected to Flexure and Axial Load". Bull. N.Z. National Society for Earthquake Engineering, Vol. 10, No. 2, June, 1977.
4. Davey, B. E., "Reinforced Concrete Bridge Piers Under Seismic Loading", Research Report 75-3, University of Canterbury, New Zealand.
5. Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley and Sons, New York, 1975.
6. Norton, J. A., "Ductility of Rectangular Reinforced Concrete Columns", Master of Engineering Report, University of Canterbury, 1972.
7. Leslie, P. D., "Ductility of Reinforced Concrete Bridge Piers", Master of Engineering Report, University of Canterbury, 1974.
8. Jennings, P. C., Housner, G. W., and Tsai, N. C., "Simulated Earthquake Motions for Design Purposes", Proc. 4th World Conference on Earthquake Engineering, Chile, 1969.

Paper received 5 November, 1977.

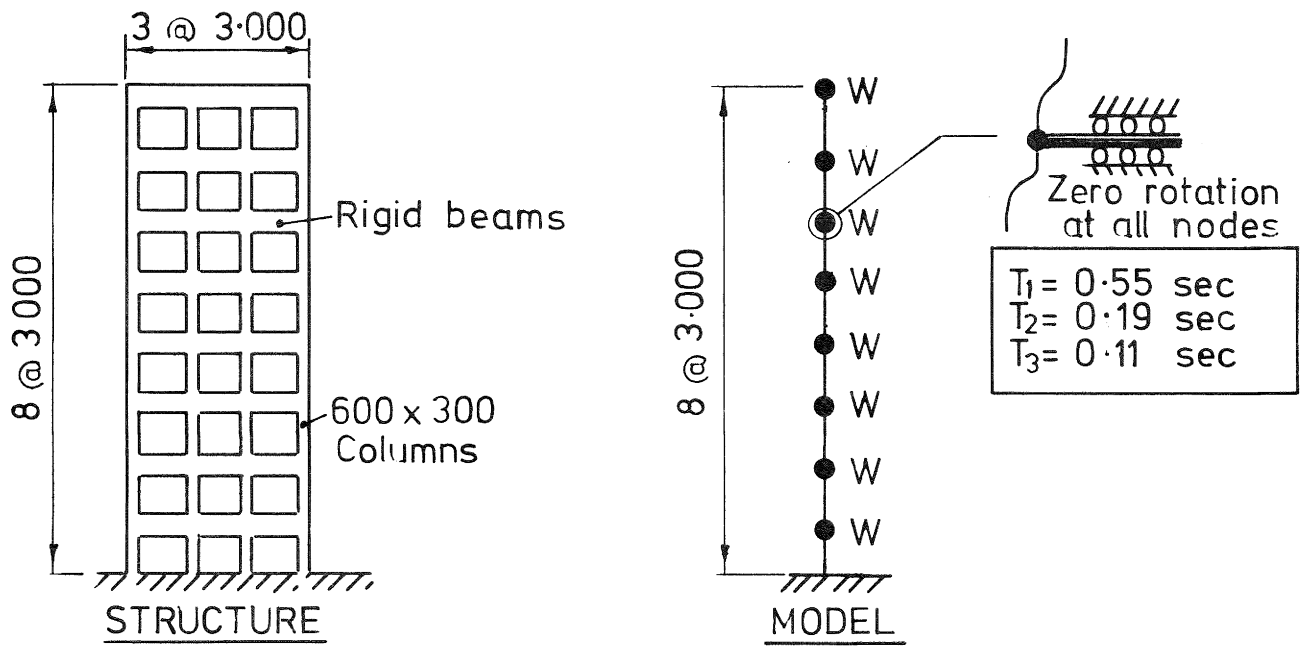
TABLE 1

SUMMARY OF RESULTS

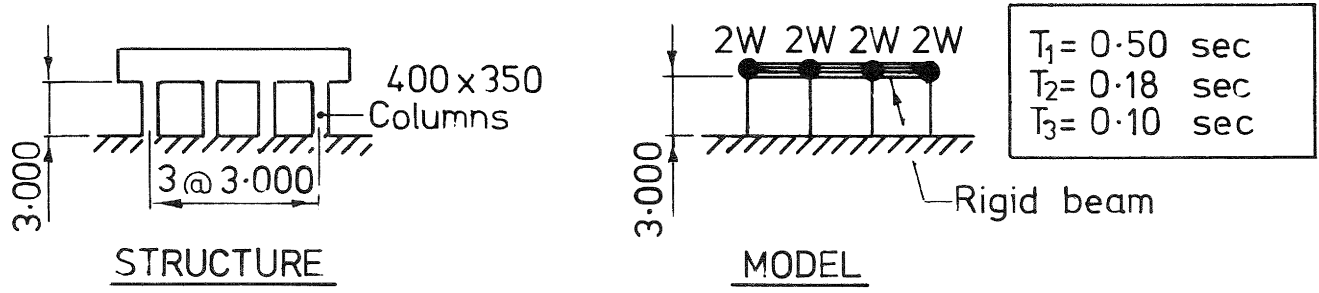
	VARYING EQ. RECORD S = 1.6, SHR = 6%			VARYING STRUCT TYPE FACTOR B1, SHR = 6%		VARYING STRAIN HARDENING RATIO B1, S = 1.6		
	EL CENTRO	B1	A1	S = 1.6	S = 1.0	SHR = 6%	SHR = 1%	SHR = 0%
<u>I WALL WITH GROUND FLOOR COLUMNS</u>								
Δ TOP MAX. (mm)	53	84	75	84	114	84	199	273
Δ i MAX. (mm) (Δ i/H)	45 (.015)	76 (.025)	65 (.022)	76 (.025)	111 (.037)	76 (.025)	193 (.064)	271 (.090)
MAX. COL. DISPL. DUCTILITY	3	5	4	5	12	5	13	18
<u>II STRUCTURE WITH RIGID BEAMS</u>								
Δ TOP MAX. (mm)	63	85	67	85	95	85	121	194
Δ i MAX. (mm) (Δ i/H)	20 (.007)	30 (.010)	27 (.009)	30 (.010)	33 (.011)	30 (.010)	63 (.021)	178 (.059)
MAX. COL. DISP. DUCTILITY								
STOREY 1	4	6	5	6	10	6	12	34
" 2	3	4	4	4	8	4	4	2
" 3	2	2	2	2	6	2	2	2
" 4	2	2	2	2	3	2	2	1
" 5	1	2	1	2	3	2	2	2
" 6	2	2	2	2	3	2	2	3
" 7	1	1	1	1	4	1	2	2
" 8	2	1	2	1	3	1	1	1
<u>III SINGLE STOREY STRUCTURE</u>								
Δ TOP MAX. (mm) (Δ TOP/H)	34 (.011)	57 (.019)	39 (.013)	57 (.019)	66 (.022)	57 (.019)	78 (.026)	86 (.029)
MAX. COL. DISP. DUCTILITY	2	4	3	4	7	4	5	6



I : WALL WITH COLUMNS



II : FRAME WITH RIGID BEAMS



III : SINGLE STORY

$E = .26,000 \text{ N/mm}^2 / I_{\text{MODEL}} = 0.75 I_{\text{GROSS}} / W = 486 \text{ kN}$

FIGURE 1: STRUCTURES ANALYSED

WALL WITH GROUND STOREY COLUMNS
 --- FIRST FLOOR DISPLACEMENT

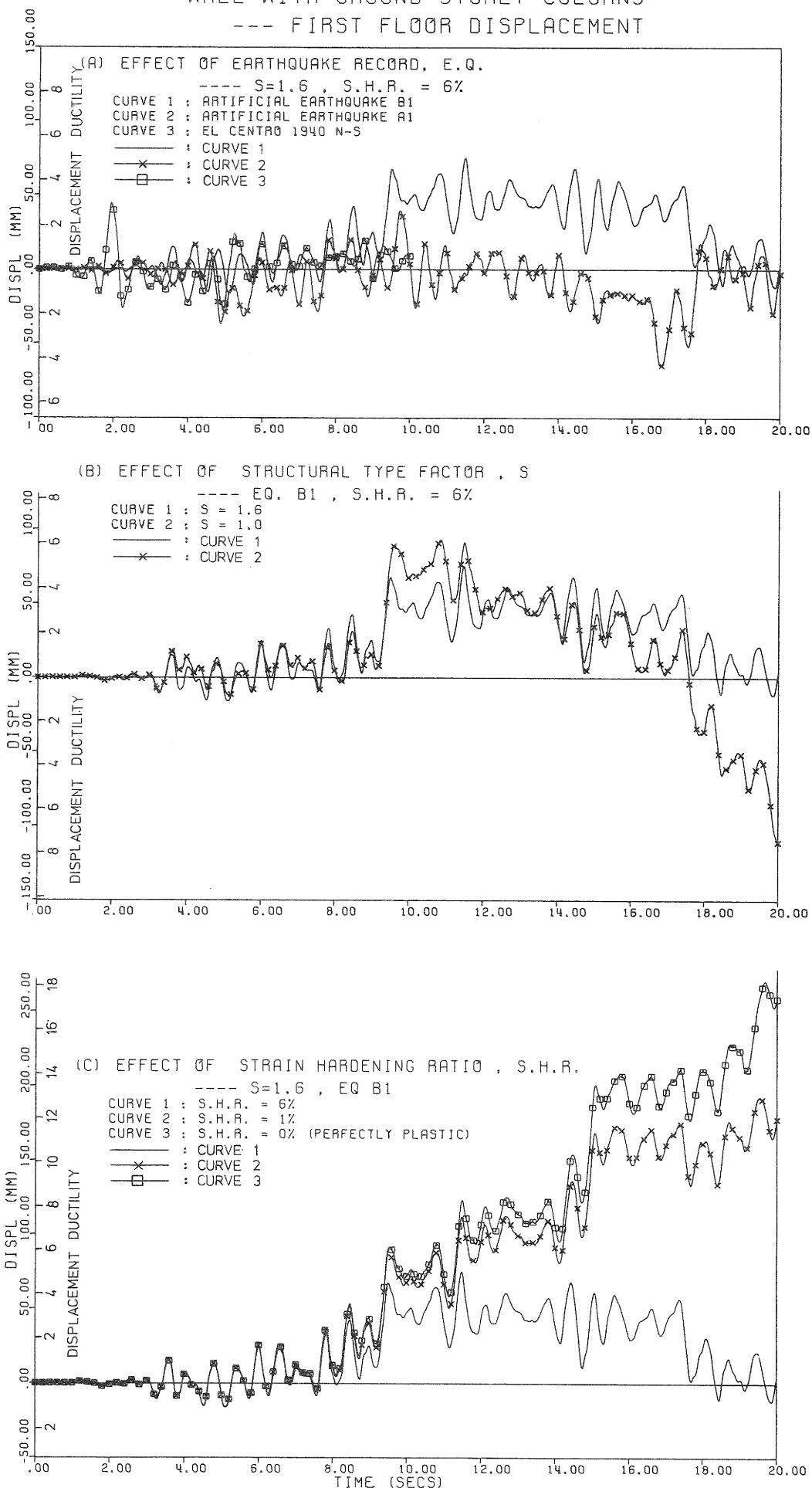


FIGURE 2: RESPONSE OF WALL WITH GROUND STOREY COLUMNS

FRAME WITH RIGID BEAMS
 --- FIRST FLOOR DISPLACEMENT

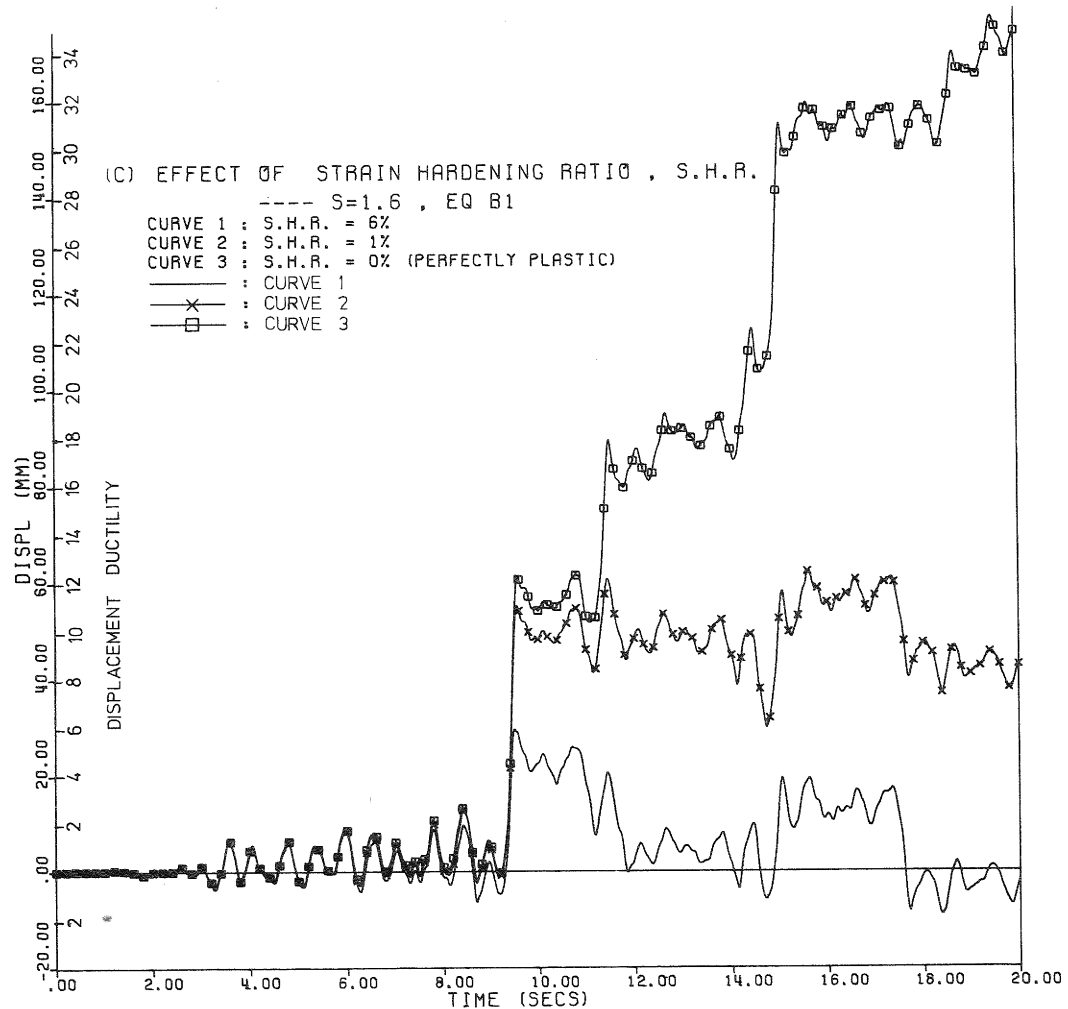
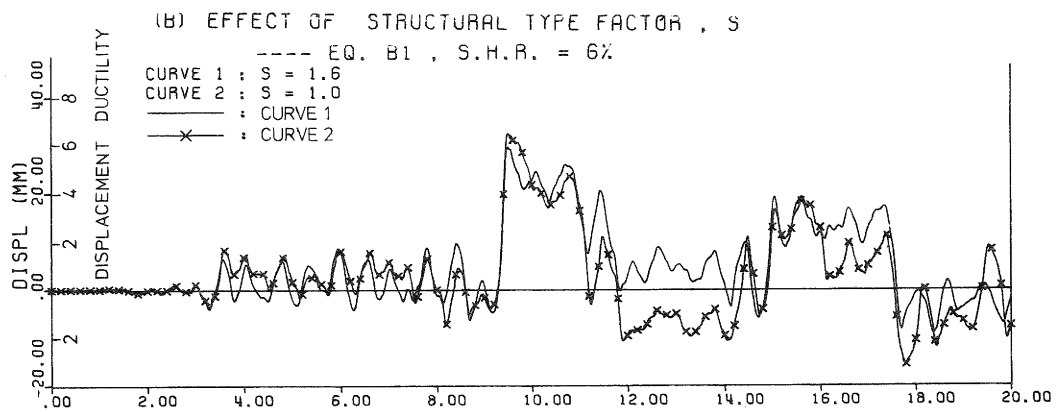
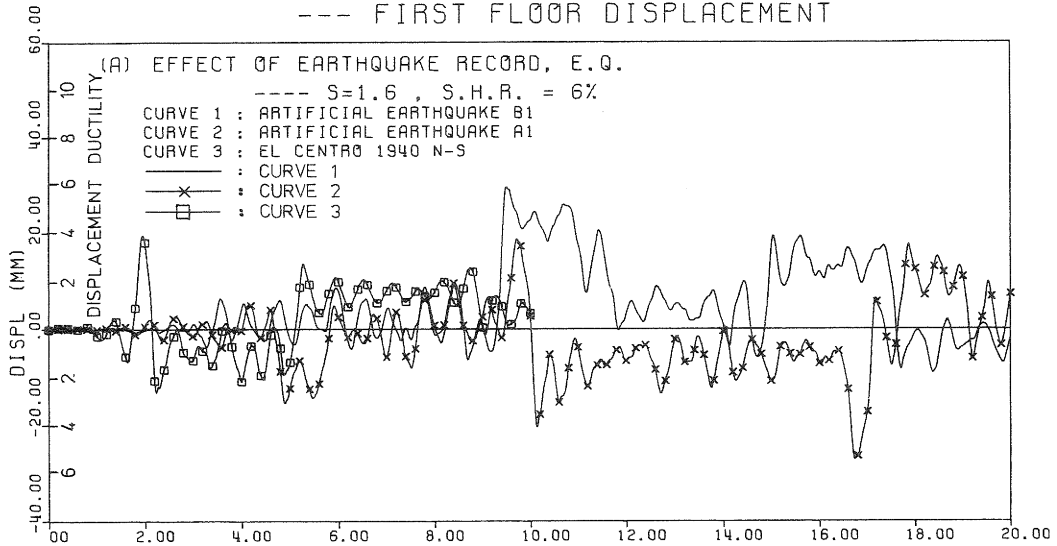


FIGURE 3: RESPONSE OF FRAME WITH RIGID BEAMS

SINGLE STOREY STRUCTURE
 --- FIRST FLOOR DISPLACEMENT

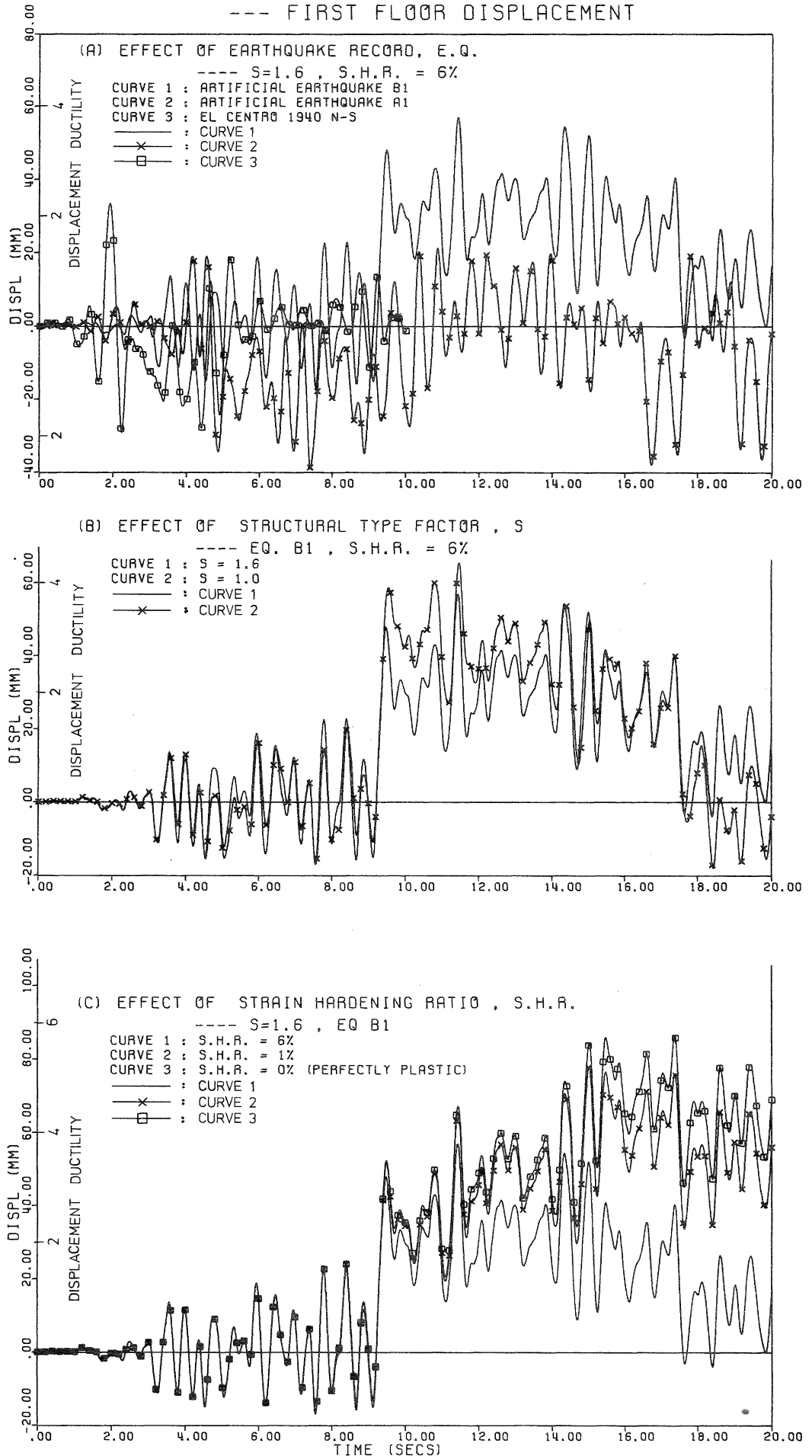


FIGURE 4: RESPONSE OF SINGLE STOREY STRUCTURE

FIGURE 5: ACCELERATION RESPONSE SPECTRA

