UNIAXIAL DYNAMIC ANALYSIS OF A SIX STOREY REINFORCED CONCRETE FRAMED STRUCTURE

T. E. Kelly*

ABSTRACT

A six storey reinforced concrete frame was analysed inelastically using a 2 dimensional dynamic analysis computer program. The program utilises the step-by-step integration method for the solution of the equations of motion. Two natural and three artificial acceleration records were used. Harmonic motions of differing frequencies and each followed by a relatively long acceleration pulse were studied using artificial records created for the purpose. A very approximate allowance for the effects of concurrent earthquake excitation in the orthogonal direction was made by reducing the available column capacities. Parameters studied were ductility requirements, deformations, and column forces.

1. INTRODUCTION

The Research and Development section of the Ministry of Works and Development Structural Design Office is currently carrying out studies on member forces caused by seismic excitations. These studies are part of continuing research to assist in quantifying design requirements for ductile reinforced concrete frames, with a view to improvements in the Department's code PW 81/10/1 "Seismic Design of Public Buildings".

One section of this research involves the design of "typical" framed structures to present code requirements. These structures are designed by current (or modified) static procedures and then analysed dynamically using the DRAIN-2D(1) inelastic analysis computer program. The program restricts analysis to two dimensional frames subjected to one horizontal earthquake component, with or without the vertical component. The structure reported in this paper is a four bay by three bay frame six storeys high. An interior transverse frame of this structure has been analysed using a wide range of earthquake records, both natural and artificial, mainly for the purpose of obtaining column moments and shear forces.

This report is limited to a special series of analyses carried out to investigate the effects on the structure of long duration acceleration pulses in an earthquake record. Bertero has recently suggested(2) that the maximum incremental velocity may be a better indication of a particular earthquake's damage potential than the maximum acceleration recorded. In other words, the length of time for which an acceleration pulse is maintained may be a governing factor in a structure's response to a particular input. To investigate this behaviour a series of analyses were computed for records containing acceleration pulses of relatively long duration. Two 'natural' earthquake records, one of which contained a long acceleration pulse, were studied together with three derived records also containing long acceleration pulses. The first of these three artificial "records" was derived by Bertero(2), while the remaining two were modifications to this original record.

* Engineer, Ministry of Works and Development.

The approximations inherent in a two dimensional analysis of a three dimensional structure cause difficulties in defining column strength. The column steel in the model structure was designed for earthquake loading along both axes simultaneously, whereas the analysis considered uniaxial loading only. For this reason a second series of analyses was carried out to study the effect on response when reduced column capacities were used in an attempt to allow for the effect of loading in the orthogonal direction on the strength in the direction under consideration.

The results of the analysis are reported with regard to building deformations, beam and column ductility demands, and column bending moments and shear forces.

The analyses and results given here are only indicative of the effects to be expected, but serve to illustrate many of the features of inelastic dynamic analyses and the problems faced in (a) attempting to quantify "response" for the purpose of comparing different records and (b) approximating actual building behaviour in a computer simulation. It is for these latter reasons that the results are fully recorded herein: they should not be applied directly to the design of specific structures.

2. STRUCTURE ANALYSED

The frame used throughout these analyses is shown in Figure 1. It is an interior 3 - bay frame of a regular 4 - bay by 3 bay reinforced concrete building. The structure was designed with a 5" floor slab, for dead and live loads as specified in NZS 4203:1976 (3) for office type loadings. Seismic coefficients were those for a public building in Zone A, with a value of C_d = 0.156, corresponding to a short period reinforced concrete frame. Provision for code torsion was omitted from the design as torsional effects could not be modelled in the two-dimensional dynamic analysis. The natural frequencies of the structure were calculated from a modal analysis, giving the first three periods as 0.57 secs, 0.17 secs, and 0.09 secs. respectively. A period of 0.55 secs. calculated by method in NZS 4203 compares well.

The seismic coefficient C_d = 0.156 was based on an estimated period of 0.5 secs. When the more accurate period of 0.57 secs. is used C_d reduces to 0.143 for a structure on rigid subsoil, so that the structure is effectively overdesigned by 9%, with respect to New Zealand code requirements.

2.1 Original Capacities

For the first series of analyses the full member capacities as designed were used. Beam steel was detailed from moments calculated under the specified code loading. The roof beams were governed by minimum steel requirements, as was the bottom steel in the fifth floor beams. Several proposals for the determination of column actions are under study by a committee set up by the N.Z.N.S.E.E. As these are not finalised an approximate method was used in line with their general trend. The column design moments were taken as the moments derived from code loading increased by a factor of 1.50. This factor was based on $\frac{1.1}{0.9} \times 1.25$ where 1.1 represents a reasonably conservative ratio of likely beam steel yield strength to column steel yield strength, 0.9 accounts for the capacity reduction factor used in beam design, and 1.25 is an arbitrary factor introduced to allow for moment distributions differing from those derived from the code specified static loading, and, also other dynamic effects. These increased moment values were calculated about each horizontal axis and the columns designed for the resultant biaxial moments. The earthquake induced axial loads taken to act with these moments were derived by assuming all beams, in each direction, to be hinging* under a moment of 1.35 times their nominal moment capability. The column was designed to these loads at each level, being governed at the top two storeys by minimum steel requirements. Actual bar layouts and sizes were used, rather than the steel areas theoretically required, so that some degree of overstrength resulted.

The dynamic analysis program used has no provision for secondary failure in flexural members, hence the shear steel, confining steel and joint reinforcement was not designed, although a check was made to ensure that requirements could be met.

2.2 Reduced Column Capacities

The structure as designed and originally analysed had column capacities based on biaxial attack, and was therefore considerably overstrength when subjected to only the uniaxial loading imposed by the dynamic analysis. Therefore a second series of analyses was carried out using column capacities reduced to approximate the effects of simultaneous loading in the orthogonal direction. Any such reduction must be arbitrary due to the time-dependent nature of such orthogonal loading and lack of data on biaxial response. However, as a basis for the reduction the following assumptions were made about the ratio of column moments in each direction:

1. Column moments in the direction analysed, M_{ux} are at full magnified values, including beam overstrength and dynamic effects, and

2. Moments in the orthogonal direction, M_{uy} are at values corresponding to code loading only, with no magnification.

From these values the ratio M_{ux}/M_{uy} for each section was calculated, and using a relationship derived by Paulay[4] the ratio M_{ux}/M_{uy} was estimated. Because of the approximations made, average values were used and the moment capacity in the direction considered, M_{ux} was taken as 0.8 Mu for column bases and 0.7 Mu for all other column sections. The moment values on the input interaction diagrams were scaled down by this amount for the second series of analyses. No change was made to the column moment of inertia.

3. GROUND MOTIONS

Plots of the records used are shown in the top diagrams of Figures 2 to 6 and details are given below.

3.1 El Centro 1940 N-S x 1.30

The North-South component of the El Centro 1940 earthquake is commonly used for dynamic analyses, and forms the basis of the design spectrum incorporated in several design codes, e.g. the New Zealand Loading Code, NZS 4203[3]. This record provided a comparison with results from other records containing long acceleration pulses. The code design spectrum is based on a reduced spectrum from the El Centro 1940 N-S earthquake. This design spectrum is for an importance factor I = 1.0. As a value of I = 1.30 was used in this design the El Centro accelerations were scaled up by 1.30 to retain the same relationship. Only the first 10 seconds of the record was used.

3.2 Pacoima Dam Record

This accelerogram was recorded during the San Fernando 1971 earthquake from an accelerogram sited on the top of a steep rock ridge, and therefore the record is not typical of design earthquakes which could reasonably be used for normal structures. It was included in this study because it contains two relatively long acceleration pulses, from T = 2.5 seconds, together with high peak accelerations at about 7.8 seconds. 10 seconds of this record was utilized. The maximum incremental velocity occurring in this record is 60.6 in/sec, compared with a value of 26.4 in/sec. for the El Centro 1940 N-S component factored up by 1.30.

3.3 Harmonic (T = 0.2 sec) with 0.5 Pulse

This is the first of three artificial records analysed. It was derived by Bertero[2], and consists of four cycles of harmonic motion with a period of 0.20

* For convenience throughout the text, the words "hinging" and "hinged" refer to the formation of plastic hinges in members by yielding of steel reinforcing in regions of confined concrete.

seconds and an amplitude of ± 0.25g, followed by a 0.5 second pulse of 0.33 g amplitude. The total record length is 1.30 seconds. The pulse size is derived to give a maximum incremental velocity of 60.6 in/sec, equal to that of the Pacoima Dam record.

3.4 Harmonic (T = 0.2 secs) with two 0.5 sec Pulses

This is the second artificial record and is similar to the previous, except that an additional acceleration pulse, also of 0.5 sec duration and 0.33g amplitude but of opposite sign, is added at the end, giving a total duration of 1.80 seconds. The maximum incremental velocity is 60.6 in/sec.

3.5 Harmonic (T = 0.5 sec) with 0.6 sec Pulse

This record is also similar to the record described in Section 3.3 but the time scale of the harmonic period of the record was increased by a factor of 2.5, giving a period of 0.5 secs to the sine curve. The length of the pulse of constant acceleration was increased to 0.6 seconds, but the amplitude was not altered. The maximum incremental velocity is therefore increased to 71.7 in/sec.

The period of the first two derived records, T = 0.2 secs, was relatively close to the second natural period of the frame (0.17 secs). The modified time scale in the third artificial record brought the period of harmonic acceleration closer to that of the structure's first mode where T = 0.57 secs.

4. METHOD OF ANALYSIS

The model structure was analysed using the DRAIN-2D computer program, a full description of which is found in Reference 1. The structure is idealized as an assemblage of beam-column elements and the equations of motion are solved by step-by-step integration using the direct stiffness method. The program allows shear deformations, $P - \Delta$ effects, rigid end blocks and gravity load cases. Yield capacities are specified as positive and negative moment capacities for beam elements, and for column elements a trilinear interaction diagram is specified. The moment-rotation relationship is bilinear, allowing provision for a strain-hardening branch after yielding. However, stiffness degradation effects cannot be modelled. For the study reported here a strain hardening modulus of 3% of the original stiffness was specified. Damping was specified at 5% of critical for each of the first two modes. Higher mode damping proportions are calculated by the program. The yield capacities input were based on a probable yield strength of $1.1f_y$ and a capacity reduction factor $\phi = 1.0$.

The program outputs displacements and forces for specified member nodes at specified time steps, together with an envelope of all maxima at the end of the analyses. The amount of inelastic deformation in each member is given by plastic rotations at hinge positions, which are permitted at member ends only.

5. RESULTS

A feature of the time-history type of dynamic analysis is the amount of output generated and the difficulties encountered in reducing this output to a manageable number of parameters which sufficiently describe the response of a structure to a given earthquake record. The ideal would be to produce a set of parameters enabling the response of a structure under any conditions to be compared with the response under standardized conditions. In this study the conditions varied were the input acceleration records and the input column strengths. The output parameters studied here fall into three categories: (1) overall displacements, (2) extent and degree of inelasticity and (3) forces on members.

In this report all parameters are described fully, with no attempt made to reduce them to a simple set.

5.1 Building Displacements

The top floor displacement history for each earthquake record is shown in Figures 2 to 6. The top floor displacement under code loading, using the same moment of inertia, was 0.93". In general the maximum top floor displacements were reduced slightly when reduced column strengths were used although as expected, the maximum interstorey displacements were increased by a proportionately greater amount with reduced column strengths (see Table 1). This is because more extensive column hinging restricts the amount of base shear input to the structure, while inelastic displacements increase the maximum deflections in some storeys. The long duration acceleration pulses caused large top floor displacements, and in the case of the Pacoima Dam record the building took a permanent set of 4" after the relatively long pulses at 2.5 secs. For records 3 and 4, in which the period of harmonic motion is 0.2 secs, top displacement is considerably less than for record 5, where the harmonic motion has a period of 0.5 secs. This reflects the structure responding largely in second mode to records 3 and 4, and in first mode to record 5, as shown by the displaced shape of the structure. For the harmonic motion with T = 0.2 secs. the displaced shape is such that at maximum top floor displacement the 3rd floor displacement is of opposite sign to that at the top floor. When the harmonic motion has a period of 0.5 secs the 3rd floor displacement is typically 60% of top floor displacement, and of the same sign.

A parameter which has been used to describe a structure's overall response is the overall displacement ductility, defined as the maximum top floor displacement in a structure divided by the top floor displacement when yield first occurs in the structure. Table 1 shows this value calculated for each analysis. The overall displacement ductility varies widely and values appear to be at variance with relative response as determined by other parameters. This difference occurs because first yield and maximum top displacement occur under conditions of different displaced shapes, indicating some higher mode response. This parameter thus proved to be a very unreliable indicator of the relative response caused by different records.

Maximum interstorey deflection of .018h, (for the case of reduced column capacities) was 11 times the interstorey deflection of .0016h occuring under code loading. NZS 4203 would require $\frac{2.0CI}{Cd}$ = .0045h, and this is exceeded in all records, supporting the commentary of NZS 4203 C3.8.1.2.

5.2 Base Shear

The maximum base shear on the structure was equivalent to C_d = 0.44 for full column capacities and C_d = 0.40 for reduced column capacities. This is over twice the base shear for which the structure was designed. There are two basic causes for this increased shear force, namely overstrength of the structure, and an actual shear distribution different from that of the code loading.

5.2.1 Sources of Overstrength

Overstrength is built into the structure both by design methods and by approximations used in the model for dynamic analysis. The most significant sources of overstrength are:

1. The beam strengths used for the dynamic analysis are 1.1/0.9 times the reliable strengths used in design. Rounding up of bar sizes gives a total of about 25% over-strength from these sources.

2. For upper floors, minimum steel requirements govern in at least one face of the beam. Also, in floors 2 and 4 the beam steel areas are set equal to the areas required in floors 1 and 3 respectively.

3. Strain hardening, taken as 3%, adds 12% to flexural strength at a ductility ratio of 5.

4. Gravity loads used in the analysis are constant at D + 0.33L compared with 0.9D and 1.0D + 1.3L_R used for design. This adds about 6% to the capacity available to resist earthquake loading.

5. Beam design was based on centreline moments, a design approximation, whereas the dynamic analysis incorporates rigid end blocks and therefore uses column face moments. This effectively gives an overstrength of about 12%.

The cumulative effect of these factors is an overstrength ranging from 1.7 in the lower storey beams to 2.2 in the top storeys. It should be noted that apart from item 5 above these factors causing overstrength would be present in "real" structures, so that overstrength factors of this order are not unrealistic.

Assuming an effective overstrength of say 1.8, and a shear distribution similar to that caused by code loading, the maximum expected value of base shear would be equivalent to C_d = 0.156 x 1.8 = 0.28, considerably less than the maximum encountered in the dynamic analyses. The extra shear is introduced into the structure by a distribution of shear forces up the height of the structure differing from the code distribution.

5.2.2 Distribution of Shear Forces

Figure 12 shows the building forces and displacements at the time step at which maximum base shear occurred during the Pacoima Dam analysis. Also shown on the diagram are the shear forces calculated from NZS 4203, normalized to have the same value at the first floor level. The shear distribution tends to be of triangular shape, rather than the parabolic distribution derived from code loading. Therefore, when the individual storey shears are extracted (Figure 12b) seismic forces tend to be equal at each level, rather than increasing up the structure as prescribed by code loading. The effect of this is that whereas base shear is 2.5 times code shear, 5th storey shear is only 1.66 times code shear. This effect appears to be consistent through all earthquake records used.

At the time step illustrated in Figure 12, all beams had hinged, together with the column bases and the tops of the 4th storey columns. Once the column bases have hinged, extra shear can be introduced into the structure only by an alteration of the shape of the bending moment diagram, as shown in Figure 12d. As shear increases, with no increase in the base moment and with beam inputs remaining essentially constant, the column bending moments move out to one side of the baseline, until yield capacity is reached at a column above the base, in this case below the fourth floor level. In addition, the moment at the top of the six storey column is constrained to the roof beam yield capacity so that shears in the upper storeys cannot increase.

When reduced column capacities are used, column hinging above the base occurs at an earlier stage, inhibiting the changes in the bending moment diagram to some degree. The base shear is reduced by up to 9%, with the greatest reductions occurring in records producing the greatest base shear.

5.3 Extent of Hinging

Periods in each analysis in which hinging occurred are marked by heavier lines on Figures 2 to 6. Also marked are the number of beam hinges (maximum possible 36) and the number of column hinges during each period of inelasticity. Table 2 gives some parameters describing the extent of hinging.

When full column strengths were used, all records containing long duration pulses formed collapse type mechanisms with all beams hinging plus four column base hinges. When the column strength was reduced, a form of storey hinging mechanism occurred, involving a block of 2 or more storeys with column hinges at the top and bottom of the block, and hinging in all beams between these column hinges.

The maximum duration of continuous beam hinging was 0.46 secs, and of column hinging 0.37 secs. The maximum duration of column hinges increased with reduced column strength, as expected, but the total periods of hinging were similar for both series of analyses. Of more importance than the extent of hinging is the degree of inelasticity, and this is studied in the following section.

5.4 Ductility Requirements

5.4.1 Beam Ductility

The maximum plastic rotations and

ductility demands in each floor are listed in Table 1, and the ductilities are also plotted in Figure 11. Both the magnitude and the distribution of ductilities varies with the acceleration record used; however, some trends are apparent. Using full column capacities the maximum ductility demand occurs at the 5th floor, with a relatively high demand at the 6th floor. When the column capacities are reduced the maximum value still occurs in the fifth floor for the 'natural' records but for the 'artificial' records the maximum occurs at lower floors.

The Pacoima Dam analysis shows a greater disparity in demand at 5th floor level than any other analysis. In Figure 14 the column moment distribution and the shear distribution are plotted at time T = 7.81 seconds for the Pacoima Dam analysis with full column capacity. At this time step only the top 2 floors were hinging, but the ductility demands at these floors were the maximum encountered in any analysis. In this case, the higher ductility demands at these floors were caused by a shear distribution greatly different from that used in design.

The distribution of the maximum plastic rotations in the beams at each floor level, given in Table 1, differs from the distribution of ductilities. In general, plastic rotations decrease up the building, particularly when reduced column strengths are used. As the strength of the beams, and therefore the yield rotation, decreased at a greater rate than the plastic rotations, the ductility demand increased.

If the aim of the design process is to achieve uniform ductility demand at each level then the coefficient of variation of beam ductilities gives an approximate measure of the degree to which this has been achieved. Although there are some reservations about the statistical validity of this parameter in this context it nevertheless gives some measure of the relative spread of ductilities. It also serves to illustrate the difficulties in achieving uniform ductility, as the values of the coefficient of variation vary widely, so that the aim may be satisfied for a particular record but not for a different record. Moreover, uniform ductility would not necessarily give an optimum design. For constant size beams with reducing reinforcement ratio - a design requirement typical of many framed structures - the members capability of undergoing plastic deformations increases at higher levels. Therefore higher ductility demands could be accommodated in upper storeys.

Figure 13 shows the moment rotation history for a hinge in a 5th floor beam during the Pacoima Dam response. The maximum ductility factor reached was 13.6, but as Figure 13 shows this is a very inadequate description of the inelasticity that occurred.

The long acceleration pulse caused a large rotation which was never recovered in subsequent cycles of hinging, and the ductility factors following the long pulse were effectively only ± 5 about an imaginary new zero rotation baseline.

5.4.2 Column Ductilities

Column ductilities are listed in Table 1, and also shown in Figure 10 for the case of reduced column capacities. The ductilities shown in Figure 10 generally occurred simultaneously under the action of the long acceleration pulse. Actual variations in column strength (changes in ϕ and f_y) will affect the pattern of demand.

When full column capacities were used column hinging occurred at the base during all analyses and also below 4th floor level in the Pacoima Dam analysis, despite the fact that the columns were designed for biaxial loading and so had considerable extra capability for uniaxial attack. This indicates that despite adoption of "capacity design" procedures there can be no guarantee of avoidance of column hinging. The varying shear distributions described in Section 5.2 caused much of the hinging to be concentrated at 4th floor level when reduced column capacities were used, with the ductility demands at this level reaching relatively high values.

The program includes P - Δ effects and so any instability caused by large plastic rotations would be shown up in the interstorey deflections. The deflections listed in Table 1 show no evidence of any such instability. The maximum degree of column hinging occurred at 4th floor level where axial load levels are low. At this level the design biaxial load reached only 5% of the compressive capability of the section, and at this level of load a confined section could undergo considerable plastic rotations.

5.5 Column Actions

5.5.1 Bending Moments

Figures 7 and 8 show the maximum bending moment envelopes, for an exterior and interior column respectively, from the five records analysed. For comparison, the envelope from El Centro 1940 N-S times 1.30 is also shown. The dashed lines show the equivalent uniaxial moments used for design, i.e. 1.5 times code loading moments, as detailed in section 2.1. The column capacity shown is based on the uniaxial capacity of the columns as detailed, at the approximate axial load acting at the time of maximum moment. The envelopes are derived from the analyses using full column strength.

Column capacity was exceeded only at the column bases and below the fourth floor level. The shear distribution in section 5.2 is evident in the shape of the bending moment envelopes. The ratios of the maximum moments reached in the dynamic analysis to the moments arising from code loading are listed in Table 3, showing an average ratio of 3.4 times code loading, with lowest values at floor levels 2, 3 and 4.

5.5.2 Shear Forces

The shear force envelopes for both full and reduced column strengths for the exterior column are shown in Figure 9. For comparison the values from the El Centro 1940 analysis are also shown. At the time of writing the question of the design shear has not been resolved by the N.Z.N.S.E.E. committee. However, one proposal is that a design shear force of $1.8M_{des}/h$ be used, where M_{des} is the design moment and h is the interstorey height. Accordingly, a uniaxial envelope of this value is also plotted on Figure 9. This value is exceeded in the lower four storeys, by amounts of 53%, 55% and 9% respectively. This is to be expected from the shear distributions discussed in section 5.2. The maximum amplification in shear does not coincide with the maximum moment amplification shown in Table 3. In fact, the moment increase is a minimum in storeys 2 and 3 where the shear increase is a maximum. This confirms that the large column moments at fourth floor level are caused by cumulative effects on the bending moments by increasing shear in the lower storeys, rather than increased shear in the fourth storey itself.

The use of reduced column capacities did not produce a uniform effect on the column shear forces: the shear in the second storey remained the same, while that in other storeys decreased by values ranging from 0 to 10%. However, the reduced column capacities assume a moment input, and hence shear, in the orthogonal direction so this does not imply a reduction in biaxial shear.

Table 4 relates the maximum shears attained in the analysis to the column strengths. The factor V x h divided by $M_{capacity}$ expresses the relative magnitude of column moments. A value of 1.0 is equivalent to hinging at one end of the column, and a value of 2.0 represents hinging at each end of a column, i.e. the maximum possible shear. The values are approximate as the axial load levels at time of maximum shear varied over the full range. Therefore, the two values shown represents the upper and lower bounds.

6. EFFECT OF LONG DURATION PULSES

The most important effect of the long duration acceleration pulses was the permanent displacement imparted to the structure. Plastic displacements were "locked in" at member ends, resulting in a permanent displacement of over four inches at the top floor in the case of the Pacoima Dam record. Although the maximum velocity increment was of similar magnitude for the four records containing long duration pulses, the maximum top displacement varied considerably. When the pulse was preceded by harmonic motion of period 0.2 secs, the maximum incremental top displacement was five inches. However, when the period of the harmonic motion was increased to 0.5 secs the corresponding incremental displacement was eleven inches. The respective values for the Pacoima Dam record, and the second pulse of the two pulse record were twelve inches and thirteen inches.

Not all maximum values recorded in Table 1 occurred under the action of the long duration acceleration pulse. For the Pacoima Dam record, the maximum ductility requirements in the upper floors occurred at the time of maximum acceleration, when the shear distribution was different from the code loading distribution. However, in general, the overall response to the records containing long duration pulses was more severe than the response to El Centro 1940 N-S x 1.30, which contained maximum accelerations of greater magnitude but shorter duration.

7. EFFECT OF REDUCED COLUMN CAPACITY

The general effect of using reduced column capacities was (1) to reduce maximum top floor displacements slightly but increase maximum interstorey deflections (2) reduce the maximum base shear (3) reduce the maximum beam ductilities and change the distribution of ductilities and (4) increase the extent and degree of column hinging. The differences were not consistent for all records, and there was a marked difference in the effect on the distribution of beam ductilities between the 'natural' records and the 'artificial' records. The distribution of beam ductility, as measured by the coefficient of variation, became more regular for the 'natural' records when reduced column strengths were used, whereas for 'artificial' records the converse was true.

8. CONCLUSIONS

Care must be exercised in interpreting results from dynamic analyses, in that trends noted for a particular earthquake record, or for a particular structure, may not hold for other records or structures. The conclusions noted below are drawn for this particular six-storey frame, and caution must be taken in extending these to other structures.

(1) Acceleration pulses of long duration cause generally more severe effects than short pulses of equal or greater accelerations. The effect of the long duration pulse on a frame structure are influenced by the acceleration history preceding the pulse.

(2) Reduced column capacities have a considerable effect on the magnitude and distribution of beam ductilities. Structure deformations and forces are effected to a lesser extent.

(3) The maximum base shear introduced into the structure increased considerably above the design value. The formation of a hinging mechanism did not prevent additional shears being introduced into the structure. This was caused partly by overstrength present in the structure and partly by a redistribution of shear forces over the height of the structure. The redistribution of shear forces caused by large changes in the shape of the column bending moment diagrams.

(4) The maximum column moments reached a value up to four times the design values, with maximum increases at the base and upper floors. The moment increases were caused by cumulative effects of increased shear in the lower storeys rather than by shear increases in the upper storeys.

(5) Difficulties are encountered in quantifying relative responses. The overall ductility factor, related to top floor displacement, was shown to be an unreliable guide to relative response when the deflected shape at first yield is of different form from the deflected shape at maximum displacement. The maximum beam ductility factors at each floor level did not give a complete picture as they did

not reflect the number of cycles at that magnitude. A full description would therefore need to include the total number of inelastic cycles and the average ductility demand associated with these cycles, i.e. a weighted cumulative ductility factor.

In future dynamic analyses it is hoped to refine the output produced to a greater degree. In particular, data on the number of inelastic cycles is required, together with time periods for which moments and shear forces are greater than predetermined values. As the number of structures studied increases together with the number of associated earthquake records it is hoped to obtain sufficient data to refine factors used in the design of ductile frames.

9. ACKNOWLEDGEMENTS

The permission of the Commissioner of Works to publish this paper is acknowledged. The writer wishes to thank members of the design and draughting staff for their assistance. Special acknowledgement is made to Mr. B. W. Buchanan, Senior Design Engineer (Research and Development) and Dr. D. L. Hutchison, Senior Engineer, of the Ministry of Works and Development, Head Office, for their many helpful comments and suggestions, and Mr. O. A. Glogau, Chief Structural Engineer, who initiated this study.

10. REFERENCES

1. Kanaan, A. E., and Powell, G. H.: "DRAIN 2D: A General Purpose Computer Program for Dynamic Analysis of Inelastic Plate Structures", University of California, 1973.
2. Bertero, V. V.: "Identification of Research Needs for Improving the Aseismic Design of Building Structures". Bull. N.Z.N.S.E.E., Volume 9, No. 1, 1976.
3. NZS 4203 Code of Practice for General Structural Design and Design Loadings for Buildings. 1976.
4. Paulay, T.: Communication to Frames Committee of N.Z.N.S.E.E. June 1976.

TABLE 1.

SUMMARY OF RESULTS

		FULL COLUMN CAPACITY					REDUCED COLUMN CAPACITY				
		RECORD NUMBER*					RECORD NUMBER*				
		1	2	3	4	5	1	2	3	4	5
Max. top Displacement (ins)		3.69	7.9	4.84	7.84	9.03	3.48	7.48	4.91	6.98	8.67
Top Displacement At 1st Yield (ins)		1.63	1.8	0.67	0.67	1.62	1.63	1.8	0.67	0.67	1.62
Overall Displacement Ductility		2.3	4.4	7.2	11.7	5.6	2.1	4.2	7.3	10.4	5.4
Max. Interstorey Deflection, δ, ins.		0.86	1.46	1.08	1.08	1.78	1.84	1.73	1.24	1.90	2.42
δ/h		(.006)	(.011)	(.008)	(.008)	(.013)	(.006)	(.013)	(.009)	(.014)	(.018)
Max. Base shear (kips)		374	439	418	435	434	369	404	396	403	405
$\div W_t$ (W_t = 1001 kips)		.374	.439	.418	.435	.434	.369	.404	.396	.403	.405
Max Beam Ductilities	Floor 1	2.2	4.2	3.0	5.0	5.3	2.2	4.9	3.5	5.3	6.7
	2	2.5	4.6	3.0	4.9	5.3	2.3	4.5	3.2	5.1	6.3
	3	2.9	6.3	3.5	5.6	6.5	3.2	5.6	3.8	5.7	6.9
	4	2.4	5.8	2.4	4.1	5.3	1.7	3.8	1.6	1.6	1.9
	5	5.3	13.6	4.2	7.5	10.5	3.5	8.6	2.6	2.8	3.7
	6	4.2	10.6	3.3	5.3	7.7	2.9	6.7	2.3	2.3	3.0
Coefficient of Variation		38%	50%	19%	21%	31%	26%	31%	29%	47%	45%
Max Beam Plastic Rotations (rads. x 10^5)	Floor 1	361	897	563	1124	1177	369	1096	676	1210	1544
	2	430	1005	566	1081	1185	396	995	615	1141	1451
	3	398	1102	509	942	1116	446	948	560	969	1178
	4	291	881	292	653	866	148	579	120	150	196
	5	367	976	274	561	816	217	657	138	159	231
	6	437	1085	319	558	849	283	726	209	209	294
Column Ductilities	Ground	1.5	3.8	2.3	4.4	4.3	1.8	5.3	3.8	6.0	7.6
	Floor 1	-	-	-	-	-	1.4	2.0	-	-	-
	2	-	-	-	-	-	-	2.7	1.3	1.8	1.5
	3	-	-	-	-	-	1.0	2.5	1.4	2.5	2.9
	4	-	1.1	-	-	-	3.0	6.6	3.9	8.6	10.2
	5	-	-	-	-	-	-	1.0	-	-	-
	6	-	-	-	-	-	-	-	-	-	-

* RECORD 1: El Centro 1940 N.S. x 1.30

" 2: Pacoima Dam

" 3: Harmonic (T = 0.2 secs) with Pulse

" 4: Harmonic (T = 0.2 secs) with 2 Pulses

" 5: Harmonic (T = 0.5 secs) with Pulse

TABLE 2.

EXTENT OF HINGING

	FULL COLUMN CAPACITY					REDUCED COLUMN CAPACITY				
	RECORD NUMBER*					RECORD NUMBER*				
	1	2	3	4	5	1	2	3	4	5
No. of Periods of Hinging	9	21	6	7	9	9	21	6	7	9
Max. Period of Hinging (Secs)	0.22	0.45	0.33	0.44	0.37	0.21	0.46	0.33	0.43	0.38
Total Duration of Hinging (Secs)	1.29	3.17	0.49	0.93	1.22	1.21	3.30	0.50	0.92	1.22
Length of Record (Secs)	10.0	10.0	1.30	1.80	2.62	10.0	10.0	1.30	1.80	2.62
% of Time Inelastic	13%	32%	38%	52%	46%	12%	33%	38%	51%	46%
Max. No. of Simultaneous Beam Hinges	36	36	36	36	36	35	36	24	32	36
Max. No. of Simultaneous Col. Hinges	3	6	4	4	4	10	16	10	15	15
Max. Duration of Column Hinging (Secs)	0.13	0.26	0.13	0.26	0.33	0.14	0.37	0.26	0.33	0.35

* Record 1: El Centro 1940 N-S x 1.30
" 2: Pacoima Dam
" 3: Harmonic (T = 0.2 secs) with Pulse
" 4: Harmonic (T = 0.2 secs) with Two Pulses
" 5: Harmonic (T = 0.5 secs) with Pulse

TABLE 3.

RATIO OF MAXIMUM COLUMN MOMENTS FROM DYNAMIC ANALYSES TO CODE LOADING MOMENTS

LEVEL	MAX. MOMENT FROM ANALYSIS / MOMENT FROM CODE LOADING	
	EXTERIOR COLUMN	INTERIOR COLUMN
GROUND	3.9	3.5
Floor 1	3.2	2.4
" 2	2.9	2.5
" 3	3.5	2.9
" 4	4.3	3.2
" 5	3.9	4.7
" 6	3.0	4.2

TABLE 4.

EXTERIOR COLUMN: MAXIMUM SHEAR FORCES RELATED TO COLUMN STRENGTHS

STOREY	ORIGINAL COLUMN CAPACITY					REDUCED COLUMN CAPACITY				
	V_{max} x h	$M_{capacity}$		V x h/M_{cap}		V_{max} x h	$M_{capacity}$		V x h/Mcap	
		at P_{max}	at P_{min}	at P_{max}	at P_{min}		at P_{max}	at P_{min}	at P_{max}	at P_{min}
1	15480	13400	10000	1.16	1.55	14688	10720	8000	1.37	1.84
2	7992	9800	6800	0.82	1.18	7992	6860	4760	1.17	1.68
3	7344	8800	7000	0.83	1.05	6480	6160	4900	1.05	1.32
4	5616	8000	7200	0.70	0.78	6048	5600	5040	1.08	1.20
5	3888	6400	5200	0.61	0.75	3888	4480	3640	0.87	1.07
6	2376	5800	4800	0.41	0.50	2376	4060	3360	0.59	0.71

NOTES:

V_{max} = Maximum column shear from dynamic analyses

h = Clear interstorey height

$M_{capacity}$ = column capacity as designed

P_{max}, P_{min} = Maximum and minimum axial loads, respectively, from the dynamic analyses

MEMBER	SIZES
Columns	30" x 30"
Beams	
Floors 1-5	24" x 18"
Floor 6	18" x 12"

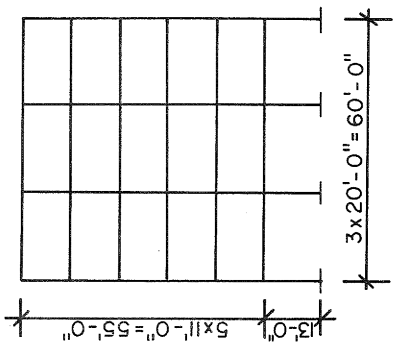


FIGURE 1: STRUCTURE ANALYSED

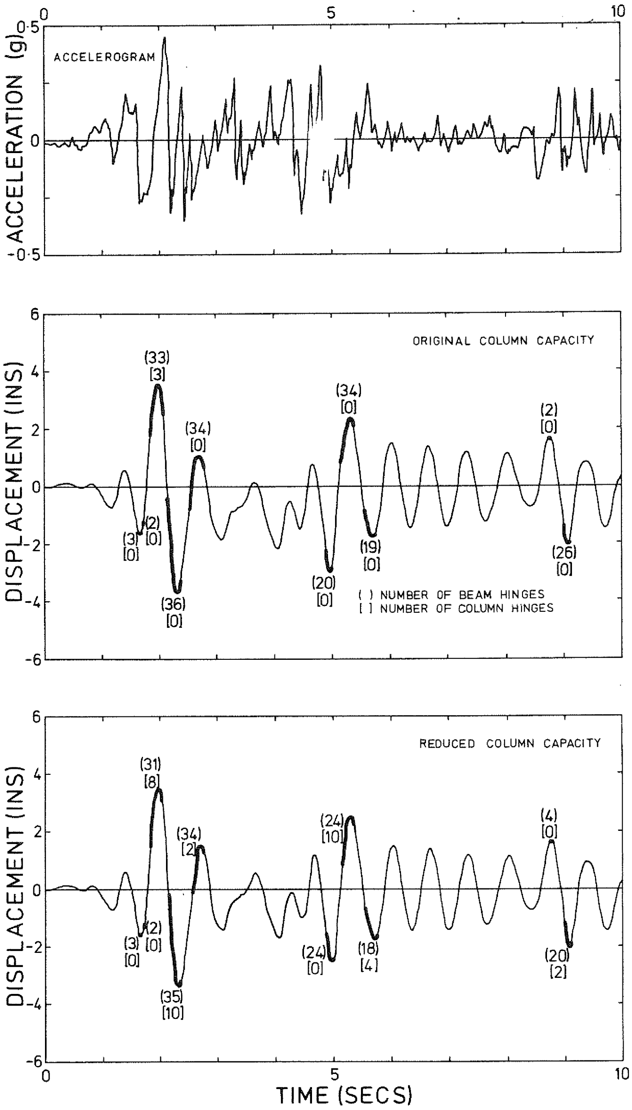


FIGURE 2: RESPONSE TO EL CENTRO 1940 N-S COMPONENT x 1.30

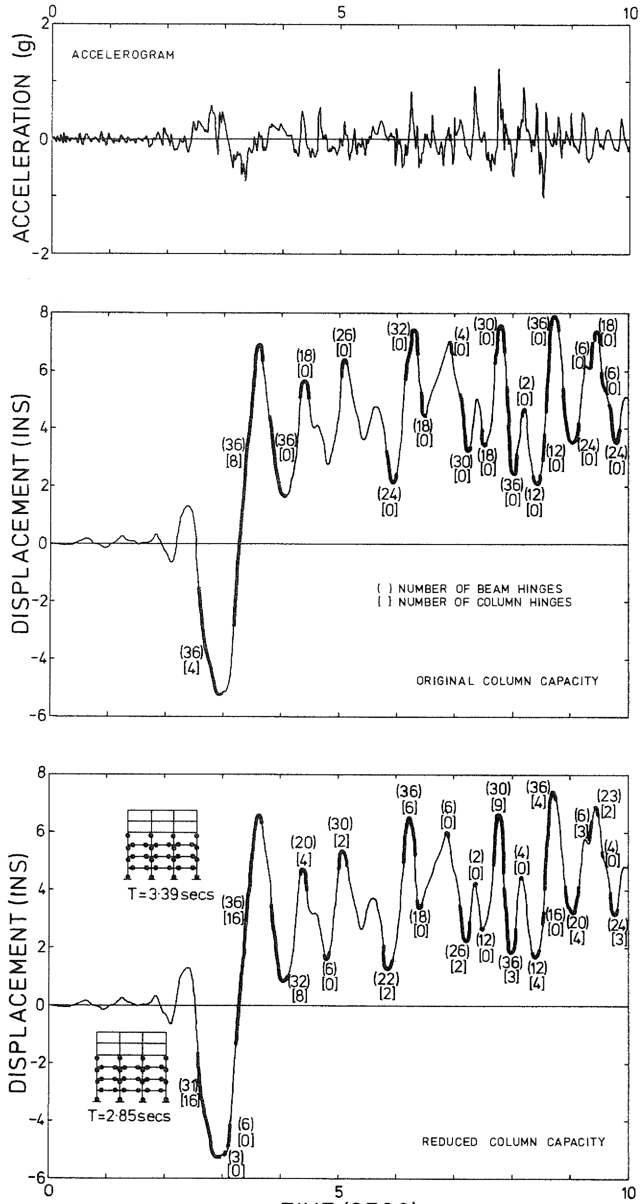


FIGURE 3: RESPONSE TO PACOIMA DAM RECORD

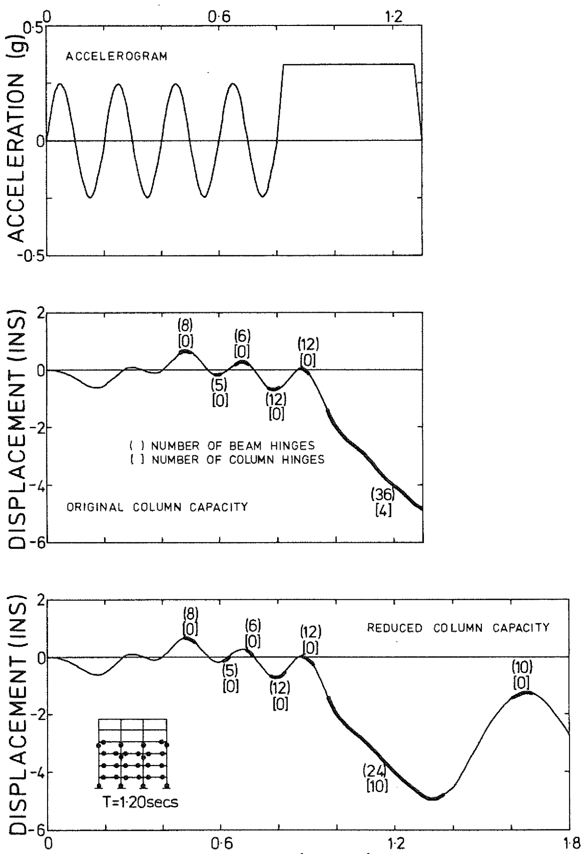


FIGURE 4: RESPONSE TO HARMONIC MOTION (T = 0.2 SECS) WITH 1 PULSE

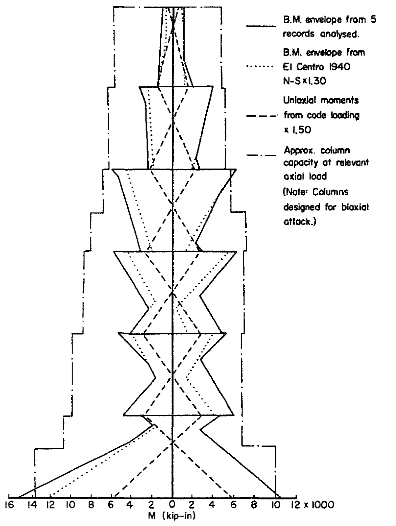


FIGURE 7: EXTERIOR COLUMN. BENDING MOMENT ENVELOPE FROM DYNAMIC ANALYSES

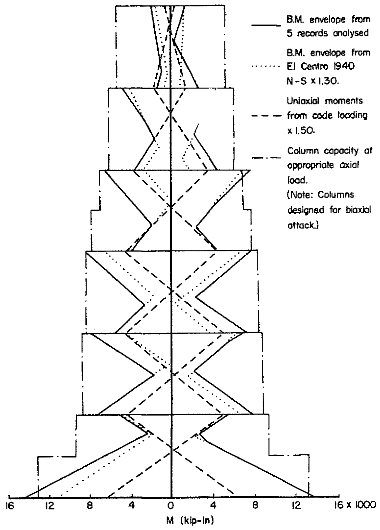


FIGURE 8: INTERIOR COLUMN. BENDING MOMENT ENVELOPE FROM DYNAMIC ANALYSES

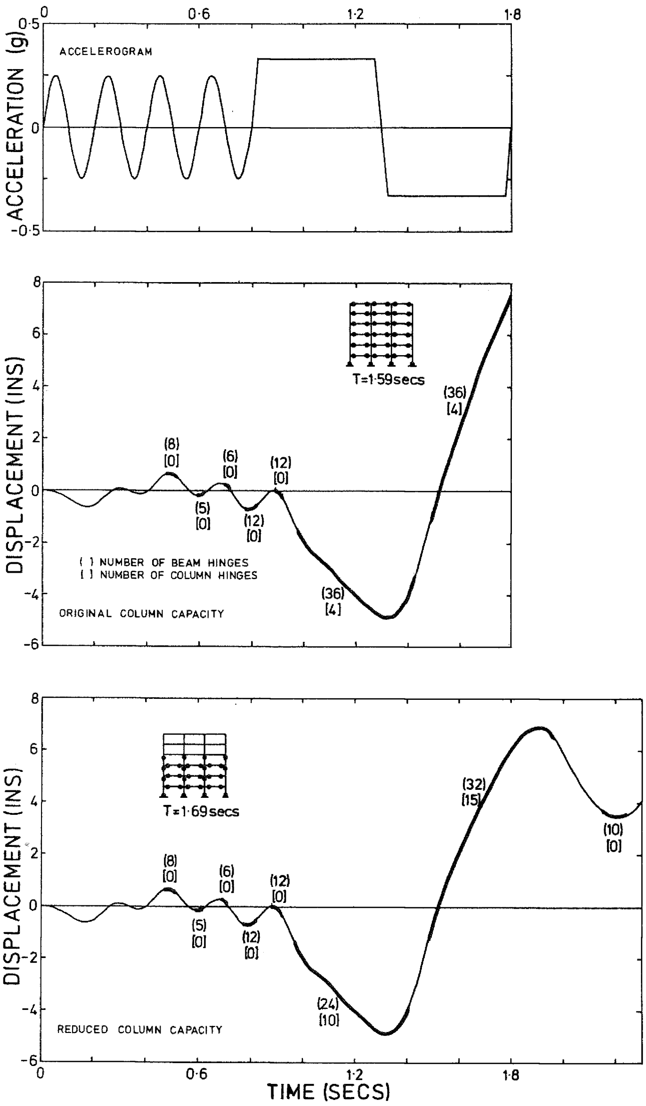


FIGURE 5: RESPONSE TO HARMONIC MOTION (T = 0.2 SECS) WITH 2 PULSES

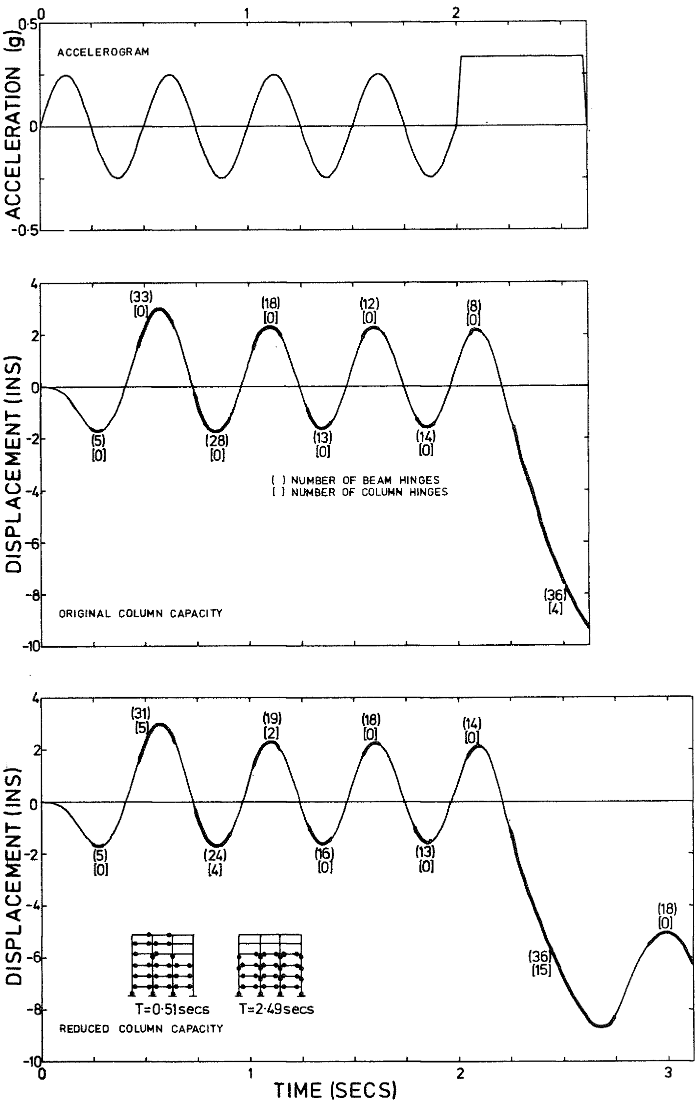


FIGURE 6: RESPONSE TO HARMONIC MOTION (T = 0.5 SECS) WITH 1 PULSE

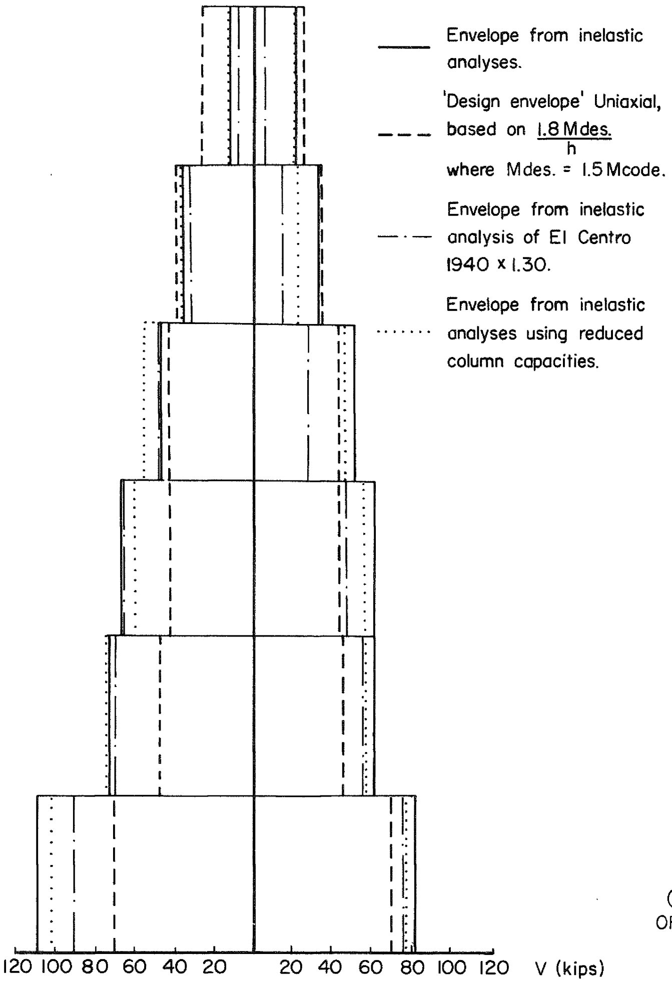


FIGURE 9: EXTERIOR COLUMN:
SHEAR FORCE ENVELOPE FROM DYNAMIC ANALYSES

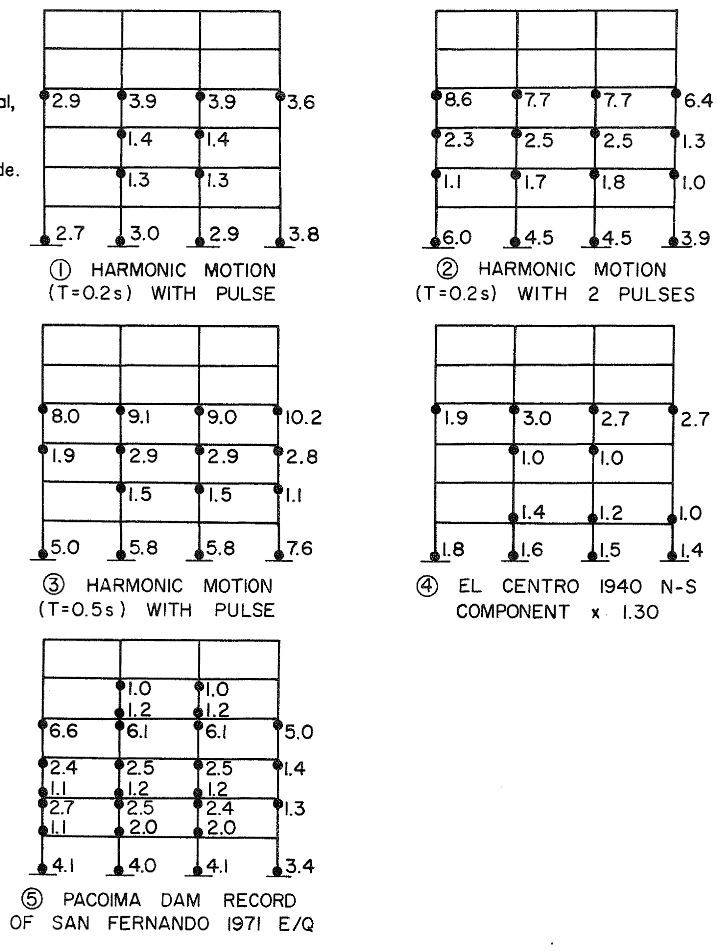


FIGURE 10: EXTENT OF COLUMN HINGING AND MAXIMUM DUCTILITY
DEMANDS USING REDUCED COLUMN CAPACITIES

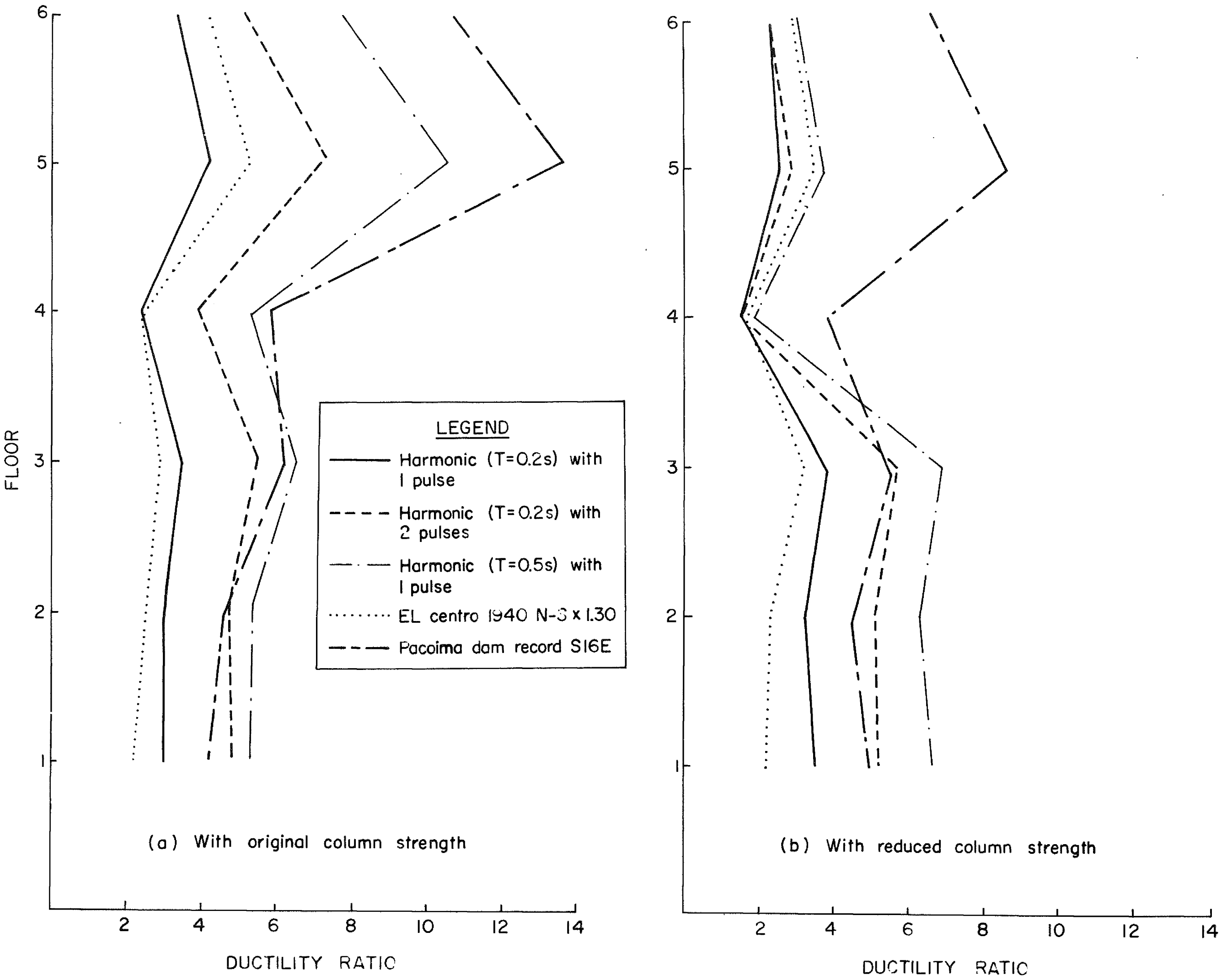


FIGURE 11: BEAM DUCTILITY REQUIREMENTS

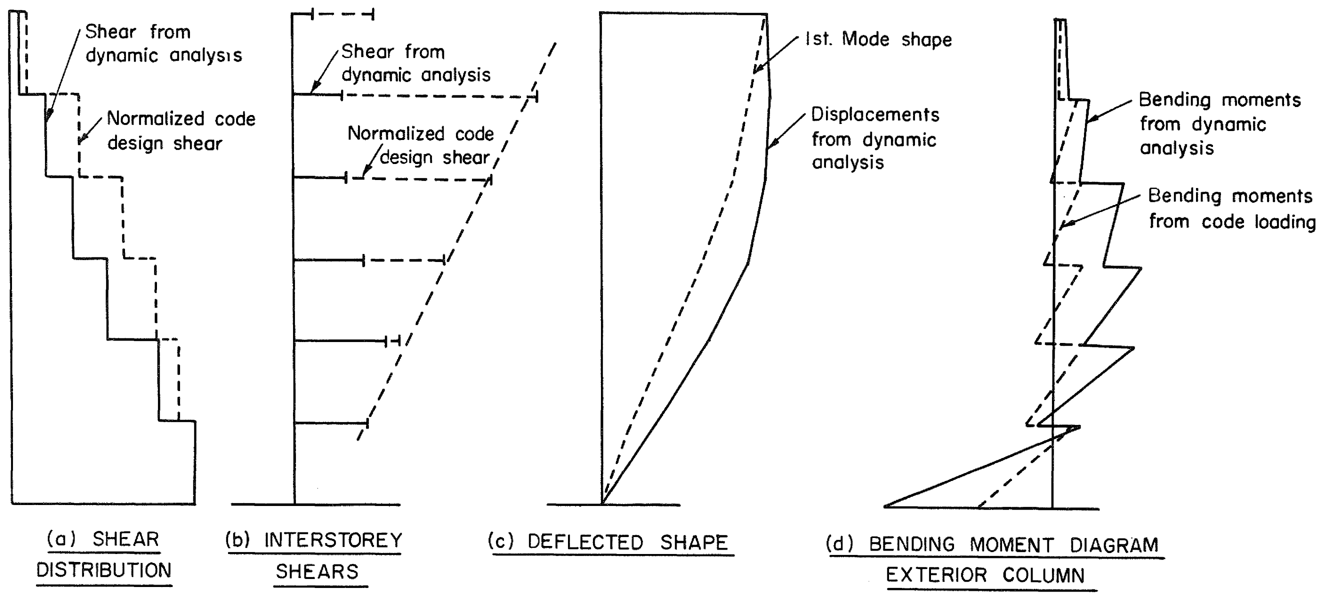


FIGURE 12: PACOIMA DAM RECORD-FORCES AT TIME OF MAXIMUM BASE SHEAR (T = 3.39 SECS)

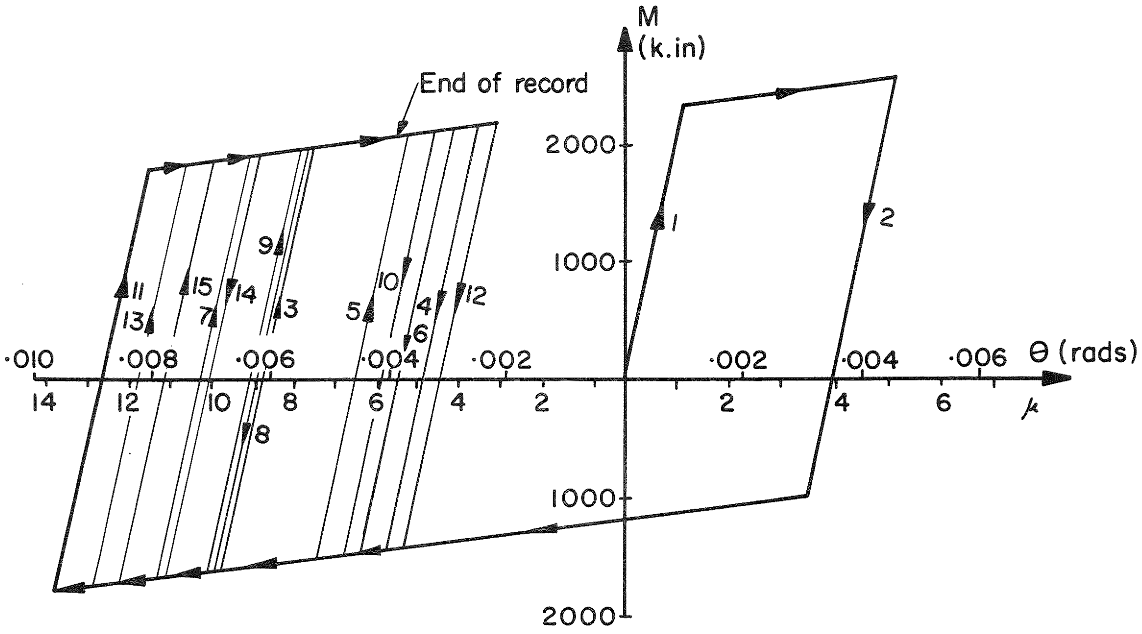
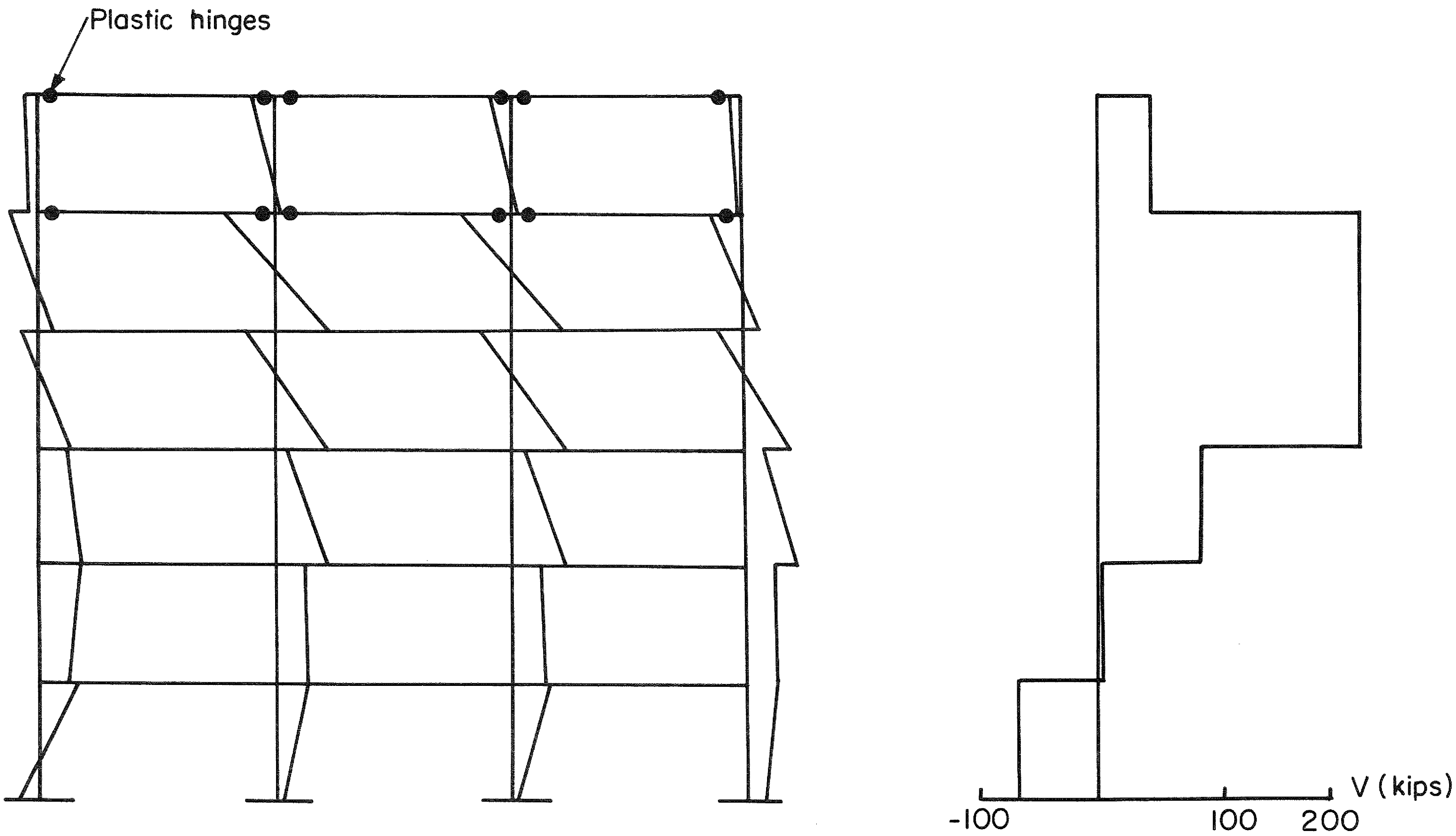


FIGURE 13: 5TH FLOOR PLASTIC HINGE ROTATION. - PACOIMA DAM RECORD



(a) Column moment distribution

(b) Shear distribution

FIGURE 14: PACOIMA DAM RECORD-FORCES AT TIME OF MAXIMUM DUCTILITY DEMAND IN UPPER FLOORS (T = 7.81 SECS)