# THE SEISMIC RESPONSE OF INELASTIC STRUCTURES

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## ABSTRACT

A thirteen storey, two bay, reinforced concrete framed structure is subjected to a series of non-linear, dynamic analyses in an attempt to find some correlation between the damaging potential of various digitised earthquakes and their relative strengths which have been computed in a variety of ways. Much of the previous work in this field has been with respect to simple one degree of freedom systems and these do not appear to give any indication of the correlation that could be expected for a non-linear multi-degree of freedom structure.

The results show the effects of the different scalings of the various earthquakes and compare these with those obtained for the familiar North-South component of the May 18, 1940 El Centro earthquake. These results highlight the difficulty of trying to relate the use of such a dynamic earthquake analysis to the present pseudo-static code requirements. Further, the results of the analyses show also the great difference between the present assumption of the plastic hinge distributions, used in the ultimate seismic design method, and those observed during the earthquake excitation with the consequences on the lower-floor column axial loads.

## INTRODUCTION

Although there has been a variety of both actually recorded and artificially produced digitised earthquake accelerograms available for a number of years, the North-South component of the May 18, 1940 El Centro earthquake has been the most frequently used for dynamic analyses. This has been largely due to the lack of knowledge of the comparative strengths and effects of the other records available and of any way of interrelating these various records. Further, the relationships between building code requirements for seismic design and the ability of a particular frame to withstand such a digitised earthquake have always been vague because of the uncertainty as to the criteria with which to gauge the strength of an earthquake in some absolute terms. For example, measurement of the maximum ground acceleration is not, by itself, a true indication of the size of an earthquake unless the duration of this acceleration can be incorporated into the assessment. Jennings (1), and others, have electronically generated artificial earthquake records to meet predetermined criteria for spectral response so that they are characteristic of the type of motion that might be expected at various distances from an epicentre. Penzien and Liu (2) subjected a non-linear single degree of freedom model to a wide range of these artificial records and attained a good correlation between the peak response for similar earthquakes. However, this gives little indication of the extent of correlation that could be expected for multi-degree of freedom systems in which varying amounts of inelastic structural behaviour may occur.

It was, therefore, desirable that an attempt should be made to gain an appreciation of what the principal earthquake parameter was in the amount of damage sustained by a tall framed structure when it was subjected to the various earthquakes.

## THE NON-LINEAR DYNAMIC ANALYSIS METHOD

The analyses were carried out using a non-linear dynamic plane frame analysis program developed by Sharpe (3). The time-wise solution of the equations of motion is achieved in a step by step manner using Newmark's Constant Acceleration method (4) as the authors have shown this to be an accurate and economical method (5) for such analyses. The beam and column members use elasto-plastic, bi-linear elastic or Ramberg-Osgood type functions to describe the moment curvature relationships at the plastic hinges that may occur at their critical sections, usually near the ends of the members. Shear deformations and rigid end blocks may also be allowed for in the members. The column members may further use a pure moment criterion or an axial-force moment interaction relationship to control the formation of plastic hinges and a program option is to exclude the formation of column hinges which, if applicable, results in a reduction of computational effort of approximately ten per cent.

Viscous damping ratios may be specified for two frequencies and the damping matrix is assumed to be of the form of a linear combination of the mass and stiffness matrices.

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The program allows for the use of a lumped or consistent mass matrix and for these analyses the former formulation was chosen.

Besides obtaining the tabulated values of the desired deflections, member moments and axial forces at specified time step intervals, a pictorial representation of the frame, showing the distribution of all sections exhibiting inelastic behaviour (plastic hinges, axial failure, etc.), is produced at the end of any time step during which a change in the distribution of such effects has occurred.

## THE STRUCTURE CHOSEN FOR THE ANALYSES

Because of the exceedingly large amounts of computational effort required for non-linear dynamic analyses of even simple plane frame structures it was necessary, in this attempt to compare and scale earthquake records, to limit severely the types of structures and range of earthquakes that could be considered. An expedient compromise between the magnitude of the problem and the resources available was the choice of a tall slender frame of a type likely to be built in New Zealand.

A thirteen storey, two bay frame (see appendix) was used for this study as the relatively large number of members would ensure a diverse distribution of inelastic effects, avoiding the all or nothing character of a one-degree of freedom system. This frame, designed to the load factors of Chapter 8, NZS 1900 (6), was considered to have elasto-plastic moment-curvature relationships at the critical sections in the beam members with no hinges occurring in the column members. Trial analyses, using the North-South and vertical components of the May 18, 1940 El Centro accelerogram, and a moment-axial force interaction relationship, indicated that the likelihood of column hinges occurring in the frame was very small, and that their very transitory nature would have very little effect on the response of the structure. This was confirmed when, by repeating the analysis, but with column hinges excluded, it was found that a four per cent increase in the strength of the critical column sections would automatically eliminate such column hinges.

Because of the diverse lengths of the various earthquake records available, and the cost of each analysis, only the first ten seconds of each record was used in the analyses.

## EARTHQUAKE SCALING

Various criteria for scaling of earthquake records are easy to evolve. They range in complexity from one derived from a simple scaling of the maximum ground acceleration to that based on the integral of the spectral velocity over a chosen period range (7). To gain an appreciation of the significance and range of these various scaling factors a large number of spectral responses were calculated. Table I lists the factors by which each acceleration component needs to be multiplied for it to be comparable with the North-South component of the May 18, 1940 El Centro record. The percentages of critical viscous damping used encompass the extremes likely in practice. It can be seen that there is no obvious correlation between the scale factors and the method by which they were derived, though the maximum spectral acceleration ratios stand out as being the most inconsistent. Even increasing the damping ratios, which may be expected to lead to an asymptotic behaviour of the scaling factors, has not reduced the variability of the earthquake scaling ratios. Part of the variations in these ratios will be due, however, to the choice of the control earthquake and it would be useful in the future to repeat the comparisons with respect to a different control assumption.

Some general comparisons may, however, be made. For example, the artificial earthquakes C1 and C2 are apparently of the order of one-fifth as intense as the control earthquake regardless of the method of scaling used.

## THE MEASURE OF EARTHQUAKE STRENGTH

A large part of the expense incurred in repairing a building following a major earthquake is spent on non-structural items. If, however, only structural repairs are considered it would appear reasonable that the cost of repair of a member would be qualitatively, a function of the size of the member and the maximum ductility ratio it has sustained. Other factors would be its accessibility and the number of excursions into the inelastic range.

The scheme selected to compare the damaging potential of several different earthquakes acting on the non-linear frame, must, however, at this stage be limited to that related to the measurement of member ductilities achieved.

## RESPONSES COMPARED WITH RESPECT TO MAXIMUM MEMBER DUCTILITIES

Of the earthquake records available four were chosen as being of a roughly similar type and were used for the comparative analyses. They were :-

(a) El Centro, December 30, 1934, North-South component,
(b) Taft, July 21, 1952, N69$^{o}$W component,
(c) Seattle, April 29, 1965, S58$^{o}$ component,
(d) Al - Jennings (1) artificial earthquake suite.

Although an inspection of the various means of scaling does not help in the decision as to whether any particular scaling methods are more applicable than others, it was decided to use two of the methods already discussed in order to gauge their success. The first type was based on the ratios of maximum ground accelerations and gave a wide range of factors for the four selected components whereas the second method was based on a comparison of the integral of the linear pseudo spectral velocity, with five percent damping, from a period of 0.1 seconds to 2.5 seconds. As a standard against which the results can be compared the North-South component of the May 18, 1940 El Centro accelerogram was used, its magnitude scaled linearly by factors of 0.7, 1.0 and 1.3.

The difficulty in quantitatively comparing responses becomes immediately clearer. Although the maximum top-storey displacement during analysis (a) was fifty percent higher than for that of the control earthquake, the resulting various ratios of corresponding member ductilities varied widely, from 0.642 to 1.74. Tables II and III give further comparisons between the various analyses. Table IV compares the three strengths of the control earthquake and shows clearly the threshold effects of the yield plateau in the moment-curvature relationships as it can be seen that the reduction in the magnitude of the ground motions does not necessarily result in a decrease in the ductility demands of every member. In this same case, the left-hand (outer) end of the twelfth-floor beam member reached an inelastic deflection sixteen percent greater in the analysis in which the excitation was factored by 0.7 than in that in which it was factored by 1.0.

An inspection of the variation of the hinge patterns in the structure, during the earthquake, disclosed that there was normally a three storey band of plastic hinges migrating up the structure. An example of this is shown in figure 2. Even when two such bands were present there was still a band, free of beam hinges, between them. It has been felt for some time that the current ultimate strength design philosophy, which requires the bottom storey columns of a frame to resist an overturning moment compatible with plastic hinges occurring concurrently in all the frame's beams, is overly conservative. It could be, therefore, that such inelastic analyses could be used to obtain a more realistic estimate of the ultimate column loads in tall building frames.

Figures 3, 4 and 5 show the maximum ductility demands at the outermost critical sections of the beam members. No simple trend is obvious though scaling by maximum ground acceleration has given less scatter than has scaling with respect to the integral of the spectral velocities. The corresponding maximum lateral displacement envelopes are shown in figures 6, 7 and 8 and also exhibit a seemingly unrelated scatter of magnitudes. Although a form of scaling appears possible in these latter figures it must be remembered that they are envelopes and as the displacements have contributions from higher modes, these figures give very little indication of the dynamic deformation and hence the ductility levels reached.

## CONCLUSIONS

Even if only a limited number of earthquake accelerograms are considered there does not seem to be a simple criterion by which the strengths of these excitations can be scaled to give equivalent damage. The problem besetting a designer trying to find a digitised earthquake that will match a required code strength specification has not been solved. This makes, at the present time, the use of non-linear dynamic analyses for seismic design purposes a very difficult and (as several different analyses will be required to satisfactorily cover the whole range of possible ductility demands) a very expensive process.

Recent developments, however, of special non-linear energy absorbing and vibration isolation devices (8,9) which may lead to the reduction of structural motion as well as limiting the non-linear effects to a few isolated points in the structure, together with the use of suitable substructure analysis methods (10,11) which then become feasible in such dynamic analyses, could largely reduce this problem by drastically lowering the cost of such computer analyses.

## ACKNOWLEDGEMENTS

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## APPENDIX A

### STRUCTURE I

A thirteen-storey, two-bay exterior frame derived from a North-East frame of the Jerningham Apartments, Wellington.

**Data**

| | | | |
|---|---|---|---|
| Elastic modulus | | $2.760 \times 10^{10}$ N m$^{-2}$ | |
| Shear modulus | | $1.200 \times 10^{10}$ N m$^{-2}$ | |
| Axial areas, Shear areas | - | exterior columns | |
| Levels | 0 - 4 | 0.6580 | 0.5550 m$^2$ |
| | 4 - 8 | 0.6070 | 0.5050 m$^2$ |
| | 8 - 13 | 0.5560 | 0.4540 m$^2$ |
| | - | interior columns | |
| Levels | 0 - 4 | 0.5660 | 0.4630 m$^2$ |
| | 4 - 8 | 0.5410 | 0.4380 m$^2$ |
| | 8 - 13 | 0.5150 | 0.4130 m$^2$ |
| | - | beams | |
| Levels | 1 - 4 | 0.4350 | 0.3600 m$^2$ |
| | 5 - 9 | 0.3800 | 0.3150 m$^2$ |
| | 10 - 13 | 0.3250 | 0.2700 m$^2$ |
| Moment of inertia | - | exterior columns | |
| Levels | 0 - 4 | $2.900 \times 10^{-2}$ m$^4$ | |
| | 4 - 8 | $2.720 \times 10^{-2}$ m$^4$ | |
| | 8 - 13 | $2.460 \times 10^{-2}$ m$^4$ | |
| | - | interior columns | |
| Levels | 0 - 4 | $4.320 \times 10^{-2}$ m$^4$ | |
| | 4 - 8 | $3.930 \times 10^{-2}$ m$^4$ | |
| | 8 - 13 | $3.770 \times 10^{-2}$ m$^4$ | |
| | - | beams | |
| Levels | 1 - 4 | $4.705 \times 10^{-2}$ m$^4$ | |
| | 5 - 9 | $4.370 \times 10^{-2}$ m$^4$ | |
| | 10 - 13 | $3.920 \times 10^{-2}$ m$^4$ | |
| Rigid end-blocks | - | columns | |
| | | 0.533 m | |
| | - | beams | |
| | | 0.410 m | |

Weight per unit beam length - $1.250 \times 10^5$ N m$^{-1}$

| | | |
|---|---|---|
| Plastic hinge lengths | - exterior columns | 0.320 m |
| | - interior columns | 0.420 m |
| | - beams | 1.06 m |

| | | |
|---|---|---|
| Natural frequencies | $\omega 1$ | - 1.076 Hz |
| | $\omega 2$ | - 1.906 Hz |

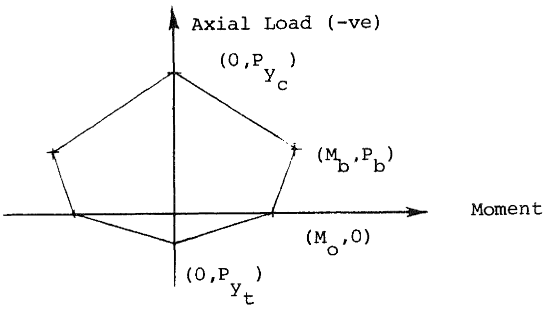
Percentage critical damping in first two modes - 5.00

Ultimate strengths

| | $M_1$ | $M_2$ | $M_b$ | $P_b$ | $P_{y_c}$ | $P_{y_t}$ |
|---|---|---|---|---|---|---|
| | $\times 10^6$Nm | $\times 10^6$Nm | $\times 10^6$Nm | $\times 10^6$N | $\times 10^7$N | $\times 10^6$N |
| Ext. cols. - | | | | | | |
| Level 0 - 4 | 1.450 | | 1.850 | -3.740 | -1.540 | 7.790 |
| 4 - 8 | 1.060 | | 1.520 | -4.010 | -1.340 | 5.250 |
| 8 -13 | 0.583 | | 1.230 | -3.740 | -1.090 | 2.670 |
| Int. cols. - | | | | | | |
| Level 0 - 4 | 1.330 | | 1.800 | -3.230 | -1.460 | 5.340 |
| 4 - 8 | 1.030 | | 1.610 | -2.880 | -1.300 | 3.920 |
| 8 -13 | 0.706 | | 1.430 | -3.160 | -1.140 | 2.630 |
| Beams - | | | | | | |
| Level 1 - 4 | 1.040 | 1.980 | | | | |
| 5 - 6 | 1.030 | 1.970 | | | | |
| 7 | 0.790 | 1.500 | | | | |
| 8 | 0.724 | 1.440 | | | | |
| 9 | 0.679 | 1.290 | | | | |
| 10 | 0.550 | 0.506 | | | | |
| 11 -13 | 0.421 | 0.785 | | | | |

The dimensions of this frame are given in figure 9.

The interaction curve for a column member-type is ...



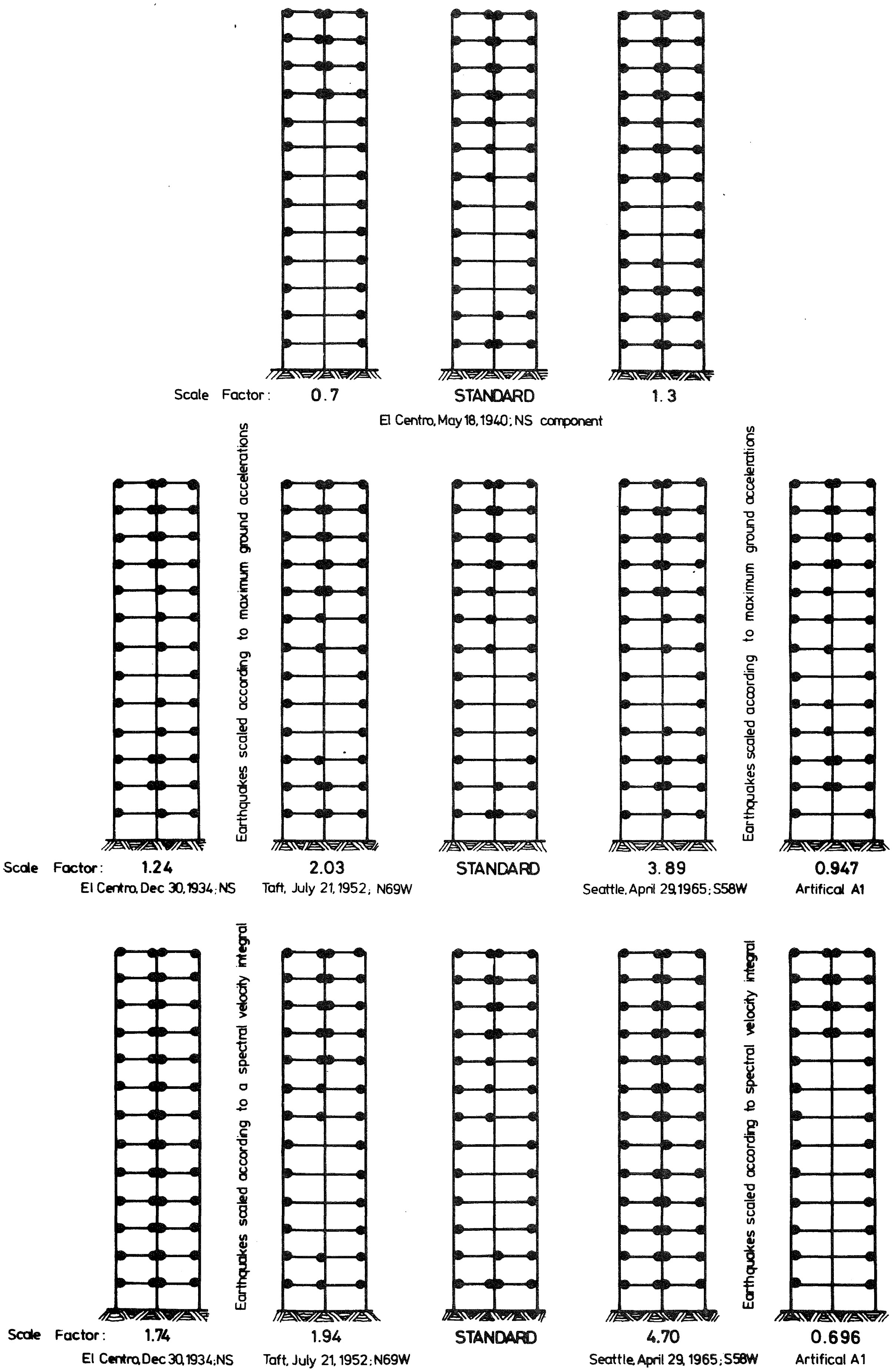


FIGURE 1 : POSITIONS AT WHICH PLASTIC HINGES FORMED DURING THE COMPARATIVE ANALYSES.

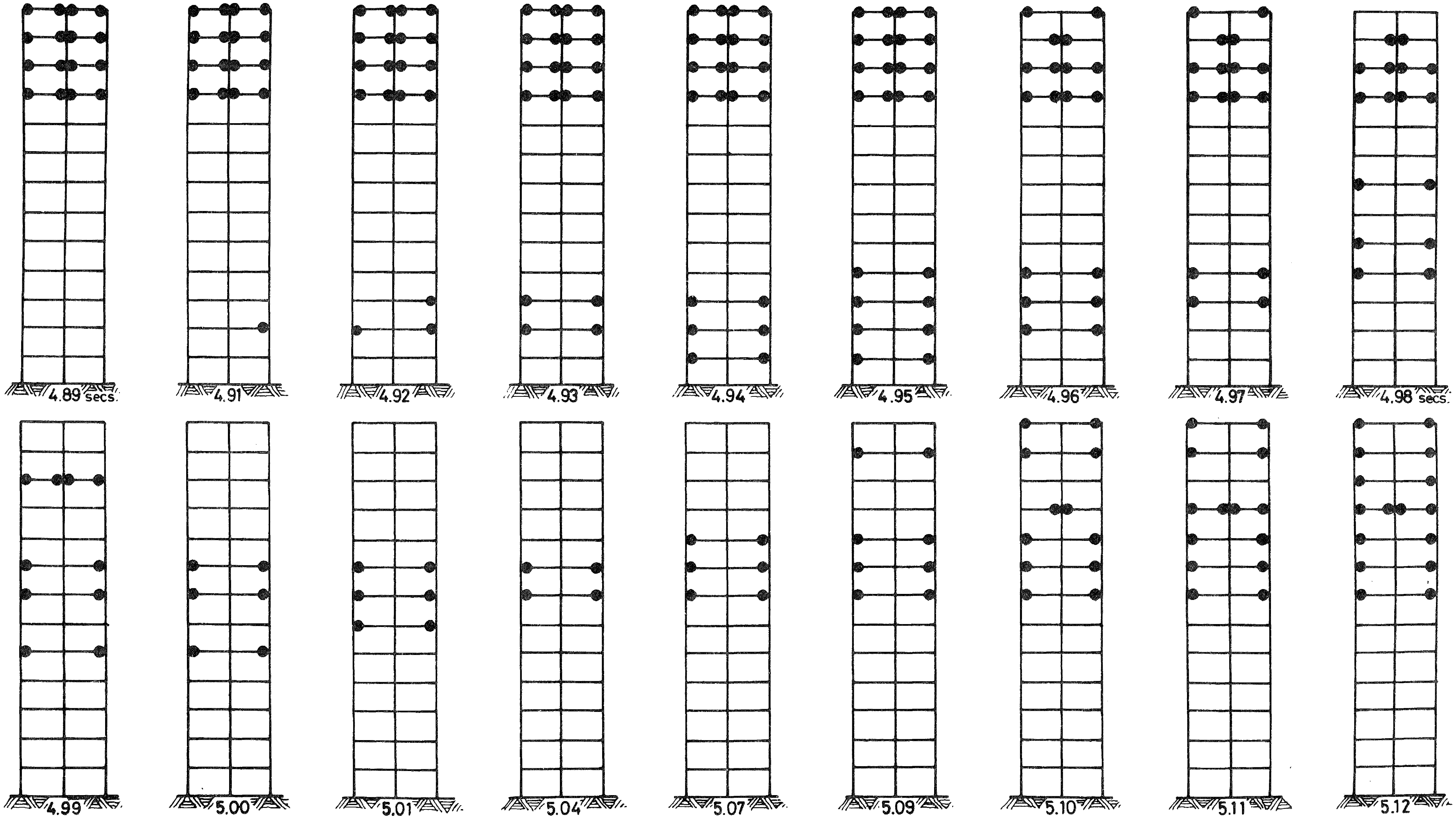


FIGURE 2 : AN EXAMPLE OF THE MIGRATION OF PLASTIC HINGES UP A FRAME DURING AN EARTHQUAKE (EL CENTRO, MAY 18, 1940, N-S).

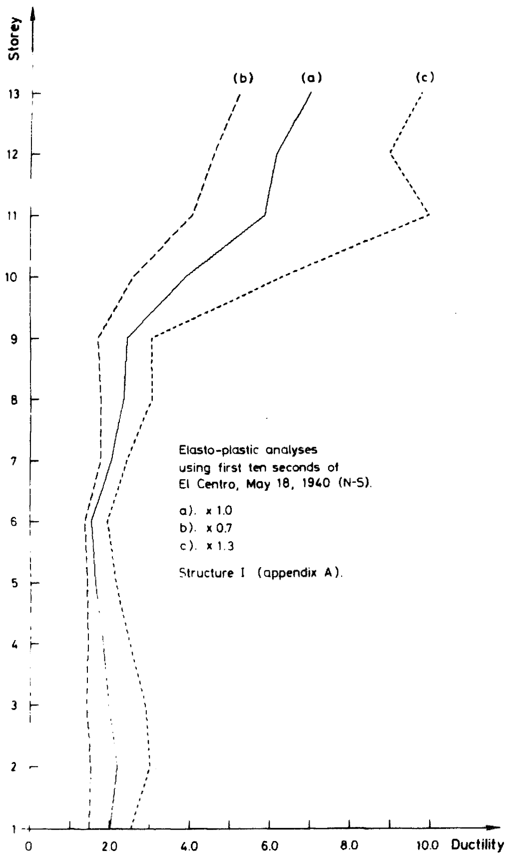


FIGURE 3 . MAXIMUM DUCTILITIES OF OUTER BEAM SECTIONS - BENCHMARKS.

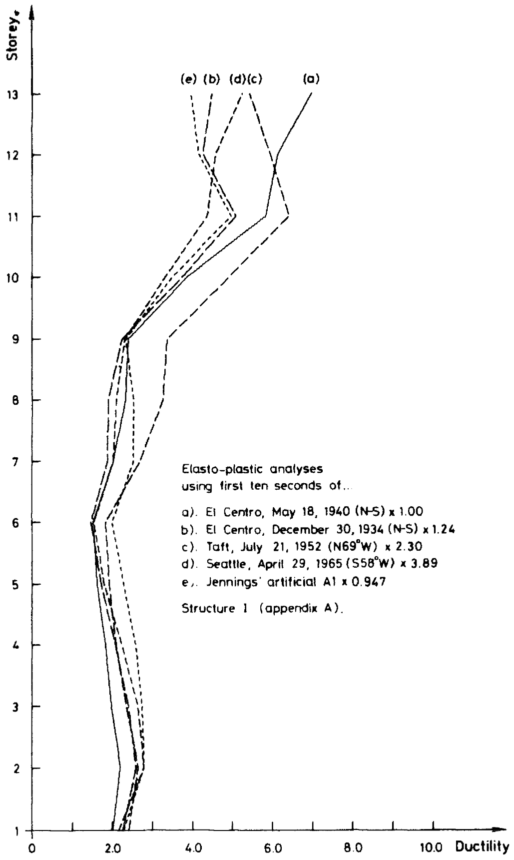


FIGURE 4 : MAXIMUM DUCTILITIES OF OUTER BEAM SECTIONS - SCALING BY MAXIMUM GROUND ACCELERATIONS.

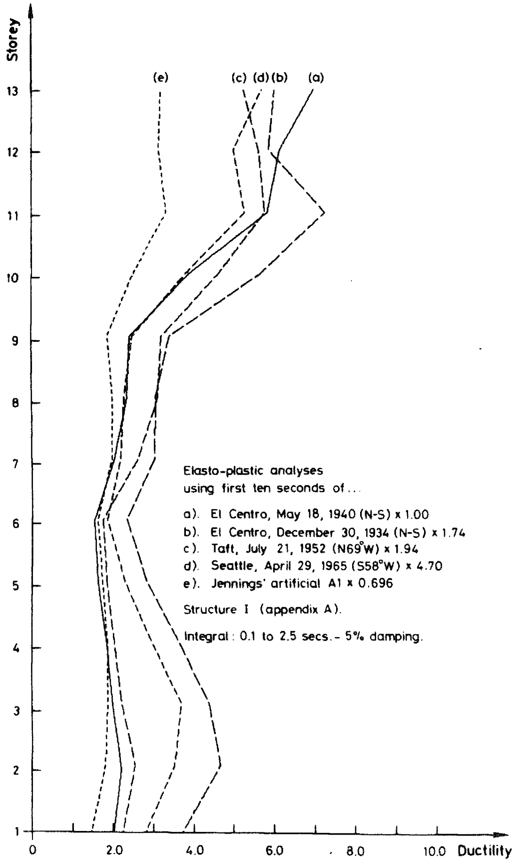


FIGURE 5 : MAXIMUM DUCTILITIES OF OUTER BEAM SECTIONS - SCALING BY A SPECTRAL VELOCITY INTEGRAL.

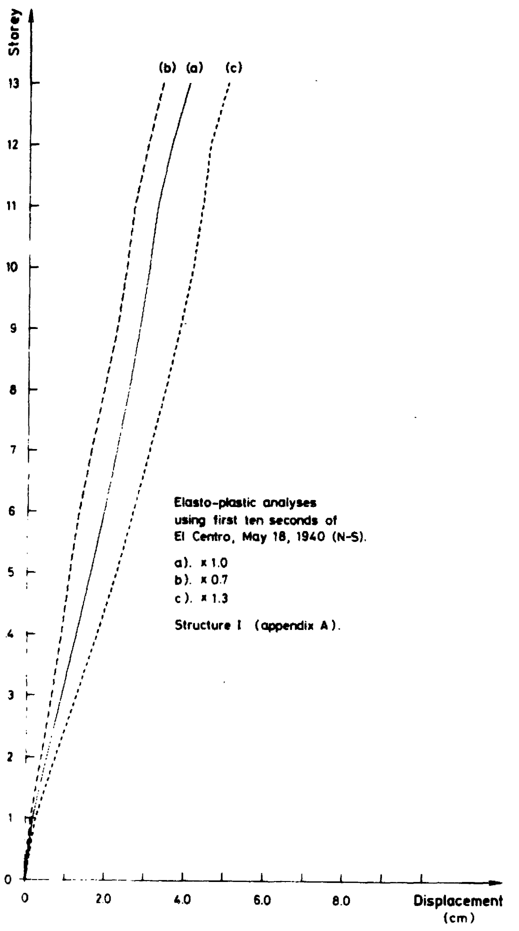


FIGURE 6 : MAXIMUM HORIZONTAL DISPLACEMENTS - BENCHMARK RESPONSES.

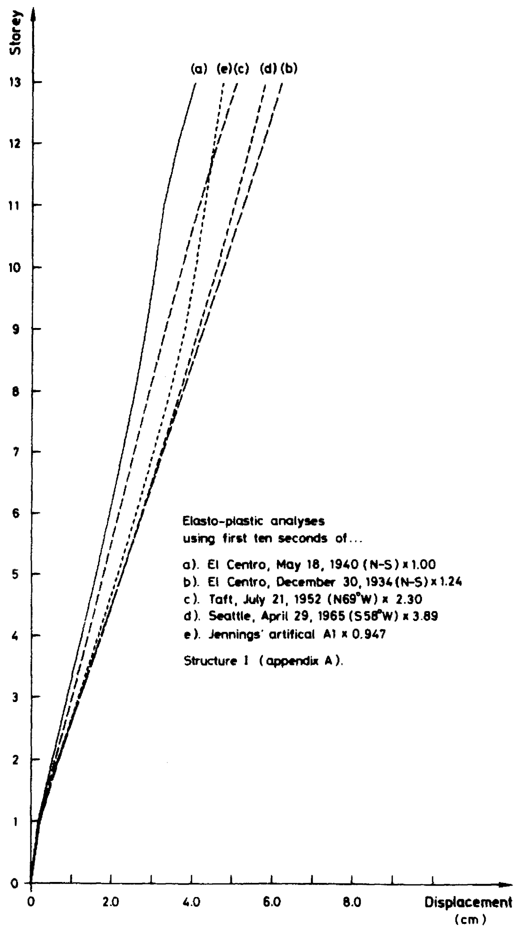


FIGURE 7 : MAXIMUM HORIZONTAL DISPLACEMENTS - SCALING BY MAXIMUM GROUND ACCELERATIONS.

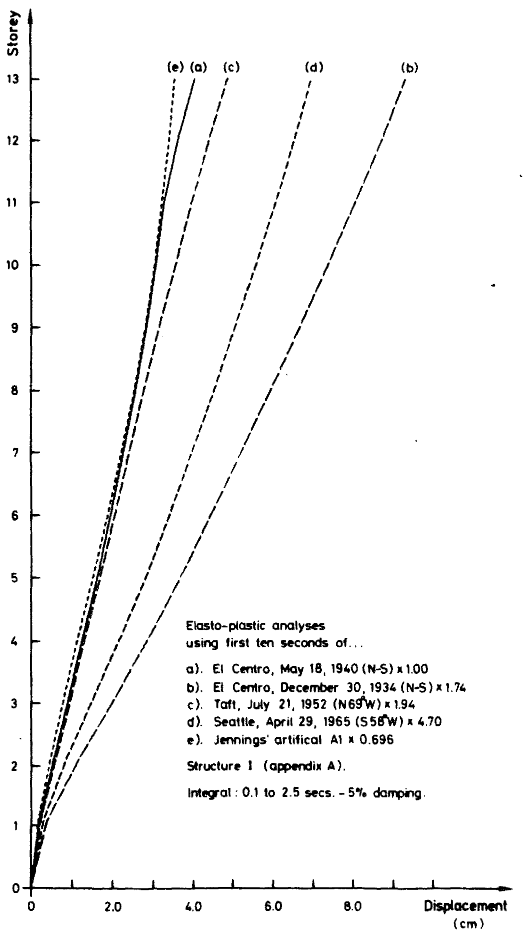


FIGURE 8 : MAXIMUM HORIZONTAL DISPLACEMENTS - SCALING BY A SPECTRAL VELOCITY INTEGRAL.

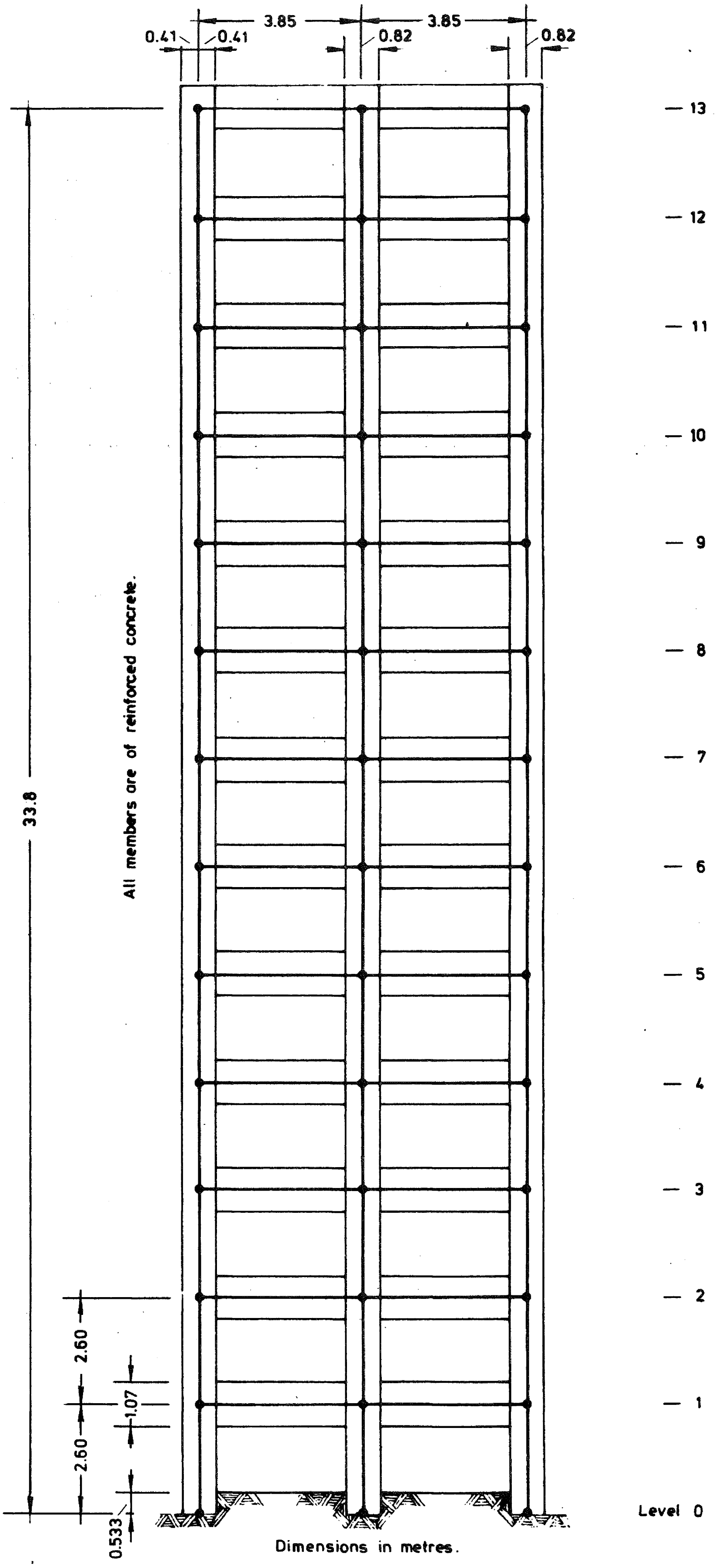


FIGURE 9 : <u>STRUCTURE I.</u>

| Ratios given as Benchmark* / E/Q component | Component | Maximum acceleration | 0% damping |  |  |  | 5% damping |  |  |  | 20% damping |  |  |  | Average |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | Maximum spectral accel. | Maximum spectral velocity | Sp. vel. integral 0.0→3.0s | Sp. vel. integral 0.1→2.5s | Maximum spectral accel. | Maximum spectral velocity | Sp. vel. integral 0.0→3.0s | Sp. vel. integral 0.1→2.5s | Maximum spectral accel. | Maximum spectral velocity | Maximum spectral displ. | Sp. vel. integral 0.1→2.5s |  |
| El Centro, Dec. 30, 1934 | N-S | 1.24 | 1.67 | 1.61 | 2.24 | 1.98 | 2.00 | 1.88 | 2.02 | 1.74 | 1.81 | 1.36 | 2.82 | 1.46 | 1.83 |
|  | E-W | 2.42 | 2.09 | 2.18 | 2.23 | 2.31 | 2.51 | 1.80 | 1.93 | 2.01 | 2.54 | 1.27 | 1.27 | 1.51 | 2.01 |
| El Centro, May 18, 1940 | N-S* | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
|  | E-W | 1.47 | 0.963 | 1.46 | 1.44 | 1.45 | 1.54 | 1.47 | 1.46 | 1.46 | 1.49 | 1.34 | 1.34 | 1.39 | 1.41 |
| Olympia, April 13, 1949 | N80°E | 1.91 | 1.29 | 1.83 | 1.90 | 2.17 | 1.47 | 1.37 | 1.76 | 1.94 | 1.78 | 1.47 | 1.46 | 1.72 | 1.70 |
|  | N10°W | 1.72 | 0.851 | 2.20 | 3.15 | 2.74 | 1.65 | 2.52 | 2.98 | 2.56 | 2.04 | 2.80 | 3.96 | 2.55 | 2.44 |
| Taft, July 21, 1952 | N69°W | 2.03 | 1.48 | 2.02 | 2.14 | 1.94 | 1.90 | 2.14 | 2.15 | 1.94 | 2.15 | 2.03 | 2.02 | 1.71 | 1.97 |
|  | S21°W | 1.79 | 1.36 | 1.70 | 2.62 | 2.34 | 1.62 | 2.42 | 2.67 | 2.33 | 2.23 | 2.48 | 3.52 | 2.08 | 2.24 |
| Olympia, April 29, 1965 | S86°W | 1.59 | 1.02 | 2.78 | 4.26 | 3.86 | 1.53 | 4.02 | 4.10 | 3.71 | 1.97 | 4.26 | 4.53 | 3.61 | 3.17 |
|  | S04°E | 1.98 | 0.781 | 2.44 | 5.42 | 4.57 | 1.53 | 3.43 | 5.34 | 4.46 | 2.22 | 4.56 | 7.31 | 4.24 | 3.71 |
| Seattle, April 29, 1965 | S58°W | 3.89 | 4.19 | 4.30 | 6.12 | 5.38 | 3.93 | 4.80 | 5.47 | 4.70 | 5.08 | 4.02 | 7.28 | 4.30 | 4.88 |
|  | S32°E | 6.02 | 4.46 | 5.00 | 11.9 | 10.8 | 4.49 | 6.76 | 11.0 | 9.99 | 6.25 | 8.39 | 10.0 | 8.92 | 8.00 |
| Artificial (Jennings') | A1 | 0.947 | 0.971 | 0.811 | 0.842 | 0.715 | 1.01 | 0.689 | 0.817 | 0.696 | 1.16 | 0.798 | 0.995 | 0.764 | 0.863 |
|  | A2 | 0.707 | 0.807 | 0.967 | 0.950 | 0.919 | 0.953 | 0.958 | 0.894 | 0.864 | 0.894 | 0.911 | 0.910 | 0.683 | 0.878 |
|  | A3 | 0.927 | 0.782 | 1.02 | 1.17 | 1.28 | 1.02 | 0.992 | 1.20 | 1.05 | 1.05 | 1.15 | 1.75 | 1.09 | 1.11 |
|  | A4 | 1.04 | 0.810 | 0.955 | 0.845 | 0.768 | 1.07 | 0.747 | 0.779 | 0.707 | 1.05 | 0.815 | 0.896 | 0.773 | 0.866 |
|  | B1 | 0.848 | 0.830 | 1.07 | 1.11 | 1.10 | 0.932 | 0.915 | 1.03 | 1.02 | 1.15 | 0.958 | 0.958 | 0.832 | 0.981 |
|  | B2 | 1.01 | 0.848 | 1.12 | 1.18 | 1.06 | 1.10 | 1.02 | 1.16 | 1.04 | 1.11 | 1.13 | 1.46 | 1.06 | 1.10 |
| First ten seconds of each component used to obtain spectra. | C1 | 4.69 | 4.03 | 4.62 | 5.92 | 4.95 | 4.63 | 5.65 | 5.76 | 4.71 | 4.81 | 4.91 | 9.45 | 4.82 | 5.30 |
|  | C2 | 5.50 | 5.16 | 5.46 | 6.88 | 5.80 | 4.61 | 6.65 | 7.08 | 5.89 | 5.35 | 6.92 | 11.8 | 6.09 | 6.40 |
|  | D1 | 0.656 | 0.789 | 1.05 | 1.45 | 1.24 | 0.715 | 1.18 | 1.48 | 1.27 | 0.690 | 1.19 | 2.01 | 1.33 | 1.16 |
|  | D2 | 0.648 | 0.915 | 1.35 | 1.62 | 1.37 | 0.745 | 1.22 | 1.44 | 1.19 | 0.726 | 1.21 | 2.43 | 1.19 | 1.23 |

**TABLE I : SCALING FACTORS FOR ACCELEROGRAMS.**

| | | E/Q scale factor $\left(\frac{E/Q}{\text{Benchmark*}}\right)$ | Max. top-storey displacement ratio | Max. ductility recorded in any beam | Max. ductility recorded in any beam - ratio | Max. ductility recorded at position 1† | Max. ductility recorded at pos. 1 - ratio | Max. ductility recorded at position 2 | Max. ductility recorded at pos. 2 - ratio | Max. ductility recorded at position 3 | Max. ductility recorded at pos. 3 - ratio |
|---|---|---|---|---|---|---|---|---|---|---|---|
| El Centro, May 18, 1940 | N-S* | 1.00 | 1.00 | 6.99 | 1.00 | 5.85 | 1.00 | 6.99 | 1.00 | 4.23 | 1.00 |
| El Centro, Dec. 30, 1934 | N-S | 1.24 | 1.54 | 7.36 | 1.05 | 5.08 | 0.868 | 4.49 | 0.642 | 7.36 | 1.74 |
| Taft, July 21, 1952 | N69°W | 2.03 | 1.26 | 6.41 | 0.918 | 6.41 | 1.10 | 5.42 | 0.775 | 5.52 | 1.31 |
| Seattle, April 29, 1965 | S58°W | 3.89 | 1.44 | 6.02 | 0.862 | 4.36 | 0.747 | 5.25 | 0.751 | 6.02 | 1.42 |
| Artificial (Jennings') | A1 | 0.947 | 1.18 | 4.95 | 0.708 | 4.95 | 0.846 | 3.94 | 0.564 | 4.40 | 1.04 |

† Structure I (appendix A) used with first ten seconds of each accelerogram;
Position 1 at outer end of 11th level beam; position 2 at outer end of 13th level beam;
Position 3 at inner end of 11th level beam

**TABLE II : DUCTILITY RATIOS - EARTHQUAKE SCALINGS BY MAXIMUM GROUND ACCELERATIONS.**

| | | E/Q scale factor $\left(\frac{E/Q}{\text{Benchmark*}}\right)$ | Max. top-storey displacement ratio | Max. ductility recorded in any beam | Max. ductility recorded in any beam - ratio | Max. ductility recorded at position 1 | Max. ductility recorded at pos. 1 - ratio | Max. ductility recorded at position 2 | Max. ductility recorded at pos. 2 - ratio | Max. ductility recorded at position 3 | Max. ductility recorded at pos. 3 - ratio |
|---|---|---|---|---|---|---|---|---|---|---|---|
| El Centro, May 18, 1940 | N-S* | 1.00 | 1.00 | 6.99 | 1.00 | 5.85 | 1.00 | 6.99 | 1.00 | 4.23 | 1.00 |
| El Centro, Dec. 30, 1934 | N-S | 1.74 | 2.32 | 10.6 | 1.52 | 7.26 | 1.24 | 6.01 | 0.861 | 10.6 | 2.51 |
| Taft, July 21, 1952 | N69°W | 1.94 | 1.21 | 5.77 | 0.828 | 5.77 | 0.987 | 5.25 | 0.752 | 5.20 | 1.23 |
| Seattle, April 29, 1965 | S58°W | 4.70 | 1.73 | 7.24 | 1.04 | 5.26 | 0.900 | 5.70 | 0.816 | 7.24 | 1.71 |
| Artificial (Jennings') | A1 | 0.696 | 0.878 | 4.34 | 0.621 | 3.30 | 0.564 | 3.17 | 0.455 | 4.37 | 1.02 |

See table II for details of positions monitored.

**TABLE III : DUCTILITY RATIOS - EARTHQUAKE SCALINGS BY SPECTRAL VELOCITY INTEGRAL.**

| | | E/Q scale factor $\left(\frac{E/Q}{\text{Benchmark*}}\right)$ | Max. top-storey displacement ratio | Max. ductility recorded in any beam | Max. ductility recorded in any beam - ratio | Max. ductility recorded at position 1 | Max. ductility recorded at pos. 1 - ratio | Max. ductility recorded at position 2 | Max. ductility recorded at pos. 2 - ratio | Max. ductility recorded at position 3 | Max. ductility recorded at pos. 3 - ratio |
|---|---|---|---|---|---|---|---|---|---|---|---|
| El Centro, May 18, 1940 | N-S | 1.30 | 1.24 | 9.95 | 1.42 | 9.95 | 1.70 | 9.79 | 1.40 | 7.88 | 1.86 |
| | | 1.00 | 1.00 | 6.99 | 1.00 | 5.85 | 1.00 | 6.99 | 1.00 | 4.23 | 1.00 |
| | | 0.70 | 0.837 | 5.21 | 0.746 | 4.00 | 0.685 | 5.21 | 0.746 | 4.89 | 1.16 |

See table II for details of positions monitored.

**TABLE IV : DUCTILITY RATIOS - BENCHMARKS.**