CAPACITY DESIGN OF EARTHQUAKE RESISTING DUCTILE MULTISTOREY REINFORCED CONCRETE FRAMES

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SYNOPSIS

A condensed step by step summary of the application of a recently developed capacity design philosophy, as applied to earthquake resisting ductile reinforced concrete frames in New Zealand, is presented. The theoretical inelastic dynamic response of three prototype frames, so designed and subjected to particularly severe seismic excitations, is then reported. It is shown how the predicted maximum actions compare with those used in the design. The design quantities, derived from a modified conventional elastic frame analysis for a code specified lateral static loading, were found to ensure a very high degree of, and yet economical and practical, protection against hinging in columns at and above the first floor. Areas in which improvements can be made in this deterministic design procedure, are also outlined. The claim is made that capacity design can ensure predictable behaviour during severe random seismic excitations and yet it is very simple to apply.

RESUME

Un résumé pour la philosophie de la conception de capacité pour le calcul sismique en Nouvelle Zélande est présenté. Une étude élasto-plastique, pour trois cadres soumis à de violents tremblements de terre, est effectuée en utilisant cette méthode. Une comparaison effectuée avec les méthodes de calcul ordinaires montre que les rotules se forment effectivement dans les poutres et non dans les colonnes. Cette conception permet de prévoir la réponse de la structure tout en étant très simple à appliquer.

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1. INTRODUCTION

It is generally recognized in seismic design that few structures will be able to respond elastically to intense seismic ground excitations. Therefore damage, associated with excursions beyond the elastic limit, that may occur during less frequent earthquakes to be expected in the locality, is accepted. The overriding criterion of the design process is an assurance that the structure will possess properties that will enable it to survive the largest expected earthquake without collapse. While existing code requirements (1, 2, 3), that specify equivalent lateral design loads of certain intensities, ensure a high degree of protection against all forms of damage during frequent small disturbances, procedures that could lead to a comparable assurance that collapse will not occur during the largest credible excitation, are not sufficiently formalized. The formulation of the "capacity design philosophy" in New Zealand (3, 4), to be reviewed subsequently, represents a significant advance in this respect. However, the process of quantifying minimum required strengths in simple terms has not yet been completed.

It is also recognized that a hierarchy in the development of energy dissipating mechanisms, necessary to provide hysteretic damping during the inelastic response of a statically indeterminate structure, is desirable. In multistorey frames, the subject of this paper, this desirable hierarchy implies that ductile plastic hinges should develop in the beams rather than in the columns (2, 4, 9, 10). It is particularly important that the formation of storey mechanisms, whereby plastic hinges could form at the top and the bottom of all columns within any particular storey, be prevented. One of the reasons for this is the difficulty in ensuring the ductility of such potential plastic hinges in the presence of axial compression loads. Moreover a nonductile column failure due to either flexure or shear is likely to have very serious consequences. The necessary energy dissipation, to control a large displacement pulse in a storey mechanism of a so called "soft storey", may be associated with excessive ductility demands in the plastic hinges of the affected columns and this may lead to collapse. Such mechanisms may also drastically reduce lateral shear resistance of the affected storey because due to excessive storey drift an excessive fraction of the potential column strength may be consumed to resist P-delta effects.

To reduce the likelihood of soft storey mechanisms, the NBC (1) recommends that at a beam - column joint the columns be made stronger than the beams. However, it is not specified how much stronger should the columns be. This paper attempts to formalize the design strength hierarchy in ductile multistorey frames and then report on the likely response of frames, so designed, to a number of large earthquake motions.

The common method of estimating the required seismic strengths in various parts of a frame utilizes specified equivalent lateral static loads and an elastic analysis (1). As an alternative to the quasi-static seismic analysis, a dynamic analysis, based on the square root of the sum of the squares of modal contributions, may be used (5). Irrespective of the method of analysis the designer at present has to use judgement if he or she wishes to quantify the hierarchy in the development of failure mechanisms.

The most reliable prediction of the likely behaviour of a frame during a specific earthquake may be obtained from a time history analysis of its inelastic response to the corresponding ground excitations. Unfortunately this is an analysis rather than a design technique. It is useful in verifying the viability of the structure. In the interpretation of the results, however, the relevance of the chosen earthquake motions to local seismicity must also be considered. Designers in general are unlikely to be in the position to carry out a series of such analyses to verify the response of the frame they designed to a range of selected ground motions.

To overcome some of these difficulties a simple, deterministic design philosophy, relevant to ductile multistorey frames, is being developed in New Zealand (6). The main feature of this technique, which utilizes the elastic response to equivalent lateral static forces prescribed by many building codes, is that the structural actions, such as bending moments, shear and axial forces, are modified for various members. This is done in recognition of the likely effects encountered during the inelastic dynamic earthquake response of the frame, and to ensure the desired hierarchy in the development in the yield mechanisms. The major steps of the procedure (6, 7) are briefly outlined in the next section.

This proposed design approach is deterministic to the extent that it imparts to the frame member strengths that have a high probability of ensuring that no inelastic deformations of any significance will occur in localities not specifically assigned to dissipate energy during severe earthquake excitations with a wide range of spectral characteristics.

An important aim of the development of the suggested design procedure was simplicity. Lack of precision was compensated for by deliberate conservatism in the selection of certain numerical values. In spite of this, case studies indicate that in comparison with current design practice some reduction in structural materials result. In particular the congestion of reinforcement in the columns studied has been reduced.

In the following section details of the frame design procedure (6,7) are restated step by step. Subsequently the response to some severe ground motions of three prototype frames, so designed, is presented.

In the light of the findings suggestions are made how further improvements in the design procedure could be achieved.

2. THE MAJOR STEPS IN THE CAPACITY DESIGN OF DUCTILE FRAMES

The enforced hierarchy in the development of mechanisms, referred to in the previous section and as developed in New Zealand, is embodied in the capacity design philosophy.

In the capacity design of earthquake-resisting structures, primary energy dissipating elements of mechanisms are chosen and suitably detailed, while other structural elements are provided with sufficient reserve strength capacity, to ensure that the chosen primary energy dissipating mechanisms are maintained at near full strength throughout the deformations that may occur (3, 4).

The major steps of the technique of capacity design philosophy, as applied to reinforced concrete ductile frames, is briefly summarized in the following:

Step 1 - Using an appropriate elastic analysis, the bending moments for beams and columns of the frame are derived for the specified factored lateral static earthquake load only (1). Approximate analyses (8) should be quite acceptable for this purpose. M_{code} refers to moments so derived.

Step 2 - The beam bending moments so obtained are superimposed upon moments induced by gravity loads that have been multiplied by the appropriate load factors (1, 9).

Step 3 - To minimize demands for excessive flexural reinforcement, particularly in negative moment zones, and to utilize flexural strength stipulated by minimum code requirements, typically involving the bottom flexural reinforcement at beam supports (9, 10), a redistribution of design moments, (7, 11, 12) is now carried out. Skilful moment redistribution along continuous beams of ductile frames will not only lead to more advantageous arrangement of flexural reinforcement, but it will also considerably reduce design quantities for the columns, resulting in appreciable saving in column reinforcement. Moment redistribution should be employed because the stipulated lateral static load, when multiplied by the load factors (9), is to be sustained by an inelastic frame. As potential plastic hinges in beams will be detailed for ductility, redistribution involving a moment decrement of the order of 30% of the maximum combined moment in any span should be acceptable. Care must be taken that the laws of statics are not violated and that the total stipulated lateral load resistance of the beams in a bent is not altered.

Step 4 - All critical beam sections are designed and details of the reinforcement for all beams of the frame are finalized. In this a capacity reduction factor $\phi = 0.9$ is considered (9, 10). The subsequent design of other elements will depend on the strength of the beams as detailed at this stage. Therefore if beams are unnecessarily overdesigned, the strength of the supporting columns will also need to be correspondingly increased.

Step 5 - The flexural overcapacity (4) of each potential plastic hinge, as detailed, is now evaluated in each span of each continuous beam for both directions of the applied lateral load. In this allowance must be for both the mean yield strength of the beam flexural reinforcement and for the possibility that the induced steel strains might be in the hardening range. For mild steel with a guaranteed yield strength of 275MPa and a long yield plateau, typically a 25% increase in strength is assumed. For steel with higher yield strength (f_y = 400MPa) usually a shorter yield plateau is available and correspondingly a larger increase for strain hardening should be considered. To avoid the undesirable consequences of the strength gain, in New Zealand where all structures are affected by seismicity, mild steel (f_y = 275MPa) is preferred in the beams of ductile frames.

From an extrapolation of bending moment diagrams extending over the clear spans, or otherwise from first principles, the corresponding beam overstrength moments at each column centre line are found and hence the associated moment induced beam shear forces, V_{oe}, in each span are determined.

Span 6 - The beam overstrength factor, ϕ_o, at the centre line of each column, for both directions of the loading on the frame are determined. For special localities specific values of ϕ_o have been suggested (6, 7).

The beam overstrength factor at a column, ϕ_o, is the ratio of the sum of the flexural overstrengths developed by the beams, as detailed, to the sum of the flexural strengths required in the given direction by the code specified lateral seismic loading alone, (as derived in Step 1) both sets of values being related to the centre line of the relevant column (7). In other words the overstrength factor simply relates the maximum feasible beam strengths that can be extracted at a joint, to the strength required to resist the code specified factored lateral load. The latter is always used as a reference strength.

Step 7 - At each floor the appropriate value of the dynamic moment magnification factor, ω, (6, 7) is established. This moment magnification intends to allow for the fact that during the dynamic excitation the moment pattern along a column may be markedly different from that which resulted from the initial elastic analyis. To illustrate the phenomenon Fig. 1 is presented, which compares bending moment patterns for one column, computed at various critical instants of the inelastic seismic response of a twelve storey frame, with the moments derived by an elastic analysis for the specified lateral static loads. It also shows that the assumption of equal distribution of beam imput moments at a joint between the column above and below that joint (2) is grossly unconservative in terms of protection against early column hinging. It is also evident that customary capacity reduction factors (9, 10), such as $0.7 < \phi < 0.9$, are inadequate to compensate for such radical moment increases in a column.

For ductile frames, the columns of which are designed to resist earthquake forces in the plane of the frame only, it has been suggested (3) that

$$1.2 < \omega = 0.6\ T_1 + 0.85 < 1.8 \qquad (1)$$

where T_1 is the computed fundamental period of the structure in seconds. At and near the ground floor and in the top storey, specific reduced values for ω have been suggested (6, 7). The maximum expected column moment, measured at immediately below or above the level of beam centre lines may then be obtained from $\phi_o \omega M_{code}$.

Step 8 - In order to arrive at probable maxima for earthquake induced column axial loads, all maximum earthquake induced beam shear forces, V_{oe}, for all floors from roof level down to the ground floor, are computed and hence at each floor the axial force $P_{eq} = R_v \Sigma V_{oe}$ is determined. The summation refers to all floors above the column section that is being considered. The reduction factor R_v intends to recognize the diminishing likelihood of the beam overstrength shear forces, V_{oe}, developing simultaneously with increasing number of floors above the level considered. R_v also intends to compensate for the reduced likelihood of locally magnified moments, thought to be due to higher mode effects, coinciding with maximum earthquake induced axial forces that are primarily due to a first mode response.

This approach is similar to that adopted in Canada and the United States (J coefficient) to reduce overturning moment effects with increasing number of floors taken from roof level (1, 2, 13).

The proposed values for R_v are given in Table I and these are to be used as follows:

(a) When $P_e/f_c'A_g < 0.4$, use Table I and the appropriate value of ω.

(b) When $0.4 < P_e/f_c'A_g < 0.7$, apply linear interpolation between the value of R_v given above in (a) and the maximum value given in Table I for the case when $\omega < 1.4$, that is shown there to be applicable when $P_e/f_c'A_g = 0.7$.

In the above expressions P_e is the total design axial force that includes P_{eq} and the appropriate factored gravity loads, f_c' is the compressive strength of the concrete, and A_g is the gross concrete area of the column section considered.

The provisions in paragraph (b) originate from somewhat increased conservatism for compression dominated columns, in which an underestimation of axial load will have a more serious effect on the reduction of flexural capacity. It is suggested that columns in which $P_e/f_c'A_g > 0.7$ should not be used in earthquake resisting ductile frames (3,14).

Step 9 - After the combination of the earthquake induced axial load, P_{eq}, with appropriately factored gravity loads, the design axial load on the column, P_e, for each direction of siesmic action can be found.

Step 10 - The column design shear force in each storey is estimated from

$$V_{col} = 1.4\phi_{o,max.}V_{code} \tag{2}$$

where $\phi_{o,max.}$ is the larger of the two beam overstrength factors relevant to the ends of a column, and V_{code} is the column shear force derived from the initial (Step 1) elastic analysis for the specified lateral earthquake loading only. The factor 1.4 has been derived from consideration of the maximum likely moment gradient along the column. It should be noted that while the design moment at one end of the column, as given in Step 7, is being approached the moment at the other end will be much smaller (see Fig. 1). The estimation of column shear force, a particularly critical quantity, is thus related to the maximum moments that could be feasibly developed at the bottom or the top end of the column, rather than to dynamic modal effects. The column design shear force will seldom be less than twice the shear force derived from the initial elastic analysis for code loading. A special consideration of first storey columns, where the formation of plastic hinges is to be expected, is discussed in Section 4.5.

The critical column design moments at the top or the soffit of beams, to be considered together with the design axial load P_e, are finally found from

$$M_{col} = \phi_o \omega M_{code} - 0.3h_b V_{col} \tag{3}$$

where h_b is the depth of the beam which frames into the column at the floor under consideration.

Step 11 - In columns under low axial compression or subject to axial tension, relatively "early" yielding is considered to be acceptable because of the larger curvature ductility that is available. Consequently a reduced column design moment, given by

$$M_{col,\ reduced} = R_m M_{col} \tag{4}$$

may be considered. The reduction factor R_m takes the axial load intensity and the dynamic moment magnification factor, ω, into account and its value is given in Table II.

For example the suggested value of R_m may be as low as 0.3 (i.e. 70% moment reduction) when in the outer column of a 20 storey frame the design axial load, obtained in Step 9, produces a tensile stress equal to or larger than $0.15f_c'$ over the gross concrete area. While the axial load carrying capacity of a column must always be assured, under such extreme circumstances a significant loss of moment capacity in some tension columns should have no adverse consequence. Significant hinging cannot occur in such a column unless other columns of the bent, for which moment reduction is not applicable, are also developing plastic hinges. This moment reduction usually leads to a significant reduction in the vertical reinforcement content in exterior columns and it allows a better balance for steel demand when earthquake induced axial compression or tension dominates. In order to ensure that the column flexural capacity "so lost" is not excessive in terms of the lateral load resistance of the storey, it is suggested (6, 7) that the moment reduction used in the design of columns of a bent should not exceed 10% of the sum of the design moments for all columns of that bent, taken at the same level. Usually only one column of a bent will qualify for a reduction of design moment.

Step 12 - With the determination of the design moments, axial and shear forces, the section properties at each level of the columns may be determined so as to give ideal strengths not less than those required. Capacity reduction factors, normally used in reinforced concrete design, are all taken as unity in this capacity design procedure, because upper bound estimates, based on realistic material properties in beams, have been used in determining the critical column design actions. There is also a small reserve strength available because the guaranteed strength for both steel and concrete, f_y and f'_c, is likely to be less than the mean value that will be made use of during a severe earthquake.

The above procedure is restricted to undirectional seismic action on plane frames. The consideration of skew earthquake effects (13, 15) on two-way framing systems is beyond the scope of this study. It has been suggested (6) that a similar degree of protection against column hinging, due to biaxial bending and axial load input from 2 to 4 orthogonally arranged beams, may be attained by considering independent unidirectional seismic attacks only in the two principal directions. To compensate for concurrency of actions on a column, a larger value for the dynamic moment magnification, ω, is recommended (6) together with the simple technique summarized in the above 12 Steps.

3. A STUDY OF PROTOTYPE FRAMES

To examine in greater detail the suitability of the capacity design procedure based on previous limited studies (15, 16) and outlined in the previous section, three prototype two-bay frames with 6, 12 and 18 storeys were chosen for detailed study (17).

Each structure was loaded in accordance with the provisions of the New Zealand loading and general design code (3) and then designed as outlined in the preceding section. The base shear force relevant to the total equivalent mass of the building is given for each structure in Table III, where all other significant data are assembled. Simple interior two-bay frames of a building, resisting earthquake loads in one direction only, were chosen. Other elements, such as shear walls, were assumed to resist seismic actions perpendicular to the plane of these frames. To avoid excessive reserve strength that could possibly mask some aspect of behaviour, care was taken in the proportioning of members to represent, as closely as possible, the minimum requirements of the proposed design procedure. In particular full use was made of moment redistribution between potential plastic hinges in the beams to minimize the flexural steel content at the critical sections.

The unreduced live load for all frames was 2.5kPa. Actions for the prescribed lateral static load (3) were derived using Muto's approximate frame analysis (8). The lateral deflections at each floor were estimated from the average of the deformations computed for two columns, which showed good correlation at all levels.

The time history studies for various ground excitations were carried out mainly using a 2-dimensional dynamic analysis developed by Sharpe (18). Certain obvious errors, attributed to overshoots in the process of successive approximations, resulted in a decision to repeat some of these analyses using a different program, developed by Powell (19). In terms of structural design the differences in the results, obtained from these two programmes, were not significant. Both analyses led to the same overall assessment of structural performance.

To compensate for the effects of cracking, 25% and 50% loss of stiffness with respect to uncracked member sections was allowed in columns and beams respectively. 8% of critical damping was assumed in the first and the nth mode of vibration, where n is the number of storeys. The programme then allocated values less than 8% critical damping for modes between 1 and n, and larger values beyond mode n.

The permanent gravity load during dynamic excitation was simulated by the dead and one third of the design live load.

Member moment capacities were expressed in terms of probable material properties, but some reduction was allowed to compensate for overestimates that are frequently made in the program before yielding is detected. The guaranteed yield strength of the main reinforcement in the beams and columns was taken as 275MPa respectively. Moment curvature relationships were represented by elastic-perfectly plastic hysteresis loops without allowance for any stiffness degradation or strain hardening.

Wherever applicable comparison of computed actions during the response are made in the diagrams that follow with those stipulated by the loading code (3). To make the comparison more meaningful the code specified actions (3) were inflated by 23%. This was considered to correspond with the probable strength of the structure that would have been designed (7, 9, 10) so as to satisfy exactly code specified lateral static load requirements, without the use of any magnifying factors of the proposed design procedure.

Because of time limits and expense, only a few selected ground acceleration records could be used in this study. The 2% damped pseudo velocity response spectra of the records that have been considered (20, 21) are shown in Fig. 2. The records to be used in the analyses, as shown in Table III and Fig. 2, were selected so as to give the maximum likely response in the region of the fundamental period for each of the frames studied. Because of its extensive use in similar studies, the El Centro 1940 N-S record was used as a bench mark for all structures.

In the assessment of the column responses, the appropriate moment-axial load interaction relationship for each column section was simulated by a cubic function that gave exact values at four equidistant points within the predictable range of axial compression load intensities, i.e. $0 < P_e/f'_c A_g < 0.6$. No axial tension in any column was expected in this study.

4. A COMPARISON OF FRAME RESPONSES TO DIFFERENT EARHTQUAKE MOTIONS

4.1 - Displacements and Interstorey Drifts

A convenient way to study the overall response of a multistorey frame to a selected earthquake ground motion, is to examine its horizontal deflection at roof level. For convenience the analytical results for all three frames studied are assembled in Fig. 3. The lateral displacement of the top floor, Δ, is shown in millimetres as well as in terms of the total building height, H. A deflection equal to H/100 is shown with a dashed line because this quantity represents closely the maximum average storey drift suggested by the New Zealand design and loading code (3) to be acceptable. For the frames studied this deflection, to be estimated in a routine design (3), is 2.5 times the elastic deflection induced by the specified lateral static loading (3). In this allowance is also made for the reduction of stiffness resulting from cracking in various members.

From Fig. 3 it is seen that this code limit on deflection has not been exceeded in any of the frames during the El Centro excitations. However, with the exception of the 18 storey frame, larger displacements were indicated by the dynamic analyses for all other of the selected ground motions.

Of particular interest is the permanent inelastic displacement imposed by both the Pacoima and the Parkfield excitations after approximately 8 seconds of duration. The permanent tilt of the 12 storey frame is of the same order as the maximum distortion encountered during 14 seconds of the Pacoima excitation. After the first major inelastic excursions of the Pacoima and Parkfield motions the six storey frame oscillated about an axis on an approximate slope of 1 in 200.

For reasons of economy the analysis for the 18 storey frame was carried out for only the first 10 seconds of the excitations. As will be seen in subsequent sections the response of this frame was the least critical in all respects. For the chosen records no inelastic distortions were expected to occur in the frames beyond the time of 14 seconds. These permanent displacements may be taken as a measure of the severe damage that would have to be expected. From several studies it became evident that these permanent drifts developed only in the later stages of the excitations and that their magnitudes were rather sensitive to the assumptions with regards viscous damping. With a reduction in the assumed fraction of critical damping the permanent drifts were found to increase.

As may be seen in Fig. 2, the Pacoima record was not expected to produce particularly critical conditions for the six storey frame. The analysis showed, however, (Fig. 3a) that these motions resulted in maximum responses in all the frames studied.

In considering non-structural damage and the relative influence of P-delta moments, the interstorey drift is of particular importance. These are presented both in absolute magnitudes and in terms of the storey height, h, in Fig. 5. Whereas the storey drifts are well within the intended code limit of h/100 (3) for the El Centro excitations, they are much larger for the other earthquake motions. The influence of P-delta moments has not been taken into account in the modelling for the inelastic dynamic frame analysis. From preliminary studies and from the work of Powell and Row (22) it appears that the increase in inelastic displacements due to P-delta effects is not very significant.

Some indication of the significance of interstorey drifts and the associated P-delta effects on these frames may be gained from the evaluation of a stability factor (23)

$$R_r = W_{tr}\delta/\Sigma M_i \tag{5}$$

where W_{tr} is the total laterally displaced gravity load considered at floor r, δ is the interstorey drift and ΣM_i is the sum of the ideal beam capacities developed at the beam plastic hinges at the floor considered. The factor simply expresses the fraction of the total beam strength required to sustain the P-delta moment in the storey. Thus the lateral load resistance of the beams at the particular floor is reduced by this fraction. For the most critical excitation and storey, the value of R_r was found to be 0.18, 0.37 and 0.21 for the 6, 12 and 18 storey frames respectively. After 11 seconds of the Pacoima excitation about one third of the ideal beam strengths in the 12 storey frame were required to resist the overturning moments caused by the large permanent inelastic storey drifts.

4.2 - Plastic Hinge Formation

The intention of the design was to ensure that no storey mechanisms will form during the largest excitation and that the likelihood of plastic hinge formation in any but the top and bottom storey columns is minimized. The 12 and 18 storey frames met this criteria to the extent that, with the exception of ground floor, the analysis has indicated no yielding in any column during any of the selected ground motions. In all frames the required energy during the very significant inelastic displacements was predominantly dissipated by plastic hinges in beams.

Hinge patterns observed in the 12 storey frame during selected instants of the Pacoima Dam excitation are presented in Fig. 4, where a distinction is made between clockwise and anticlockwise relative hinge rotations that were required to mobilize the probable flexural capacity at the end of that member. For the framed instants in this figure, shown under a particular set of plastic hinges, the approximate distorted shape of the structure has also been plotted. These are seen in the insert of Fig. 4. These deflection and hinge patterns should be studied in conjunction with the roof level displacements shown for this excitation in Fig. 3b.

Contrary to expectations, at one instant (3.05 seconds) 96% of the possible beam plastic hinges were mobilized. It is also seen that there were only 4 brief periods during which plastic hinges at the base of the columns of this frame were predicted.

Because of the significant inelastic distortions that occurred during the first major pulse, at approximately 3 seconds, the subsequent deflection and the hinge patterns can not be related to higher mode shapes.

The hinge patterns, when studied in conjunction with Fig. 3, confirm that the number of load reversals, involving the formation of plastic hinges, was relatively small during the vigorous response of the frame during the first 9 seconds.

Similar hinge patterns were obtained for the 18 storey frame (17). Hinge developments at the base of the columns were, however, less frequent.

The six storey frame developed at least at one instant (3.71 seconds) of the Pacoima motions a complete failure mechanism with 27 plastic hinges. During both the Pacoima and the Parkfield excitations, yielding, particularly at the top end of a number of upper storey columns, occurred simultaneously with hinge formation at the base of the columns. This was the first indication that the suggested design procedure was least conservative when applied to the six storey frame, It was also noted that maximum displacement at roof level did not coincide with the development of the maximum number of plastic hinges.

4.3 - Column Moment Demands

In order to assess the relevance of the design intentions to the "observed" behaviour in this analytical study, the maxima of column bending moments at beam centre lines are compared in a series of figures. The dark central shaded pattern in Fig. 6 shows the bending moment diagrams for the interior column of the six storey frame, that resulted from an elastic analysis for the code specified lateral static loading. As described in Section 3, those moments have been scaled up by 23% to correspond with the ideal flexural strength that would have been obtained for the various column sections. The horizontally shaded broken line indicates the moment values that were obtained from the suggested design procedure, summarized in Section 2. Only the peak values are to be considered in these diagrams as the slopes, selected only for convenience, have no relevance to any specific moment gradient.

It is seen that in the lower storeys the moments induced during the El Centro motions are of the same order as the design moments. For the Parkfield and particularly for the Pacoima excitation, the "observed" moments exceed the design moments at all levels. As a consequence, with the exception of the first floor, plastic hinge formation at some instants was indicated in the columns at all floors. As is seen these "observed" hinging moments are at times considerably larger than the design moments. The main reason for this was that, to be practical, at some sections a little more reinforcement was provided in the columns than required by the design moments. It should be noted that in the columns of the 6 storey frame only the minimum 1% vertical reinforcing content, traditionally specified by codes (7, 9, 10), was used.

Fig. 7 shows a similar comparison for the moments along the exterior column of the 12 storey frame. For this long period frame the maximum value of the dynamic magnification factor, ω, given by Eq. (1), resulted in design moments considerably larger than those resulting from the specified code loading (3). With the exception of the bottom and top storeys, the moments generated during the El Centro motions were much smaller than the proposed design moments. However, during the Pacoima excitation the "observed" peak moments were of the same order as those used in the design. The moments induced at the tops of the columns, up till the 7th floor, have slightly exceeded the design moments. However, because of some excess column reinforcement and an advantageous interacting axial compression at these sections, no yielding was "observed". It may be said that for the 12 storey frame the design moments predicted very satisfactorily the column flexural demand for the critical Pacoima ground motions.

The manner in which the inelastic dynamic response may affect the bending moment patterns, is shown in Fig. 1 for an exterior column of the 12 storey frame. The instantaneous moment patterns and associated hinge formation, shown there for the Pacoima excitation, may be combined with the information given also in Fig. 4. The moment pattern and the deflected shape at 2.70 seconds indicates a distinct second mode response. Similar moment patterns at 7.80 and 8.00 seconds, however, can no longer be related to a corresponding modal shape. At neither of these instants did critical column moments occur. It is seen that at 3.09 seconds a predominantly first mode moment pattern has been affected and hence distorted by a second mode response, resulting in very large column moments between the 2nd and the 5th floors. It is this phenomenon, which the dynamic magnification, ω, is intended to compensate for.

Only the results of the Pacoima excitations are compared in Fig. 8 with the code specified flexural strengths for the exterior columns of the 18 storey frame. The maximum moments during the El Centro and the artifical A2 motions were found to be much smaller and, for the sake of clarity, corresponding plots have been omitted. It is seen that the design moments predicted very satisfactorily the peak moments for the lower half of the frame. In spite of structural symmetry the peak moments for the two exterior columns are not the same. This is due to the difference in beam hinging moment input at a floor (the positive and negative flexural strengths of the beams at a plastic hinge are not necessarily the same), and the great difference in earthquake induced axial forces that occur in the two columns at the same instants. Note that the hinging base moment in the compression column (column 3 at 2.81

seconds) is some 30% larger than a similar moment in the tension column (column 1 at 2.76 seconds). These "observed" base moments are much larger than the original design moments because the latter required less than 1% total reinforcement content, which has been provided. Only two load reversals, involving plastic hinge moments at the base of these columns, were "observed".

4.4 - Demands for Inelastic Deformations

One of the aims of this study was to estimate the ductility demand imposed on various plastic hinges during selected excitations, so that comparisons can be made with realistic values obtained from experiments. It has been found that in plastic hinges of beams of normal geometric proportions, such as used in these ductile frames, which have been suitably detailed to ensure that failure due to shear, anchorage and buckling of compression reinforcement does not interfere with the development of flexural capacity, a total rotation of 0.035 radians i.e. 2 degrees, can be attained. In laboratory studies rotations of this order were imposed on beams in both directions of loading with insignificant reduction of strength after 4 to 6 reversals of loading. Plastic hinge rotations so measured include deformations from all sources i.e. flexure, shear and anchorage slip, while the ideal strength of the beam is maintained.

Fig. 9 represents the envelopes for the maximum hinge rotations for all three frames, encountered at any of the four beam hinges at a floor. It is seen that the maximum hinge rotations, which occurred at the lower floors, did not approach 0.035 radians during any of the earthquakes used in the analyses. It may be said that with standard (7) seismic detailing, the predicted ductility demand could have been comfortably met by these beams. It is likely, however, that more significant strength degradation would have occurred in the first floor beams of the 6 storey frame, particularly during the Pacoima motions, because up to 18 load reversals, involving full flexural strength, were encountered in the first 10 seconds. However, only 7 instants involved ductility demands of significance. The El Centro excitation caused consistently small inelastic deformations.

The maximum plastic rotation at the base of the columns may be considered to be the most critical aspect of this study. A base hinge rotation of 0.0155 in the 12 storey frame "occurred" during the Pacoima excitation, when the total axial compression load on the column produced an average stress of $0.55f_c'$ over the gross sectional area.* The column section in question was therefore compression dominated, and concrete compression strains considerably in excess of 0.003 would have been required in the plastic hinge zone in order to develop the necessary plastic rotation. However, this could have been achieved with suitable confining hoop reinforcement in the end zone of the column (4, 14). Axial compression of similar intensity coexisted with the plastic hinge formation at the base of the 18 storey columns, where the ductility demand was, however, considerably less. The axial compression on the columns of the 6 storey frame was not critical and this would have

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* f_c' is the assumed compressive strength of the concrete.

allowed the use of less confining hoop reinforcement (14) to ensure an even larger rotational ductility.

The ductility demand at the base of the columns was not critical in any of the frames during the El Centro excitation. Even though intermittent column hinging occurred in the upper storeys of the six storey frame during the Pacoima Dam motions, the associated plastic deformations were insignificant, as may be seen in Fig. 9a.

4.5 - Shear Forces Across Columns

Dynamic analyses of inelastic structures for typical earthquake motions usually predict induced shear forces across columns which are considerably larger than those predicted by procedures prescribed by existing building codes (1, 2, 3). Some codes have recognized this discrepancy and hence they stipulated that shear forces obtained from conventional seismic analyses be increased with the use of a specified load factor (2). Instead of using a uniform increase of shear for all cases, the procedure employed in the design of these frames considers the realistic beam moment input to the column at each floor, and also makes allowance for the possible increase of moment gradient along each column due to local effects of higher mode dynamic responses. Eq. (2) embodies these considerations. Figs. 10, 11 and 12 compare various shear envelopes for the three frames studied, details of which are discussed in relation to the six storey frame (see Fig. 10).

The innermost stepped line represents the shear force in each storey for the interior column of the 6 storey frame, derived from the code (3) specified equivalent lateral loading. The shear forces so obtained have been increased by 23% to represent the corresponding probable shear strength of the column and these are the values shown in Fig. 10. The outermost stepped and shaded line shows the ideal shear strength obtained from Eq. (2), and it must be assumed that the column has been designed to possess at least this ideal shear strength. It is seen that at all floors the Pacoima Dam excitation produced the largest shear forces. To enable a quantitative evaluation of the critical nature of the shear forces shown, the contribution of beam shear resisting mechanisms other than stirrups (4), at a nominal shear stress of $0.17\sqrt{f'_c}$(MPa) and with a probable compression strength of the concrete, f'_c, is also shown by the shaded area. This is purely a reference strength which shows for the column under study that nominal web reinforcement is likely to be sufficient in the 6 storey frame to provide the additional shear strength required.

The design of first storey columns for shear is different because plastic hinge formation at the base, possibly developing the flexural overstrength capacity of that section, $M_{o,col}$, must be considered. Therefore the evaluation of the ideal shear strength, to be provided in first storey columns, was based on

$$V_{col} = \frac{1.15}{\ell_c}(M_{o,col} + \omega\phi_o M_{code,min}) \tag{6}$$

where $M_{code,min}$ is the theoretical value of the column moment at the centre line of the first floor beams due to the code specified lateral static loading. These values are incorporated into the relevant figures.

Fig. 11 makes a similar comparison of shear envelopes for the interior column of the 12 storey frame. It is seen that the proposed ideal design shear strengths, i.e. Eq. (2) and Eq. (6), consistently exceed the dynamic shear forces "observed" in the analytical studies. The conservatism demonstrated is not excessive. The minimum reinforcing content of 1% at the base of this interior column can develop a moment considerably larger than contemplated (see also Fig. 7). To be consistent in the established hierarchy of failure mechanism, allowance must be made for the possibly large shear force associated with the large base moment capacity. This explains the seemingly excessive design shear shown for the 1st storey in Figs. 11 and 12. The dynamic analysis used did not allow for strength increase in plastic hinges due to strain hardening of the column reinforcement. Therefore shear forces across the first storey columns of the real frame may be slightly larger than the "observed" values.

Fig 12 combines the shear forces for all three columns across each storey of the 18 storey frame. This is termed the storey shear. The critical nature of the Pacoima Dam excitation and the conservatism of the proposed procedure are again evident. The reasons for the increase in the first storey design shears are the same as outlined for the 12 storey frame. All columns of this frame develop excess flexural capacity at the base with minimum steel content. In spite of this apparent conservatism the amount of transverse steel at the base of these columns will not be governed by shear demand but rather by the confinement requirements.

4.6 - Column Axial Forces

The envelopes for the maximum and minimum axial compression forces for the exterior columns of the 12 and 18 storey frames are compared in Fig. 13 and Fig. 14. No net tension could be induced in columns of this study.

The design axial load envelopes were based on the load combinations $D + 1.3L_R + R_vE^\circ$ and $0.9D - R_vE^\circ$, where D and L_R represent dead and reduced live loads respectively, E° considers earthquake induced axial loads, originating from the plastic hinge moments at overstrength in all beams, and R_v is an axial load reduction factor discussed in Step 8 of Section 2.

It is seen that a good agreement exists between the proposed design values and the "observed" axial forces. The maximum compressive forces during the Pacoima Dam motions slightly but consistently exceed the design intensities. On the other hand the minimum design axial compression which, when combined with the design column moments, is likely to govern in the requirements for the principal column reinforcement, is generally less and therefore it is more critical than the "observed" values.

As axial forces are considered together with likely concurrent bending moments, a high degree of accuracy in their determination is not warranted. An error of the order seen in Fig. 13, will only slightly affect the flexural capacity of the column sections, which has been adequately provided for, as may be seen in Figs. 7 and 8.

The reduction factor R_v has a negligible effect on the column forces of the 6 storey frame. Therefore the conservatively predicted and less critical axial forces for this frame are not reproduced here.

5. SUMMARY AND CONCLUSIONS

5.1 - The Aims of the Design Procedure

The principal aim of the proposed design procedure is to quantify a capacity design philosophy, as applied to multistorey ductile frames. Thereby, with the exception at ground level, columns are provided with a high degree of protection against possible hinging during a very large earthquake. The study of three prototype frames indicates that this aim was largely satisfied.

By using magnification factors that recognize the probable strengths of the primary energy dissipating mechanisms, i.e. the beams, as built, and the dynamic characteristics of a frame, a more consistent protection of columns may be achieved than with the use of existing global factors.

5.2 - Overall Frame Behaviour

The predicted behaviour of the 12 and 18 storey frames was seen to be very satisfactory with respect to all of the selected earthquake motions. The largest damage was indicated for the 6 storey frame, particularly during the Pacoima Dam motions. It is thought that the reason for this is primarily the relative conservatism with respect to long period structures inherent in the specified tri-linear spectrum of the New Zealand loading and design code (3). The two taller frames were rather flexible and thus they were placed well beyond the longest fundamental period of 1.2 seconds, beyond which no reduction of the base shear is permitted by the above code (3). This resulted in considerable reserve strength in comparison with the six storey frame. The use of the seismic response factor of the National Building Code of Canada (1) would have allowed a reduction by 14% and 21% in the minimum design lateral load strength in the 12 and 18 storey frames respectively, in comparison with the strength required for a frame with 1.2 seconds of fundamental period of vibration.

The vigorous shaking imposed by the Pacoima Dam motions on the 6 storey frames was surprising. The response spectra presented in Fig. 2 did not indicate this and for this reason this excitation was not considered in the initial study (17).

It is pointed out that the dynamic magnification factor, ω, of the design procedure employed, implied 28% and 80% increase of the initial column moments, derived from the elastic analyses for the specified lateral static load, for the 6 storey and the two taller frames respectively. Thus columns of the 6 storey frame possessed considerably less reserve strength. Moreover, the higher mode responses appeared to have a larger influence on the column moment pattern for this smaller frame than anticipated.

The responses to the El Centro excitation indicate that probably all frames would have been repairable. Unexpectedly, dominant higher mode shapes did not affect the critical column moments. Rather, first mode distortions of the frames were somewhat aggravated by second mode shapes that superimposed themselves. This resulted in critical moments being developed at the bottoms or the tops of columns in a number of adjacent storeys. An example of this can be seen in Fig. 1.

5.3 - Beam Behaviour

During instants of the most severe motions, beams in most storeys developed plastic hinges, even in the 18 storey frame. As stiffness degradation of the beams was not considered in the analysis, the number of simultaneously hinging beams was probably overestimated by the analysis.

The predicted plastic hinge rotations remained in all cases below 0.035 radians, a quantity considered to be readily attainable in well detailed reinforced concrete beams. Stiffness degradation in the real structure may, however, result in some increase in the displacement response and as a consequence in some increase of ductility demand.

5.4 - The Response of Columns

With the exception of the base, no column yielding was indicated by the analysis for the 12 and 18 storey frames during any of the selected ground motions. The moment envelopes indicate, however, that at least in the lower half of these frames, at some instants during the very severe excitations, the moment demand for the columns was close to the design value.

Ductility demands in column hinges, that developed in the upper storeys of the 6 storey frame during the Pacoima Dam excitation, were small. It is likely that with a small increase in column strength the likelihood of column yielding at upper storeys of this structure could be eliminated even for this extreme disturbance.

The high degree of protection against column hinging was achieved with the use of near minimum reinforcement content in relatively small columns. To comply with stipulated drift limitations (3) these columns could not have been reduced significantly in size. The flexibility of these frames is also emphasized by the fact that the computed fundamental periods, as shown in Table III, were well in excess of those predicted by customary code procedures i.e. 0.6, 1.2 and 1.8 seconds for the 3 frames respectively (1, 2).

For the taller frames the design appears to indicate unnecessary conservatism for column moments in the upper storeys.

Hinge formation at column bases points to very careful detailing for confinement, particularly for the exterior columns, where the total axial compression approached the maximum intensity considered to be acceptable (7, 14). Significant ductility demand at column hinges of the taller frames was predicted only for the Pacoima Dam excitations.

The design axial loads used are considered to be well within acceptable proximimities of those predicted by the dynamic analysis. Inevitable errors in the estimation of earthquake induced axial loads affect only the flexural capacities of the column sections which, in general, have been generously catered for by the proposed design procedure.

As intended the design procedure predicted conservatively shear demand for all columns. The conservatism did not result in unnecessary shear reinforcement.

The lightly reinforced columns of these frames, designed in accordance with these principles, are likely to result in an easing of existing construction difficulties.

5.5 - Further Developments

It is intended to examine similar frames, designed to possess lower seismic resistance, corresponding with requirements for zones of lower seismic risks, to ensure that a similar degree of protection against column hinging can be maintained.

A re-examination of low rise frames, typically 6 storeys, designed with increased dynamic magnification factors, ω, and that of taller frames with reduced ω factors for the top storeys, is indicated, to arrive at a more uniform protection of columns for the entire range of multistorey frames.

Analyses indicated that the lateral load resistance of these frames was not seriously affected by P-delta effects. However, further studies are required to examine the influence of stiffness degradation and storey drift on the inelastic dynamic response of frames, before additional specific strength requirements (23), to accommodate P-delta moments, can be recommended for incorporation into building codes.

The continuation of this study may lead to improved dynamic column moment magnification and, as a consequence, to a drastic relaxation in the requirement for the confinement of upper storey columns. Moreover, the elimination of plastic hinge formation in upper storey columns could result also in a reduction of shear reinforcement in the end regions and in improved performance of beam-column joints. It appears that the savings in transverse column reinforcement and the increased working space at end regions, would more than offset the increase in longitudinal column reinforcement that may occasionally be required.

ACKNOWLEDGMENT

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Table I Axial Load Reduction Factor R_v

Number of floors above the level considered	Dynamic Magnification Factor ω				
	1.4 or less	1.5	1.6	1.7	1.8
2	0.97	0.97	0.96	0.96	0.96
4	0.94	0.94	0.93	0.92	0.91
6	0.91	0.90	0.89	0.88	0.87
8	0.88	0.87	0.86	0.84	0.83
10	0.86	0.84	0.82	0.80	0.79
12	0.83	0.81	0.78	0.76	0.74
14	0.80	0.77	0.75	0.72	0.70
16	0.77	0.74	0.71	0.68	0.66
18	0.74	0.71	0.68	0.64	0.61
20 or more	0.71	0.68	0.64	0.61	0.57
	Use these values when $P_e/f'_cA_g = 0.7$				

Table II Moment Reduction Factor R_m

ω	P_e/f'_cA_g										
	-0.150	-0.125	-0.100	-0.075	-0.050	-0.025	0	0.025	0.050	0.075	0.100
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.1	0.85	0.86	0.88	0.89	0.91	0.92	0.94	0.95	0.97	0.98	1.00
1.2	0.72	0.75	0.78	0.81	0.83	0.86	0.89	0.92	0.94	0.97	1.00
1.3	0.62	0.65	0.69	0.73	0.77	0.81	0.85	0.88	0.92	0.96	1.00
1.4	0.52	0.57	0.62	0.67	0.71	0.76	0.81	0.86	0.90	0.95	1.00
1.5	0.44	0.50	0.56	0.61	0.67	0.72	0.78	0.83	0.89	0.94	1.00
1.6	0.37	0.44	0.50	0.56	0.62	0.69	0.75	0.81	0.88	0.94	1.00
1.7	0.31	0.38	0.45	0.52	0.59	0.66	0.73	0.79	0.86	0.93	1.00
1.8	0.30	0.33	0.41	0.48	0.56	0.63	0.70	0.78	0.85	0.93	1.00
	Tension							Compression			

Table III Principal Data for the Prototype Frames Studied

Data		6 Storeys	12 Storeys	18 Storeys
Spacing of Frames	(m)	5.50	9.20	9.20
Span of Beams	(m)	5.50	9.20	9.20
Floor Heights	(m)	3.35	3.65	3.65
Total Height	(m)	20.10	43.80	65.70
Total Seismic Weight	(kN)	2908	18312	31803
Estimated Period	(sec)	0.70	1.75	2.34
Computed Period	(sec)	0.79	1.88	2.31
Total Base Shear	(kN)	291	1100	1908
Size of	1 - 3 floor	600 x 350	900 x 400	1000 x 550
Beams	4 - 6 floor	580 x 350	900 x 400	1000 x 550
(mm x mm)	7 - 8 floor		850 x 400	950 x 550
	9 floor		800 x 400	950 x 550
	10 - 12 floor		800 x 400	900 x 500
	13 - 15 floor			850 x 450
	16 - 18 floor			800 x 400
Size of	1 - 3 floor	500 x 450 (1.0%)	775 x 500 (1.2%)	1000 x 650 (1.0%)
Exterior	3 - 6 floor	450 x 450 (1.0%)	775 x 500 (1.2%)	1000 x 650 (1.0%)
Columns	6 - 8 floor		750 x 500 (1.4%)	800 x 650 (1.2%)
(mm x mm)	8 - 9 floor		650 x 500 (1.4%)	800 x 650 (1.3%)
and Steel	9 - 12 floor		650 x 500 (1.0%)	750 x 650 (1.3%)
Content	12 - 15 floor			700 x 650 (1.2%)
(ρ_t %)*	15 - 18 floor			650 x 650 (1.0%)
Size of	1 - 3 floor	550 x 550 (1.0%)	800 x 800 (1.0%)	1000 x 1000 (1.0%)
Interior	3 - 6 floor	500 x 500 (1.0%)	800 x 800 (1.1%)	1000 x 1000 (1.0%)
Columns	6 - 8 floor		725 x 725 (1.6%)	1000 x 1000 (1.0%)
(mm x mm)	8 - 9 floor		675 x 675 (1.4%)	1000 x 1000 (1.0%)
and Steel	9 - 12 floor		675 x 675 (1.0%)	900 x 900 (1.1%)
Content	12 - 15 floor			800 x 800 (1.4%)
(ρ_t %)*	15 - 18 floor			700 x 700 (1.1%)
Slab Thickness	(mm)	120	160	160
Concrete Strength	(MPa)	28	28	35 (28**)
Acceleration Records Used		El Centro Parkfield Pacoima Dam	El Centro Pacoima Dam	El Centro Pacoima Dam A2

* Total longitudinal reinforcement as an approximate percentage of the gross column sectional area.

** Above 7th floor

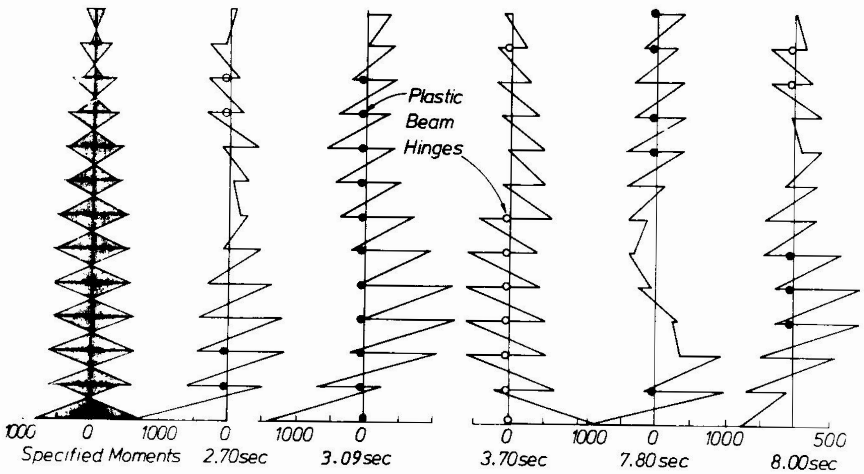

Fig. 1 - A comparison of bending moment patterns for an exterior column of a 12 storey frame encountered at instants of the Pacoima earthquake motions with moments derived from code specified lateral static loading.

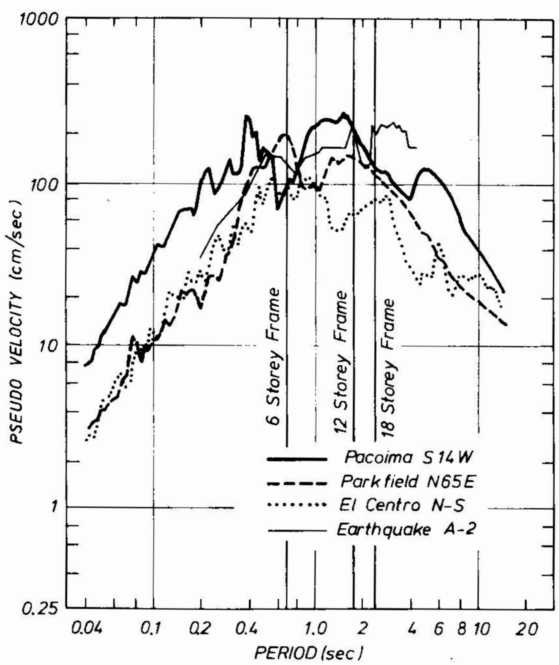

Fig. 2 - Pseudo-velocity response spectra for earthquakes used in this study.

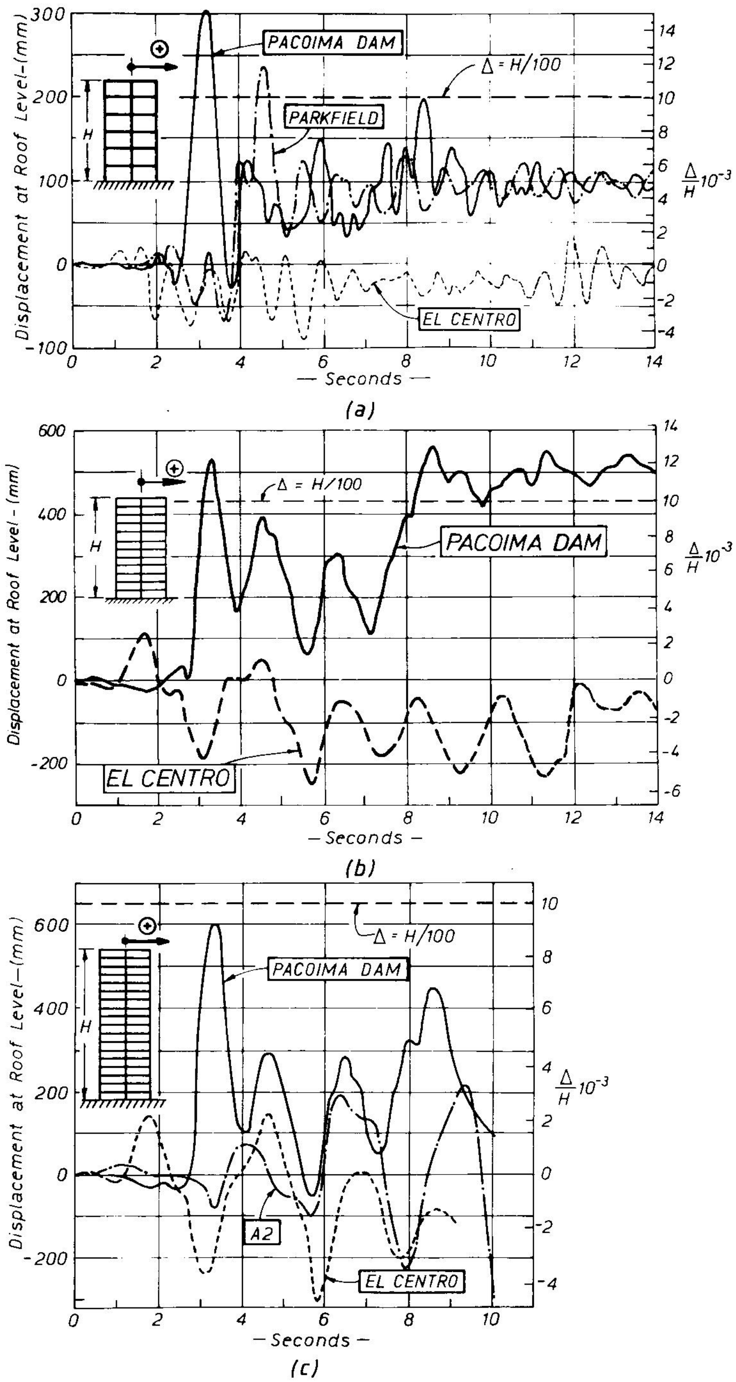

Fig. 3 - Horizontal displacements at roof level during the selected earthquake motions for the (a) 6 storey, (b) 12 storey and (c) 18 storey frames.

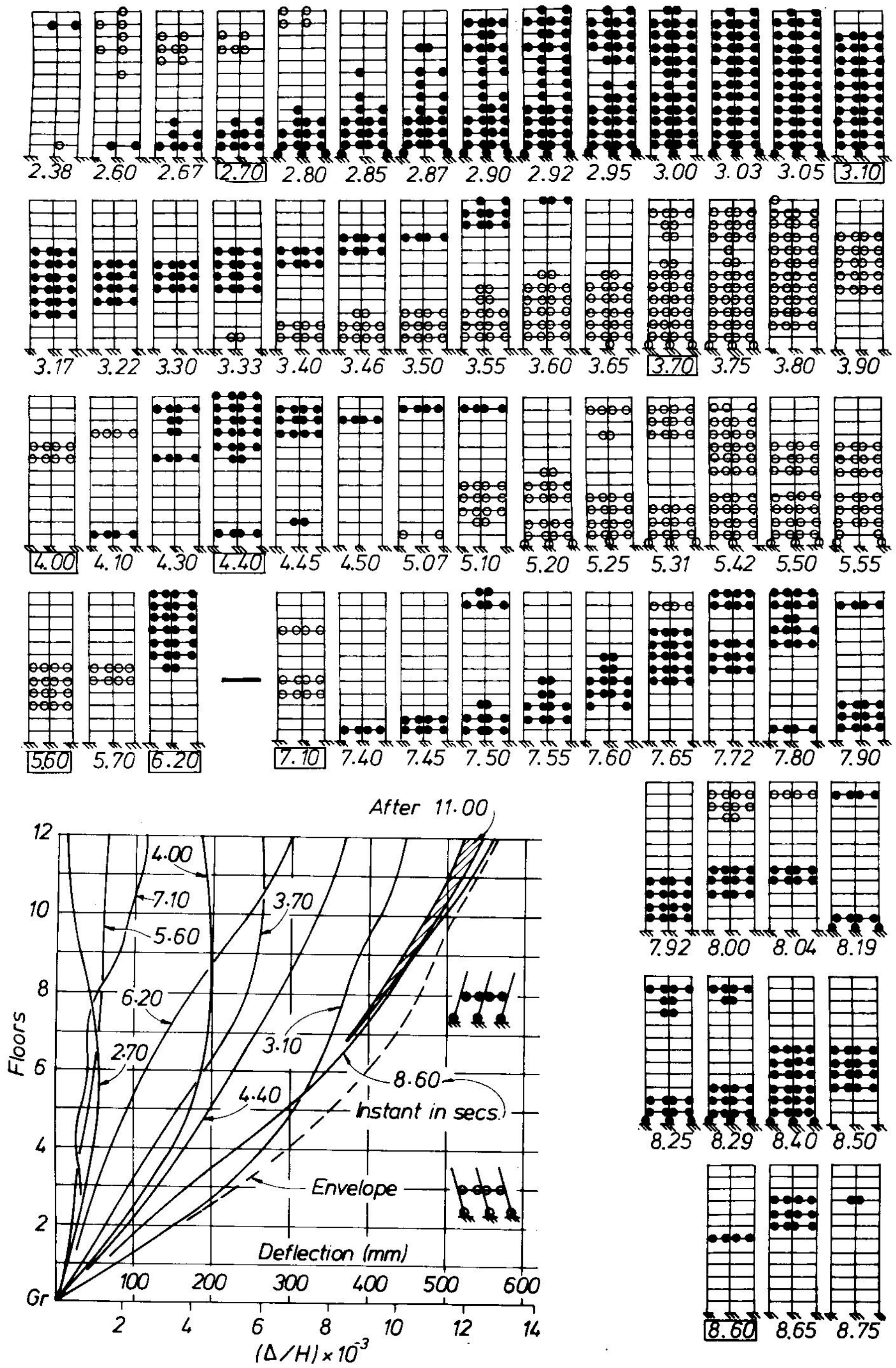

Fig. 4 - Plastic hinge developments in and storey deflections of the 12 storey frame during the Pacoima Dam earthquake motions.

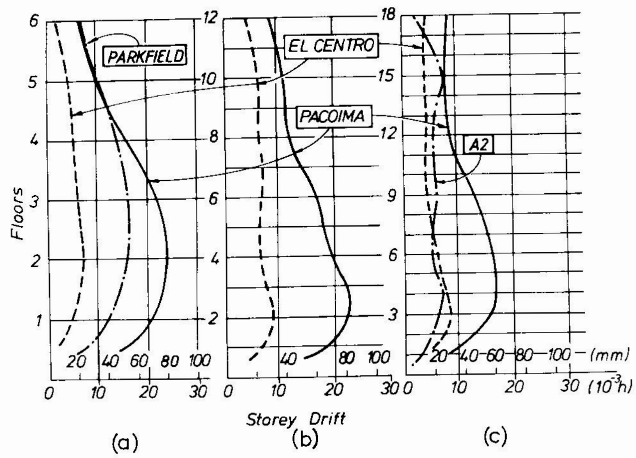

Fig. 5 - Interstorey drift envelopes for the (a) 6 storey (b) 12 storey (c) 18 storey frames.

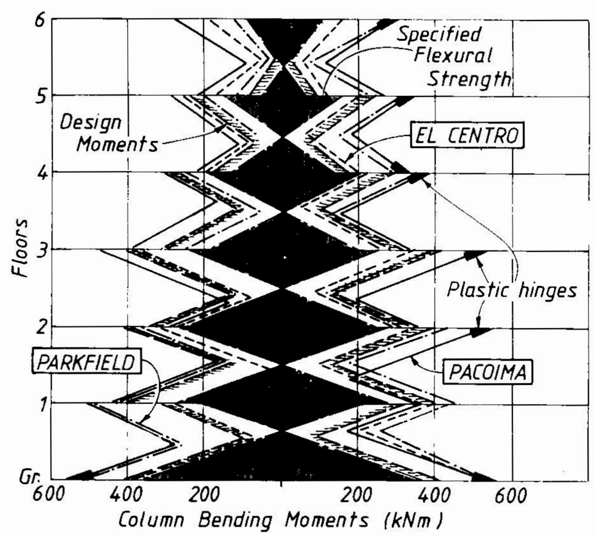

Fig. 6 - Bending moment envelopes for the interior column of the 6 storey frame.

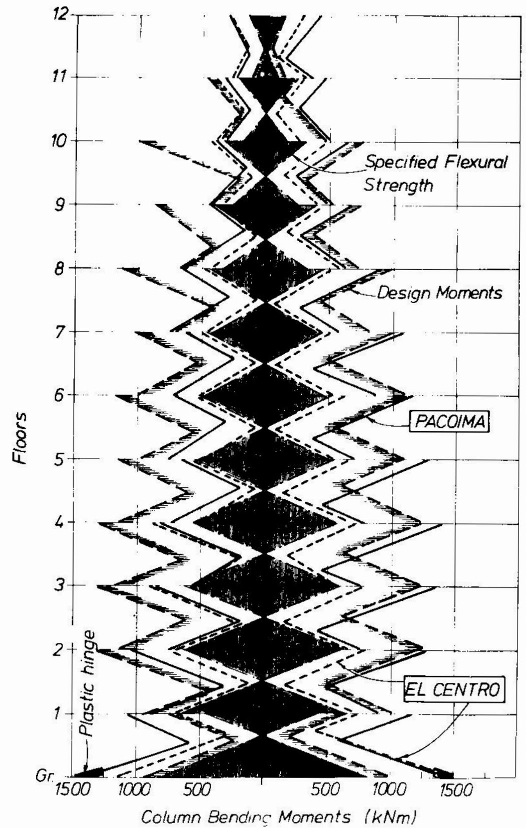

Fig. 7 - Bending moment envelopes for the exterior column of the 12 storey frame.

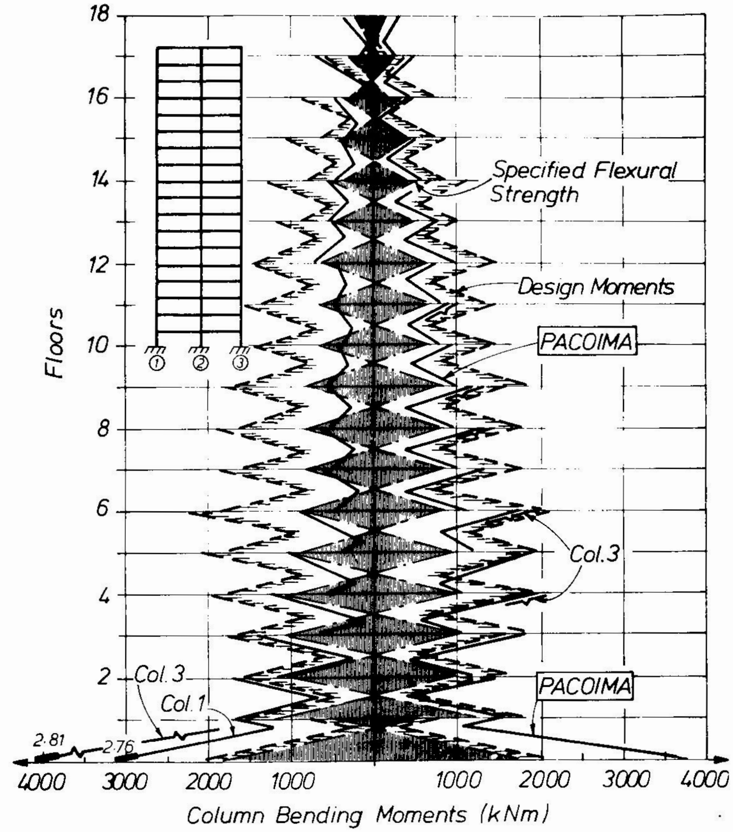

Fig. 8 - Bending moment envelopes for the exterior columns of the 18 storey frame.

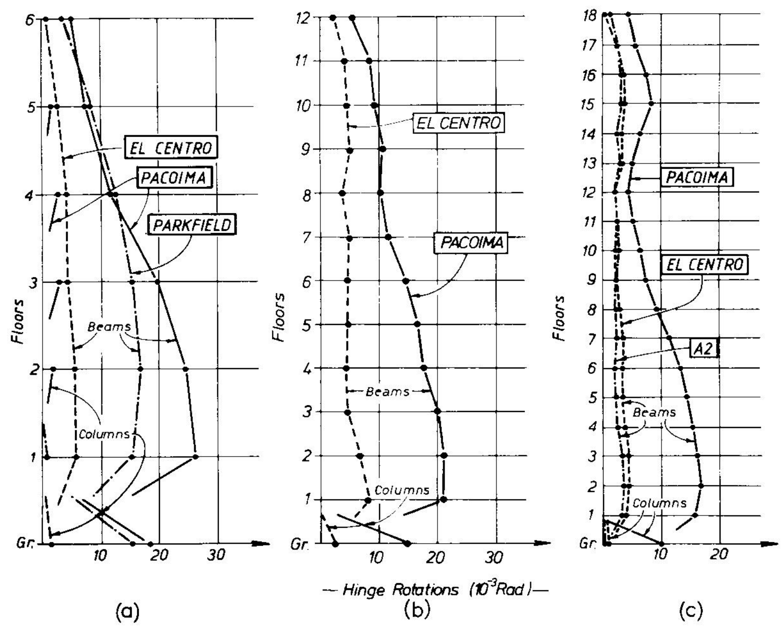

Fig. 9 - The distribution of maximum plastic hinge rotations in beams and columns of the (a) 6 storey (b) 12 storey (c) 18 storey frames during the selected earthquake motions.

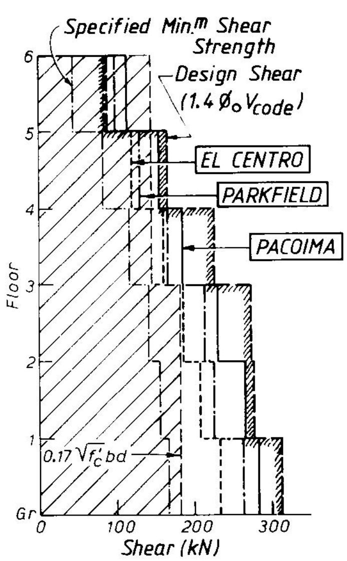

Fig. 10 - A comparison of envelopes for the shear force across the interior column of the six storey frame.

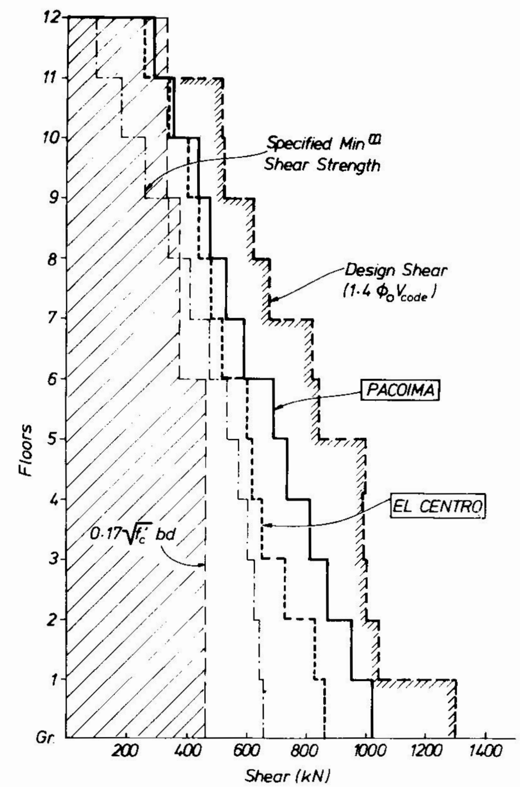

Fig 11 - A comparison of envelopes for the shear forces across the interior column of the 12 storey frame.

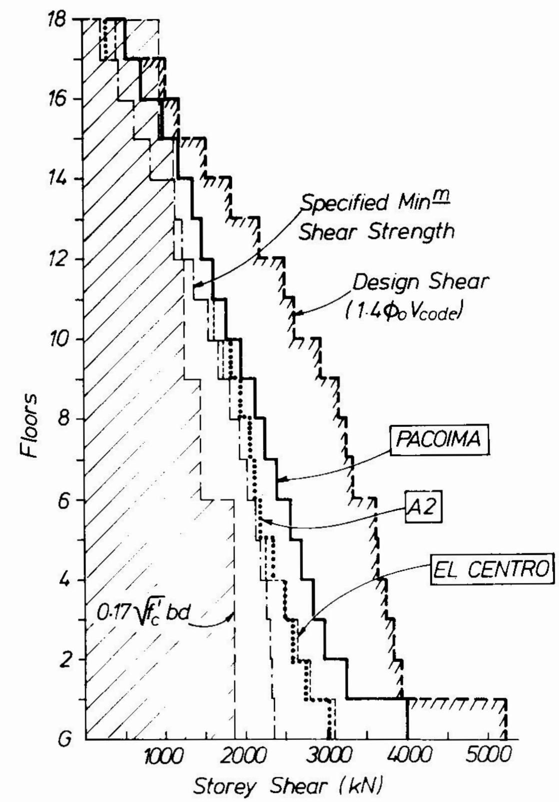

Fig. 12 - A comparison of envelopes for the sum of the shear forces across all three columns of the 18 storey frame.

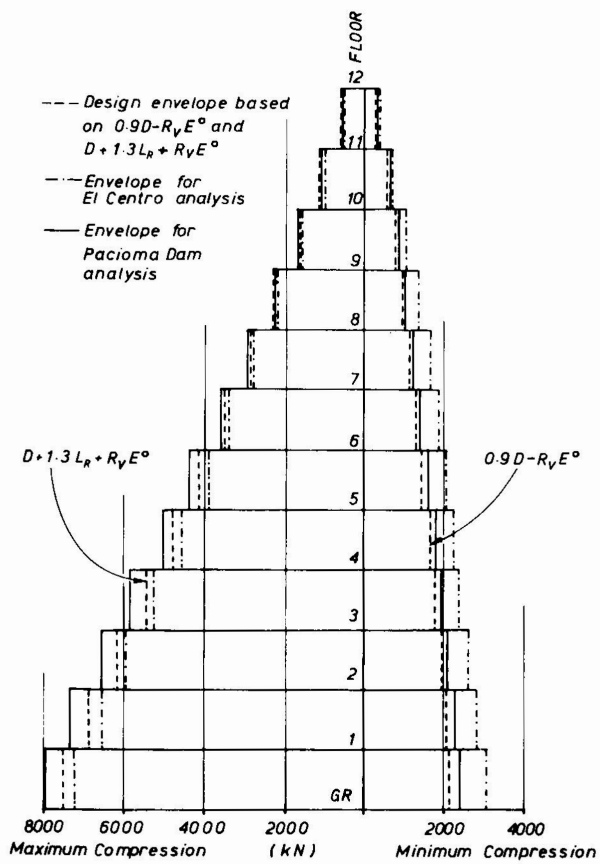

Fig. 13 - Envelopes for the maximum and minimum axial compression forces in the exterior column of the 12 storey frame.

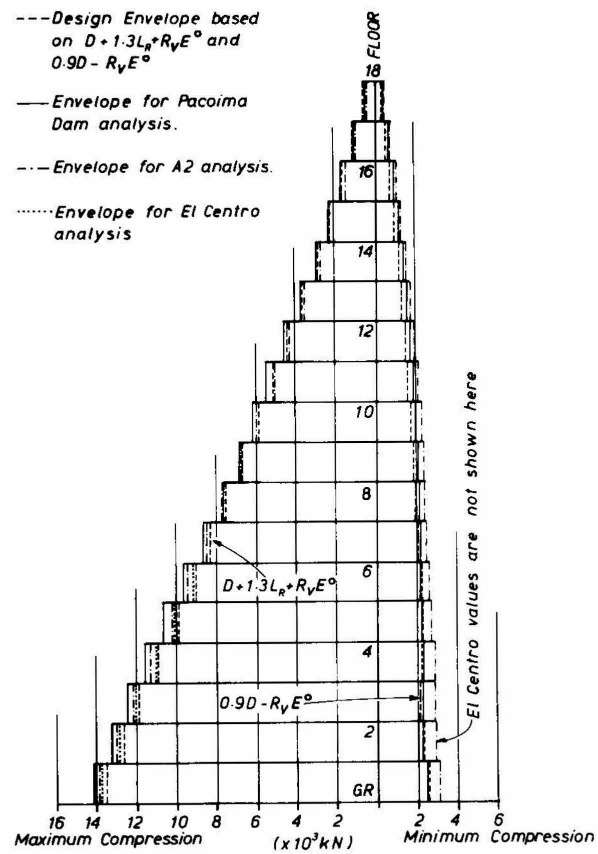

Fig. 14 - Envelopes for the maximum and minimum axial compression forces in the exterior column of the 18 storey frame.