ENG 236-(EQC 1991/)

Seismic Response of Multi-Storey Buildings

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ISSN 0111-0136

UNIVERSITY OF AUCKLAND

SCHOOL OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

REPORT No. 495

THE SEISMIC RESPONSE of MULTI-STOREY BUILDINGS

by
R.C.Fenwick and B.J.Davidson

September 1991

PRIVATE BAG, AUCKLAND, NEW ZEALAND

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School of Engineering
Report No. 495

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ABSTRACT

The draft loading codes, DZ4203, produced in 1986 and 1988, specified different response spectrum curves for different values of structural ductility factor. This increased the complexity of the design as an analysis based on an elastic response spectrum, which is required for the serviceability limit state, could not be scaled for the ultimate (severe seismic) limit state.

To reduce this complexity a change in the shape of the response spectra used for modal analysis of ductile structures is proposed. The effects of changing the spectra style is investigated in analyses of a series of elastic analyses of walls and frames. The results of these analyses are compared with time history analyses of these structures for a number of different ground motions.

The predictions of the equivalent static, modal response spectrum and modal equivalent static methods of analysis are compared for a series of regular wall and frame structures. From these it was concluded that the equivalent static method of analysis is acceptable for regular structures providing the fundamental period does not exceed 2 seconds. For other structures the modal response spectrum method is recommended.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support provided by the Earthquake and War Damage Commission for this project. We also wish to acknowledge the contributions made by Dr. A.G. Gillies, of Lakehead University, Ontario, Canada and of Dr. V.K.M. Lam in carrying out the analyses reported in this project.

Very sincere thanks are also extended to the Department of Civil Engineering of the University of Canterbury for the help provided in preparing this report. Particular thanks are due to Mrs. V. Gray, who traced the many diagrams in this report and to Mrs. L. FitzGibbon who did most of the word processing.

We thank also Miss Eileena Sucich who patiently made corrections, and juggled the pages and figures to obtain the final format of this report.

CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 - BACKGROUND AND SCOPE OF REPORT

1.1 Background to Project

The shape of the response spectra used to determine seismic actions in the current loadings code[1] and the proposed code[2,3] differ markedly, as is illustrated in Fig. 1.1. There are two aspects to the changes which need to be noted.

(1) Compared with the existing loadings code the draft code requires a large increase in the seismic forces to be used in design for structures with short periods. However, for structures with long fundamental periods it allows considerably smaller design forces to be used.

(2) The shape of the response spectra in the draft code varies with the structural ductility factor selected for the design.

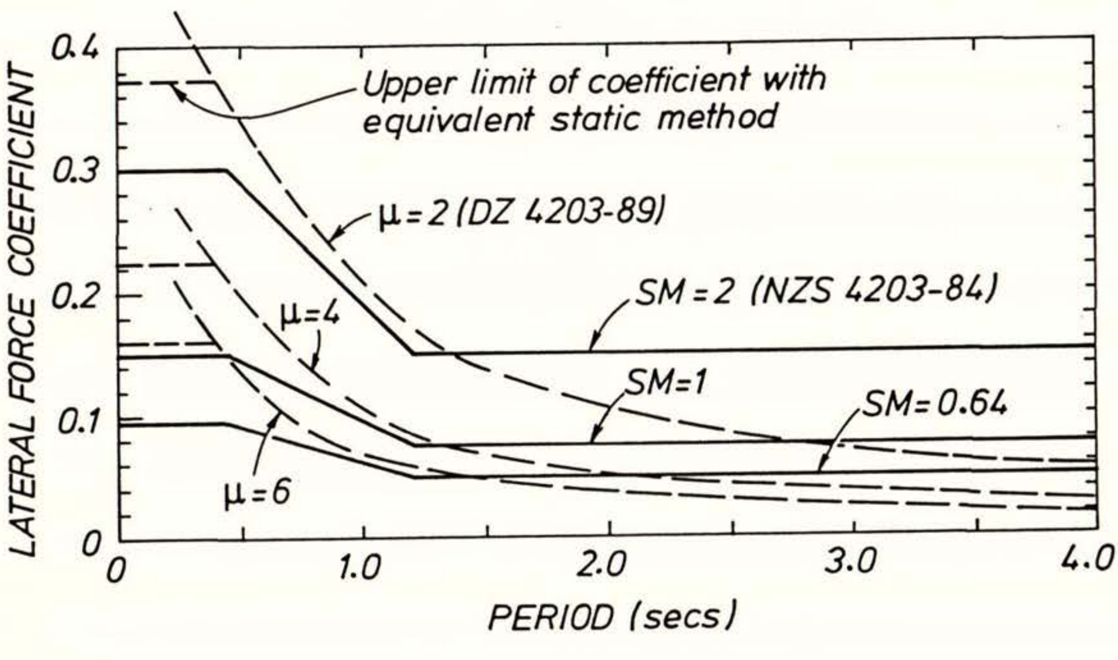

Fig. 1.1 Comparison of response spectra in NZS 4203-84 and DZ 4203-89 for the Wellington Region.

In this report the primary concern is with the second of these aspects, that is the change in shape of the response spectra with the structural ductility factor.

In the proposed code[3], the response spectra are used as part of two basic methods of analysis to obtain seismic design forces; the equivalent static method, and the model response spectrum method. The first step in the modal response spectrum analysis of a structure is to determine the modes of vibration and participation factors. The response for each mode is then derived from the spectral coefficient defined in the design spectrum. The most probable response of the structure may then be determined by a combination of the individual modal contributions. These may be combined by the "square root of the sum of the squares" unless the periods of the modes are close, in which case some other technique such as the CQC method is required.

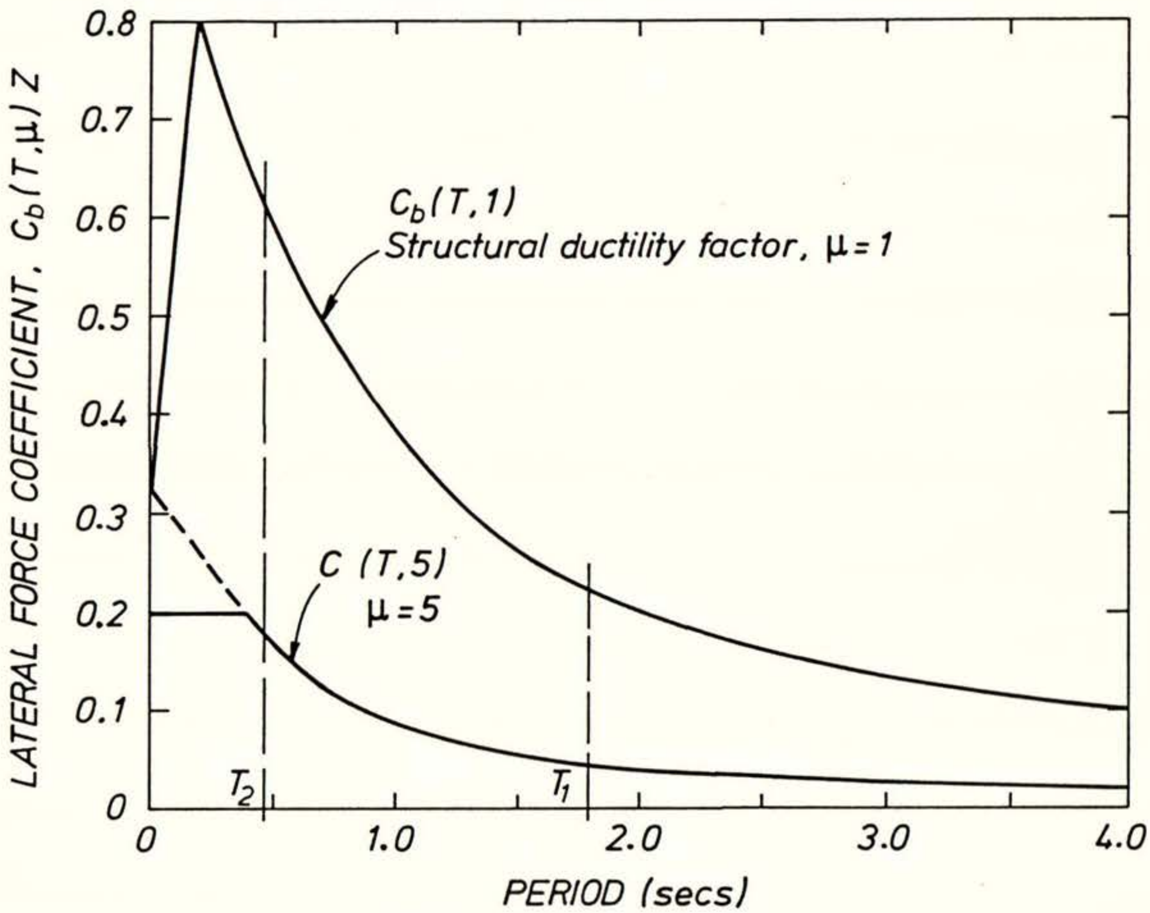

Fig.1.2 A comparison of response spectra for elastic response ($\mu = 1$) and ductile response ($\mu = 5$) as given in the draft code (DZ 4203-89).

In Fig. 1.2 the response spectra for structural ductility factors of $\mu = 1$ and 5 are shown for normal soils as given in the draft code[3]. It is evident from these two curves that linear scaling does not exist between them. The ratio of the response coefficient at a short period (T_2) to that at a long period coefficient (T_1) is higher for the $\mu = 5$ spectrum than for the $\mu = 1$ spectrum. If these spectra are retained for the new code it means that "ductile" structures must be designed for a greater relative contribution of higher mode effects than "elastic" structures. It also implies that a minimum of two modal analyses would be required in the design, the first for the serviceability limit state, which should be based on the $\mu = 1$ (elastic) spectrum and the second for the ultimate (severe seismic)* limit state, which should be based on the response spectrum corresponding to the appropriate structural ductility factor. It also should be recognised that the use of a ductile response spectrum in the draft code represents a substantial change from the existing code (NZS 4203-84) and if adopted would lead to some change in the distribution of the relative strength provided over the height of the structure. In particular, there would be some increase in strength provided in the upper storeys compared with the lower storeys due to the increased higher mode contributions.

A review of a number of overseas codes, such as ATC-03[4], UBC-88[5], Eurocode- 8[6], SEAOC-90[7] and the Canadian National Building Code[8], has provided no support for the proposal described in the draft loadings code[3]. In all these, the response spectra for ductile design are obtained by linear scaling of the elastic spectrum. This approach seems to be followed in all current seismic design codes[9].

This report proposes an alternative spectrum shape for the new loadings code, in which the response spectrum for a ductile structure is taken as a scaled version of the elastic spectrum, with the scale factor being the ratio of coefficients for the first mode for the structural ductility factor (μ) to the corresponding value for the elastic response. Hence the scale factor, SF, is given by -

$$SF = \frac{C_0(T_1, \mu)}{C_0(T_1, 1)} \tag{1.1}$$

With this scale factor the proposed spectrum is consistent with the equivalent static force procedure and the smaller reduction factor appropriate to structures with fundamental periods of less than one second is accommodated. This proposed spectrum has two advantages over the spectra in the draft code. Firstly, structures where the ductility factor is greater than one are not penalised in the ultimate limit state by a proportionally greater "higher mode" response than an elastic structure and secondly, as the elastic and ductile response spectra are the same shape the results of an elastic analysis carried out for the serviceability limit state can be scaled to give the appropriate values for ultimate limit state.

The proposed spectrum for a structural wall, which has a fundamental period of 1.8 seconds, is illustrated in Fig. 1.3.

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* The term "severe seismic" was introduced in the second draft code. This may be replaced by the terms "ultimate" or "strength" in the new loadings code.

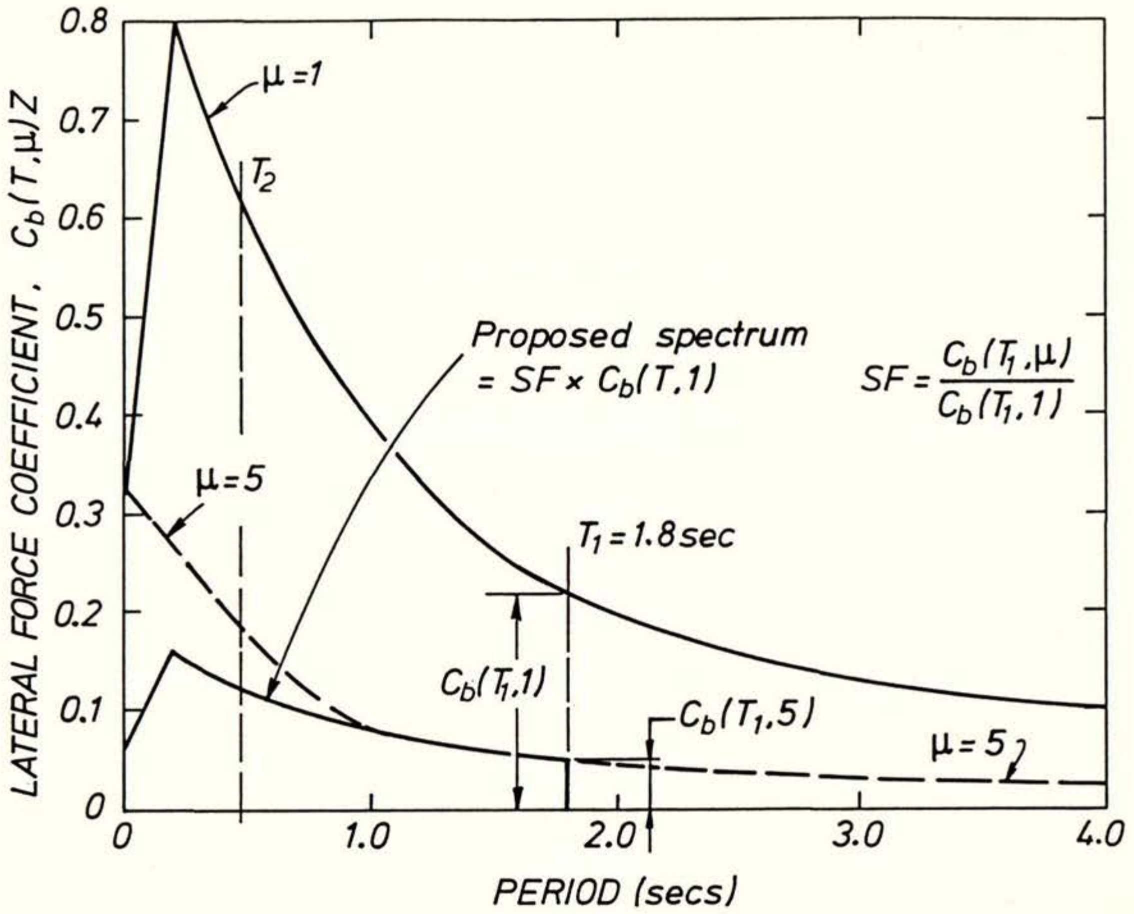

Fig. 1.3 Proposed response spectrum, illustrated for a structural wall with a fundamental period of 1.8 seconds and a structural ductility factor of 5.

1.2 Methods of Analysis for Seismic Actions

1.2.1 General

A number of different methods of analysis are currently used for the design of multi-storey structures. The background to these is briefly outlined in this section. The step-by-step time history analysis using seismic ground motion is not included here as it is believed that this technique is principally suited to checking the adequacy of a structure which has already been designed using some other method.

1.2.2 Equivalent Static Method

With this approach the base shear, which is equal to the sum of the lateral seismic design forces, is given by the expression -

$$V = C_b(T_1, \mu) W_t \tag{1.2}$$

where $C_b (T_1, \mu)$ is the acceleration ordinate, expressed as a proportion of gravity, of a response spectrum whose value depends on the fundamental period of the structure T_1, the ductility (structural ductility factor (μ)) and the geotechnical characteristics of the site. The value of W_t is taken as the total seismic weight of the structure above the base level.

The distribution of the seismic design forces applied at each level should ideally be proportional to the absolute of acceleration the level sustains in an earthquake. If it is assumed that the fundamental mode of vibration dominates then the maximum acceleration of any level is proportional to the maximum deflection sustained by that level. By assuming that the lateral deflections are proportional to the height above the base it follows that the seismic design force, F_x, at level x is given by:

$$F_x = V \frac{W_x h_x}{\sum_1^n (W_i h_i)} \tag{1.3}$$

where W_x is the seismic weight and h_x is the height of level x above the base. This form of expression, first adopted in the San Francisco code in 1956, is used in many codes of practice.

A number of modifications to the equivalent static method have been proposed as a result of theoretical studies. In multi-storey structures it has been found that the storey shears in the upper levels of the building corresponding to the forces defined by Eq. 1.3, are low when compared with the values found in a modal response spectrum analysis. This is due to the significant contribution of the higher modes to the storey shears in this region of a structure. To improve the accuracy of the equivalent static method it has been suggested that a portion of the total design lateral force be concentrated at the top of the building. This procedure was adopted in the UBC[5] code, where this force, F_t, which is applied to the top level, increases with the fundamental period of the structure. Its value is given by:

$$F_t = 0.07\ V T \tag{1.4}$$

The upper limit to F_t is taken as 0.25 V. With this modification the seismic design force at level x is given by

$$F_x = (V - F_t) \frac{W_x h_x}{\sum (W_i h_i)} \tag{1.5}$$

and at the highest level of the structure the lateral force F_t is added to the value given by Eq. 1.5.

A number of simple modifications to Eqs. 1.3 and 1.5 have been adopted in some codes in an attempt to provide a better match between the distribution of forces and the observed deflected shapes. For example in the ATC3-06[4] recommendations the lateral forces are given by:

$$F_x = V \frac{W_x h_x^k}{\sum_1^n W_i h_i^k} \tag{1.6}$$

where the value of k depends upon the fundamental period. Where this is less than 0.5 seconds it is taken as 1.0, but when T_1 is equal to or exceeds 2.5 seconds it is taken as 2.0. Interpolation may be used for intermediate values. The Indian code[9] assumes k is equal to 2 for all periods.

In some codes the deflections found from an initial set of forces defined by an equations such as Eqs. 1.4 and 1.5 may be used to calculate a more accurate set of design forces. With this approach the revised force, F_x, at level x is given by:

$$F_x = (V - F_t) \frac{W_x \delta_x}{\sum_1^n (W_i \delta_i)} \tag{1.7}$$

At the top level, F_t, given by an equation such as Eq. 1.4, is added to the value given by Eq.1.7. In this equation δ_i is the lateral deflection of level i found from the initial set of forces. The revised set of forces are based on the concept that the maximum acceleration in a given mode is proportional to the maximum deflection at that level.

The equivalent static method of analysis predicts larger overturning moments and deflections than the corresponding values found from the modal response spectrum method. This phenomenon can be explained in terms of the proportions of the mass included in the analysis. As previously noted the equivalent static method assumes that the total seismic mass of the structure vibrates in a fundamental mode. From the modal response spectrum theory, the proportion of the total mass of a structure, p, which acts in any mode is given by -

$$p = \frac{\sum_1^n (W_i \delta_i)^2}{\left[\sum_1^n W_i \delta_i\right]^2} \tag{1.8}$$

For a multi-storey building, which has a uniform mass distribution and a first mode shape which increases linearly with height above the base, the value of p is 0.75.

For this case the equivalent static method can be seen to overestimate the first mode contributions by approximately 1/0.75, or 1.33. Thus actions which are dominated by the first mode contribution, such as the deflections and the overturning moments near the base of the structure tend to be overestimated by a factor of 1.33 when calculated using the equivalent static method. In the upper storeys of multi-storey buildings where the second and higher modes contribute to the storey bending moments and interstorey deflections, this trend may not be so apparent[10]. A number of correction factors have been introduced in codes to compensate for this discrepancy. For example, in the New Zealand loadings code[1] (NZS 4203) the ultimate interstorey and lateral deflections are scaled from the values found in an elastic analysis to allow for the effects of inelastic deformation. Where an equivalent static method is used the scale factor is 0.91 times the corresponding scale factor used with a modal analysis is applied. The ATC3-06[4] and a number of other codes of practice[9] compensate for the overestimate of the storey bending moments, M_x, found by the equivalent static method by introducing a reduction factor into the equilibrium equation, such that

$$M_x = k \sum_{1}^{x} F_i (h_i - h_x) \tag{1.9}$$

where k = 1.0 for the top 10 storeys,
= 0.8 for the 20th storey and below and intermediate values may be found by interpolation.

The use of the equivalent static method in different codes of practice is generally limited to structures which are reasonably regular in both plan and elevation and satisfy either a stated height limitation or their estimated fundamental period is less than a critical value. The ATC3-06[4], UBC-88[5] and Eurocode 8[6] codes contain criteria which enable a structure to be classified as regular, in which case the equivalent static method may be used, or irregular in which case a response spectrum approach is required. The ATC3 and UBC codes give height limits for the equivalent static method, while Eurocode 8 limits the approach to structures which have a fundamental period of 0.8 seconds or less for structures founded on rock, and to 1.6 seconds for structures on soft foundations.

1.2.3 Modal Response Spectrum Method

In this method the different modes of vibration of a structure are considered. For each one of these a single degree of freedom oscillator, which has the same period, T_i, and the same effective weight, W_{ei}, can be envisaged. From the design response spectrum the maximum base shear for each of these oscillators, V_i, is given by:

$$V_i = C\,W_{ei} \tag{1.10}$$

where C the response spectrum may be $C_b(T, \mu)$ .Z.R*, as defined in DZ 4203-90, or SF x $C_b(T, 1)$ .Z.R, as proposed in this study. From the base shears for each of the modes the corresponding structural actions such as the bending moments, shears, deflection and accelerations of each level can be found.

While this step in the analysis is straight forward a number of practical problems arise in combining the individual modal responses to obtain a design value. The first point which needs to be identified is how many modes need to be considered in the design of a structure. A number of different criteria have been used for the response in a particular direction as indicated in the following:

(i) All the modes with a period longer than a specified minimum value should be considered.
(ii) A specified number of modes should be considered.
(iii) The number of modes considered should be such that the sum of the masses participating in each of the modes equals or exceeds 90% of the total mass of the structure.

The last criterion, which is contained in the UBC[5] and a number of other codes of practice[9], appears to be the most reliable. The first two criterion do not always identify critical modes in irregular structures, (such as a tower with a podium)[10]. No scaling is required to account for the missing 10 percent of mass as it has been found that introducing the additional modes makes no significant difference to the resultant values[10].

The second practical problem arises in the method of combination of the different modal contributions to give a design value. The maximum response of each mode in a structure for a given earthquake ground motion occurs at a specific time, which differs from that of all other modes. Consequently the numerical sum of all the modal values would be unrealistically high in most cases. On the basis of statistical theory it is generally recognised that the maximum probable value for any structural action (bending moment, shear, deflection or interstorey deflection) is given by the square root of the sum of the squares (SRSS) of the individual values in the different modes, provided the mode periods are well separated[11, 12, 13, 14]. Where the modes are close, such that T_{i-1}/T_i is more than 0.9, some other appropriate combination should be used (such as the CQC technique).

The combination of the individual mode values by the SRSS or other appropriate technique has been adopted by many codes[9] including UBC-88[5], ATC3[4], Eurocode 8[6], and NEHRP-85[15].

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* Z and R are Zone and Risk factors respectively.

The values obtained by combining the modal values for some action, such as the bending moment along a member, gives an envelope of the most likely maximum values to be sustained in an earthquake that has the design response spectrum. The envelope values are not sustained simultaneously and consequently individual values cannot be assumed to be in equilibrium with other values. For example, the envelope shears cannot be integrated to give the design bending moments.

Often when a model response spectrum analysis is made for a frame structure the combined base shear is found to be less than the corresponding value found in an equivalent static analysis. Where this occurs many codes of practice[9] require the base shear, together with the corresponding structural actions to be scaled. For example the UBC-88[5] code requires these values to be scaled to the corresponding percentage of the equivalent static base shear given below:

(a) for irregular buildings, 100 percent, and
(b) for regular buildings, where the fundamental period has been calculated from the Rayleigh (or equivalent method), 90 percent.

Where the base shear found in the analysis exceeds the corresponding equivalent static method the code permits the values to be scaled down. For certain structures, such as slender walls, this can appear to lead to unconservative design values[10].

1.2.4 Modal Equivalent Static Method

This method, which was adopted in the First Draft Code[2], appears to have come from a suggestion made by Newmark and Rosenblueth[16]. A modal response spectrum analysis is carried out to find the modal storey shears and these are combined by the SRSS, or other appropriate method such as the CQC technique, to give the seismic design storey shears. By finding the differences between these an equivalent set of static forces can be found, which when applied to the structure generates the storey shear envelope. The set of structural actions found by applying this set of forces to the structure, is used for the design. The advantage which is seen for this approach is that the structural actions are all consistent and in equilibrium with each other.

The modal equivalent static method violates the principles of structural dynamics in that it assumes that all the maximum shears are sustained in one direction simultaneously. Since the design bending moments are effectively found from the integration of the storey shear force envelope, this incorrect assumption leads to an overestimate of the storey bending and deflections[14,16,17]. For frame structures, which behave in a shear mode (that is, the deflection arises predominantly from the storey shear rather than the storey bending moment), the values of principal concern in the design are the shears and bending moments in the beams and columns and the interstorey deflections. These depend mainly on the magnitude of the storey shear and consequently the modal equivalent static approach generally gives acceptably accurate values for these quantities. However, it overestimates the lateral defections and the axial forces induced in the columns. With structures which behave in a flexural mode, such as braced frames, structural walls or frames with closely spaced columns, significant errors usually arise in the storey bending moments and the deflections. While these values are on the high side of the corresponding modal response spectrum values they are not necessarily conservative. For example, when used as a basis for capacity design the storey shear force to storey bending moment ratios are too low. A wall designed on the basis of such an analysis might fail in shear in a non-ductile mode due to this discrepancy.

With the modal equivalent static method of analysis the error in the lateral deflections is considerably greater for the flexural structures (walls etc) than it is for the shear type structures (frames). Consequently where a building contains both walls and frames the distribution of the actions between the two different types of lateral force resisting elements is incorrect.

The modal equivalent static method of analysis used in the NZ code[1] is not permitted in any of the major overseas codes of practice[9]. This includes UBC-88[5], ATC3[4], NEHRP-85[16], Eurocode 8[6], and SEAOC[7] codes.

1.3 Scope of Project

In this project a series of frames and walls were analysed by different methods using both the response spectra given in the draft code and the modified spectra for ductile structures as proposed in the previous section. There were two aims with these analyses:-

(i) to allow the effect of changing the response spectra to be ascertained, and
(ii) to allow the influence of using the different methods of analysis to be assessed.

The frames and walls were given the strengths required on the basis of a response spectrum modal analysis using the proposed response spectra. These were then modified so that the structures complied with the generally accepted requirements for capacity design. Time history analyses were then carried out for three different earthquake ground motions to allow their performance to be assessed and to see if there was any indication which of the response spectra should be used in design.

In chapter two the details of structures considered are described together with the methods of analysis and the details relating to the time history analyses. In chapter three the results of the different methods of analysis with the different response spectra are given, while in chapter 4 the time history results are compared with the values predicted in the analyses. Chapter 5 contains the conclusions to the study.

CHAPTER 2 - STRUCTURES ANALYSED AND METHODS OF ANALYSIS

2.1 Dimensions of Structures

A number of structural concrete frames and walls sized to provide the lateral resistance for a series of multi-storey buildings were analysed in this project. Except for a few minor changes the structures were the same as those used in a previous project[10]. The floor plan for the buildings was kept constant, as is shown in Fig.2.1. The analyses were restricted to seismic attack in the x direction. The internal frames 2, 3, and 4 were proportioned to support the gravity loads but to be flexible with respect to lateral forces. With this arrangement the floors, which act as rigid diaphragms, distribute the lateral seismic forces to the two perimeter frames located on lines 1 and 5. The gravity loads acting on the beams in the external frames were for the purpose of this project assumed to be negligible. This assumption was made to avoid any complications associated with the redistribution of gravity actions in a severe earthquake[18].

For the wall structures, the perimeter frames on lines 1-1 and 5-5 were replaced by two uncoupled pairs of structural walls.

The interstorey height for all storeys was kept constant at 3.4m. The seismic weight associated with each level was taken as 3 400 kN. This gave a seismic weight of 1 700 kN at each level for each perimeter frame and of 850 kN for each wall.

In this project the emphasis was on the results predicted by different methods of analysis. For this reason a number of assumptions were made to simplify the analyses as far as possible and to prevent differences in the analyses from being disguised by nominal design requirements. The assumptions relating to the member dimensions and strengths are outlined in the following list.

(1) The member sizes are selected to satisfy the seismic related requirements given in the draft codes[2,3]. The corresponding values resulting with wind forces were not considered. In practice these could be expected to determine the minimum strength requirements for the higher structures.

(2) The torsional component of the seismic forces specified in the draft code[3] was not considered. This enabled the walls and frames to be analysed as two dimensional assemblages.

(3) For the inelastic numerical integration time history analyses of the frames, the initial flexural yield strengths of the potential plastic hinge zones in the beams were taken as the corresponding combined bending moments found in the modal response spectrum analysis using the proposed spectrum. No allowance was made for minimum strength levels, which might in practice be required as a result of the minimum steel contents specified in the concrete code[19].

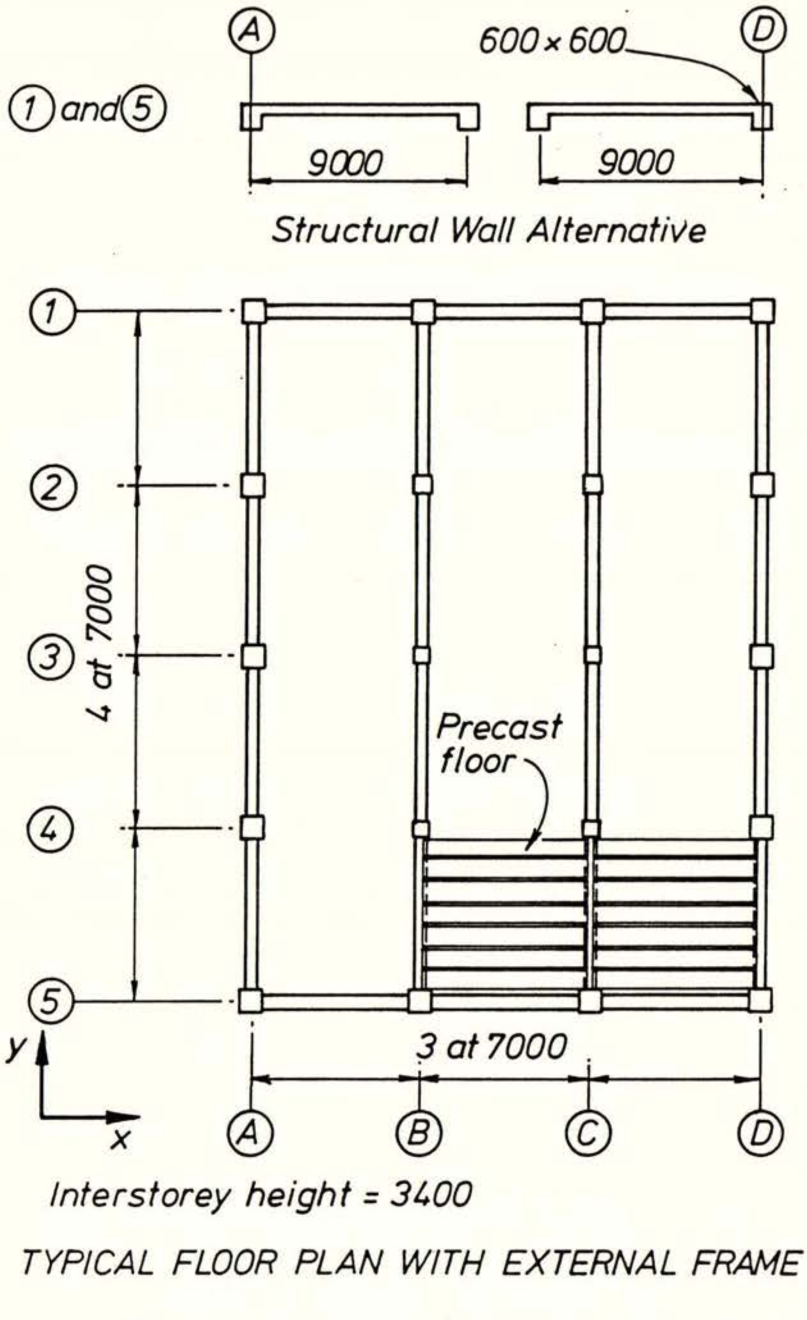


Fig. 2.1 Floor plan for multi-storey frame and wall structures.

(4) The initial flexural yield strengths used at ground level in the frame columns were taken as 1.4 times the corresponding bending moments found in the modal response spectrum analysis. As the flexural strengths given to the beams corresponded to the dependable strengths, the column values could have more correctly been taken as 0.9 x 1.4 times the modal value. The 1.4 factor is the multiplier that is suggested for calculating the ideal strength associated with capacity design procedure given in the commentary to the NZ concrete code[19]. No attempt was made to model the interaction between axial load and flexure in these hinge zones. The flexural strength in the remainder

of the columns was set at a sufficiently high level to ensure that they remained elastic.

(5) The initial flexural yield strengths used at the base of the walls, M_b, was taken as the combined modal bending moment at this location found in the modal response spectrum analysis using the proposed spectrum.

(6) The flexural strength envelope used for the walls was taken on the conservative side of that proposed in the commentary to the NZ concrete code[19], and an allowance, which is not included in the code, was made for the anticipated increase in flexural resistance at the base of the wall for strain hardening effects. In the commentary the strength envelope is defined by offsetting a line drawn between the required flexural strength at the base of the wall, M_b, to a value of zero at the top of the wall by one wall length up the wall. This is illustrated in Fig. 2.2(b). For these analyses the required flexural strength at the base of the wall, M_b, was multiplied by 1.1 to give the anticipated maximum value that would be sustained with strain hardening. A line joining this point (1.1 M_b) to the top of the wall was drawn and then offset by half a wall length up the wall. This line provided one limit to the strength envelope. The other limit is the flexural strength M_b. The resultant envelope is illustrated in Fig. 2.2(c).

The offset of one wall length used to obtain the design bending moment envelope in the commentary in the code is to allow for tension lag associated with diagonal cracking and other unspecified effects. For walls sustaining low axial loads, in which the shear reinforcement is sufficient to resist the entire shear, the tension lag is approximately equal to half the wall length, as is shown in Fig. 2.2(a). As tension lag effects are not modelled in the wall, to be equivalent to the proposals in the commentary, analyses took the offset up the wall as one half of the wall length.

(7) The strain hardening characteristics of all the potential plastic hinge zones were chosen to give the structure a nominal displacement strain hardening characteristic of 2½ percent. The individual characteristics for each hinge zone were assessed using the following steps.

(a) The displacement at the top of the structure resulting from the modal analysis was defined as a ductility one displacement. The design sway mechanism for each structure was selected as shown in Fig. 2.3. The strain hardening characteristics of the hinge zones were assessed so that when the structure displaced in its chosen sway mechanism to accommomdate an increase in displacement ductility of one the bending moments sustained at all the plastic hinge zones increased by 2½ percent of their initial yield values.

(b) The maximum displacement anticipated at the top level of each structure was calculated by multiplying the displacement found in the modal analysis, by the structural ductility factor.

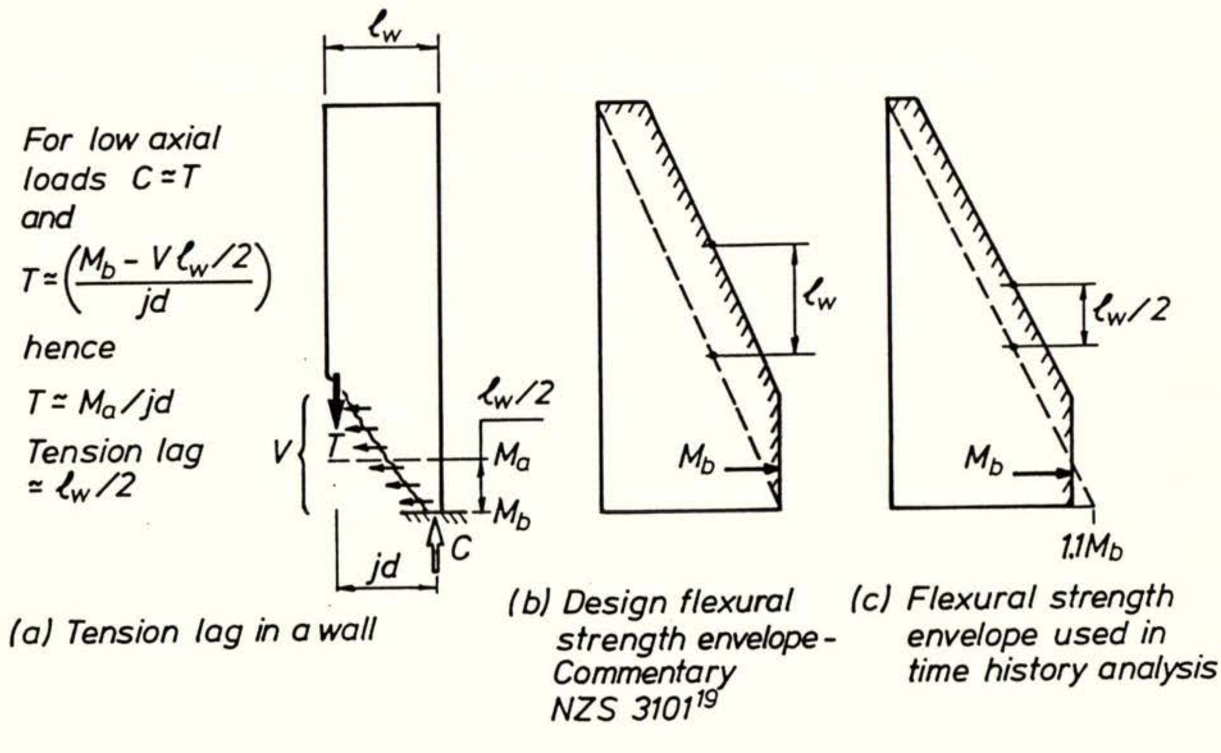


Fig. 2.2 Tension lag and flexural strength envelope in structural walls

(c) A push over analysis was carried out using the equivalent static forces and a plot of base shear against displacement of the top level of the structure was made. The slope of this plot when the mechanism had fully formed was required to be 2½% of its initial elastic stiffness.

(d) To achieve this the strain hardening ratios found in (a) were then scaled by a factor so that the 2½% slope in the plot (c) was obtained.

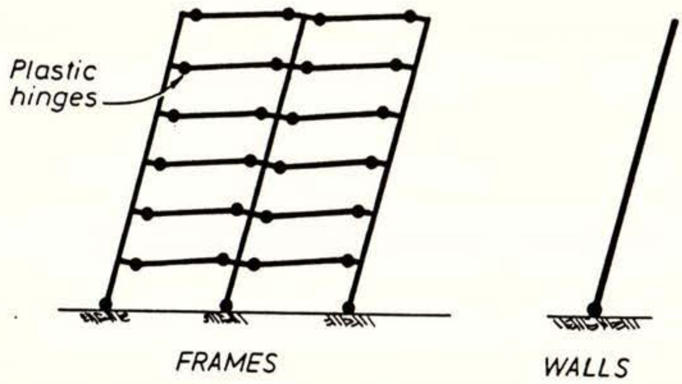


Fig.2.3 Sway mechanisms for frames and walls.

2.2 Details of Frames and Walls

Frames of 6, 12, 18 and 24 storeys were analysed in this project. Typical dimensions are shown in Fig. 2.4 for the 12 storey frame. The column sizes for all the frames are listed in Table 2.1, with the section properties being based on the gross section neglecting steel content, as recommended in the concrete code[19]. The assumed beam dimensions, which are shown in Fig. 2.4, were kept constant for all the frames. As indicated, some allowance was made for the composite action of the in situ concrete in the floor slab. As recommended in the concrete code commentary[19] the section properties for the beams were based on the gross concrete section and multiplied by a half to allow for stiffness loss associated with flexural cracking.

Table 2.1 Column sizes for frames.

Storeys	6 storey	12 storey	18 storey	24 storey
0 - 6	450 x 450	550 x 550	700 x 700	800 x 800
7 - 12		550 x 550	600 x 600	700 x 700
13 - 18			500 x 500	600 x 600
19 - 24				500 x 500

In this project the beam-column zones were assumed to be rigid. This unrealistic assumption was made so that the results obtained in the modal analysis could be compared directly with the results obtained from the time history analyses, where there was no simple means of allowing for joint zone flexibility.

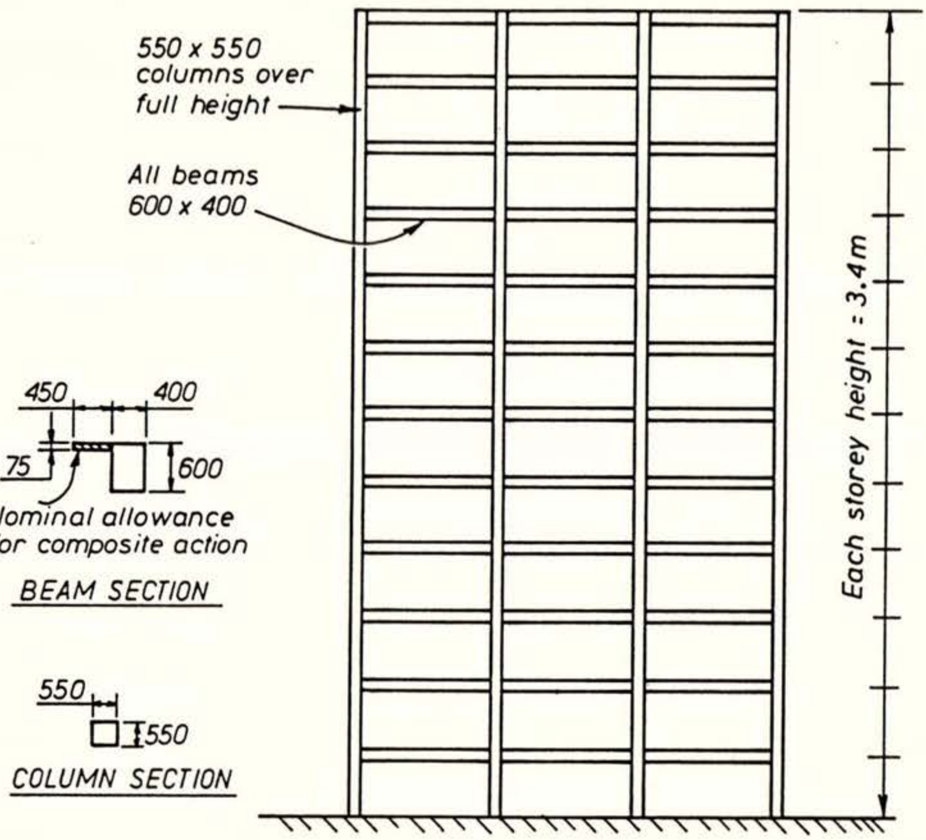


Fig. 2.4 Typical details of a frame.

Walls, with the section dimensions shown in Fig. 2.1, were analysed for 12, 18, 24 and 30 storey structures. The section properties were taken as sixty percent of the values calculated from the gross section dimensions. The sixty percent factor was used to make an allowance for the stiffness reduction associated with flexural cracking. In all cases the base of the wall was taken as rigid.

The elastic modulus of the concrete was assumed to correspond to an average cylinder strength of 39 MPa. This value was based on a specified strength of 30 MPa. The additional 9 MPa was added on to make an allowance for both the average strength being greater than that specified and the strength increase expected with age. On this basis and the expression given in the code[19] the elastic modulus was taken as 28 350 MPa and the shear modulus was taken as forty percent of this value.

Appendix A tabulates the numerical values of beam and column properties used in the computer analyses.

2.3 Methods of Analysis

2.3.1 Equivalent Static Method

The set of lateral equivalent static forces used in these analyses were calculated in accordance with the method specified in the draft code[3]. At each level, i, a lateral force, F_i, was applied. This value was given by:

$$F_i = 0.92 \ V \frac{W_i \ h_i}{\sum (W_i \ h_i)} \qquad (2.1)$$

and at the top level an additional force of 0.08V was added to the value given by Eq. 2.1. In this expression W_i is the seismic weight, h_i is the height of level i above the base and V is the base shear. This value is given by the equation:-

$$V = C_b \ (T_1 \mu) \ R.Z. \ W_T \qquad (2.2)$$

where W_T is the sum of the seismic weights in the structure (Σ W_i), R is the risk factor, which was taken as unity, Z was the zone factor, which was taken as the maximum value of 0.8 and C_b (T_1 μ) is the lateral force coefficient for normal soils for a structure with a fundamental period of T_1 and structural ductility factor of μ, as given in the draft code[3]. For the walls and frames the structural ductility factors were taken as 5 and 6 respectively.

2.3.2 Modal Response Spectrum Method

In the modal response spectrum method the contributions of the different modes were combined by the square root of the sum of the squares. The program ETABS[24] was used for these analyses. The response spectra were taken from the draft code[3] for normal soils for the structural ductility factors of 5 and 6 for the walls and frames respectively, or where the proposed response spectrum was used it was obtained from the structural ductility 1 spectrum scaled as suggested in section 1.1.

2.3.3 Modal Equivalent Static Method

The method is defined in the first edition of the draft code, NZ 4203-86[2] and in section 1.2.4. The structure is subjected to a modal response spectrum analysis with the storey shears being combined as outlined in 2.3.2. By taking the differences of these values a set of equivalent static forces are found, which are then applied to the structure to give the design structural actions (deflections, shears, bending moments and axial loads).

2.3.4 Push-over analysis

Push-over analyses were carried out using the set of equivalent static forces defined in Section 2.3.1. These were incrementally increased until the deflection of the top level of the structure reached the value obtained in the modal response spectrum analysis times the structural ductility factor. The stiffness, strength, and strain hardening values used for the members in these analyses were the same as the values used in the numerical integration time history analyses (Section 2.3.5).

2.3.5 Numerical Integration Time History Analyses

The computer program DRAIN2DX[25] was used to carry out non-linear time history analysis of the walls and frames. The program is able to model two dimensional finite element models, which in this project were formed from beam-column elements. Yielding as a result of excess bending moment and/or a defined combination of axial load and bending moment is allowed at the ends of the members or at the defined position representing the face of a beam or a column in these members.

Structural Damping

Viscous damping in the analyses was assumed to take the form of Rayleigh damping. With this system the damping matrix is formed by a linear combination of the mass and initial structural stiffness matrices. This is the most common assumption made in programs which perform step by step integration of the equations of motion. It can lead to artificially high amounts of damping to vibrations associated with the so called "higher modes". In this project the analyses were performed with the equivalent of 5 percent of critical viscous damping applied to the first two modes.

Hysteretic Model

The DRAIN2DX beam and column elements allows for concentrated plastic hinges to form at the element ends. In this study interaction between axial load and bending moment was not considered and a bilinear moment - rotation relationship used. The slope of the strain hardening line was adjusted as described in Section 2.1.

Ground Motions

To make valid comparisons between the predictions based on modal response spectrum and time history analyses it was considered that one of the earthquake ground motions should have a similar response spectrum to that used for the modal response spectrum analyses. To satisfy this requirement an artificial ground motion, that had been developed in a previous project[10], was used. This ground motion was generated using the SIMQKE[20] program. The target response spectrum for the motions as taken as the elastic spectrum in the draft code[3] for normal soils for a structural ductility factor of 1 and a zone factor of 0.85. For use in the project it was scaled by 0.8/0.85 to correspond to a zone factor of 0.8. With this scaling the peak ground acceleration was 0.32g. The length of the record was effectively 28 seconds, with the intensity of the shaking rising from zero to unity in the first three seconds, where it was held constant until 25.5 seconds, when it decreased rapidly over the next 2½ seconds. In this report this ground motion is referred to as the ART-1 record.

The target velocity response spectrum is compared with the one produced by the artificial record in Fig. 2.5. In general the agreement between the two curves is acceptable. However, the discrepancy exceeds 15 percent in the period ranges of 1.7 to 1.8, 2 to 2.6, 2.8 to 2.9 and 4.6 to 4.7 seconds.

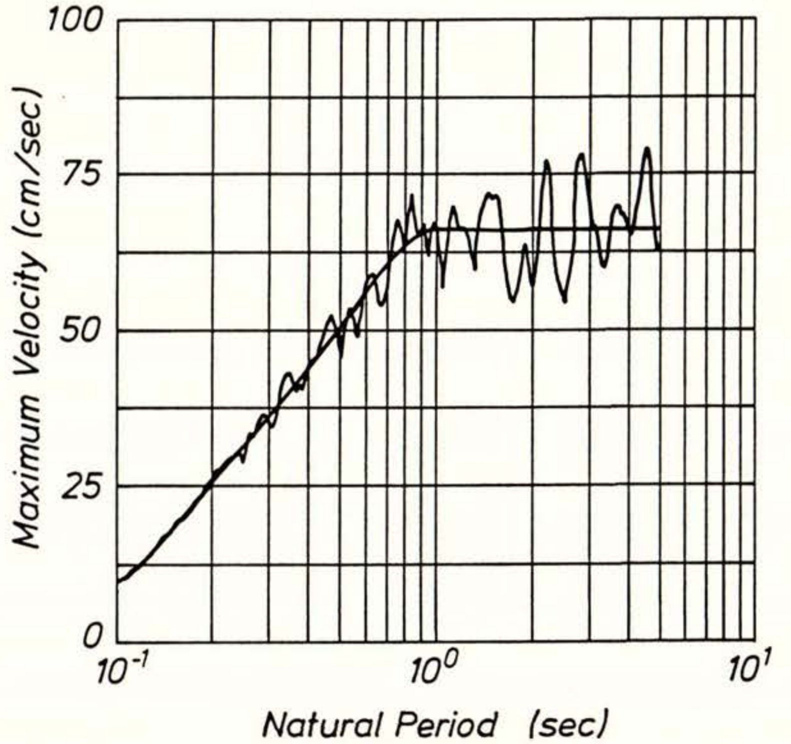


Fig. 2.5 Velocity response spectrum for the artificial ground motion, ART-1

In a previous project[10] it was found that with this ground motion the resultant response spectra for different structural ductility factors were in close agreement with those given in the draft code[3].

The other ground motions used in the project were -

El Centro 1940 NS, and
Parkfield 1966 (Chalome Shandon Array No.2) N6SE.

These were scaled to give the same value of spectrum intensity[26] as is implied by the code lateral coefficient given in the draft code[3] for the period range of 0.25 to 2.5 seconds. Thus the time history of these earthquakes were scaled so that their resulting pseudo acceleration response spectra S_a satisfied

$$\sum_{T=.1}^{2.5} S_a\,(T,\ 0.05)\ T.\ \Delta\ T\ =\ \sum_{T=.1}^{2.5} C_b\,(T,\mu=1).T.\Delta\ T \qquad (2.3)$$

The required scale factors were; 1.02 for El Centro, and 0.56 for Parkfield. The response spectra for the scaled ground motion records are shown in Fig. 2.6 together with the fundamental periods of the 12 to 30 storey walls (W_{12} - W_{30}) and the 6 to 24 storey frames (F_6 - F_{24}).

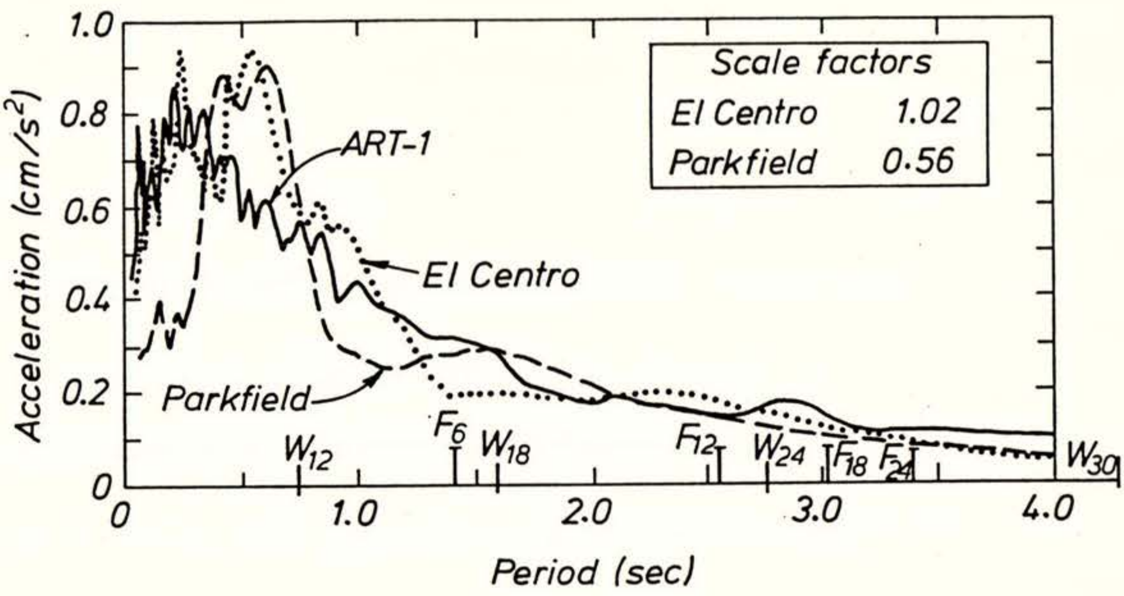


Fig. 2.6 Acceleration response spectra for the ground motions used in this project.

CHAPTER 3 - RESULTS OF ELASTIC ANALYSIS

3.1 Abbreviations Used With Methods of Analysis and Response Spectra

In this project a number of different methods of analysis were used with two different sets of response spectra. The abbreviations which are used to identify these in the remainder of this report are given in Table 3.1. The methods of analysis are described in detail in sections 2.3.1 to 2.3.4 and the two sets of response spectra are defined in section 1.1.

Table 3.1 Methods of Analysis and Response Spectra.

Item	Abbreviation
Equivalent static method of analysis as defined in DZ 4203-89*	Eq. S
Modal response spectrum method where the mode contributions are combined by SRSS	M
Modal equivalent static, where combined modal storey shears are used to define a set of equivalent static loads that are then applied to the structure.	M Eq. S
Response spectrum for a given structural ductility factor (μ) as defined in DZ 4203-89	DZ (μ)
Response spectrum scaled for a given structural ductility factor (μ) as proposed in section 1.1.	P (μ)

* Except the deflections have not been multiplied by the factor (0.85) given in

Clause 3.7.3.1 of NZ 4203-89.

For example the notation M Eq. S - DZ (6) stands for a "modal equivalent static analysis using the response spectrum defined in DZ 4203-89 with a structural ductility factor of 6".

3.2 Dynamic Properties of the Structural Walls and Frames

The dynamic elastic properties of the walls and frames structures analysed in this project are set out in Table 3.2.

Table 3.2 Properties of Walls and Frames

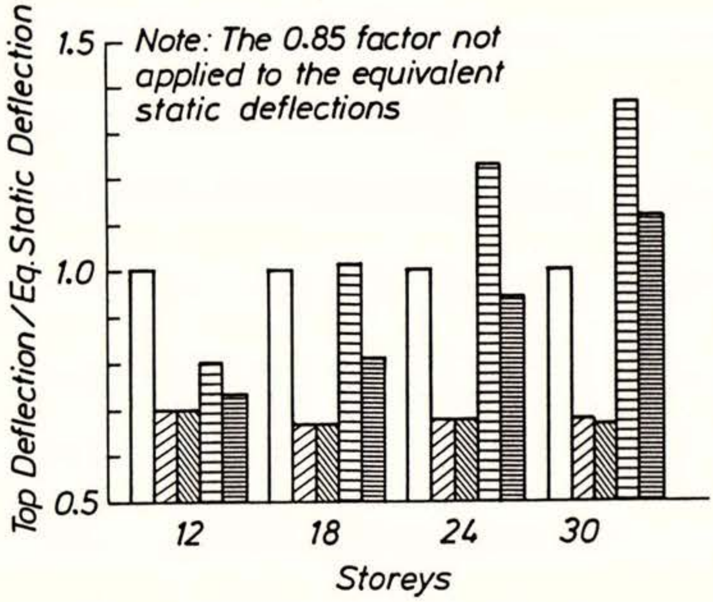
Item	Symbol	Walls - No. of storeys			
		12	18	24	30
Periods of first 3 modes (seconds)	T_1 T_2 T_3	0.73 0.13 0.05	1.58 0.26 0.10	2.76 0.45 0.17	4.28 0.70 0.26
Proportion of mass participating in mode	M_1 M_2 M_3	0.65 0.21 0.07	0.63 0.20 0.07	0.63 0.20 0.07	0.63 0.20 0.07
	ΣM	0.93	0.90	0.90	0.90
Seismic Weight (kN)	W_t	10 200	15 300	20 400	25 500
Response spectrum value x R x Z at T_1 for $\mu = 1$	$C_b(T_1,1)$	0.485	0.254	0.145	0.094
and $\mu = 5$	$(C_b(T_1,5)$	0.120	0.054	0.029	0.018
		Frames - No. of storeys			
		6	12	18	24
Periods of first 3 modes	T_1 T_2 T_3	1.43 0.46 0.25	2.55 0.83 0.47	3.01 1.02 0.59	3.36 1.16 0.67
Proportion of mass participating in mode	M_1 M_2 M_3	0.84 0.10 0.04	0.81 0.10 0.04	0.78 0.11 0.04	0.76 0.12 0.04
	ΣM	0.98	0.95	0.93	0.92
Seismic Weight (kN)	W_t	10 200	20 400	30 600	40 800
Response spectrum value x R x Z at T_1 for $\mu = 1$	$C_b(T_1,1)$	0.279	0.152	0.133	0.119
and $\mu = 6$	$C_b(T_1,6)$	0.0465	0.0261	0.022	0.020

Table 3.3 Key results of Elastic Analyses.

Method of Analysis and spectrum	Item	Walls - No. of storeys			
		12	18	24	30
Eq. S - DZ (5)	Deflection	34	75	133	207
M - DZ (5)	at top of	23.5	50	89	138
M - P (5)	structure	23.4	50	89	137
M Eq. S - DZ (5)	(mm)	27	76.3	164	293
M Eq. S - P (5)		25	61	126	231
Eq. S - DZ (5)	Base shear	1 220	826	624	503
M - DZ (5)	(kN)	1 039	997	954	891
M - P (5)		869	729	687	680
M Eq. S - DZ (5)		1 039	997	954	891
M Eq. S - P (5)		869	729	687	680
Eq. S - DZ (5)	Overturning	35 900	35 900	36 000	36 100
M - DZ (5)	bending moment	24 900	25 900	27 100	27 800
M - P (5)	at base (kNm)	24 400	24 500	25 400	26 300
M Eq. S - DZ (5)		28 800	37 200	45 400	51 800
M Eq. S - P (5)		25 700	29 500	34 800	41 100
		Frames - No. of storeys			
		6	12	18	24
Eq. S - DZ (6)	Deflection	37.0	71	90	105
M - DZ (6)	at top of	31	54.5	68	79
M - P (6)	structure	31	54.5	68	79
M Eq. S - DZ (6)	(mm)	34.5	62.9	79.3	94.1
M Eq. S - P (6)		32.1	59.9	76.5	90.9
Eq. S - DZ (6)	Base shear	474	534	677	808
M - DZ (6)	(kN)	454	484	609	739
M - P (6)		427	464	590	717
M Eq. S - DZ (6)		454	484	609	739
M Eq. S - P (6)		427	464	590	717
Eq. S - DZ (6)	Overturning	7 510	15 600	29 400	46 600
M - DZ (6)	bending moment	6 420	12 100	22 400	35 600
M - P (6)	at base (kNm)	6 200	12 100	22 400	35 500
M Eq. S - DZ (6)		6 740	13 800	25 700	41 200
M Eq. S - P (6)		6 250	13 100	24 900	40 000

3.3 Key Results by Analyses

The principal results of the analyses of the frames and walls are presented in Table 3.3. In Figs. 3.1 and 3.2 the values are shown as a proportion of the corresponding value obtained in an equivalent static analysis. As indicated in the table a structural ductility factor of 5 was used for the walls and 6 for the frames.



(a) Lateral deflection of top level as a proportion of the equivalent static deflection for the walls

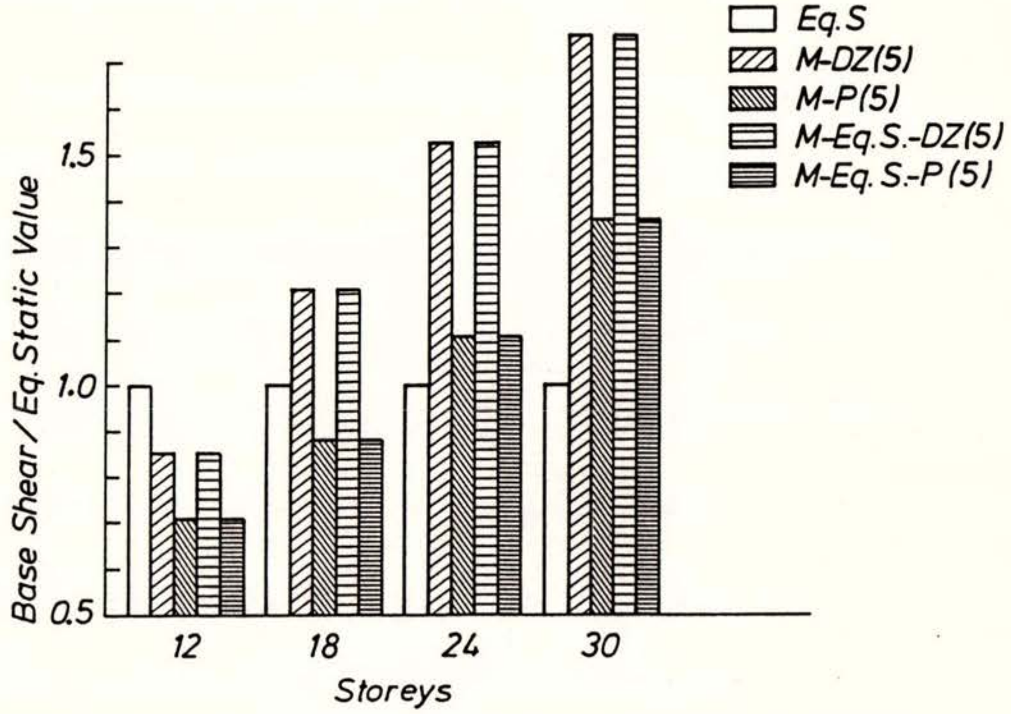
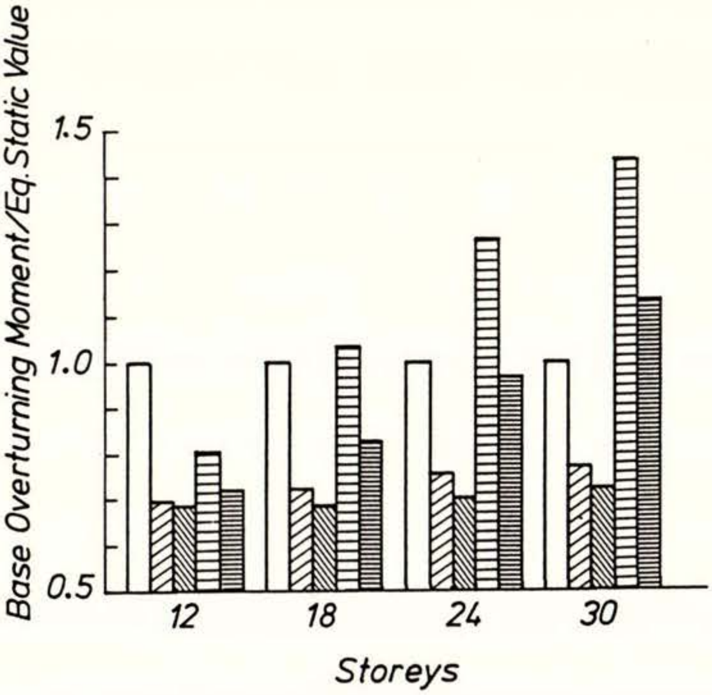


Fig. 3.1 (b) Base shear as a proportion of equivalent static shear for the walls - continued



(c) Base over-turning moment as a proportion of the equivalent static value.

Fig. 3.1 Key results for structural walls obtained using different methods of analyses - concluded.

From Figs 3.1(a) and (c) and 3.2(a) and (c) it can be seen that the equivalent static method of analysis consistently predicts higher deflections and overturning moments than the modal methods of analysis (M-DZ (5) & M-P (5)). This difference arises from the assumptions made with regard to the effective mass (Section 1.3.2). In the modal analysis it is found that nearly all the deflection and the majority of the overturning moment comes from the first mode contribution. For the walls the proportion of mass contributing to the first mode is approximately 0.64 while for the frames it is approximately 0.8.

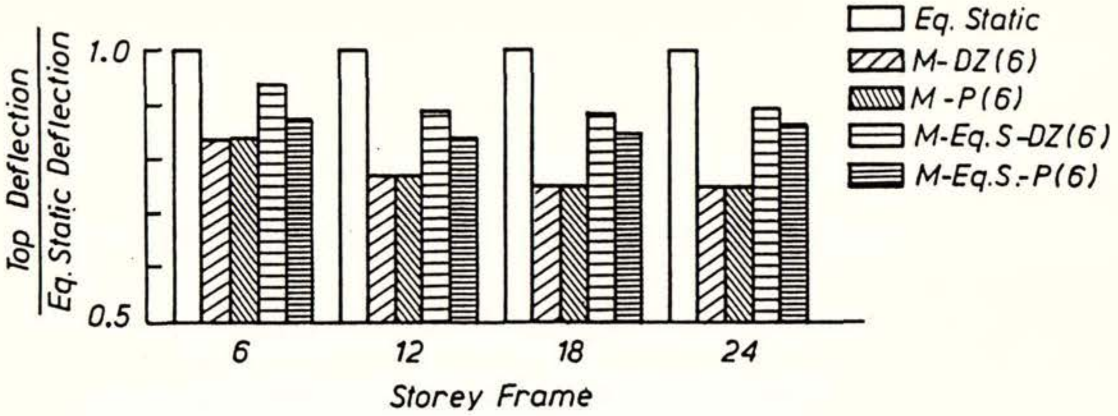
The equivalent static method assumes that the total mass contributes to a first mode type of action. A close estimate of both the deflection and the overturning moments attained by the modal response spectrum method can be found by multiplying the corresponding equivalent static values by the proportion of mass contributing to the first mode (see Tables 3.2, 3.3 together with Figs. 3.1 and 3.2)$^{+}$.[1]

---

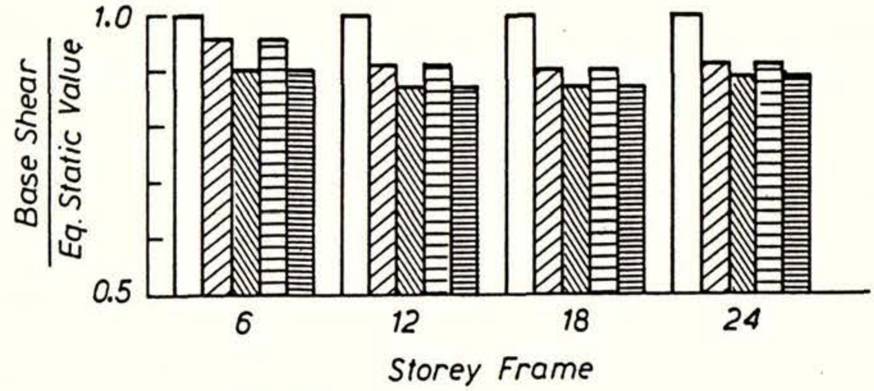
1 The proportion of mass, p_1, contributing to the first mode may be assessed with reasonable accuracy from the equivalent static method. It is given by the

expression, $p_1 = \dfrac{\sum_1^n (W_i\,\delta_i)^2}{\left[\sum_1^n W_i\,\delta_i\right]^2}$ where W_i and δ_i are the seismic weights

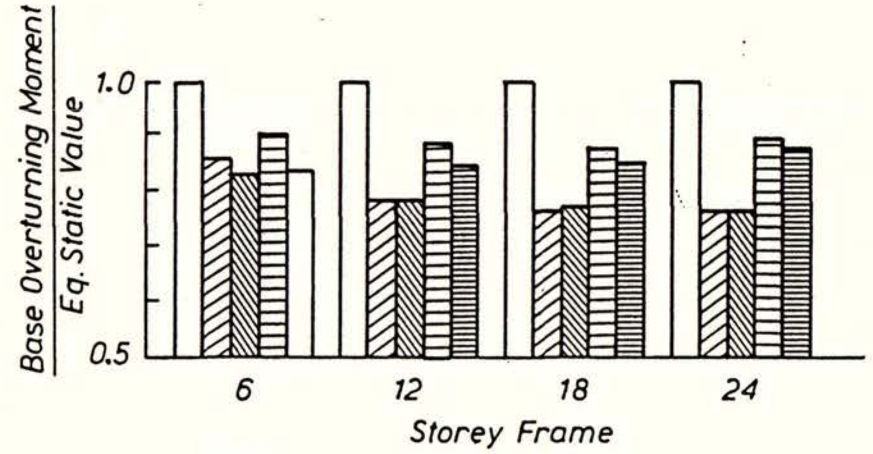
and deflections at level i.



(a) Lateral deflection of top level as a proportion of the equivalent static deflection



(b) Base shear as a proportion of the equivalent static shear



(c) Base Overturning moment as a proportion of the equivalent static values.

Fig. 3.2 Key results for frames obtained using different methods of analysis.

The change in response spectrum from the one in the code, DZ (μ), to the proposed, P (μ), can be seen to make very little difference to the magnitudes of the deflections or the overturning moments where these are calculated by the modal response spectrum method. However, this change does effect the magnitude of the base shear in the walls, see Fig. 3.1(b), where the values are 20 to 40 percent greater with the DZ spectrum than with the proposed spectrum. For the frames there is little difference, see Fig. 3.2(b).

The modal equivalent static method with both response spectra can be seen to predict higher deflections and overturning moments than those obtained with the modal methods of analysis. The difference is particularly marked in the more slender walls, with the deflection and overturning moment for the 30 storey wall being close to twice the corresponding values for the modal analysis with the DZ (5) spectrum and 1.6 times for the proposed spectrum. This discrepancy arises from the assumption inherent in the method that the maximum storey shear force envelope is sustained simultaneously over its full height (see 1.3.4).

3.4 Storey Shear Force Envelopes

The shear force envelopes for the walls are shown in Fig. 3.3 (a → d). With the 12 storey wall the equivalent static values are everywhere greater than the other values. However, with the more slender walls the modal values at both the base and in the upper reaches of the wall increase above the equivalent static values by an appreciable amount. In all cases the modal values found from the proposed spectrum are smaller than those determined from the draft code spectrum.

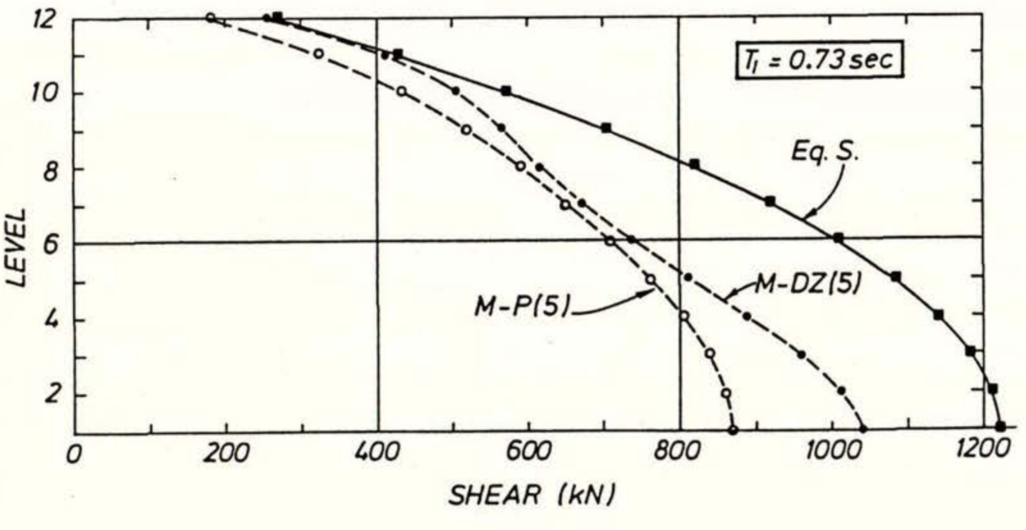
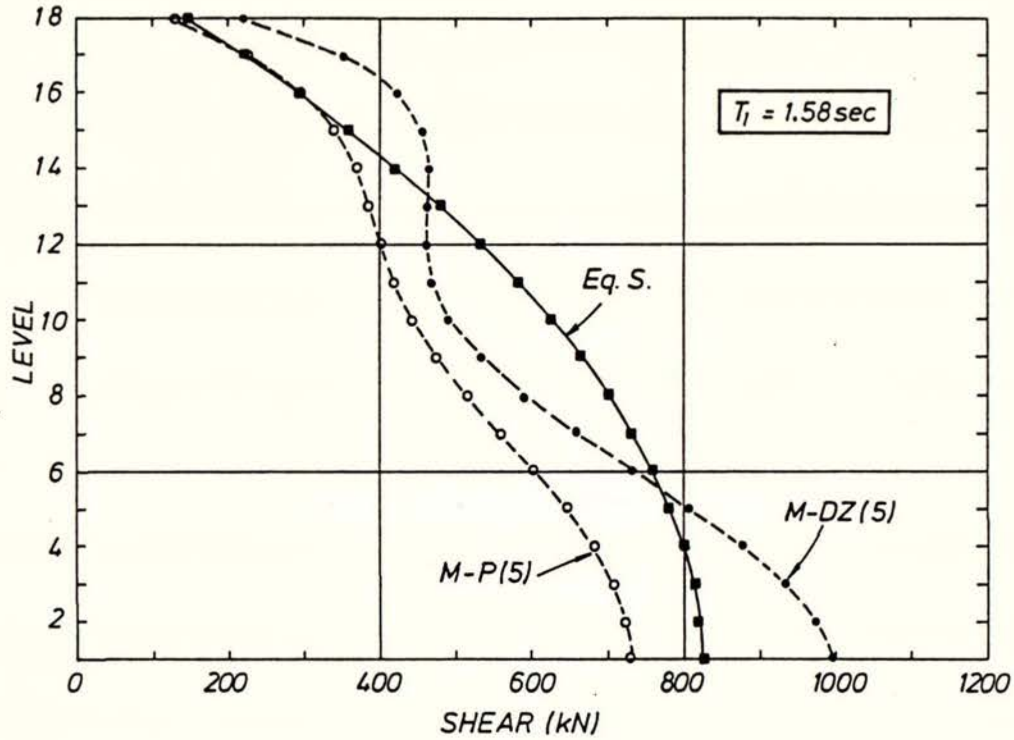


Fig.3.3(a) Shear force envelope in 12 storey wall - continued



(b) Shear force envelope for 18 storey wall.

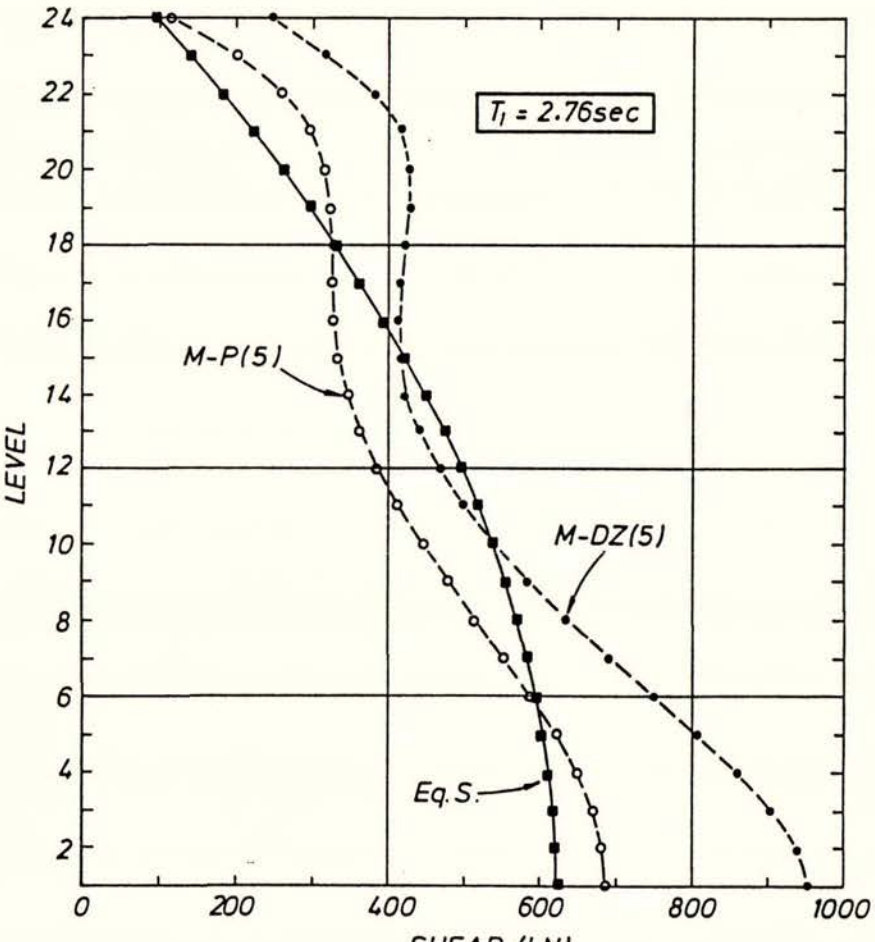


Fig.3.3 (c) Shear force envelope for 24 storey wall - continued

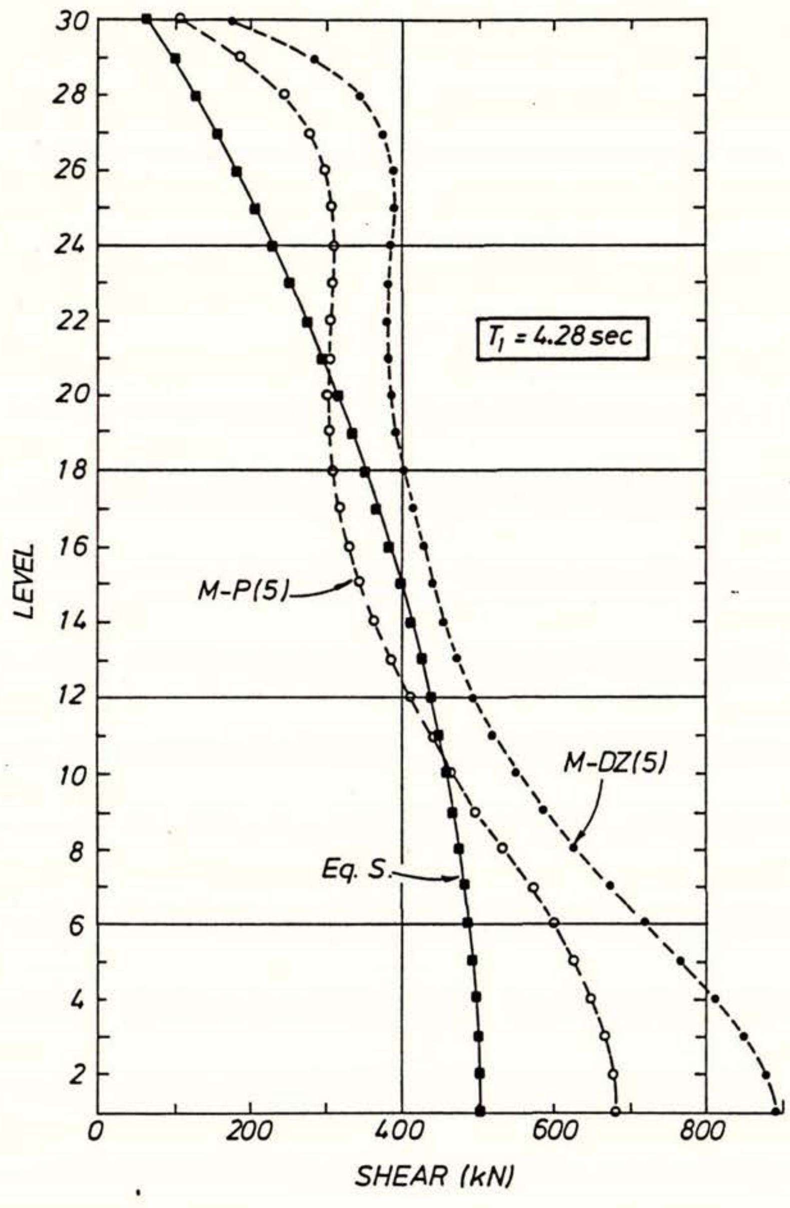
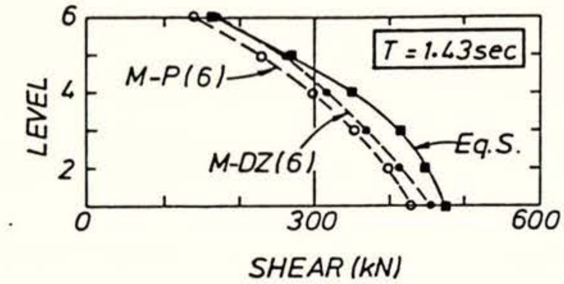


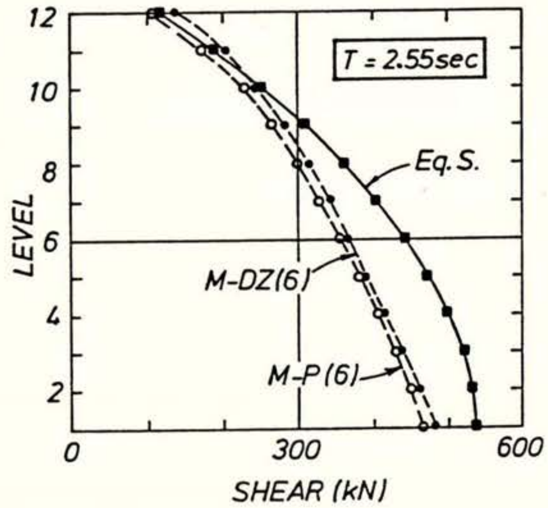
Fig. 3.3(d) Shear force envelopes for 30 storey wall - concluded.

The distributions of the storey shears for the frames are shown in Fig. 3.4 (a → d). With these structures the second, third and higher modes make only a small contribution. Consequently there is little difference between the values found from the modal analyses with the two spectra. Except for the top few storeys the equivalent static values exceed the combined modal shears.

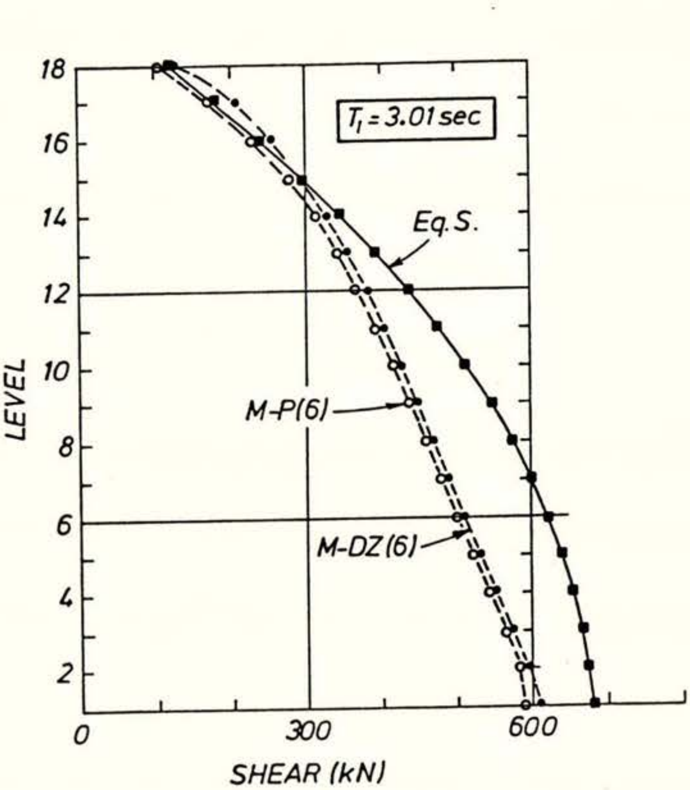
As the modal equivalent static method is based on the combined modal shears the storey shear forces with this method of analysis are identical to the values found in the corresponding modal analysis.



(a) 6 storey frame



(b) 12 storey frame



(c) 18 storey frame

Fig. 3.4 Storey shear force envelopes - continued

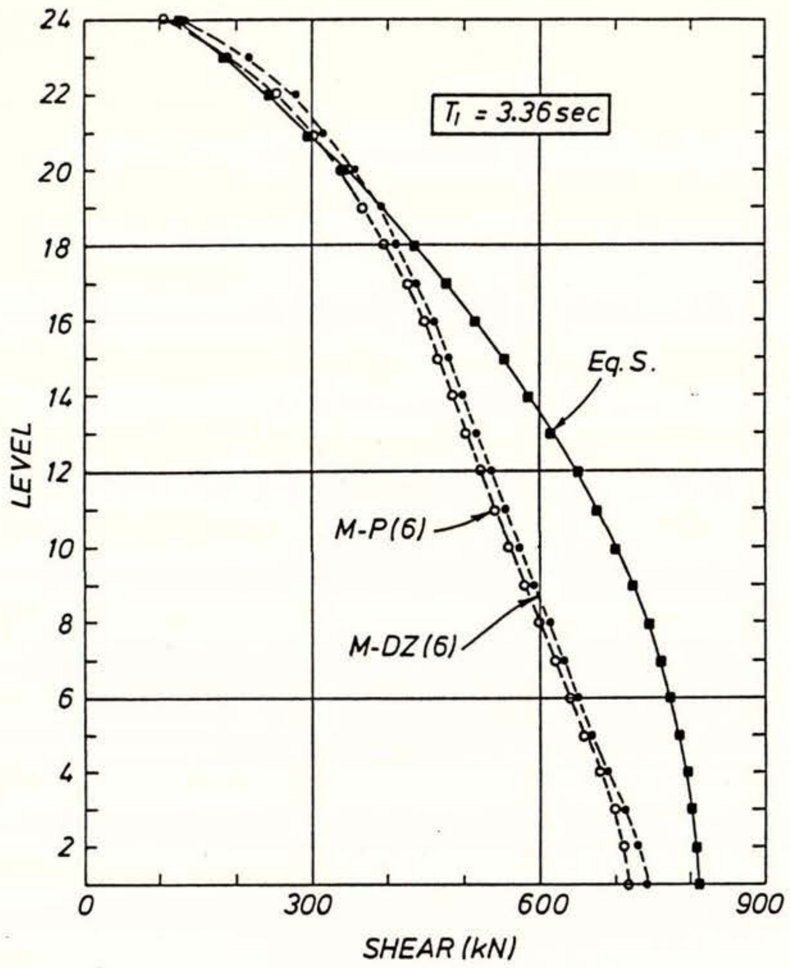


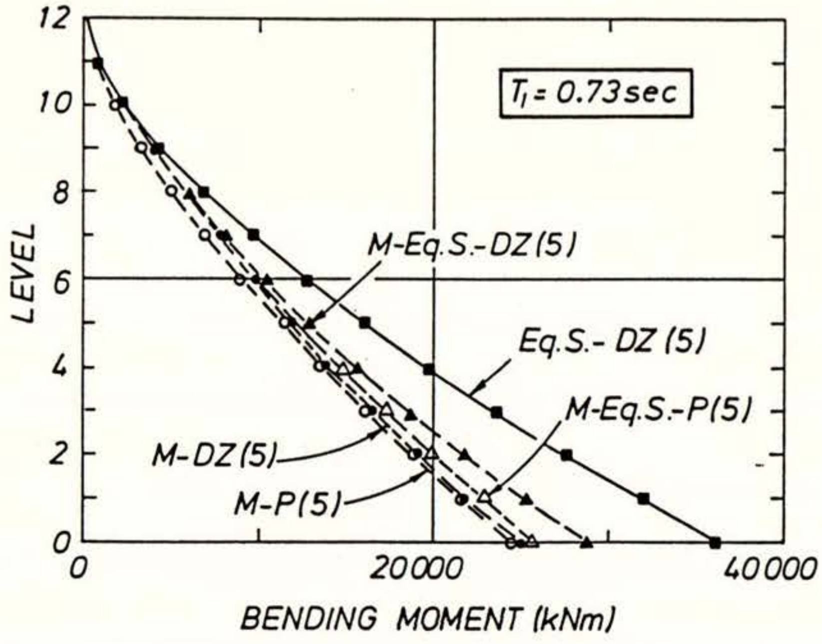
Fig. 3.4 (d) Storey shear force envelopes in 24 storey frame - concluded.

3.5 Distribution of Storey Bending Moments in Walls

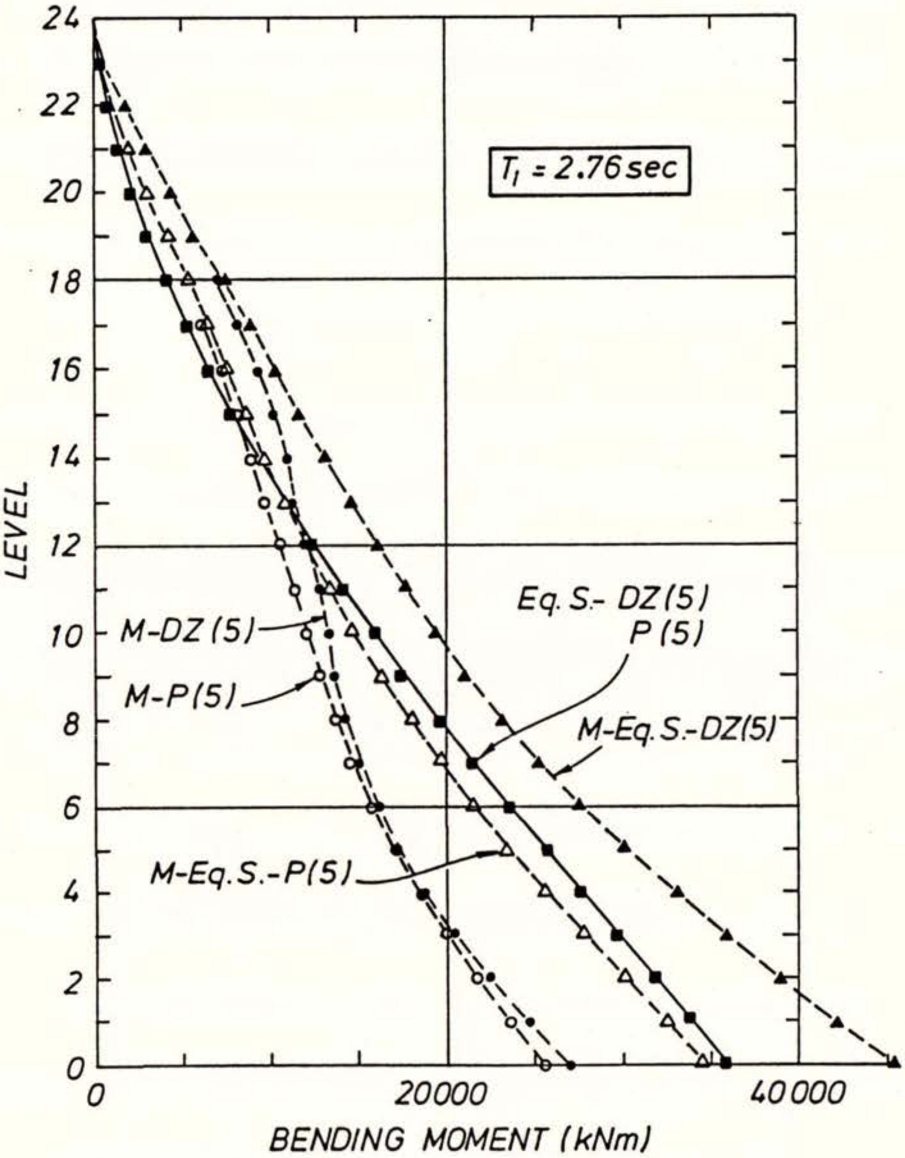
The distributions of the storey bending moments in the 12 and 24 storey walls found by the different methods of analysis are reproduced in Fig. 3.5 (a, b). With the 12 storey wall the shape of the diagrams produced by all the analyses is similar. It should be noted there is very little difference between the two modal analysis results. As indicated previously the greatest values arise with the equivalent static method of analysis.

With the 24 storey wall greater differences occur between the results of the analyses when the response spectrum is changed than was the case with the 12 storey wall. In this case the modal equivalent static bending moments with the DZ spectrum exceed the corresponding bending moments predicted by the equivalent static method. With the modal method of analysis there is little difference in the bending moments in the lower half of the wall with the two spectra. However, in the upper half the values found using the DZ(5) spectrum are appreciably greater than the corresponding values found using the proposed spectrum.

As indicated in Figs. 3.1 and 3.2 the storey bending moments in the frames are not as sensitive as those in the walls to the method of analysis.



(a) 12 Storey wall



(b) 24 Storey wall

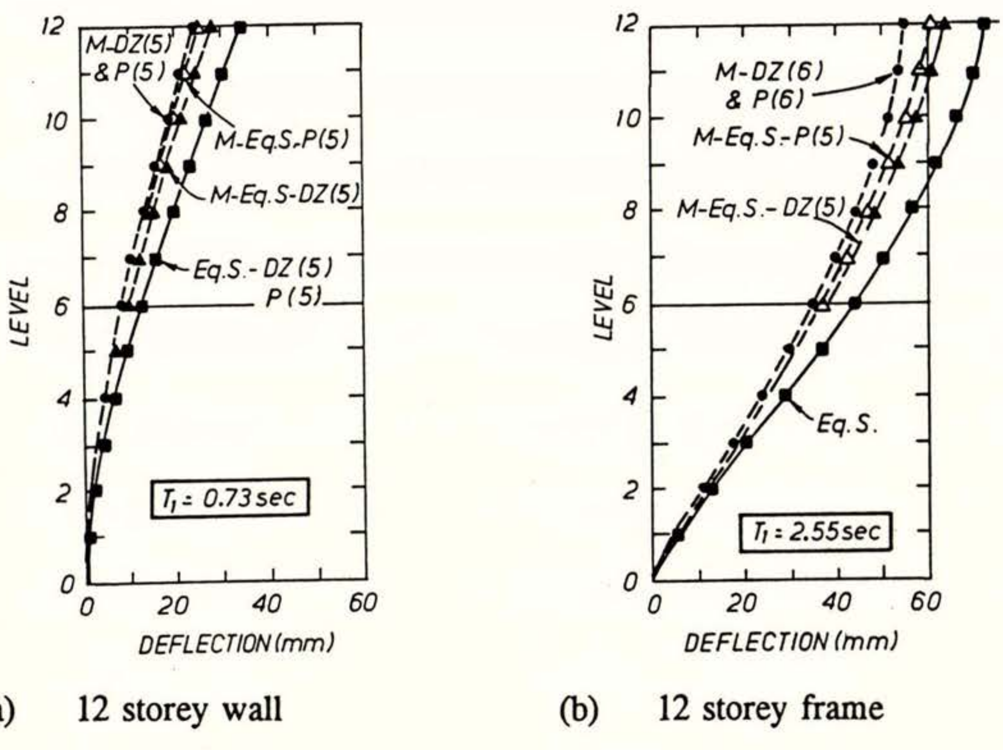
Fig.3.5 Envelope of storey bending moments in 12 & 24 storey walls.

3.6 Deflected Shapes and Interstorey Deflections

The deflected shapes of the 12 and 24 storey walls and frames as predicted by the different methods of analysis are shown in Fig. 3.6 (a - d). In all cases these are similar to each other and only the magnitudes vary with the different methods of analysis. Due to the very small contribution of the higher modes to the combined modal deflected shape envelope with the modal method of analysis it is not possible to show the difference in the deflections obtained with the two response spectra. However, a major difference does occur in these values with the modal equivalent static method, particularly with the 24 storey wall.

The maximum interstorey deflections found by the different methods of analyses are listed in Table 3.4 for the frames. For the modal method these values were found by taking the square root of the sum of the squares of the interstorey deflections for each mode. The maximum values in each frame found in this way are generally about five percent greater than the corresponding (but less rigorous) values found by taking the difference of the SRSS of the storey deflections. However, this second method of calculation was found to underestimate the smaller interstorey deflections in the higher storeys by up to 35 percent. Provided only the maximum interstorey deflection in the structure is required the simpler approach of taking the difference in the modal combined interstorey deflections appears to be sufficiently accurate for the purposes of design.

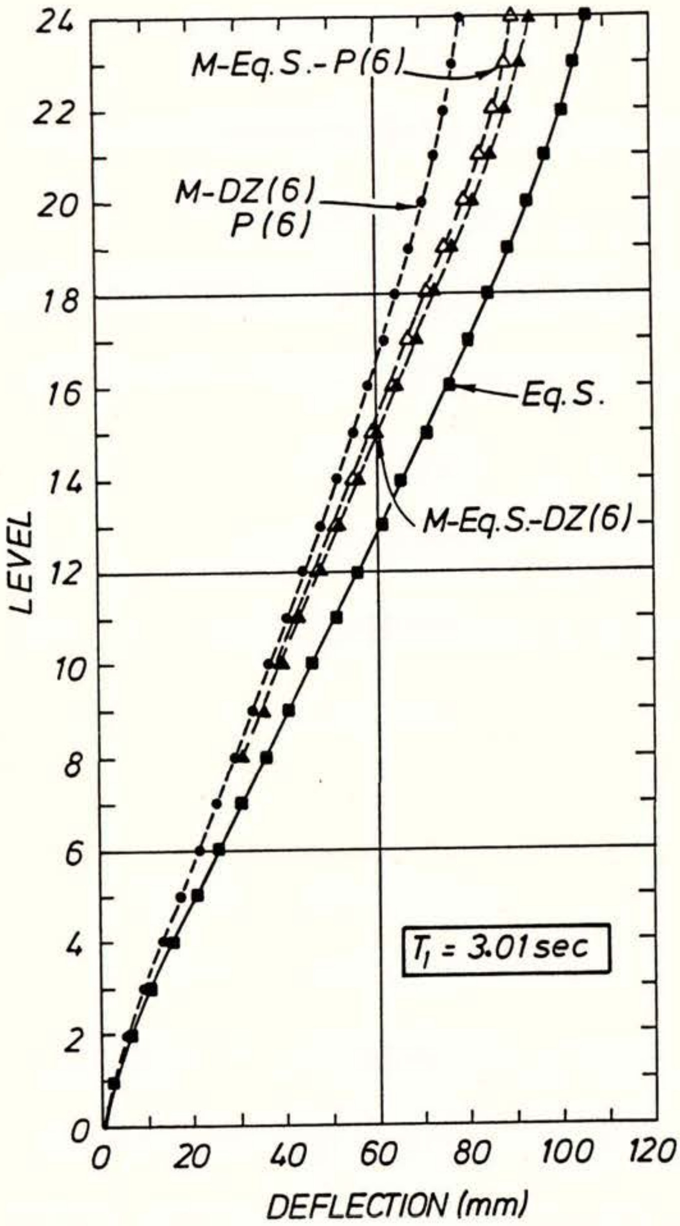
There is little difference in the magnitude of the interstorey deflections found by the modal method with the two response spectra. These values are close to 83 percent of the corresponding equivalent static interstorey deflections in all cases.



(a) 12 storey wall

(b) 12 storey frame

Fig. 3.6(a) & (b) Lateral deflection envelopes for 12 storey walls and frames - continued



(c) Lateral deflection envelopes for the 24 storey wall - continued

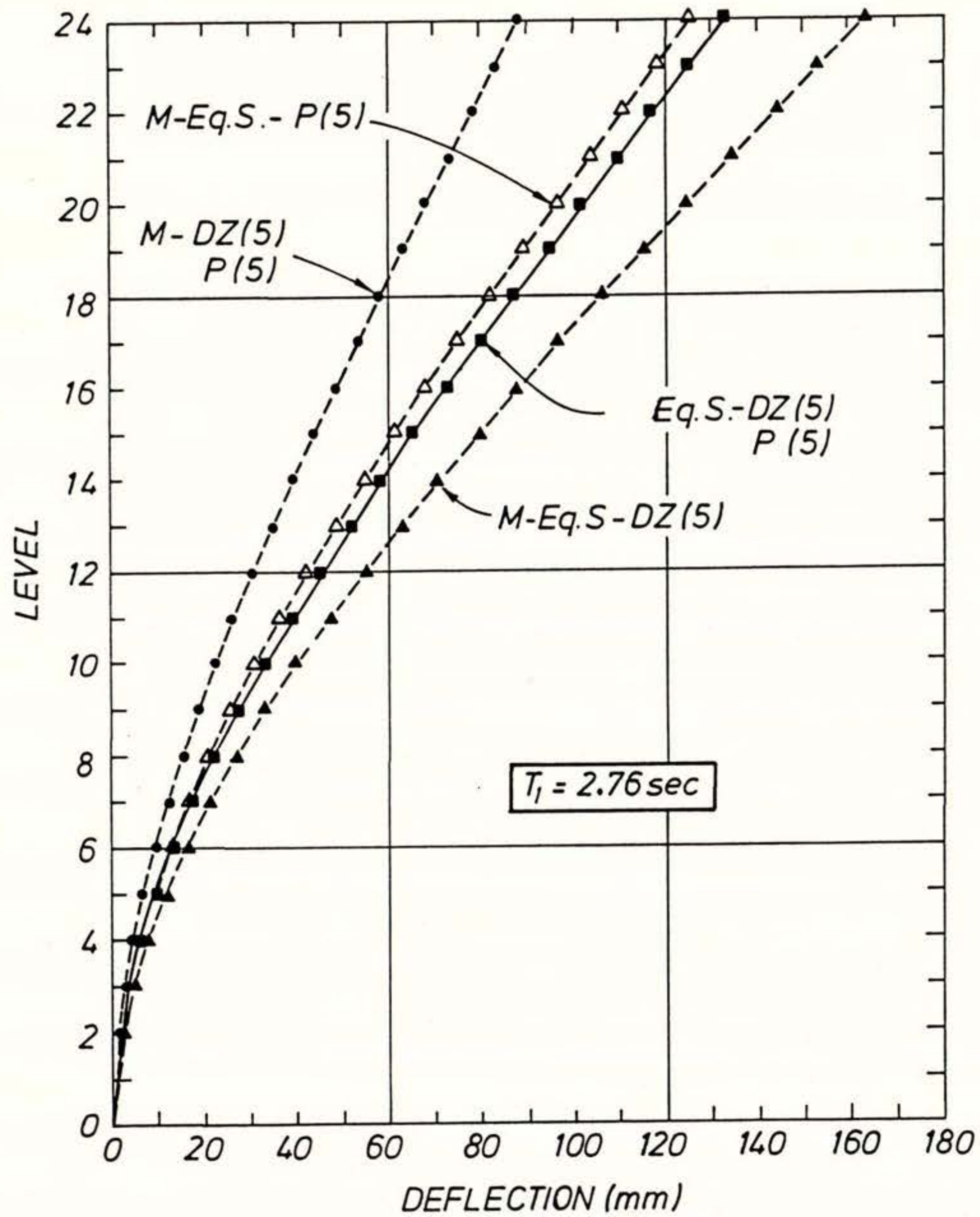


Fig.3.6 (d) Lateral deflection envelopes for the 24 storey frame - concluded.

Table 3.4 Maximum Interstorey Deflections in Frames given by diffferent methods of analysis

Method of Analysis and spectrum	Interstorey deflections (mm)			
	6	12	18	24
Eq. S - DZ (6)	8.21	8.14	6.36	5.28
M - DZ (6)	7.33	6.74	5.07	4.23
M - P (6)	7.32	6.92	5.02	4.18
M Eq. S - DZ (6)	7.40	6.93	5.12	4.33
M Eq. S - P (6)	7.19	6.80	5.05	4.28

3.7 Discussions and Conclusions

(a) Influence of response spectra on values

Of the methods of analysis used in this project the modal method with individual mode contributions combined by the SRSS is accepted in the literature (Section 1.2) as giving the best estimate of likely seismic induced actions in elastically responding structures. Consequently in assessing the influence of the response spectra on the structures only the results of this method are considered.

The difference in the values of the base overturning moments, deflections, and interstorey deflections obtained using the two response spectra are small for all the structures considered. However, significant differences do occur in the wall storey shears and the wall storey bending moments at higher levels. The values are greater when the draft code spectrum is used due to the increased contribution of the higher modes. The base shear in the walls was on average 32 percent greater with the draft code spectrum than it was with the proposed spectrum.

(b) Influence of methods of analysis

Compared with the modal method of analysis the equivalent static method over-estimates the storey bending moments, the deflections, and the interstorey deflections. This over-estimate of response can be explained in terms of one of the basic assumptions in the method. With the equivalent static approach it is assumed that the entire seismic mass acts in a manner similar to the first mode. However, in the modal analyses of the walls and frames approximately only 75 percent of the mass participates in the first mode[10]. As a result, values which depend almost entirely on the first mode response, such as deflections and overturning moments, tend to be over-estimated by the equivalent static method by approximately 1/0.75, or 33 percent. The current loadings code[1] compensates for this by scaling the deflections found by the equivalent static method by a factor of 0.91 times the value used for deflections found by the modal method. From the results of these analyses and previous work[10] it is apparent that a factor of 0.85 could conservatively be used for both the deflections and the interstorey deflections found by the equivalent static method.

The equivalent static method under-estimates the base shears in the 24 and 30 storey walls by 10 and 35 percent respectively. There are similar short falls in the shear in the upper few levels of these walls. With the frames the equivalent static base shears are conservative. However, the values in the upper few floors are a few percent low. This discrepancy could be reduced by increasing the proportion of the lateral force applied to the top of the structure. Currently 8 percent of the base shear is applied at the top level (F_t) with the remaining 92 percent being distributed by equation 1.3. The UBC and many other codes require the proportion of the base shear added to the top level, (F_t), to be increased with the period, as given by the equation: -

$$F_t = 0.07\ V\ T \qquad (3.1)$$

Adopting this value would reduce this discrepancy but it would be at the expense of complicating the analysis.

From the results of the analyses on both the frames and the walls it can be seen that for structures with fundamental periods of up to 2 seconds the equivalent static method predicts values of actions which are either close to or on the conservative side of the corresponding modal values found with the proposed spectrum.

Compared with the modal method the modal equivalent static method over-estimates the deflections, storey moments and the interstorey deflections. The greatest discrepancies occur in the slender walls. For example with the proposed spectrum the base overturning moments for the 24 and 30 storey walls are 37 and 56 percent greater than the modal values respectively and the corresponding values for the top deflections are 41 and 68 percent in excess of the corresponding modal values. As explained in Section 1.2.4 these errors arise as this method of analysis does not make a rational allowance for higher mode effects. This approach cannot be relied upon to produce realistic design values in structures in which the higher mode effects make a significant contribution to the combined modal actions.

CHAPTER 4 - RESULTS OF TIME HISTORY ANALYSES

4.1 Key Results of Analyses

The key results of the time history analyses for the frames and walls are compared with the predictions obtained from the equivalent static and modal methods of analysis using the proposed spectrum in Table 4.1. The ground motions and the scaling used with these in the analyses are described in Section 2.3.4. As the artificial record, ART-1, was generated from the target response spectrum given for the normal soils in the draft code a direct comparison can be made between time history analysis results for this ground motion and the modal response spectrum values.

From Table 4.1 it can be seen that a reasonable estimate of the deflection at the top of the structure for the ART-1 ground motion can be obtained by multiplying the corresponding value attained in a modal response spectrum analysis by the structural ductility factor. For the walls this gives an over-estimate of the deflection, with the average value for the ART-1 record being 0.83 times the value estimated from the modal analysis. For the frames the discrepancy is in the opposite direction with the corresponding value being 1.07. The overall average of 0.95 for all the structures is reasonable given the scatter expected in such cases. The corresponding value for the equivalent static method is 0.69. This is in line with what would be expected in terms of the over-estimate that this approach makes of the storey bending moments and deflections (Section 1.2.2 and 3.7).

The deflections with the El Centro and Parkfield records varied considerably from those obtained with the ART-1 ground motion. Generally the change in magnitude is what could be anticipated from the acceleration response spectra for these earthquakes, see Fig. 2.6. For the long period structures (30 storey wall and 24 storey frame) the El Centro and Parkfield spectra lie well below the ART-1 values and the order of deflections is consistent with this. For the short period structures the Parkfield record gives the highest displacements.

The maximum values of base shear sustained by each wall with each of the three ground motions are of similar magnitude to each other. The ART-1 values range from 2 to 3.6 times the corresponding modal value with the average value being 2.8. With the equivalent static method the corresponding ratios are no more consistent with the ratio ranging from 1.5 to 3.2 with the average value being 2.6.

For the frames the maximum base shear sustained in the ART-1 record ranged from 1.46 to 1.87 times the corresponding modal response spectrum analysis value, with the average ratio being 1.7. With the equivalent static method the corresponding average ratio is 1.53 with the individual values ranging from 1.32 to 1.63. For the different ground motions the maximum base shear for each frame is reasonably constant.

The maximum overturning moment sustained by each of the walls is in all cases close to the design value (see Table 3.3 - the M - P(μ) values). These bending moments were limited by the strengths given to the base of the structure plus a small

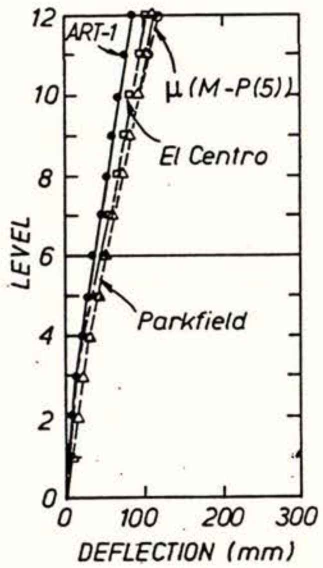
Table 4.1 Key Results of Elastic and Time History Analyses

Item	Structural Walls			
	12	18	24	30
Top Deflection (mm)				
(Eq. S. - P(5)) x μ	170	375	665	1 035
(M - P(5)) x μ	117	250	443	685
ART-1	83	212	343	793
El Centro	104	108	374	340
Parkfield	112	232	365	195
Base shear (kN)				
Eq. S. - P(5)	1 220	826	624	503
M - P(5)	869	729	687	680
ART-1	1 780	2 620	1 810	2 070
El Centro	1 650	1 800	2 150	2 240
Parkfield	1 670	1 740	1 560	2 110
Overturning Moment (kNm)				
M - P(5)	24 400	24 500	25 400	26 300
ART-1	26 200	26 600	27 000	27 700
El Centro	26 800	25 700	26 500	26 700
Parkfield	26 800	26 400	26 200	26 600
	Frames			
	6	**12**	**18**	**24**
Top deflection (mm)				
(Eq. S - P(6)) x μ	222	428	541	635
(M - P(6) x μ	188	326	406	474
ART-1	173	363	443	522
El Centro	115	258	269	287
Parkfield	256	248	236	223
Base Shear (kN)				
(Eq. S - P(6))	474	534	677	808
(M - P(6))	427	464	590	717
ART-1	624	822	1 106	1 306
El Centro	788	788	1 104	1 304
Parkfield	744	740	1 190	1 190
Overturning moment (kN.m)				
Eq. S. - P(6)	7 930	16 160	29 900	47 650
M - P(6)	6 200	12 100	22 400	35 500
ART-1	7 500	15 600	31 000	49 900
EL Centro	7 400	14 700	27 200	43 600
Parkfield	7 700	14 600	27 800	44 800

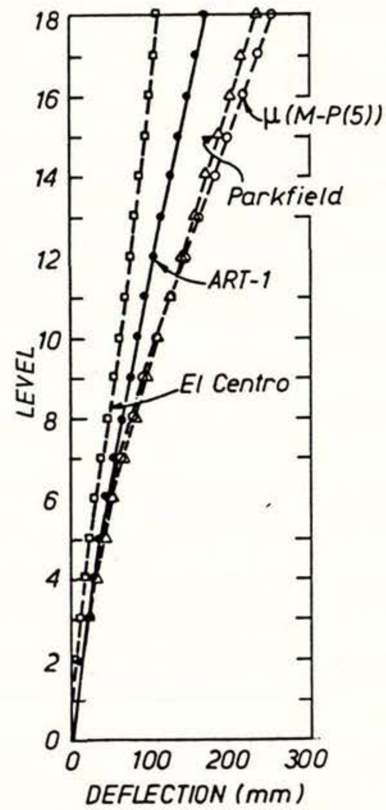
increase which occurs due to strain hardening. The overturning moment for the frames was similar for three earthquake records, but in this case the values observed varied between 1.2 to 1.4 times the modal response spectrum values.

4.2 Deflected Shapes of Frames and Walls

The deflected shape envelopes for the walls for the three different ground motions are shown in Figs. 4.1. (a) to (d). For purposes of comparison the deflected profiles found from the modal method of analysis with the proposed spectrum multiplied by the structural ductility factor are also shown. For the less slender walls the majority of the deflection comes from the rotation of a plastic hinge at the base of the structure with the remainder of the wall being reasonably straight. With the more slender structures some additional deflection arises from flexural hinging to the region of a third to two thirds of the height. In general the deflected profiles are poorly represented by the scaled deflections obtained from the modal analyses.



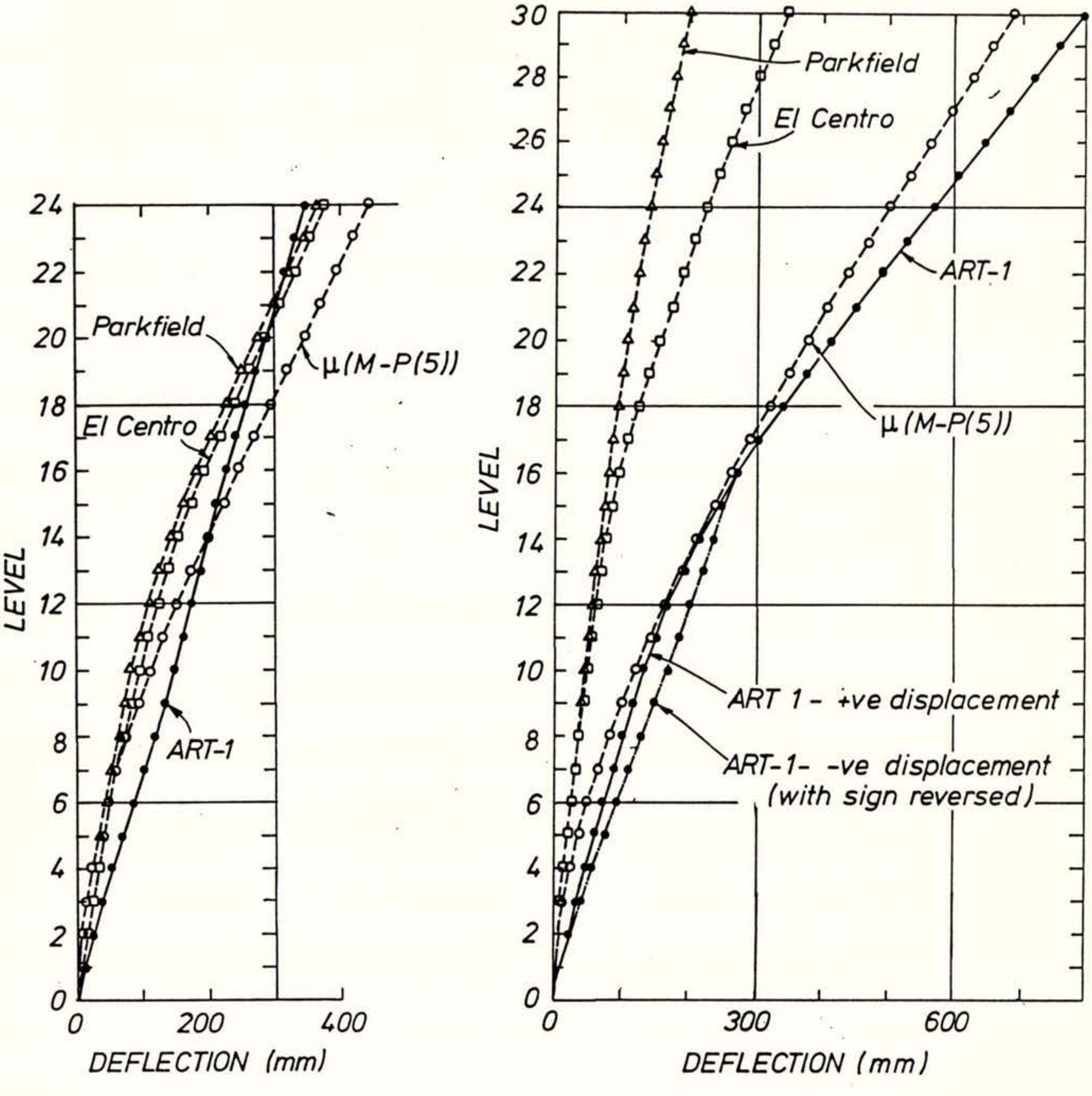
(a) 12 storey wall



(b) 18 storey wall

Fig. 4.1 Deflection envelopes for the walls - continued

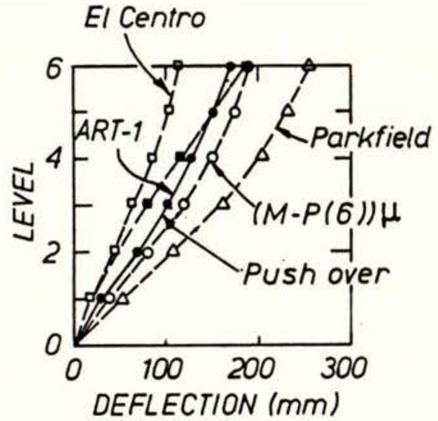
The deflected shape envelopes for the frames are shown in Figs. 4.2 (a) to (d) for the three different ground motions. Two further deflected profiles are also shown. The first of these is obtained by multiplying the modal response spectrum deflection envelope by the structural ductility factor and the second one is obtained from a push-over analysis, where the equivalent static forces were increased incrementally until the deflection at the top level was equal to the corresponding modal response spectrum deflection times the structural ductility factor.



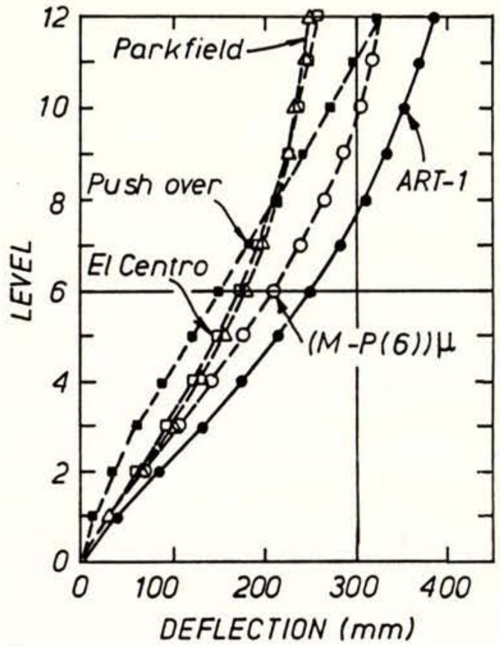
(c) 24 storey wall (d) 30 storey wall

Fig. 4.1 Deflection envelopes for the walls - concluded.

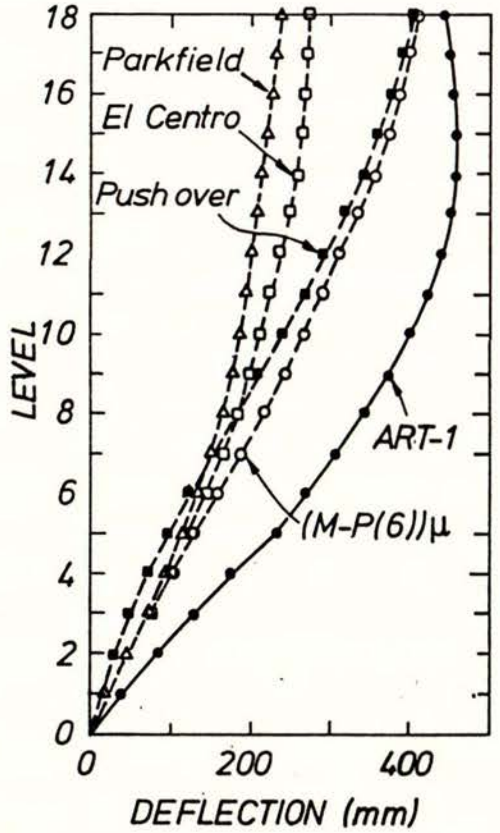
For each of the frames the form of the deflected profile obtained by the three earthquake records is similar, though the magnitudes vary. For the six storey frame the deflected profile is close to linear with height. However, with the 18 and 24 storey frames it is generally parabolic in form. The 12 storey frame is intermediate between these two. Similar observations have been made in previous analyses[10,21,22,23]. It can be seen that the scaled elastic modal analysis values, μ(M - P(6)), with the exception of the six storey frame, provide a poor representation of the deflected profiles. It was felt that the push-over analysis might give a better representation. However, as can be seen from Fig. 4.2 this is not the case and it provides an even poorer match than the scaled modal value.



(a) 6 storey frame

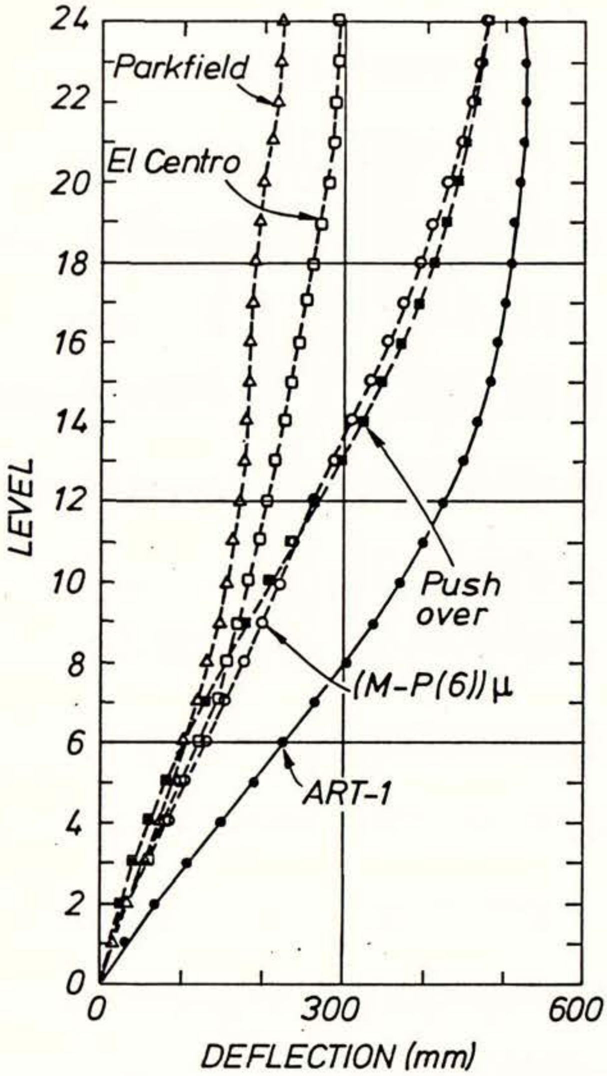


(b) 12 storey frame



(c) 18 storey frame.

Fig. 4.2 Deflection envelopes for frames - continued



(d) 24 storey frame

Fig. 4.2 Deflection envelopes for frames - concluded

As clearly indicated in Figs. 4.1 and 4.2, scaled modal or push-over analyses can not predict the deflected shape obtained in a severe earthquake. This has important implications for predicting interstorey deflections, seismic gaps and P-delta effects. In Table 4.2 the maximum interstorey deflections obtained from the modal response spectrum analyses (M-P(μ)) scaled by the structural ductility factor are compared with the maximum values sustained with the different ground motions for the four frame structures. It can be seen that the scaled modal values for the 18 and 24 storey frames are only of the order of 50 percent of the values found for the corresponding seismic ground motion (ART-1).

Table 4.2. Maximum interstorey deflections for frame structures.

Method of Analysis or Earthquake Record	Interstorey Deflection (mm)			
	6 storey	12 storey	18 storey	24 storey
μ (M - P(6))	44	40	24	25
ART-1	37	46	47	40
El Centro	23	32	22	21
Parkfield	57	32	21	19

To see how the structural ductility factor, used in determining the strengths of the frames, influenced the deflected profile, a series of analyses were made for the 12 and 24 storey frames. The yield strengths in the plastic hinge zones were initially found assuming a structural ductility factor of 6 with the proposed spectrum. These were scaled to correspond to structural ductility factors of 1 (elastic response), 2 and 4. The frames were the subjected to the ART-1 ground motion. The resulting deflected shape envelopes are shown in Figs. 4.3 (a) and (b). To allow comparisons to be made the scaled modal response spectrum profiles are also shown.

A number of trends are apparent from Fig. 4.3. The lateral deflection at the top of the building generally increased with the structural ductility factor in both frames. However, the trend is not consistent and in both cases the deflection sustained when the structural ductility factor was 2 was less than for fully elastic response. For the 24 storey frame the roof deflections ranged between 83 and 112 percent of the scaled modal response spectrum value. The corresponding values for the 12 storey frame were 67 and 115 percent.

The scaled modal response spectrum values are generally in close agreement with the deflected shape envelopes obtained assuming either elastic response or structural ductility factor of two. However, with the higher structural ductility factors the agreement is poor, with the lateral deflections in the lower half of the frames being seriously under estimated by the scaled modal response spectrum values. The discrepancy in these is due to a "higher mode" behaviour that is associated with plastic hinging in the column bases and the lower level beams (Section 4.6).

Table 4.3 shows how the maximum interstorey deflection changed in the 12 and 24 storey frames with the structural ductility factor. These values show that the modal analysis underestimates the interstorey deflection for the 24 storey frame when a high structural ductility factor is used. However, for the 12 storey frame the predictions are reasonable. The same conclusion can be deduced from the deflected shapes shown in Figs. 4.2 and 4.3.

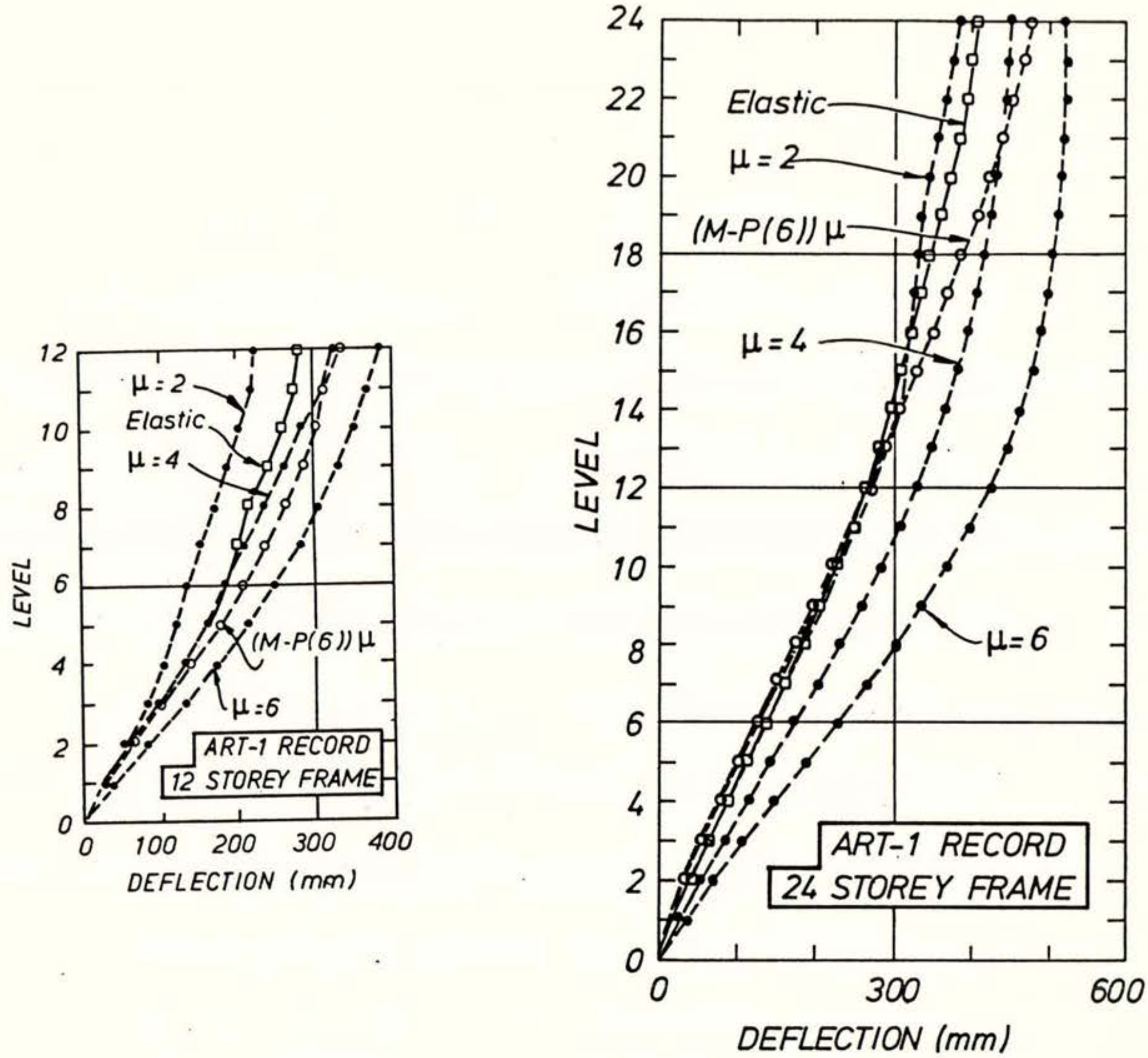


Fig. 4.3 Deflection envelopes for 12 and 24 storey frames with strengths corresponding to structural ductility factors of 1, 2, 4 and 6.

Table 4.3. Ductility factor influence on maximum interstorey deflections

Analysis	12 storey frame	24 storey frame
μ = 1	38	26
2	31	25
4	37	32
6	46	40
μ (M-P(6))	40	25

4.3 Storey Shear Force Envelopes

The shear force envelopes for the walls for the three different earthquake records are shown in Figs. 4.4 (a) to (d) together with the envelope predicted by the modal response spectrum method of analysis. Generally the storey shear force envelopes found from the three time history analyses lie close to each other. The exceptions to this are in the 18 storey wall where the ART-1 ground motion induces a high localised shear at the base of the wall and in the 24 storey wall where a similar action occurs with the El Centro record. In all cases the shear force envelope found from the modal analysis based on the proposed spectrum is smaller than the time history values. If these values are scaled by a factor of two they are generally in reasonable agreement with the time history values except for the narrow band at the bases of the 18, 24 and 30 storey walls. Here the values are on the low side by about 50 percent. With a more realistic method of modelling of the softening of the shear stiffness that occurs in the plastic hinge zones at the bases of these walls it is likely that these values would decrease.

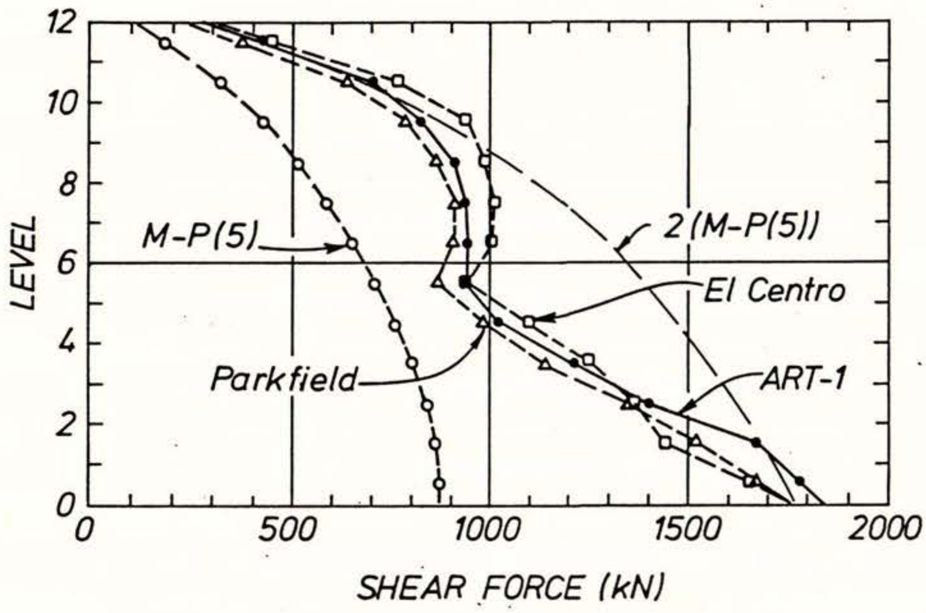
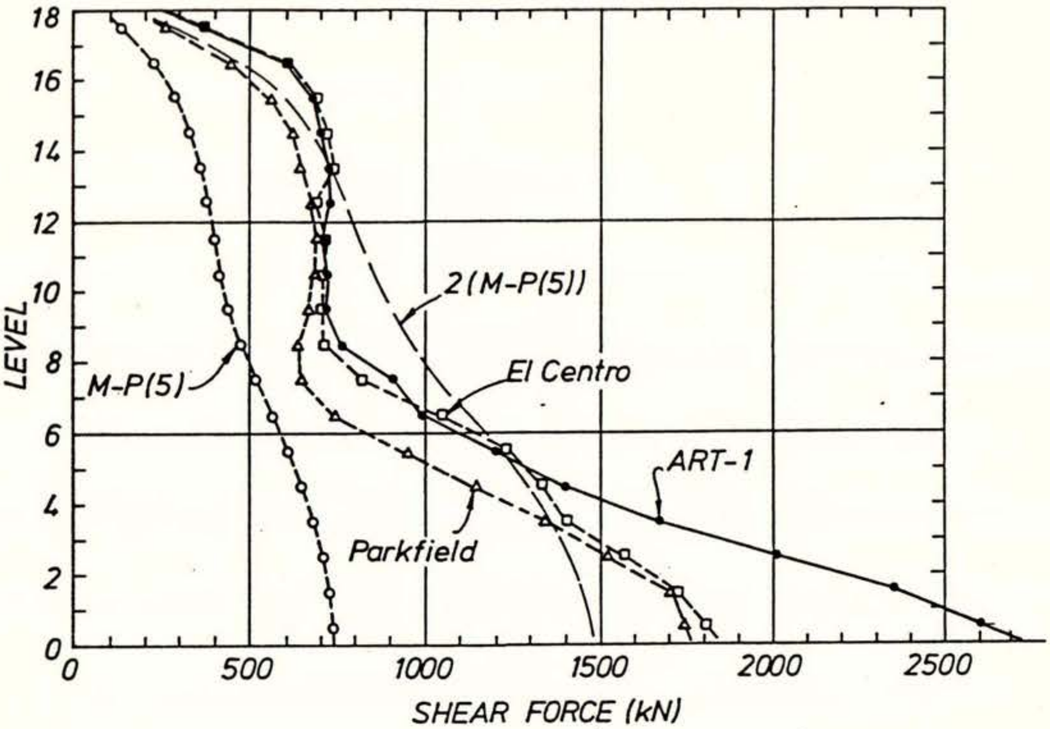


Fig. 4.4 (a) 12 storey wall - continued

The shear force envelopes for the frames for the three different earthquake ground motions are shown in Figs. 4.5 (a) to (d), together with the corresponding values predicted by the modal response spectrum method. The envelopes of storey shear give very similar values for the three earthquake records. As in the case for the walls the time history envelope shears are apparently greater than the corresponding values predicted by the modal analysis. Generally multiplying the modal values by 1.5 gives a conservative estimate of the time history values except in the lower third of the frame. In this zone the scaled values are typically low by 20 percent.



(b) 18 storey wall

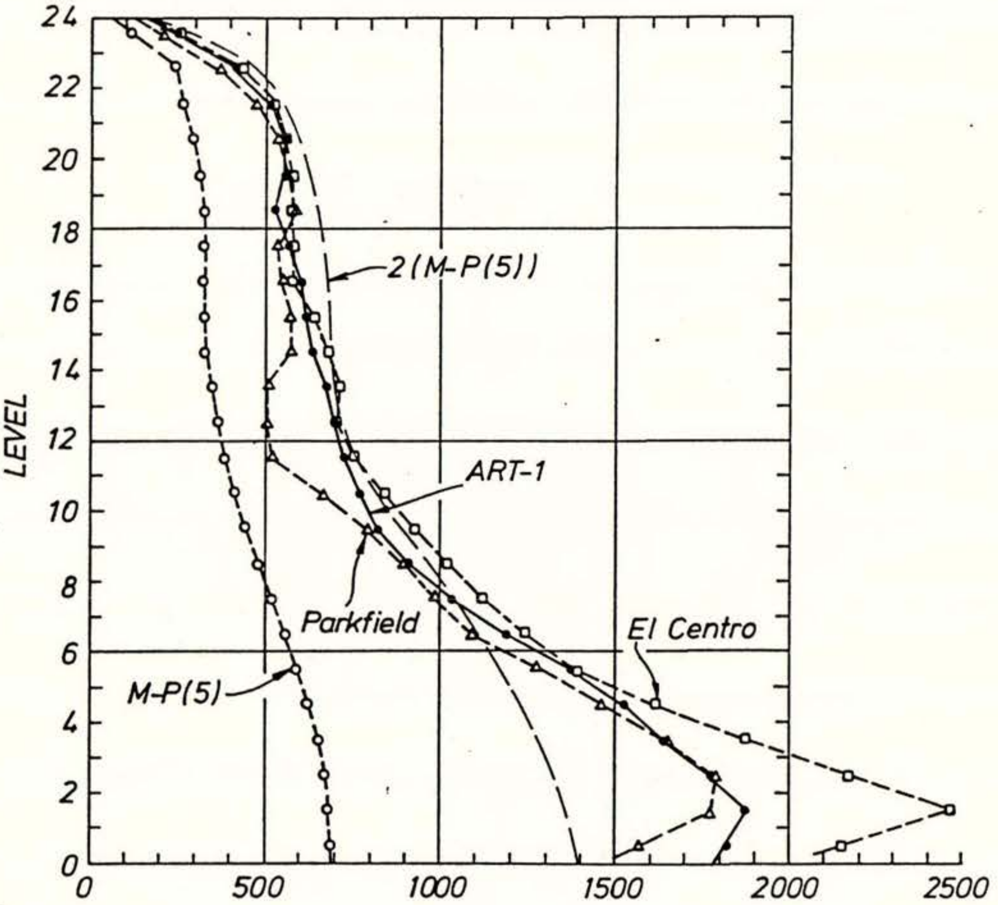
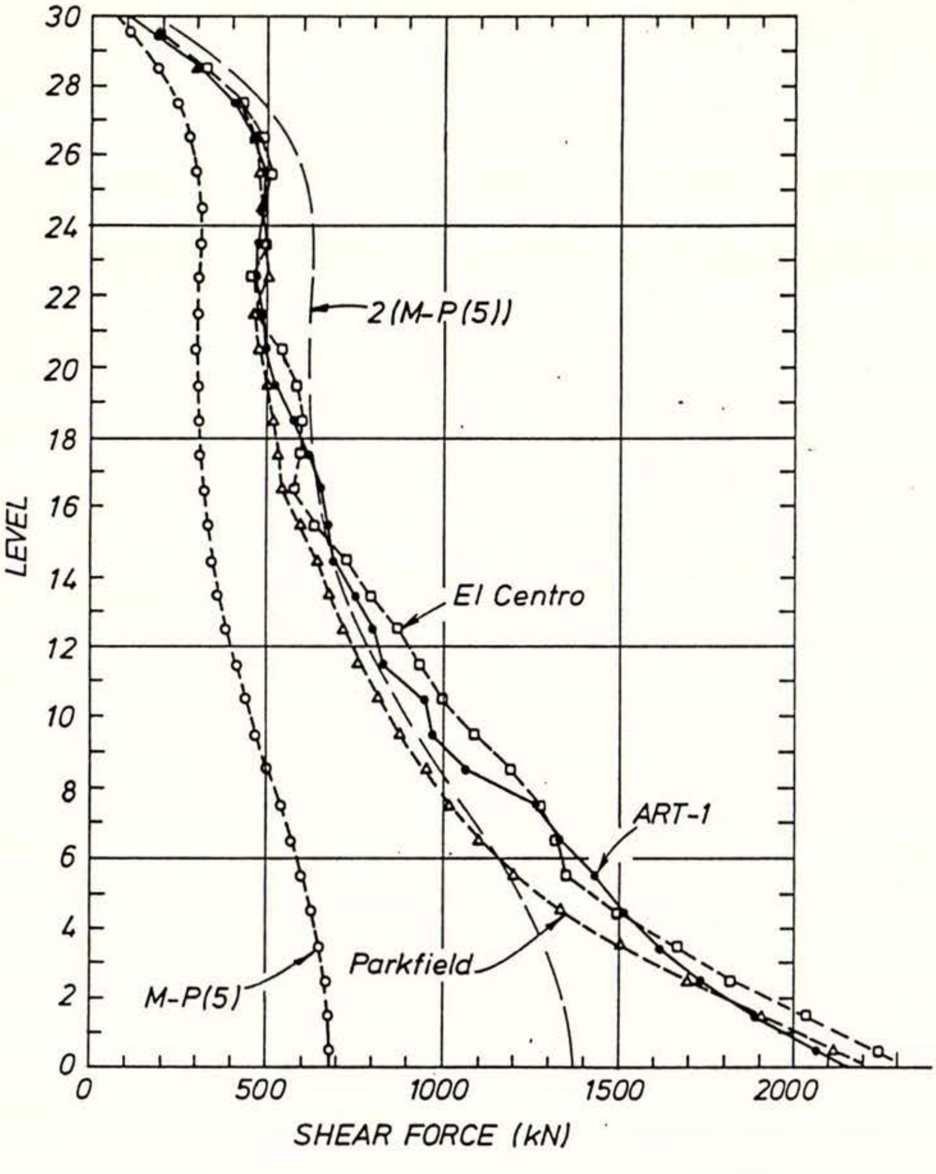
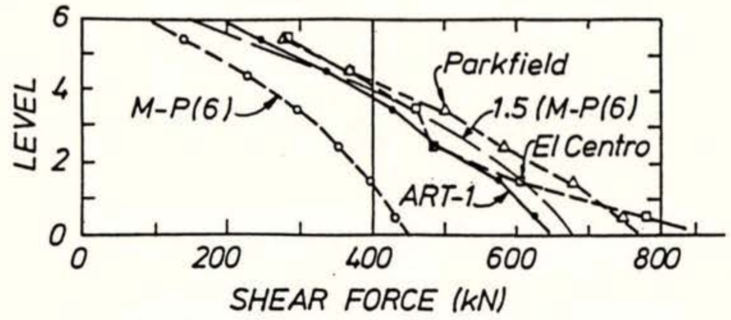


Fig. 4.4 (c) 24 storey wall - continued

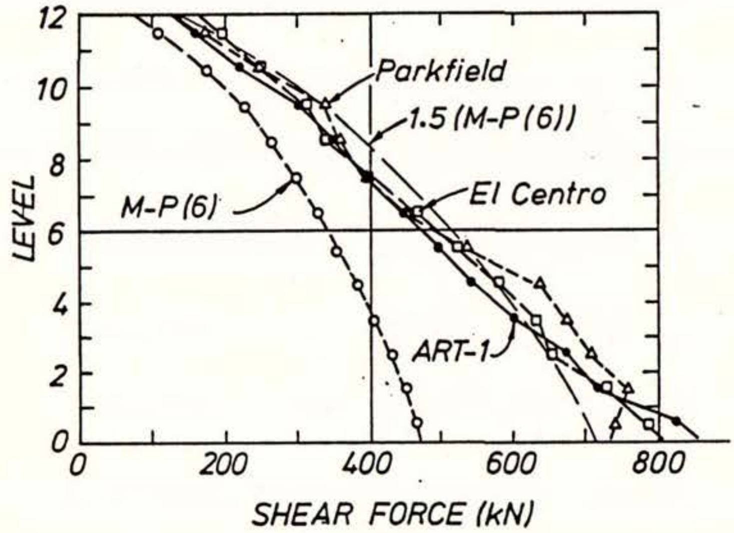


(d) 30 storey wall

Fig. 4.4 Shear force envelopes in walls - concluded



(a) 6 storey frame



(b) 12 storey frame

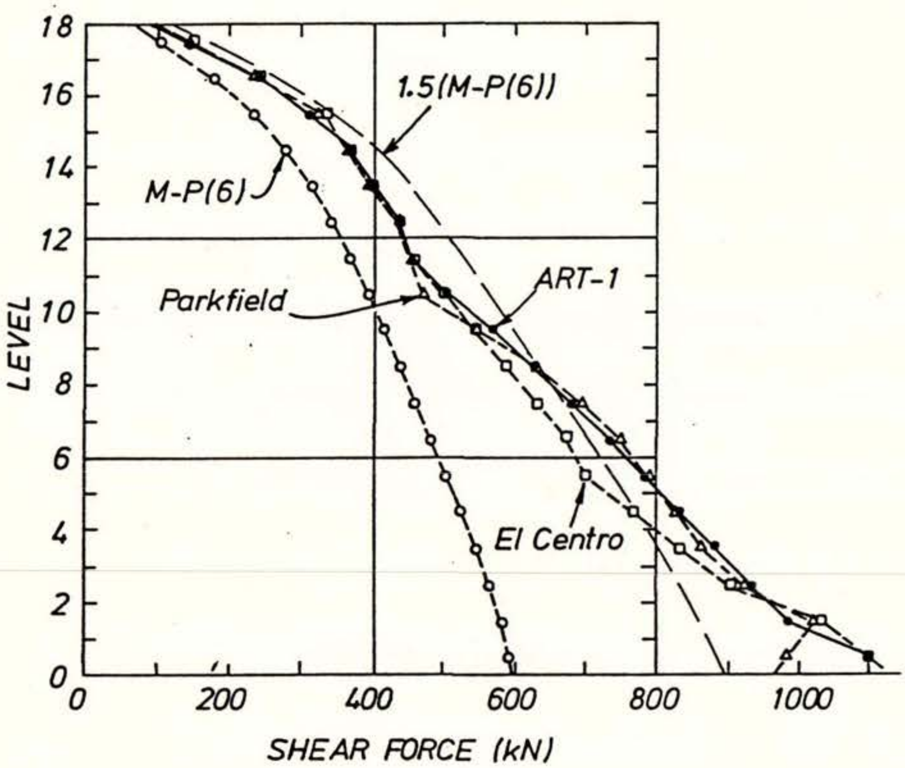
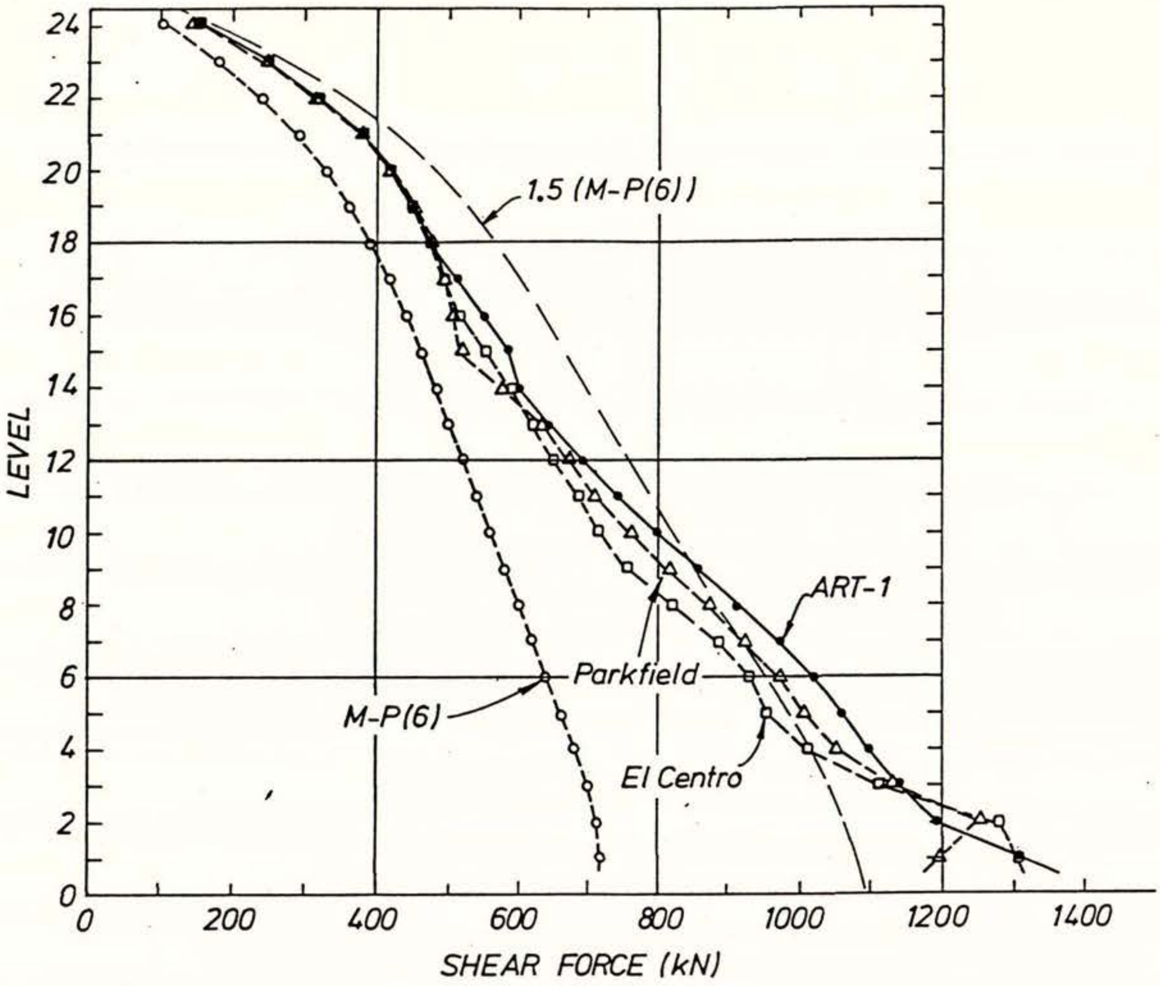


Fig. 4.5 (c) 18 storey frame - continued

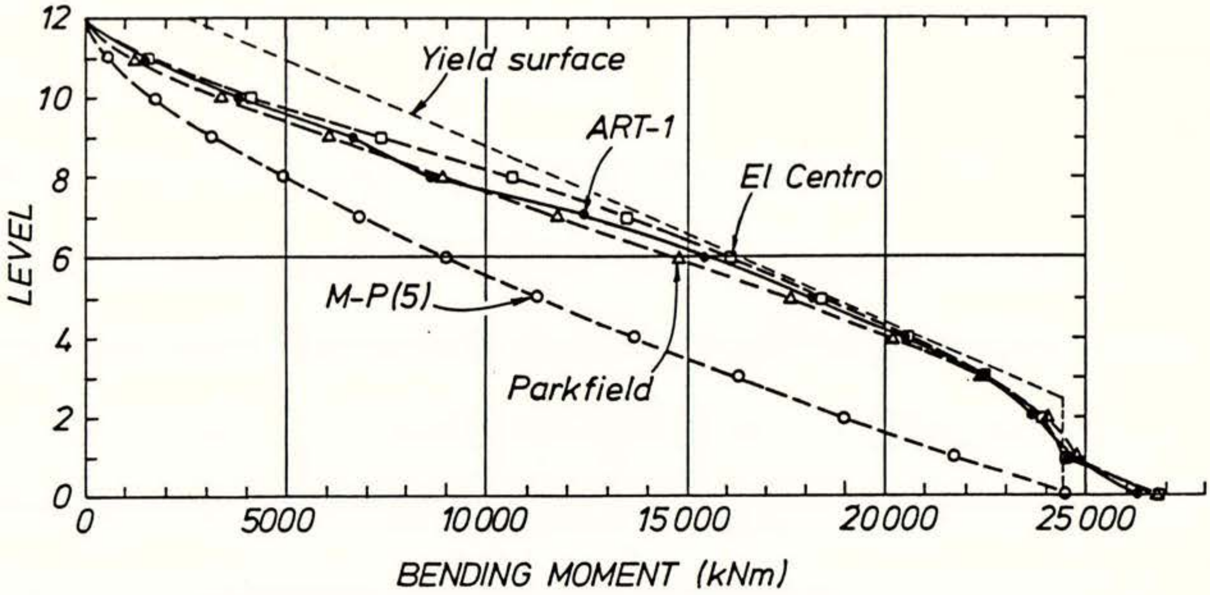


(d) 24 storey frame

Fig. 4.5 Storey shear force envelopes for frames - concluded

4.4 Bending Moment Envelopes in Walls and Inelastic Rotations

The bending moment envelopes for the walls for the three different ground motions are shown in Figs. 4.6 (a) to (d). For the purposes of comparison the modal response spectrum bending moment envelope is also shown together with the initial flexural yield strength of the wall. It can be seen that the values obtained from the three ground motions differ markedly from the modal response spectrum values. In the lower half to two thirds of the wall the maximum bending moments are slightly greater than the initial yield values. This indicates that the envelope was limited by the yield strength of the wall. A "higher mode" response associated with the formation of plastic hinges is believed to cause the departure from the modal response spectrum values (Section 4.6).



(a) 12 storey wall

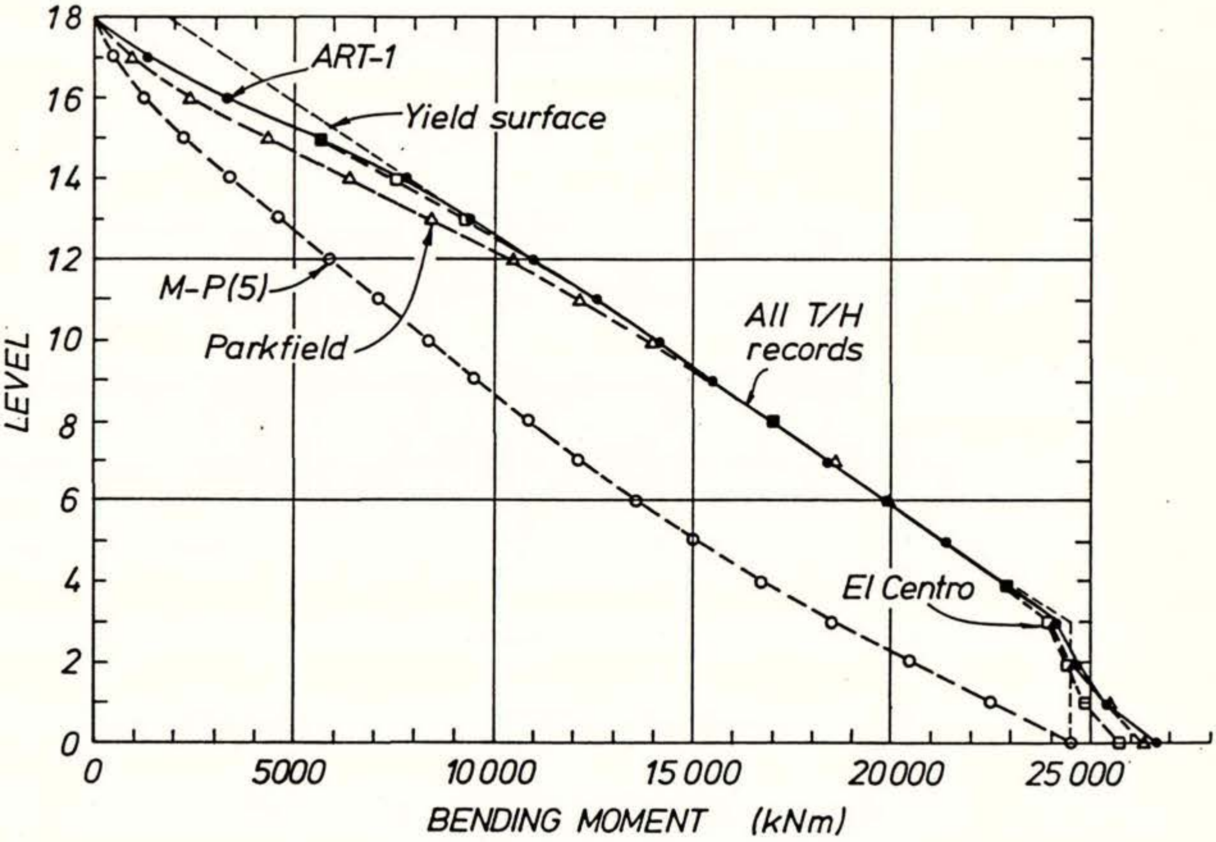


Fig. 4.6 (b) 18 storey wall - continued

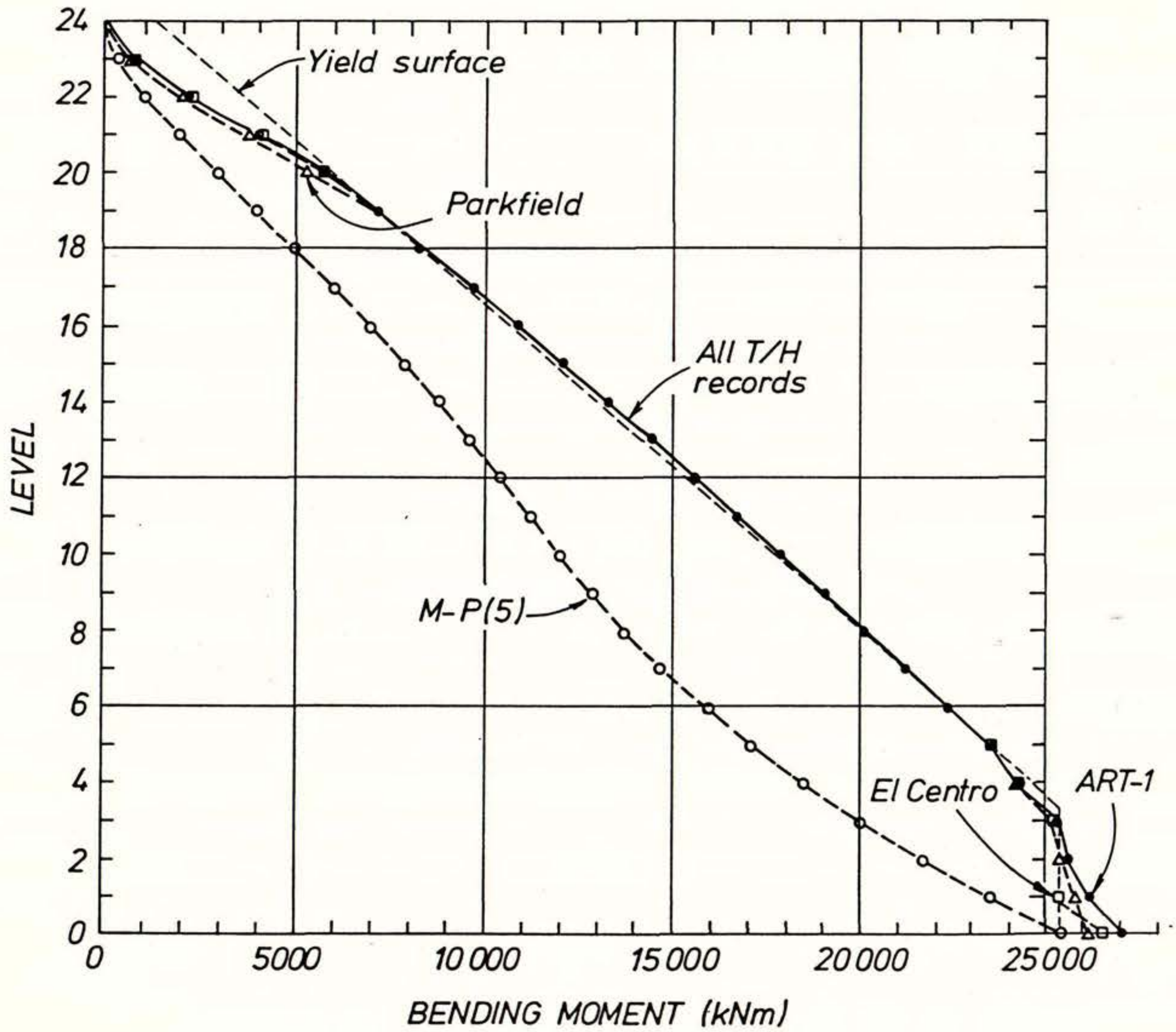
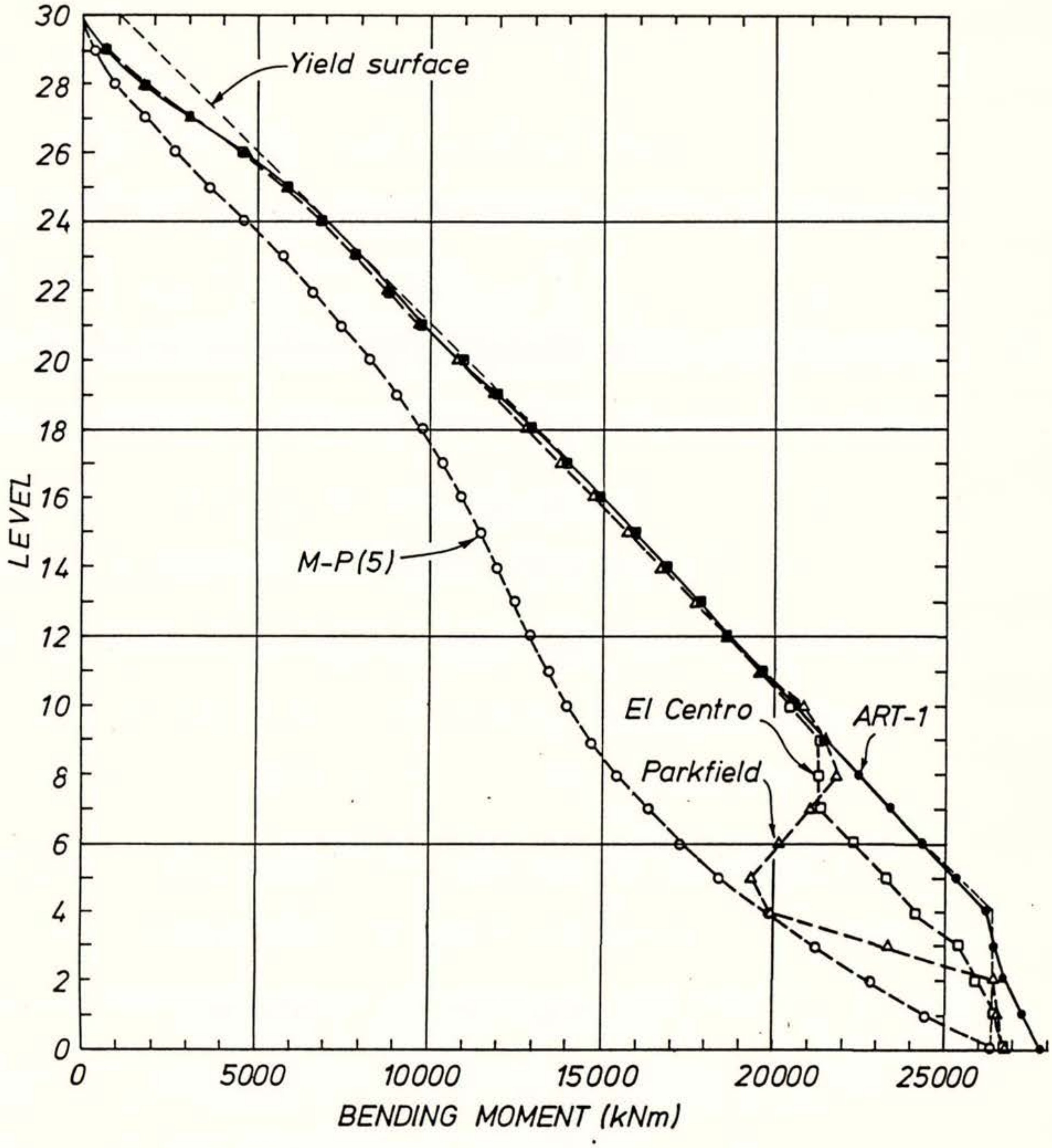


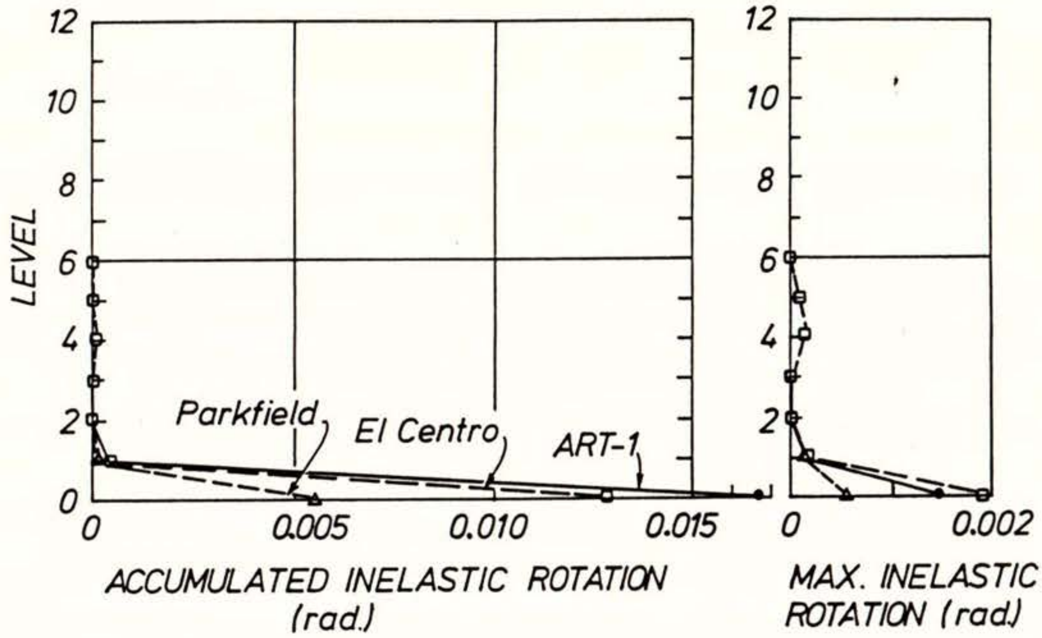
Fig. 4.6 (c) 24 storey wall - continued



(d) 30 storey wall

Fig. 4.6 Bending moment envelopes in walls - concluded

The maximum inelastic rotations in the walls together with the sum of all the inelastic rotations without regard to sign sustained in each ground motion are shown in Figs. 4.7 (a) to (d). These latter values are referred to as the accumulated inelastic rotations and are plotted as rotations sustained at each node point. To convert the maximum inelastic rotations into curvatures all the values except the one at the base need to be divided by the distance between the node points, which is 3 400 mm. In all cases, except for the 12 storey wall, the ART-1 ground motion induces the critical rotations. However, the general pattern obtained with all the ground motions is similar.



(a) 12 storey wall

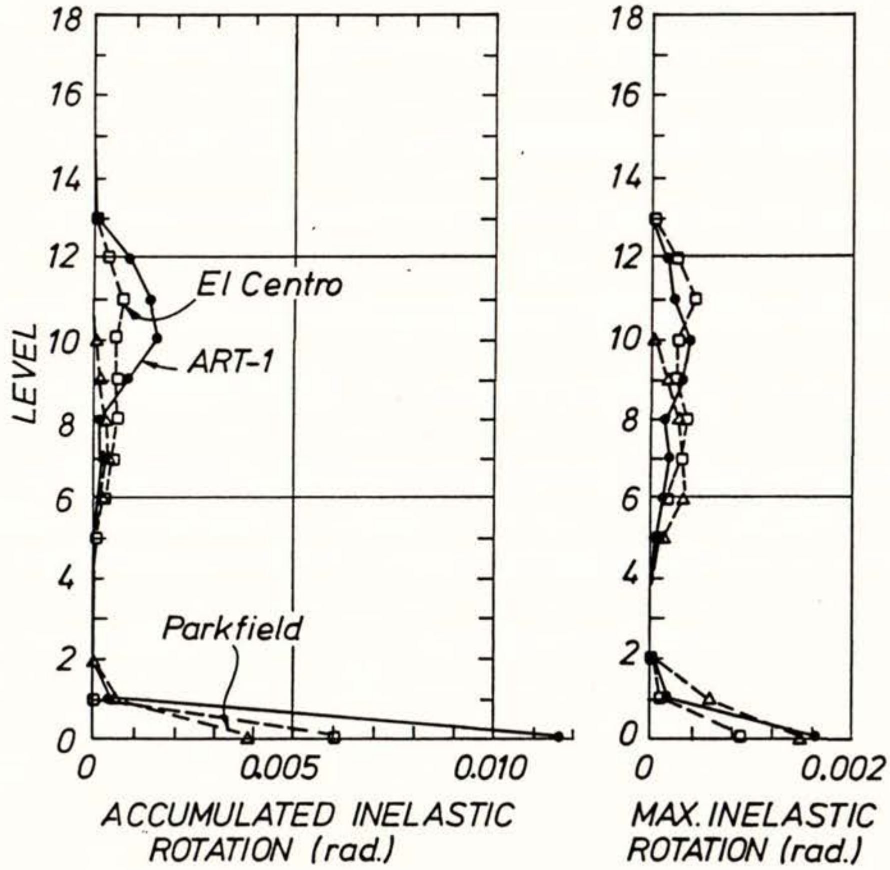


Fig. 4.7 (b) 18 storey wall - continued

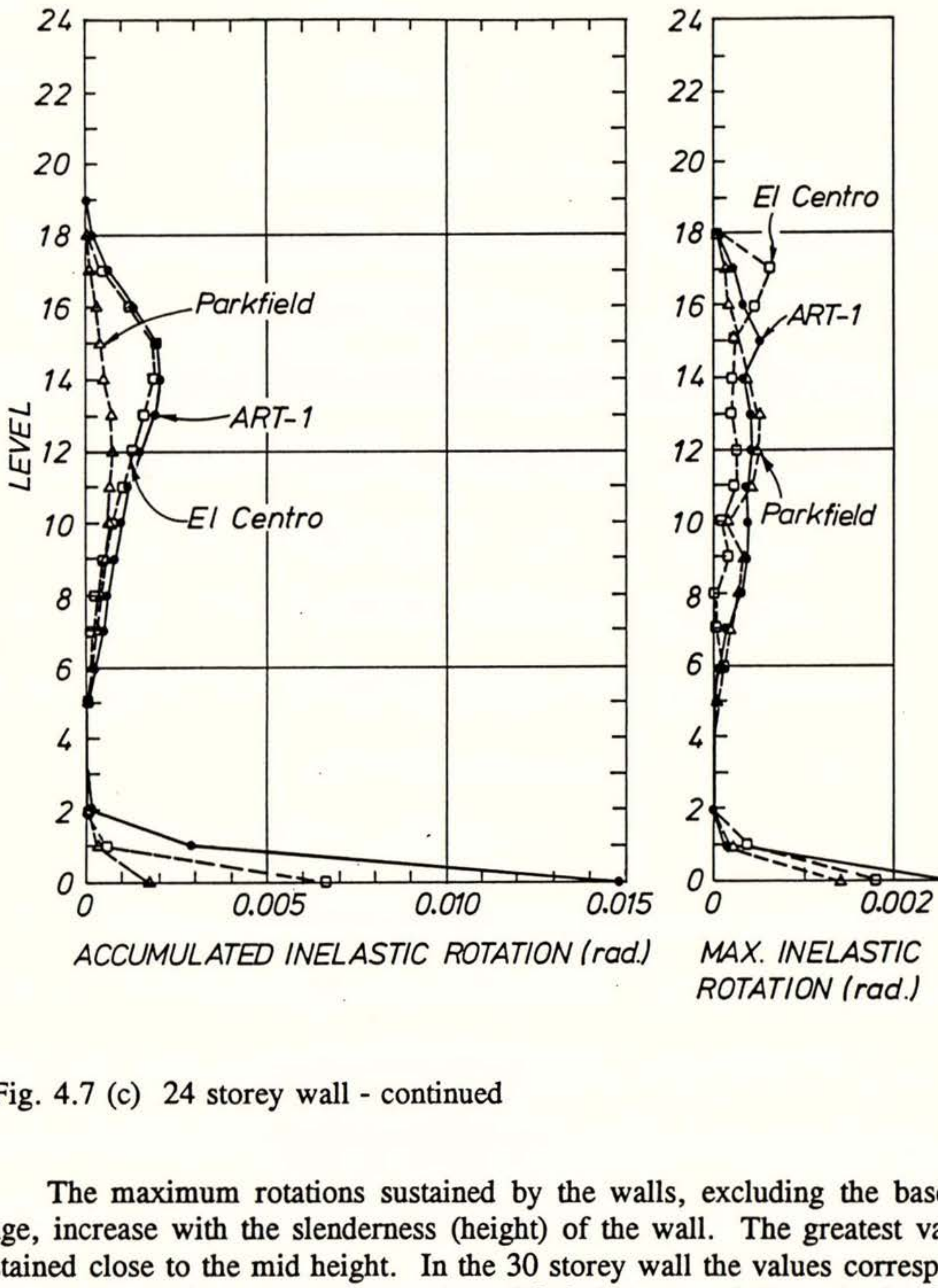
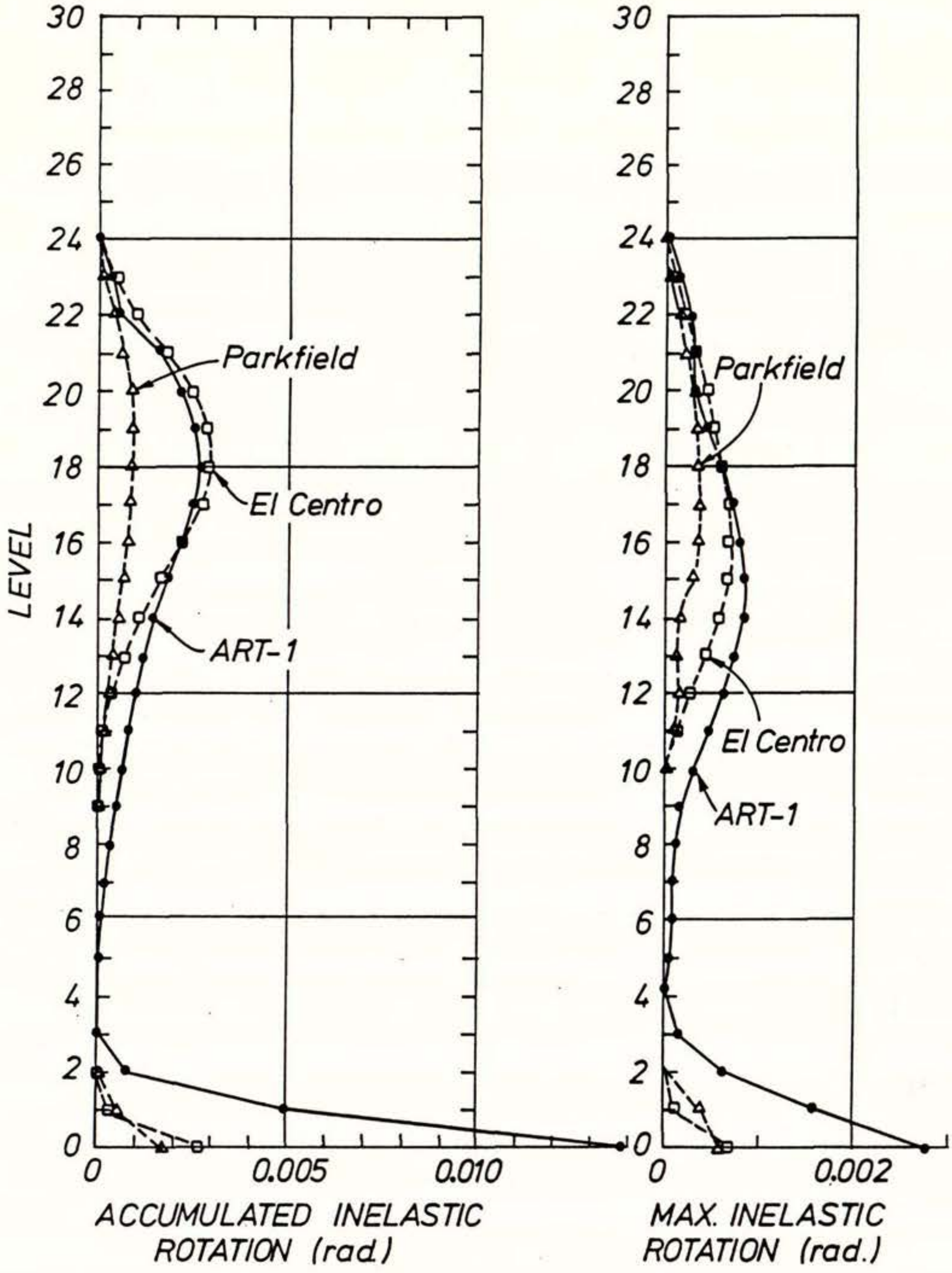


Fig. 4.7 (c) 24 storey wall - continued

The maximum rotations sustained by the walls, excluding the base plastic hinge, increase with the slenderness (height) of the wall. The greatest values are sustained close to the mid height. In the 30 storey wall the values correspond to a strain in the reinforcement of the order of 2½ times the yield strain for 300 MPa steel. The maximum curvature at the base is of the order of two to three times this value.

The accumulated inelastic rotation sustained by the walls indicates that an appreciable portion of the energy is dissipated by yielding away from the base. To suppress this action and confine the energy dissipation to the chosen location some increase in strength is required in the mid-height region. As the strength envelope used in these analyses was based on a conservative interpretation of the design envelope given in the concrete code (Section 2.1) this implies that some revision to this design guide is required for slender walls.



(d) 30 storey wall

Fig. 4.7 Maximum inelastic rotations and accumulated inelastic rotations in walls - concluded

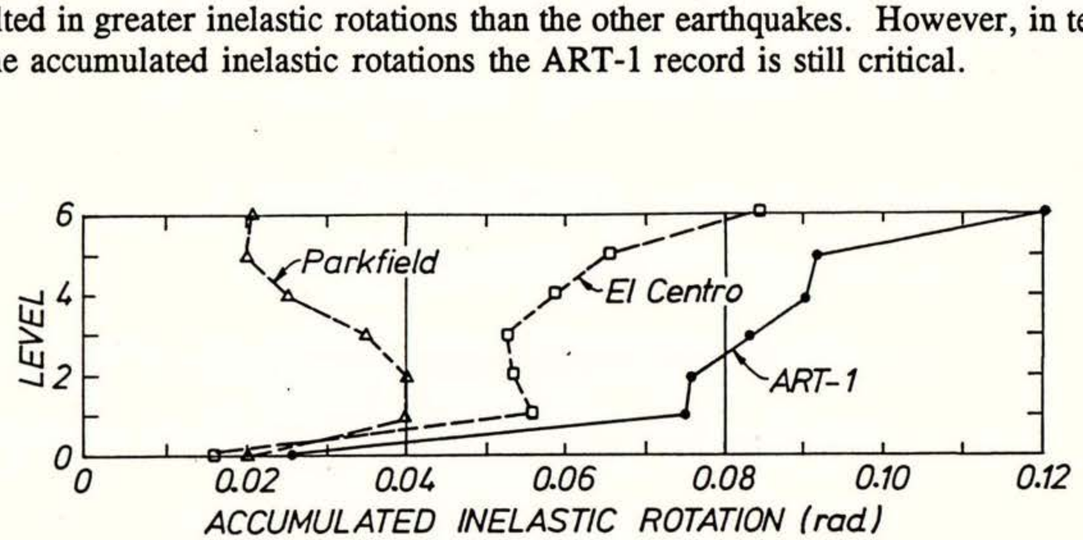
Table 4.4 gives the ratio of the accumulated inelastic rotation sustained at the base of the wall to the maximum rotation required for each earthquake record. The ratios are greatest for the ART-1 record and smallest for the Parkfield ground motion. There is a general trend in that the ratios decrease with an increase in the height of the wall.

Table 4.4. Ratio of accumulated inelastic rotations to the maximum inelastic rotation required a the base of the walls.

WALL	GROUND MOTION		
	ART-1	El Centro	Parkfiel d
12	11.3	6.7	2.9
18	7.1	6.6	2.6
24	5.6	3.6	1.2
30	5.0	4.2	3.0
Average	7.3	5.3	2.4

4.5 Plastic Hinge Rotations in Frames

In Figs. 4.8 (a) to (d) the maximum plastic hinge rotations and accumulated inelastic rotations in the beams where they frame into the external columns are shown for the three ground motions. At the ground level the rotation corresponds to that at the column base. For the 12, 18 and 24 storey frames the ART-1 earthquake ground motion gives the critical values of inelastic rotation and accumulated inelastic rotation. For the six storey frame the Parkfield motion resulted in greater inelastic rotations than the other earthquakes. However, in terms of the accumulated inelastic rotations the ART-1 record is still critical.



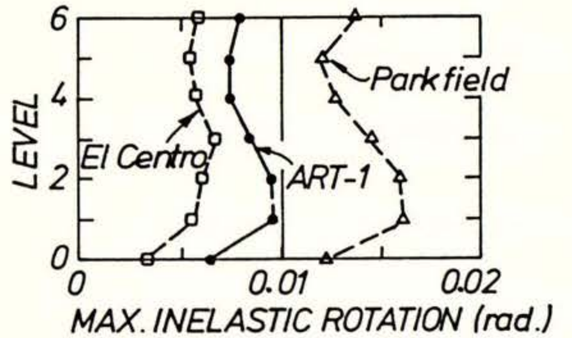
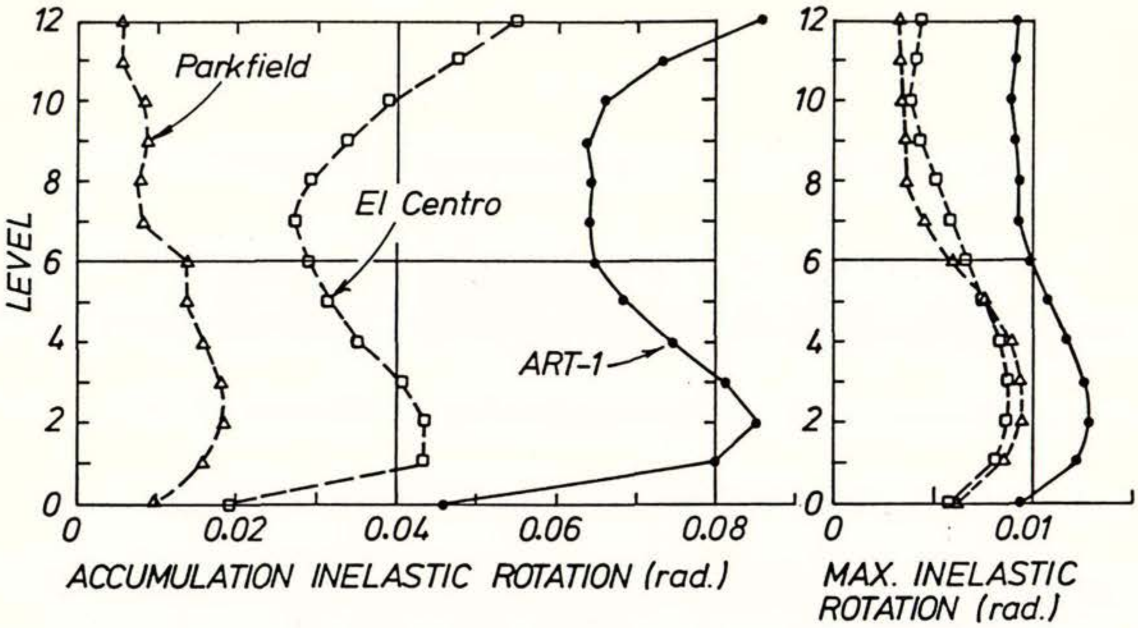


Fig. 4.8 (a) 6 storey frame - continued



(b) 12 storey frame

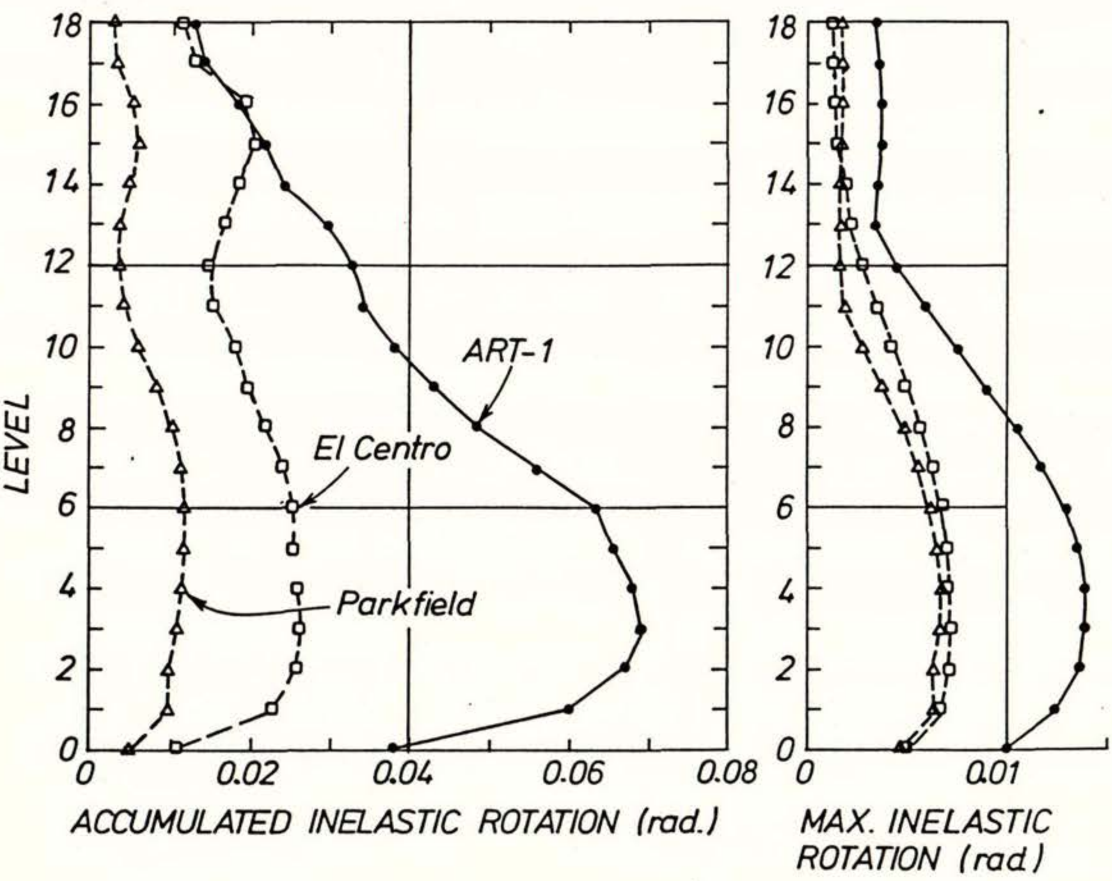
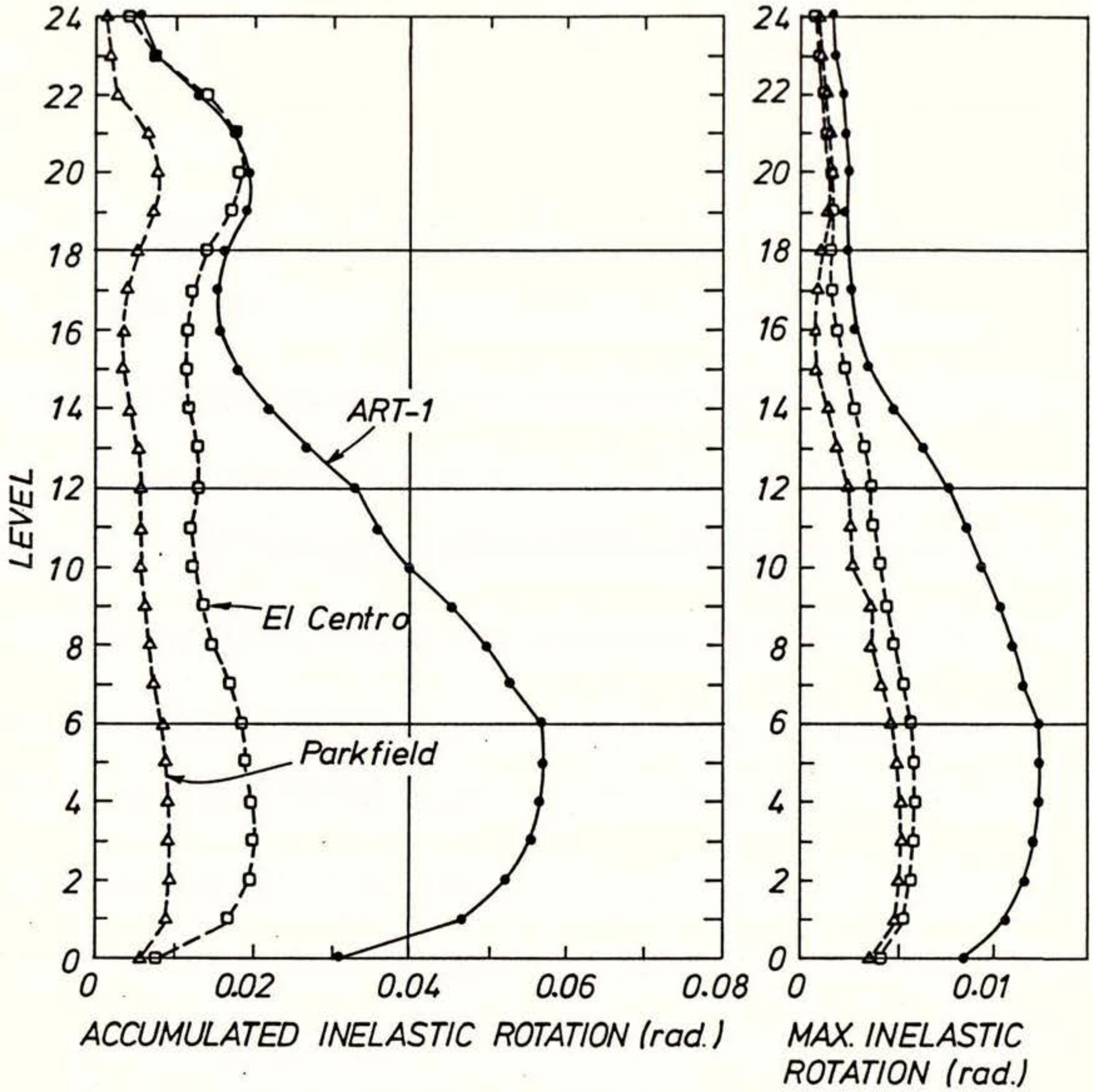


Fig. 4.8 (c) 18 storey frame - continued



(d) 24 storey frame

Fig. 4.8 Maximum inelastic rotations and accumulated inelastic rotations in frames - concluded

Comparing the maximum inelastic rotations required for the three earthquake records a marked difference can be seen between the six storey and the more slender frames. With the six storey structure the inelastic rotations sustained at each level are reasonably uniform with height. However, with the 18 and 24 storey frames greater inelastic rotations are sustained in the lower half of the structure than in the upper reaches. The ART-1 record tends to emphasise this characteristic.

It should be noticed that the inelastic rotation demands placed on the beam hinges in the upper levels of the 18 and 24 storey frames are low compared with other regions of the frame. Consequently an underestimate of the strength of these regions, which was indicated might occur with the equivalent static method (Section 3.6), would have little significance for the structure for the ultimate strength limit state.

For the frames the ratios of the accumulated inelastic rotation to the maximum inelastic rotation were found for the three ground motions for the beam sustaining the greatest inelastic rotation. The values are shown in Table 4.5. The average values are greatest for the ART-1 and lowest for the Parkfield ground motions. Generally they appear to decrease as the fundamental period of the structure increases. The trends are similar to those found for the walls.

Table 4.5. Ratio of accumulated inelastic rotations to maximum inelastic rotation in beams.

FRAME	**GROUND MOTION**		
	ART-1	**El Centro**	**Parkfiel d**
6	7.8	7.7	2.5
12	6.5	5.0	2.0
18	4.9	3.7	1.6
24	4.6	3.8	1.9
Average	6.0	5.0	2.0

4.6 Discussion

A comparison of the results of analyses using the ART-1 ground motion and the corresponding values from the modal response spectrum method shows that the agreement between them is generally poor. The exception to this is the deflection of the top level, which it appears can be estimated from the modal response spectrum analysis.

As illustrated in Figs. 4.3(a) and (b) the discrepancy between the time history envelope values and the corresponding modal values increases with the structural ductility factor. It is suggested that this occurs due to a "higher mode effect" associated with the formation of plastic hinges in the structure. This is illustrated for walls in Fig. 4.9. In Fig. 4.9(a) a plastic hinge forms at the base of the wall with the overall deflections and bending moments corresponding to first mode type of behaviour. With the plastic hinge acting for a length of time any further increase in bending moments cannot be sustained at this point. Hence for higher mode actions the base acts like a pin for the duration of time that the base hinge exists. During this interval the ground motion may excite the first mode (or higher mode) of the modified structure inducing the deflected shape and bending moments shown in Fig. 4.9(b). As the higher mode bending moments increase so the sum of the first mode and modified higher mode values start to look like the values found in the time history analysis. The contribution that this "modified higher mode action" can make to the resultant bending moment envelope should increase with the time that a plastic hinge is active at the base of the wall and hence the structural ductility factor.

A similar "modified higher mode behaviour" can be predicted for frame structures. Fig. 4.10(a) shows plastic hinges forming in response to a deflected shape similar to the first mode, with the structure swaying to the left hand side. The "modified higher mode" behaviour is illustrated in Fig. 4.10(b). With the deflection in the direction shown, the hinge rotations below line A-A are additive to those induced by the first mode type behaviour. Consequently the inelastic rotations increase in this zone. However, above this level the rotation is in the opposite direction and hence this region of the structure is likely to be returned to elastic behaviour. This would reduce the inelastic deformations in this part of the structure.

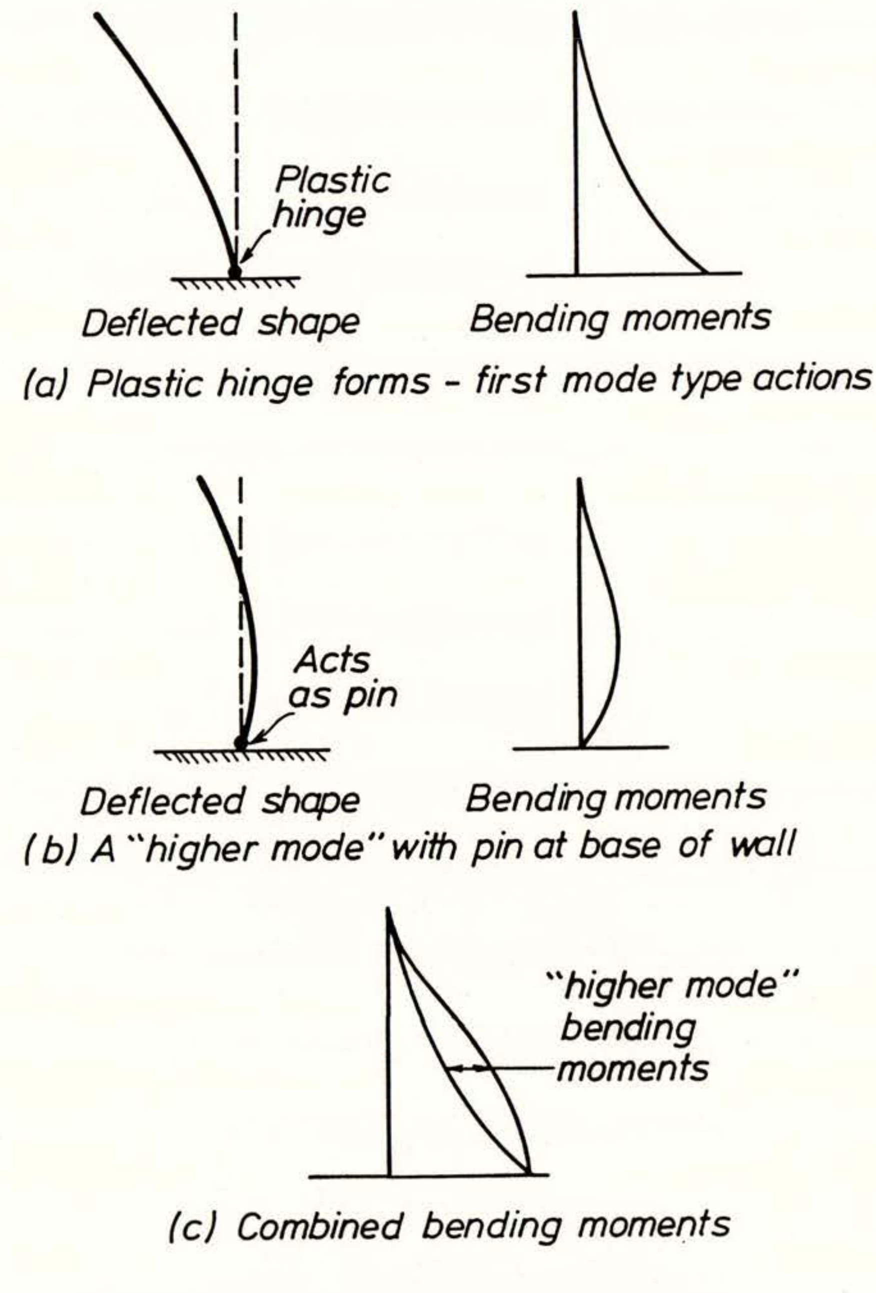


Fig. 4.9 "Higher Mode" Effects for a Structural Wall.

When the "modified higher mode" deflections develop in the opposite direction to that shown in Fig. 4.10 (b) the inelastic rotations of the hinge zones below A-A are reduced. The reversal of rotation is likely to cause an elastic response for this "modified higher mode" deformation in that direction. The non-symmetrical response of the structure in this modified mode, elastic in one direction and inelastic in the other, could well explain why the more slender ductile frame structures tend to develop appreciably greater later deflections in the lower storeys in the time history analyses than those predicted in a response spectrum modal analysis.

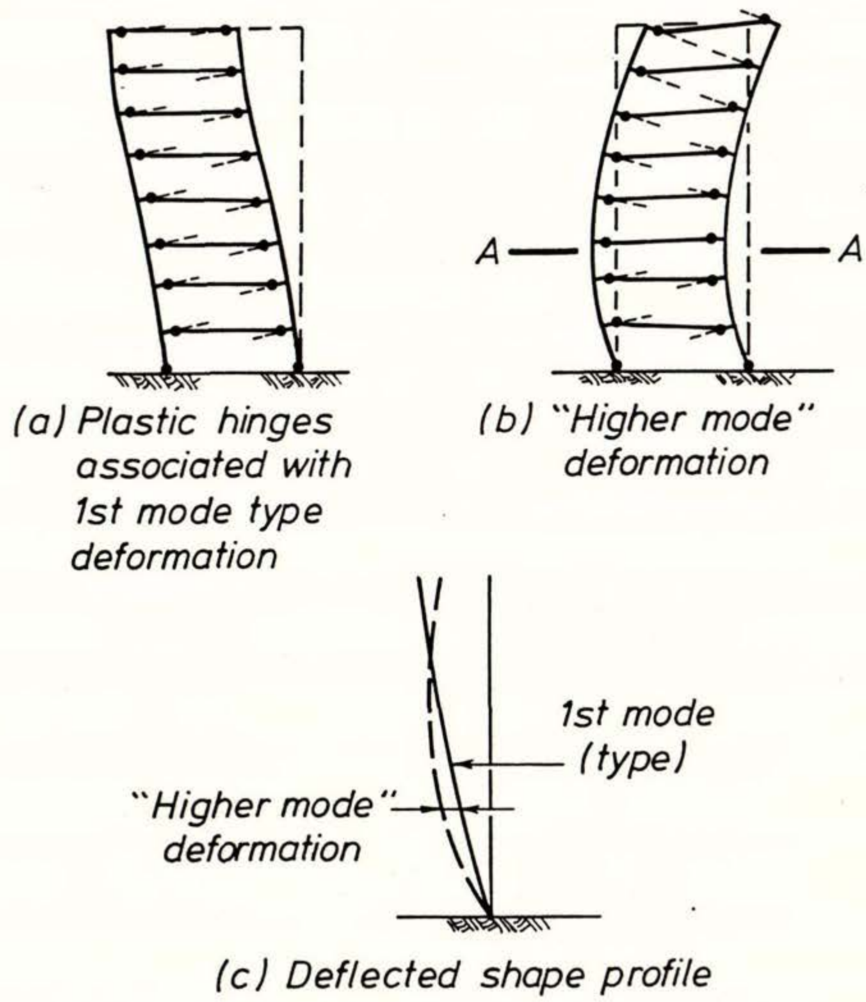


Fig. 4.10 "Higher Mode" Effects for a Structural Frame.

CHAPTER 5 - CONCLUSIONS

5.1 Overview

This project has two main objectives.

(i) To investigate how the use of the design spectra proposed in Section 1.1 alters the calculated structural response compared with the response calculated using the design spectra in the draft code[3].

(ii) To compare the seismic response of structures as predicted by using different linear analysis methods.

To help measure the accuracy, and the adequacy of the linear analyses, it has been assumed that the nonlinear time history analysis provides the most accurate estimate of the structural responses during an earthquake. This is done acknowledging that when using this method large variations of the structural response can arise from different earthquake ground motions and variations in the nonlinear modelling of the structure.

The assessment of the methods of analysis, the Equivalent Static method, the Modal Response Spectrum method, and the Modal Equivalent Static method with their incorporation of the Draft Code[3] spectrum or the Proposed Spectrum is assisted with the restatement of the reasons for their use. They are used in design to obtain primary actions, the values of which are used in the detailed design of structural sections which govern the non-linear behaviour of the structure. For example, the overturning moment at the base of a wall. They also provide estimates of other structural behaviour such as the shear in a wall or interstorey deflections. The magnitude of these secondary responses enable the adequacy of the design to be checked and may influence the redesign of the primary actions. So for the assessment of the linear methods we ask:

(i) Are they theoretically correct?

(ii) Are they easy to use?

(iii) Do they provide a consistent set of primary member actions so that a structure designed to them behaves adequately during an earthquake?

(iv) Do they accurately predict the response of the structure when it is designed using the primary member actions?

5.2 The Influence of the Proposed Spectrum on Structural Response

The draft code spectra were developed correctly for single degree of freedom systems and as such would be expected to provide good guidance in the design of single degree of freedom structures. The use of the modal response spectrum method for non-linearly behaving structures has no theoretical validity hence there is no theoretical reason to incorporate the use of the draft code spectra with it. The proposed spectra are a modified version of the draft spectra, and they give, for linearly behaving structures, the theoretically correct results. They are also easier to use that those in the draft code in the design of ductile structures.

To provide answers to questions (iii) and (iv) above the influence of the different spectra on the results of the linear analyses are summarised.

The Equivalent Static method of analysis uses only the acceleration ordinate of the fundamental mode $C_b(T_1, \mu)$ which takes the same value for both sets of spectra. Hence the choice of spectra does not influence the results.

If the Modal Response Spectrum method of analysis is used, the choice of spectra does not greatly affect the structural responses. Deflections and overturning moments of walls and frames are dominated by first mode effects and as a consequence of this, the influences of differences in the spectra are small. A similar argument holds for the interstorey and base shear in the frames. In walls however, second and higher modes influence the distribution of shear. Since the proposed spectra have relatively lower values at the periods of the higher modes, the use of these provides a relatively smaller higher-mode contribution. This results in a smaller value of shear down the height of the wall and a distribution of shear which is smoother than that calculated using the draft code spectra. A typical example of these effects are shown in Fig. 3.3(b) for the 18 storey wall.

5.3 The Influence of the Method of Analysis on Structural Response

The Modal Response Spectrum Method of analysis is the only method which is theoretically correct for linearly behaving structures, so if one method of analysis can be used for both linear and nonlinear responding structures it would be a logical choice. The Equivalent Static and Modal Equivalent Static Methods have evolved however because of their ease of use in the design process.

The structural responses discussed in this section are those calculated by the various methods of analysis using the spectra proposed in this project. The relative influence of the spectra on the responses is small as described in the previous section.

The Equivalent Static Method of analysis gives rise to the largest deflections. As discussed in Section 2, this is because this method assumes that the total seismic weight of the structure acts while the structure responds in a primarily "first mode" behaviour. The Response Spectrum methods of analysis in contrast calculate that approximately 70% of the seismic weight acts in the first mode, and since this mode dominates the deflection, these methods give rise to correspondingly smaller values. The Modal Equivalent Static method calculates a set of inertia forces from the Modal Response Spectrum interstorey shears and applies these statically to the structure. As a result, the use of this method implies, incorrectly, that the maximum probable interstorey shears occur at the same time. This gives rise to deflections that are greater than those calculated from the Modal Response Spectrum method. In the taller structures where the higher modes contribute relatively more to the shear, this error is amplified to a degree that for the 30 storey wall, the top floor deflection calculated is greater than that calculated by using the Equivalent Static Method.

The trends in the magnitudes of the overturning moments calculated by the different methods follow closely those discussed for the deflections in the preceding paragraph. That is, in general the Equivalent Static method gives rise to the largest values, the Modal Response Spectrum method, the smallest values, and the Modal Equivalent Static method values which increase from marginally greater that Modal Response Spectrum method values for the shorter structures, to values greater than those calculated by the Equivalent Static method for the tallest structures.

The distribution of shear up the structure is, by definition, the same whether it is calculated by using the Modal Response Spectrum or the Equivalent Static method of analysis. For the shorter structures where first mode behaviour dominates, the Equivalent Static method with its over estimation of first mode seismic weight gives rise to larger shears. However, for the taller wall structures, where the higher modes influence the shear, the two Response Spectrum methods calculate the shear to be greater.

5.4 The Adequacy of the Analysis Methods

The time history analyses described in Section 4 were based upon structures designed using the actions obtained from the Modal Response Spectrum method and the proposed spectra. As such, in a correct sense, they can only be used to judge the adequacy of that analysis method and that spectrum. However, for some structures the methods of analysis predicted the same or similar primary design actions and in these cases they too can be compared.

5.4.1 Structural Walls

The primary design action for the walls is the moment at the base and the Modal Response Spectrum method calculates the same value for it using either spectra. The secondary responses upon which to judge the adequacy of this choice are the top floor deflection and the shear distribution up the structure.

The top floor deflections predicted by the Modal Response Spectrum method, using either spectrum, are practically the same (Table 3.3) and when scaled by the structural ductility factor give a reasonable estimate of the time history results as shown in Table 4.1. The Equivalent Static and Modal Equivalent Static method however, predict larger deflections and require larger design base moments. These two results are in conflict, as we would expect walls designed to larger base moments to result in relatively smaller top floor deflections.

The distribution of shear calculated from the Modal Response Spectrum method using either spectrum is less than that determined by time history analyses and this needs to be scaled for design purposes. As current design practice based upon the responses obtained from the Equivalent Static method requires a scaling of the shear this concept is acceptable. The factors required to scale the shears derived from the draft spectrum are slightly less than those for the proposed spectrum.

5.4.2 Structural Frames

The interstorey shear is a good measure of the primary design actions, the beam moments used in the frames. Table 3.3 and Fig. 3.4 show that the Modal Response Spectra method calculates similar values for design using either spectra.

The interstorey deflection and shear can be used as a measure of the adequacy and accuracy of the analysis methods. As seen in Fig. 4.5 the shear in the frame needs to be scaled by a value of 1.5 to approximate the values calculated in the time history analyses. The scaled (by μ) interstorey deflections tend to be amongst a large scatter of time history results as shown in Table 4.2. It appears to be very difficult to be able to predict interstorey deflections using the Modal Response Spectrum method and it would be reasonable to estimate the envelope values by using an additional scale factor equal to that chosen for the interstorey shear; proposed here as 1.5.

5.5 Summary

5.5.1 Choice of Response Spectra

Using the modal response spectrum method with both the draft code and the proposed spectra was found to lead to minor differences in the primary design actions in the frames and walls. On the basis of the time history analyses it was not possible to judge one spectra to be more accurate than the other. However, in terms of both simplicity and consistency with seismic design codes in the rest of the world the proposed spectra are preferred over the draft code spectra.

5.5.2 Method of Analysis

The modal method of analysis is accepted as the best routine approach for elastically responding structures. Consequently the Equivalent Static and Modal Equivalent Static are judged against this method.

The Equivalent Static method generally over-estimate the deflections and storey moments compared with the modal method. The exception to this is that it under-estimates storey shears in flexure type structures (Walls). However, provided the fundamental period is less than 2 seconds the discrepancy is generally acceptable for design purposes.

The Modal Equivalent Static method over-estimates bending moments and deflections. The discrepancy increases with longer period structures and it is most severe with flexural type structural frames such as walls. Where structural walls and frames are mixed it gives an incorrect distribution of load between the two forms. Generally little is gained in accuracy if this method is used instead of the Equivalent Static method.

5.5.3 Time History Analyses

With the exception of the deflection at the top of the structure the structural actions found in time history analyses could not be predicted from elastic based Modal, Equivalent Static or Modal Equivalent Static analyses. To cover such actions as the shear in walls or bending moments in the columns some scaling of the predictions is required.

One of the reasons for the failure of the elastic based methods of analysis to predict the results of inelastic time history analyses is identified as "higher mode responses" which develop when plastic hinges form in key parts of the structure. It is hoped that the study of these "higher modes" will enable modal methods to be modified to give more realistic predictions.

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APPENDIX A Numerical Values of Properties of Beams and Columns

TABLE A.1 Member Sizes for Regular Walls

	12 STOREY	18 STOREY	24 STOREY	30 STOREY	FD30 WALL
I_e (m^4)	17.65				11.55
A (m^2)	3.24				1.98
A' (m^2)	2.88				1.65
α_{sr}	0.0061	0.0043	0.0034	0.0028	0.002

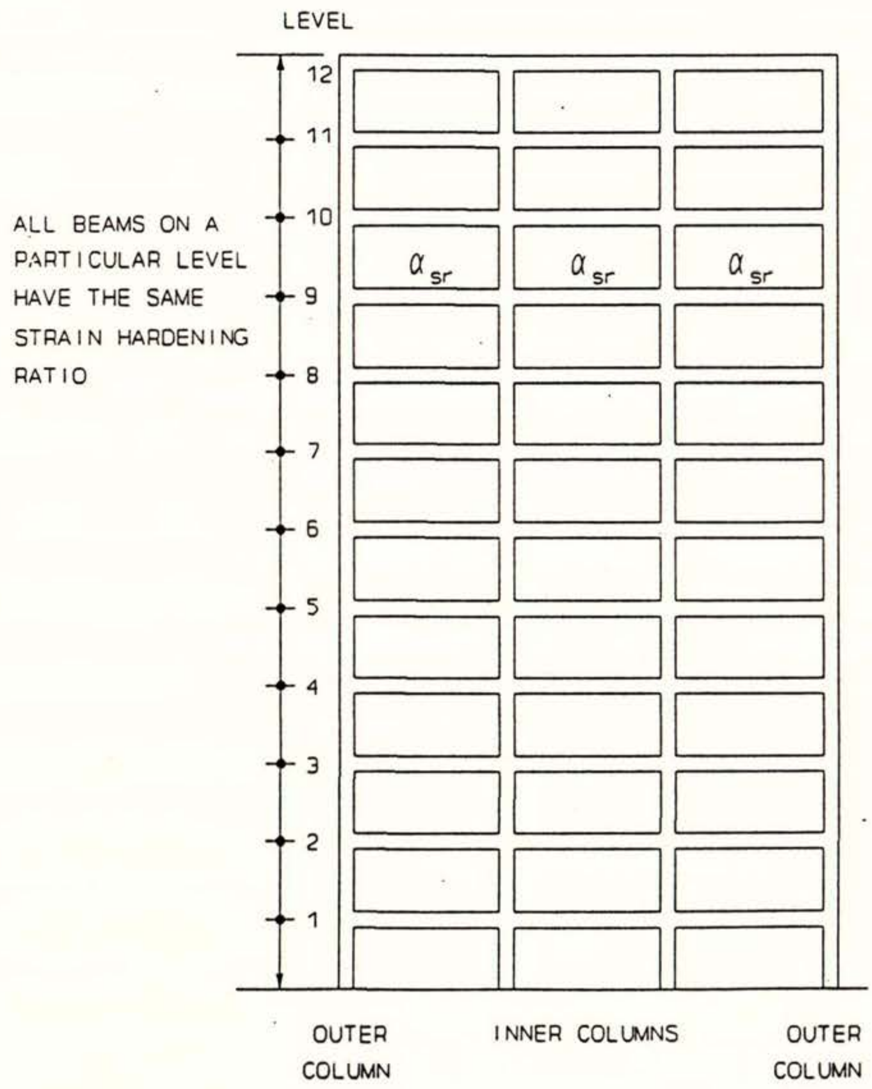

Fig. A.1 Typical Elevation of a Regular Frame Showing Column and Beam Locations as Used in Tables A.3 to A.6.

TABLE A.2

Member Sizes and Properties for Regular Frames

	6 STOREY	12 STOREY	18 STOREY	24 STOREY
BEAMS				
SIZES	600 x 400	600 x 400	700 x 400	800 x 400
I_e (m^4)	0.0046	0.0046	0.0072	0.011
A (m^2)	0.24	0.24	0.28	0.32
A′ (m^2)	0.2	0.2	0.117	0.133
COLUMNS				
COLUMN LEVELS				
0 - 6	450 x 450	550 x 550	700 x 700	800 x 800
I_e (m^4)	0.0034	0.0076	0.02	0.0341
A (m^2)	0.203	0.303	0.49	0.64
A′ (m^2)	0.169	0.252	0.408	0.533
6^+ - 12		550 x 550	600 x 600	700 x 700
I_e (m^4)		0.0076	0.0108	0.02
A (m^2)		0.303	0.36	0.49
A′ (m^2)		0.252	0.3	0.408
12^+ - 18			500 x 500	600 x 600
I_e (m^4)			0.0052	0.0108
A (m^2)			0.25	0.36
A′ (m^2)			0.208	0.3
18^+ - 24				500 x 500
I_e (m^4)				0.0052
A (m^2)				0.25
A′ (m^2)				0.208

TABLE A.3

6 Storey Frame; Summary of Strain Hardening Ratio Values, α_{sr}

	6 STOREY FRAME - Strain Hardening Ratio Values (α_{sr})						
DOF →	1	2	3	6	12	18	24
BEAMS							
Level							
6	0.0002	0.0011	0.0064	0.0049			
5	0.0016	0.0073	0.0111	0.0103			
4	0.0116	0.0139	0.0142	0.0149			
3	0.0215	0.0184	0.0175	0.0186			
2	0.0226	0.0225	0.0194	0.0213			
1	0.0210	0.0212	0.0198	0.0215			
COLUMNS							
INNER	0.0884	0.0865	0.0815	0.0909			
OUTER	0.0736	0.0748	0.0702	0.0787			

TABLE A.4

12 Storey Frame; Summary of Strain Hardening Ratio Values, α_{sr}

	12 STOREY FRAME - Strain Hardening Ratio Values (α_{sr})						
DOF →	1	2	3	6	12	18	24
BEAMS							
Level							
12	0.0005	0.0007	0.0026	0.0066	0.0054		
11	0.0005	0.0024	0.0093	0.0102	0.0096		
10	0.0005	0.0103	0.0163	0.0128	0.0136		
9	0.0009	0.0183	0.0179	0.0157	0.0169		
8	0.0038	0.0201	0.0185	0.0176	0.0197		
7	0.0169	0.0206	0.0208	0.0197	0.0220		
6	0.0300	0.0212	0.0246	0.0213	0.0240		
5	0.0331	0.0246	0.0256	0.0231	0.0260		
4	0.0337	0.0290	0.0260	0.0244	0.0278		
3	0.0336	0.0299	0.0276	0.0259	0.0294		
2	0.0325	0.0293	0.0297	0.0262	0.0300		
1	0.0277	0.0249	0.0259	0.0232	0.0265		
COLUMNS							
INNER	0.0550	0.0497	0.0517	0.0467	0.0537		
OUTER	0.0489	0.0441	0.0461	0.0415	0.0479		

TABLE A.5

18 Storey Frame; Summary of Strain Hardening Ratio Values, α_{sr}

	18 STOREY FRAME - Strain Hardening Ratio Values (α_{sr})						
DOF →	1	2	3	6	12	18	24
BEAMS							
Level							
18	0.0006	0.0009	0.0010	0.0068	NOT ANALYSED	0.0004	
17	0.0006	0.0010	0.0012	0.0114		0.0079	
16	0.0006	0.0010	0.0082	0.0121		0.0114	
15	0.0006	0.0015	0.0153	0.0143		0.0142	
14	0.0006	0.0110	0.0163	0.0172		0.0165	
13	0.0006	0.0203	0.0163	0.0176		0.0183	
12	0.0030	0.0212	0.0164	0.0186		0.0196	
11	0.0126	0.0213	0.0187	0.0203		0.0208	
10	0.0225	0.0213	0.0219	0.0208		0.0220	
9	0.0246	0.0213	0.0228	0.0218		0.0232	
8	0.0250	0.0216	0.0229	0.0234		0.0244	
7	0.0249	0.0243	0.0231	0.0237		0.0254	
6	0.0246	0.0286	0.0248	0.0245		0.0263	
5	0.0244	0.0294	0.0269	0.0257		0.0272	
4	0.0242	0.0294	0.0274	0.0261		0.0281	
3	0.0239	0.0272	0.0272	0.0265		0.0286	
2	0.0226	0.0275	0.0258	0.0261		0.0280	
1	0.0183	0.0222	0.0209	0.0214		0.0231	
COLUMNS							
INNER	0.0360	0.0439	0.0411	0.0423		0.0459	
OUTER	0.0330	0.0401	0.0377	0.0388		0.0420	

TABLE A.6

24 Storey Frame; Summary of Strain Hardening Ratio Values, α_{sr}

	24 STOREY FRAME - Strain Hardening Ratio Values (α_{sr})						
DOF →	1	2	3	6	12	18	24
BEAMS							
Level							
24	0.0006	0.0010	0.0010	0.0057	NOT ANALYSED	NOT ANALYSED	0.0028
23	0.0007	0.0012	0.0015	0.0100			0.0064
22	0.0007	0.0012	0.0015	0.0104			0.0095
21	0.0007	0.0011	0.0063	0.0105			0.0119
20	0.0008	0.0087	0.0139	0.0122			0.0139
19	0.0008	0.0164	0.0169	0.0146			0.0155
18	0.0008	0.0169	0.0169	0.0146			0.0166
17	0.0017	0.0169	0.0169	0.0146			0.0176
16	0.0102	0.0169	0.0169	0.0155			0.0187
15	0.0187	0.0169	0.0169	0.0170			0.0197
14	0.0201	0.0169	0.0183	0.0173			0.0206
13	0.0202	0.0168	0.0211	0.0173			0.0213
12	0.0199	0.0165	0.0224	0.0179			0.0218
11	0.0198	0.0167	0.0227	0.0190			0.0225
10	0.0197	0.0186	0.0226	0.0193			0.0233
9	0.0198	0.0221	0.0227	0.0195			0.0241
8	0.0197	0.0230	0.0228	0.0203			0.0249
7	0.0195	0.0230	0.0243	0.0213			0.0256
6	0.0193	0.0228	0.0266	0.0214			0.0261
5	0.0192	0.0226	0.0271	0.0216			0.0267
4	0.0190	0.0225	0.0271	0.0220			0.0272
3	0.0186	0.0220	0.0266	0.0225			0.0274
2	0.0176	0.0208	0.0251	0.0215			0.0264
1	0.0142	0.0167	0.0202	0.0174			0.0216
COLUMNS							
INNER	0.0257	0.0304	0.0367	0.0315			0.0392
OUTER	0.0237	0.0279	0.0337	0.0290			0.0361

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The New Zealand National Society for Earthquake Engineering

Administrative Secretary : MICHAEL BRICE

30 January 1992

The General Manager
Earthquake & War Damage Commission
P O Box 31-342
LOWER HUTT

Attention: Demetra Kennedy

Dear Demetra

Enclosed please find a copy of the University of Auckland School of Engineering Report No. 495 **'The Seismic Response of Multi-Storey Buildings'**, the result of a Commission funded Society Study Group on "The choice of Response Spectra for the Seismic Design of Multi-Storey Structures".

A copy of the report has also been sent to the editor of the Society's quarterly 'Bulletin' for publicity and review purposes.

The Society is again indebted to the Commission for its financial support of the work of this Study Group.

Sincerely

Michael Brice

Michael Brice

ington New Zealand Telephone & Fax 0-4-476 6866