A STUDY OF THE INELASTIC SEISMIC RESPONSE OF REINFORCED CONCRETE COUPLED FRAME–SHEAR WALL STRUCTURES

W.J. Goodsir*, T. Paulay** and A.J. Carr**

SYNOPSIS:

Three 12 storey frame-shear wall buildings with varying wall size were designed according to capacity design principles. Their response to the El Centro NS 1940 and Pacoima Dam S15^{o}W 1970 accelerograms was investigated using a 2-dimensional nonlinear dynamic analysis program. Particularly significant observations were the high levels of wall shear forces and generally low levels of column bending moment encountered. Widespread beam flexural yielding involving moderate levels of inelastic deformations and interstorey drifts controlled by the walls indicate the potentially desirable inelastic performance of these hybrid structures. Design schemes are proposed whereby improved estimation of maximum inelastic actions can be made using the traditional elastic analysis for equivalent lateral static loading on the structure.

1. INTRODUCTION

Coupled frame-shear wall structural systems combine the advantages of their constituent elements. Gravity load is most efficiently taken by frames, while shear walls provide excellent lateral load resistance, stability and the deflection control necessary in a seismically active region such as New Zealand. Despite the attractiveness and indeed the existence of many such structures in this country, comparatively little research effort has been directed to them, and for design purposes, the New Zealand Concrete Design Code (1) draws the designer's attention to the need for "special studies" of these "ductile hybrid structures". Existing research deals almost exclusively with elastic load distributions despite the obvious importance of response in the inelastic regime.

This study was initiated with the intention of ultimately formulating a series of semi-empirical rules, whereby the results of the requisite static elastic analyses, that result in "code" actions, could be modified and subsequently designed for, resulting in a structure that should exhibit good inelastic behaviour in the event of a major earthquake. Such a scheme should ideally permit a smooth transition between existing design rules for space frames (1) and those for shear wall structures (2), as a function of the relative proportions of frame and shear wall stiffness.

2. GENERAL DESCRIPTION OF THE BUILDING AND MODELLING ASSUMPTIONS

2.1 General Description of the Buildings

A series of 3 hypothetical 12 storey, reinforced concrete structures based on those investigated by Carter (3) was selected for study (Figure 1). Although these structures are unusually regular, it is thought that sufficient realism has been retained to make the investigation of practical interest. The structural height of 12 storeys was chosen in the hope that the effects of high mode participation in response, often negligible for low buildings, could be examined. Each structure consisted of 7 two-bay frames, 8 one-bay frames and a pair of vertically tapering cantilever shear walls, the size of which (ℓw = 4, 6, 8 m) provided variation in frame wall stiffness ratio. The frame components of the buildings, in part adapted from structures studied by Jury (4), was held constant. Overall centreline plan dimensions are 18.4 x 73.6 m, being 2 bays by 8 bays at a bay length of 9.2 m. The structures had a constant floor height of 3.65 m for a total height of 43.8 m, with 160 mm thick floor slabs throughout. Details of member sizes are given in Figure 1.

2.2 Modelling Assumptions

(a) For the purposes of computer assisted analyses, the 3 dimensional structure was modelled in 2 dimensions by

(i) the additive lumping of stiffness parameters of similar lateral load resisting elements (7 type 1 frames, 2 walls etc), and

(ii) the connection in series of the 3 types of load resisting element via the slaving of horizontal degrees of freedom of adjacent nodes.

Such modelling (Figure 2) allows the transfer of horizontal forces only, with total vertical load on each lumped element remaining constant and implicitly assumes the floor slabs to be infinitely rigid diaphragms in their own planes.

(b) Full fixity at foundation level of both wall and column members was assumed.

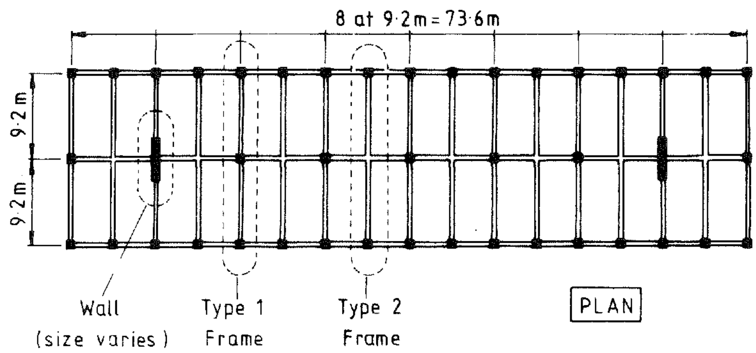
(c) Unidirectional earthquake excitation only, perpendicular to the long axis of the building, was considered with no allowance made for a skew earthquake or torsional effects.

(d) The torsional resistance of beams

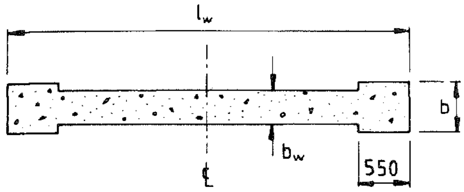
*Graduate Student, Department of Civil Engineering, University of Canterbury, Christchurch.

**Department of Civil Engineering, University of Canterbury, Christchurch.

FIGURES



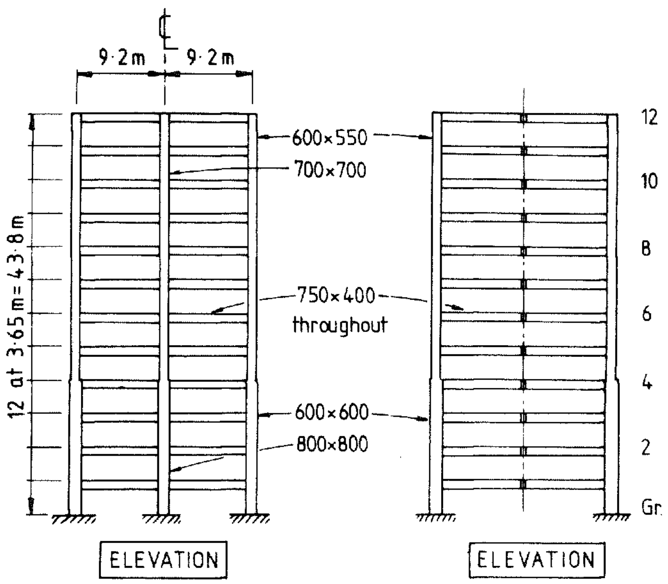
(a) General plan of structure



Wall Type	4m	6m	8m
l_w	4·00	6·00	8·00
b	0·45	0·40	0·35
b_w			
Level 1-3	0·45	0·40	0·35
4-6	0·40	0·35	0·30
7-9	0·35	0·30	0·25
10-12	0·30	0·25	0·20

Note: All dimensions are in metres.

(b) Wall dimension variation



(c) Type 1 frame

(d) Type 2 frame

Fig. 1 - Dimension Details of Building Studied

framing into the plane of type 1 or 2 frames, and the load resistance of the exterior columns in the plane of the shear walls were both ignored in the analyses.

(e) Assumed moduli of elasticity for steel and concrete were 200 and 25 GPa respectively. Allowance was made for both steel and flexural deformations in structural members and these deformations were assumed not to occur in beam column joints, these latter items being modelled by providing members with rigid end blocks.

(f) Non structural elements, present in a real building, were assumed not to modify the structural responses.

3. DESIGN OF STRUCTURES

3.1 General

The paramount consideration of any proposed design scheme is that it must provide a building so designed with the potential to exhibit good seismic performance. However, to be practical such a process must be relatively straightforward, rapid and computationally as simple as possible; considerations which as yet preclude the routine use of inelastic analyses. However, most organisations engaged in the design of multistorey structures do have routine access to static elastic analysis packages.

In view of the factors stated above, the research program reported herein was constructed as follows:

(a) A series of structures were designed using the results of a static lateral load analysis, modified in a suitable manner.
(b) The structures were subjected to simulated seismic attack (via computer based modelling) and extreme member actions were recorded, and
(c) Factors and procedures relating maximum observed dynamic actions to code (static load) actions were derived, enabling an improved and generalised design method to be proposed.

In this manner a scheme may be developed whereby the designer may derive design actions that are critical because they are not directly associated with ductile energy dissipating mechanisms, simply by the appropriate factoring of the code (static load) distribution of the corresponding actions. Typical actions are shear forces in walls and columns where flexural yielding (plastic hinges) are being developed in these components.

The proposed design philosophy for frame-shear wall structures is by the application of the previously postulated capacity design approach (5). The major feature of this approach is the 'a priori' selection and appropriate detailing of primary energy dissipating elements (plastic hinges) and the provision of other structural elements with sufficient reserve strength to ensure that significant inelastic deformations occur only at sections detailed for this purpose.

Thus a desirable hierarchy of hinge formation was decided upon, which, for the multistorey buildings under consideration involves beam hinging with a high degree of protection afforded the columns. Base level yielding in columns and walls is expected, as this is necessary for the full development of a collapse mechanism. Detailed design of only flexural reinforcement was carried out with brief checks made as to the practicability of supplying other reinforcement, i.e. for shear resistance, confining of the compressed concrete etc. The floor slab was not subject to detailed design. Assumed material properties were: concrete compression strength f'_c = 25 MPa and steel yield strength f_y = 275 MPa for beams and walls, and 380 MPa for columns.

Many aspects of frame-shear wall behaviour can be deduced from a consideration of the way in which the deflected shapes of isolated frames and shear walls subject to the same lateral load, are modified by coupling. Figures 3(a) and (b) show the "shear" mode of deformation typically adopted by a laterally loaded frame structure, and the essentially "flexural" deformation of a shear wall under the same loading. The coupling of these 2 structural forms leads to an overall deflection profile as shown in Figure 3(c), indicating that the wall, although controlling deformation at low levels, is restrained by the frame in the upper levels.

In accordance with the requirements of the Code of Practice for General Structural Design and Design Loadings for Buildings (6), an elastic analysis for the prescribed lateral static load may be used for design by the "strength" method described in NZS 3101 (1). Such analyses were made for each of the 3 structures under consideration using the elastic static analysis option of the computer program RUAUMOKO (8) (see also Section 4.1).

3.2 Loading

Dead weight calculations were based on an assumed equivalent weight of 23 kN/m^3 for concrete structural members, and a uniformly distributed dead load of 0.5 kPa representing finishes, service ducting, partitions etc. A live load of 2.5 kPa was chosen, and reduced for design purposes according to tributary areas. Static earthquake load was calculated in accordance with the provisions of Part 3: NZS 4203 (6) for loading in the short direction of the building only. Seismic live load for the purpose of estimating equivalent floor masses, was taken as L/3 (L being the unreduced live load as above) and was added to the dead load D. The seismic coefficient C_d = CISMR was calculated as follows:

C = 0.075, for a building of fundamental period assumed greater than 1.2 sec, in Seismic Zone A and founded on rigid subsoils. Subsequent checks

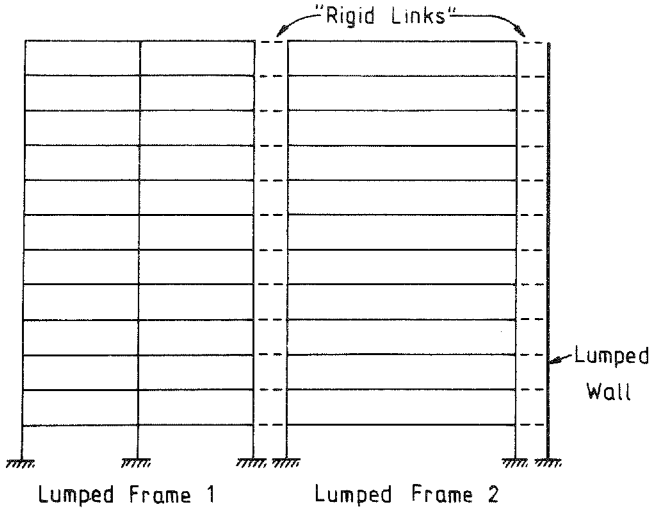


Fig. 2 - Lumped Model for Computer Analysis

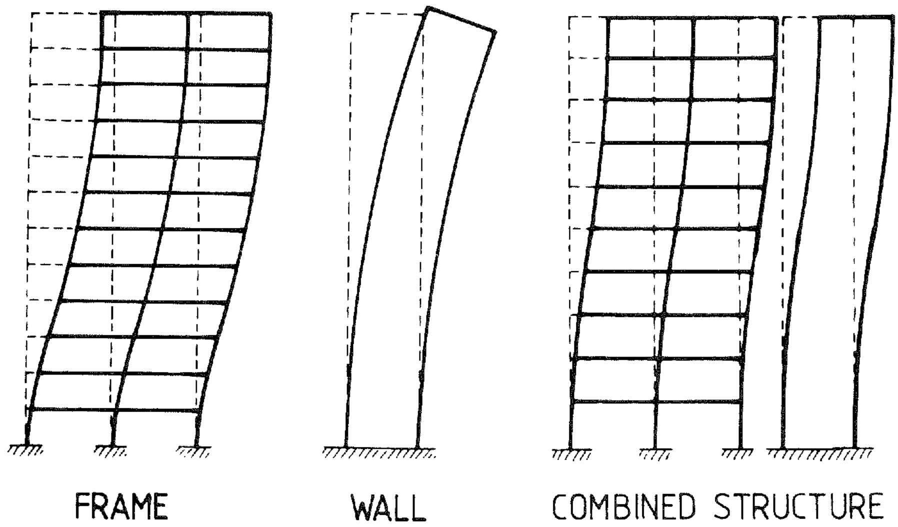
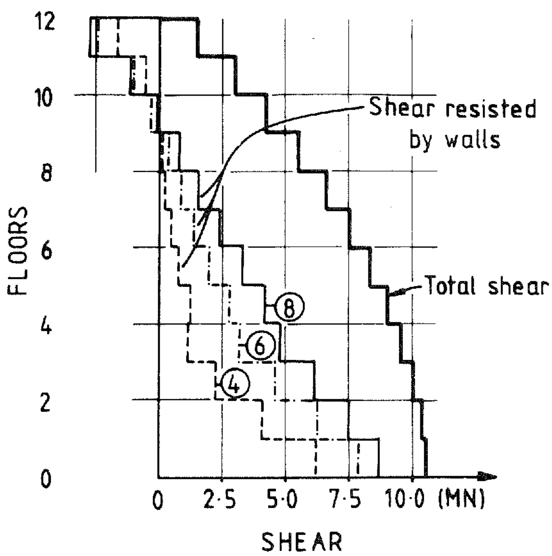


Fig. 3 - Typical Deflected Shapes of Frame, Wall and Coupled Frame-Wall Structure



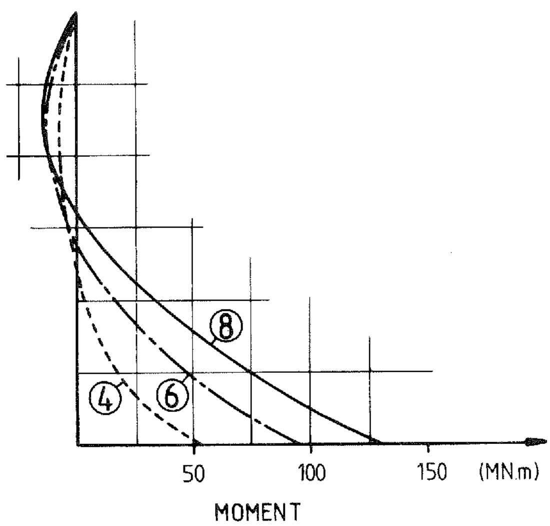


Fig. 4 - Shear Force Distribution Under Code Lateral Load

Fig. 5 - Bending Moment Distribution Along Walls Under Code Lateral Load

confirmed the validity of the period assumption for all structures (see Table 1).

The terms I and R were both assigned a value of unity, as was the product S x M, resulting in a net C_d value of 0.075.

3.3 Elastic Analysis - Code Actions

Figure 4 shows the internal distribution of storey shear forces between walls and frames. The well known phenomenon, whereby the wall shear acts in the same direction as the external load in the top few storeys, is evident. The "code" wall shear force in the ratio $(V_{wall,base}/V_{total,base})_{code}$, defined as the Shear ratio is used as an index of relative wall stiffness, and has values 0.58, 0.75 and 0.83 for the 4, 6 and 8 m walled structures respectively. Wall bending moment distribution with height exhibits trends predicted in Section 3.1 and Figure 5 illustrates the effect of changing wall size on wall moment patterns.

Figure 6 shows the variation of beam shear forces with height. This is informative when viewed in the context of laminar shear distributions commonly met in the analysis of coupled shear walls (2).

3.4 Beam Design

The design of beams was carried out for load cases prescribed in NZS 4203 (6), using conventional frame design methods, with the load combination of D + 1.3 L_R + E generally being the critical case. Use was made of redistribution of beam moments as permitted in NZS 3101 (1), which often resulted in equalization of positive and negative moment demands at a given floor. It was decided, in keeping with the recommended practice for coupled shear walls (2), to allow also up to 20% vertical redistribution of beam moments keeping the aggregated moment demand for the 12 floors unaltered. This allows considerable repetition of beam reinforcement layouts in a given structure. This is illustrated in Figure 7. Reinforcement ratio demands (ρ) varied from 0.65% to 1.80% for the uniformly sized 750 x 400 beams.

3.5 Column Design

Columns were designed according to the spirit, if not the letter, of the Appendix to the commentary of Section 3 of NZS 3101 (1), with column design moments calculated according to the formula $M_{col,red} = R_m (\phi_o \omega M_{code} - 0.3\ h_b V_{col})$. (The terms are defined in the Notation). In this study the most important variable was deemed to be the column dynamic magnification factor, ω, which recognises the higher mode effects capable of dramatically altering (5) first mode ("code") member actions in a frame. In view of the control on deflections (and hence high mode participation) expected to be provided by the walls, values of ω lower (ω = 1.45) than those suggested for a pure space frame (1) (ω = 1.8) were used, as illustrated in Figure 8. Design axial loads $P_{e,max}$ and $P_{e,min}$ were obtained in the manner suggested in Ref. 1, using the standard tributary area approach for gravity load and the modified summation of beam shear forces at beam flexural overstrength due to earthquake effects. The necessary column reinforcement demand was found using interaction charts (7). Supplied ρ_t values ranged from code minimum 0.8% to a maximum of 2.20%.

3.6 Wall Design

The cantilever shear walls were designed according to the methods suggested in Ref. (2), and using conventional strain compatibility methods with an assumed extreme fibre compression strain of 0.003. Wall plastic hinge length ℓ_p was taken equal to ℓ_w. The critical design axial load was due to 0.9D. The distributed vertical web reinforcement was supplied in a minimum area ratio of 0.3%. Flexural bars in the walls were curtailed with height as indicated in Figure 9. Required reinforcement contents are shown in Table 2. Full base fixity of walls was assumed in the analyses, and no detailed check was made as to the feasibility of such restraint being provided by the foundations.

4. DYNAMIC ANALYSES

4.1 General

The 2 dimensional inelastic time history program RUAUMOKO (8) developed in original form by Sharpe (9) and substantially modified by Carr (8) was used to investigate the response of the 3 frame-wall structures to simulated seismic attack. Input data (11), not presented in detail here, consisted essentially of:

(a) The geometry of the structure, as described in terms of 78 nodes.

(b) Member properties, consisting of both stiffness and flexural strength data for each of the 108 members. Stiffnesses were calculated in accordance with the assumed state of cracking detailed in Table 3. For beams and walls, yield data was simply a yield moment, based on the probable member strength. Column members had yield properties in the form of a slightly simplified interaction diagram. A strain hardening factor of 3% was selected for the walls, but elastic-perfectly plastic hysteresis for beams and columns was assumed.

(c) Lumped nodal weights (estimated at D + L/3) and rotational inertias (10), used for the calculation of inertia forces.

(d) One one-hundredth of a second was used throughout the analyses ensuring stability of the numerical process. A Rayleigh damping model was chosen and after some initial problems (11), a scheme assigning 5% of critical damping to modes 1 and 10 was adopted.

(e) The two earthquake records selected

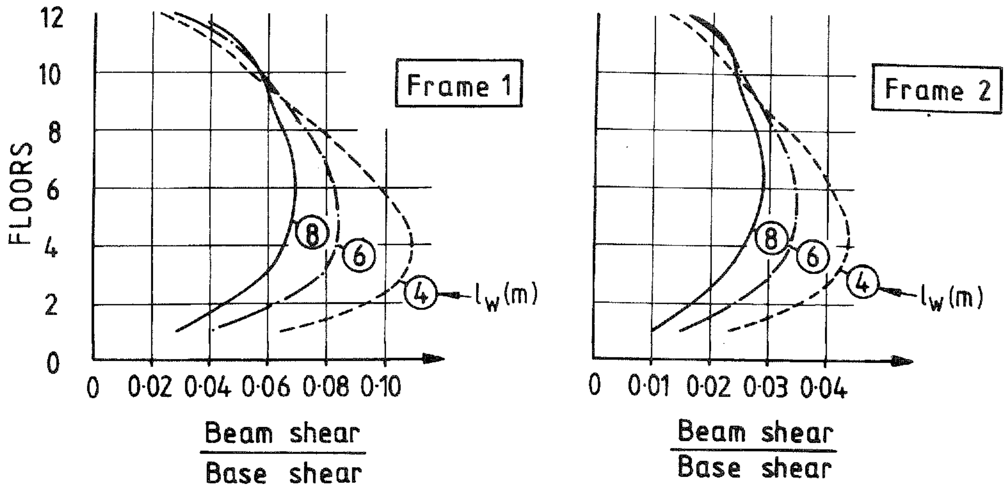


Fig. 6 - Beam Shear Force Distributions Under Code Lateral Load

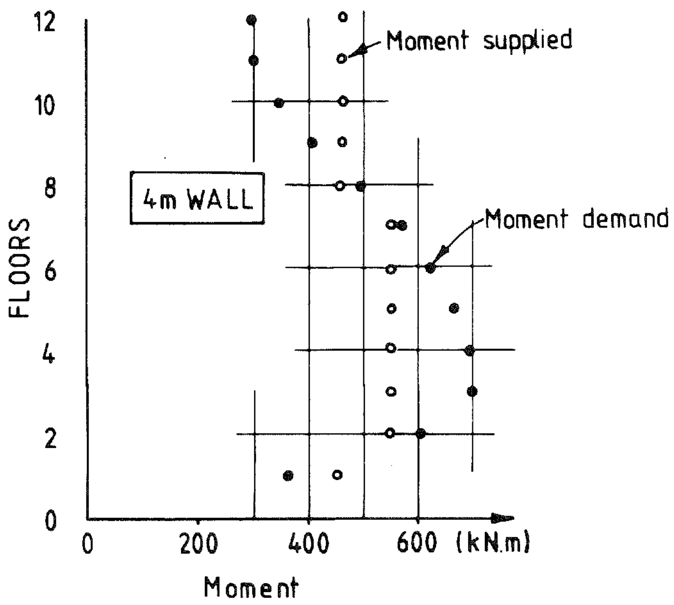


Fig. 7 - Horizontally Redistributed Beam Moments (Type 1 Frame) and Supplied Capacities Showing Vertical Moment Redistribution

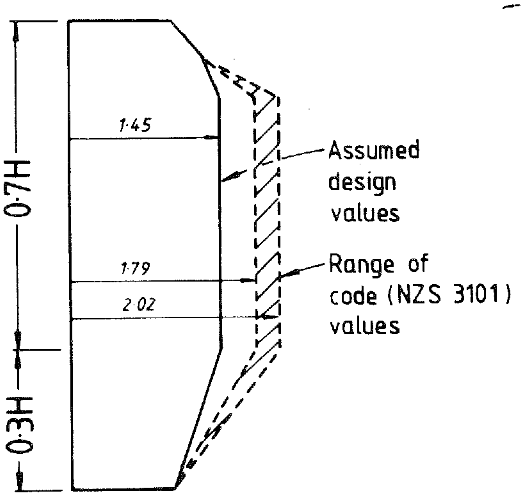


Fig. 8 - Assumed Distribution of Column Dynamic Magnification Factor (ω)

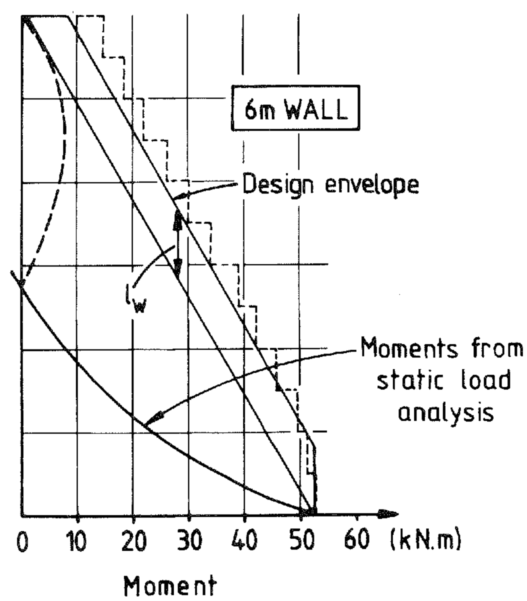


Fig. 9 - Wall Bending Moment Demand and Supplied Capacity for the 6 m Walled Structure

were the El Centro N-S 1940 and Pacoima Dam S15^{o}W 1970 events. The former, a complex multiple event quake, has become inseparably linked with studies of structural response and, as a result, is a banchmark for comparison purposes despite its shortcomings. The Pacoima Dam record is one of the most severe accelerograms in existence and, as such, constitutes an upper bound to probable ground motion in New Zealand. Despite arguments as to the likelihood of such an event, it is contended that a structure that can survive the Pacoima record without catastrophic collapse demonstrates a wholly acceptable design, while inability to survive does not rule out acceptability of the design. For reasons of economy in computation, the first 10 seconds only of each record was used.

4.2 Response of Frame - Shear Wall Structure to Horizontal Excitations

Although a great deal of information was generated in each analysis, space limitations permit the reproduction herein of only a small amount of typical material, specifically, information relevant to the 4 m walled structure. (A more detailed exposition of the results of the dynamic analyses may be found in Ref. 11). The information shown regarding structural deformations and member actions are, however, generally typical of the responses of the 6 and 8 m walled structures.

4.2.1 Displacement Response

Figure 10 shows time histories of horizontal deflection of the 3rd, 6th, 9th and 12th floors of the structure to the 2 excitations. Part (a) indicates the sinusoidally oscillating response under El Centro, with a period of about 1.9 sec. and the development of a locked in plastic displacement. The main feature of the response to Pacoima Dam is the extremely large deflection pulse at 3.2 seconds and subsequent relatively stable oscillation of the building. Envelopes of maximum horizontal deflection and interstorey drift are presented in Figures 11 and 12. The latter shows maximum drift indices of 0.0085 (El Centro) and 0.0198 (Pacoima Dam). When these are compared with the values of 0.0110 and 0.0233, obtained by Jury (4) for similar 12 storey pure frames, the deflection control afforded by this relatively flexible shear wall becomes evident.

4.2.2 Wall Shear

For the purpose of comparison Figure 13 shows 4 levels of shear strength at each floor of the 4 m walled structure, namely

(a) $(1.13/\phi_v)\ V_{code}$, $\phi_v = 0.85$ - a probable level of wall shear strength that would be obtained from the application of "strength design".

(b) $\omega\phi_o\ V_{code}$, $\phi_o = \omega = 1.45$ - wall shear strength based on the recommended capacity design procedure for the cantilever shear walls (2).

(c)(d) Maxima in shear strength demand encountered during the first 10.0 sec. of the El Centro and Pacoima Dam records respectively.

The inability of both (a) and (b) to predict the necessary wall shears strength in the upper storeys is evident. Although El Centro base shear demands are adequately met in this structure, those for Pacoima Dam are not. An unexpected observation in all analyses was that, even at ground floor level, wall and frame shears acted often in opposite senses, so that walls resisted a shear greater than the total applied (external) force. Observed shears under El Centro were on average about 40% less than those for Pacoima Dam, although on occasions, the El Centro level of shear did exceed that of the Pacoima Dam record. (e.g. positive shear, levels 3 and 4).

4.2.3 Wall Moment

Envelopes of maximum (positive or negative) bending moment encountered during the 10 seconds in wall, together with probable yield moments, are presented in Figure 14. While the Pacoima Dam moment envelope is nearly linear and closely matches the design envelope (1), that for El Centro is more reminiscent of the moment diagram resulting from the code static load. The observed yielding in the upper regions of the wall was generally insignificant, as witnessed by the recorded plastic rotations (see Section 4.2.7).

4.2.4 Column Moment

Figure 15 shows both envelopes of maximum recorded column moment and corresponding probable yield strength in flexure, calculated using the axial load observed in the column at the same time as the maximum moment was recorded. Under the El Centro loading, columns clearly enjoy generally high levels of protection and although the extent of this protection is reduced during the Pacoima Dam excitation, it is still significant. This evident conservatism design was echoed in the analyses performed for the 6 and 8 m walled structures.

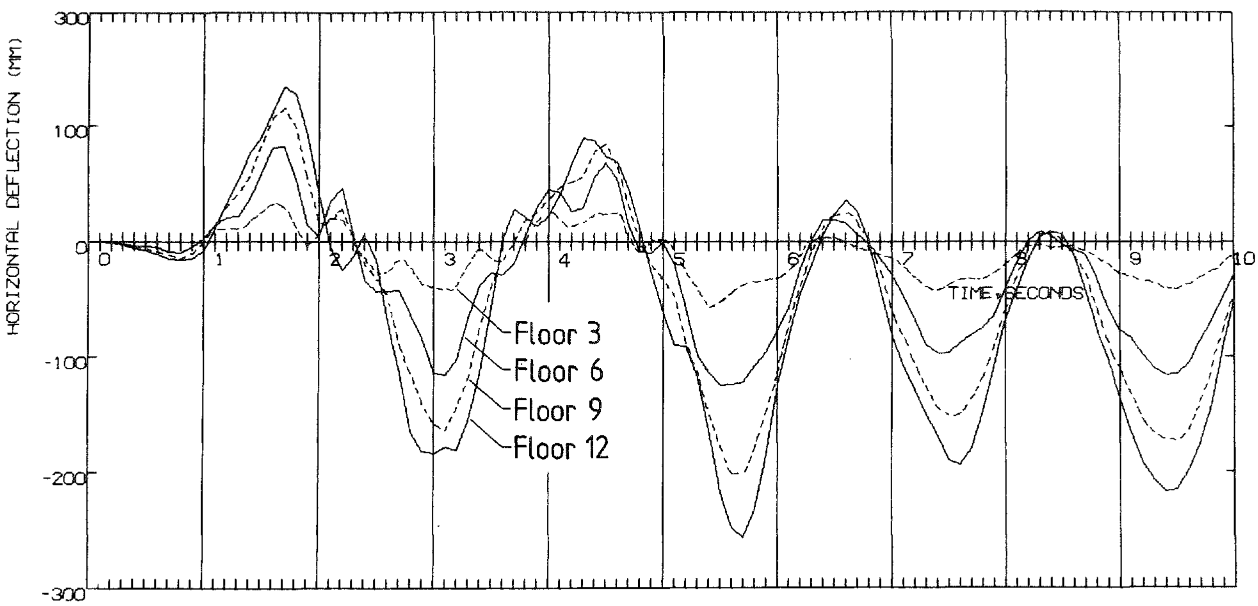
4.2.5 Column Shear

Column shear forces under both excitations were of similar orders, with no definite pattern of larger shear induced by the stronger excitation, as seen in Figure 16. Design level shears (calculated according to the rules of Appendix I NZS 3101 Part 2 (1) are also shown. These adequately estimate observed shears except at the extreme top and bottom of the structure.

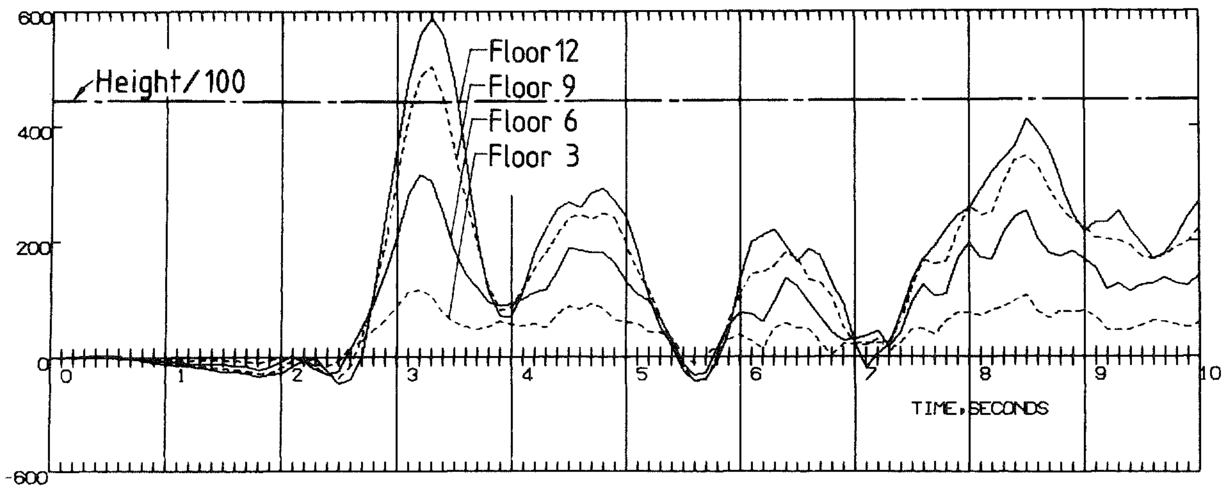
4.2.6 Column Axial Forces

As shown in Figure 17, observed extremes of column axial load compare well with the design level forces ($P_{e\ max} = D + L + P_{eq}$, $P_{e\ min} = 0.90 - P_{eq}$).

Actions observed during the Pacoima Dam event were, as would be expected, slightly more extreme. Axial loads for the interior column of type 1 frames are not shown, as these were subject to little variation in magnitude.



(a) 4M WALL EL CENTRO



(b) 4M WALL PACOIMA DAM

Fig. 10 - Horizontal Deflection History for the 4 m Walled Structure

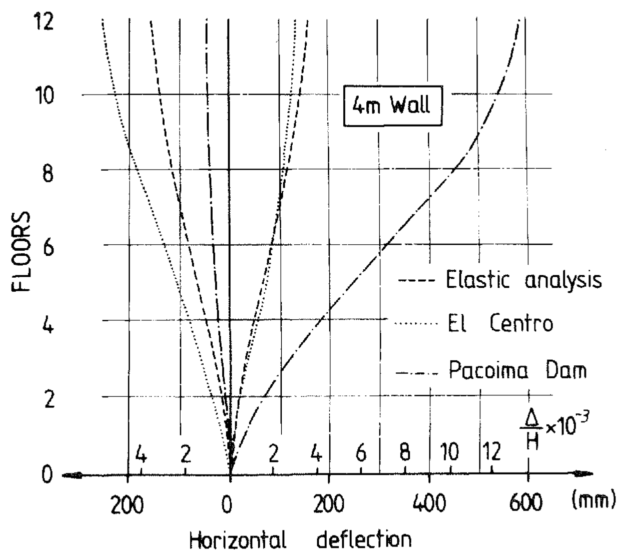


Fig. 11 - Extreme Horizontal Deflection Envelope for the 4 m Walled Structure

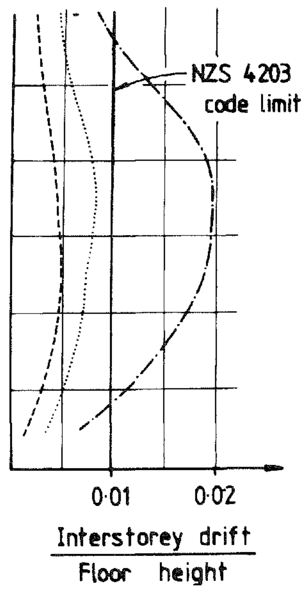


Fig. 12 - Extreme Interstorey Drift Envelope for the 4 m Walled Structure

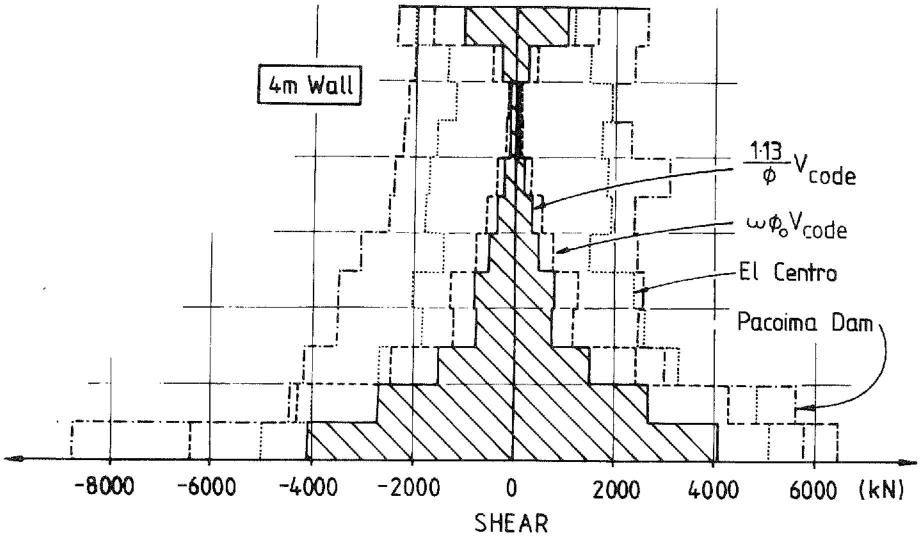


Fig. 13 - Extreme Wall Shear Force Envelope

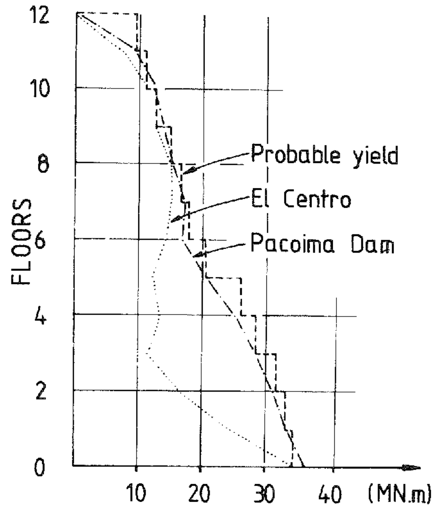
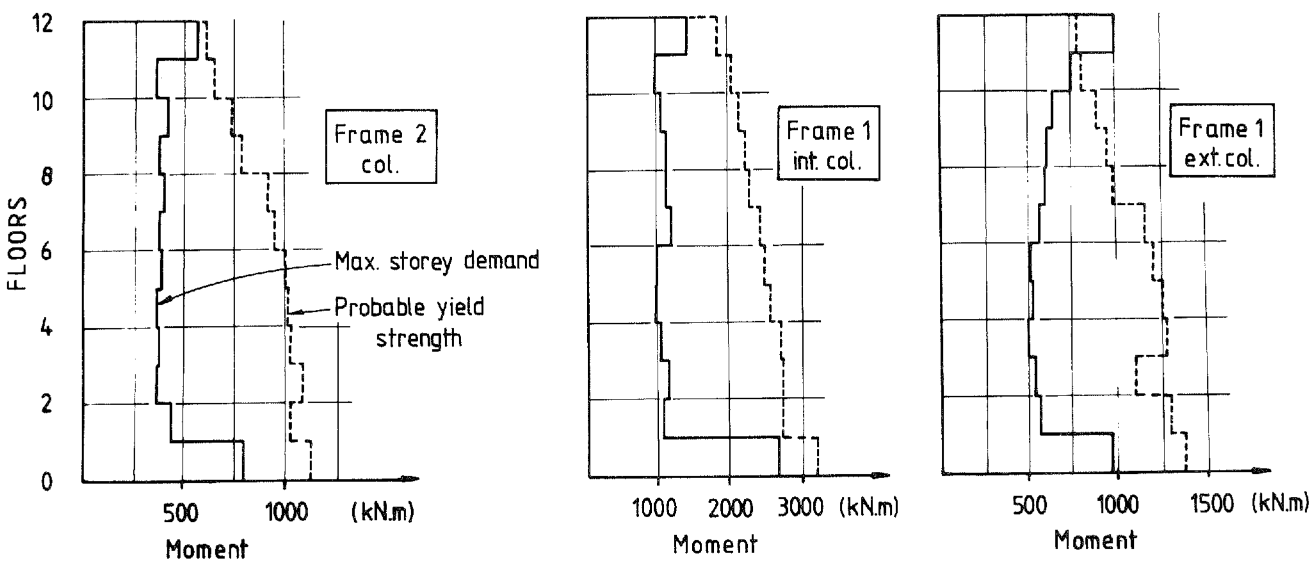
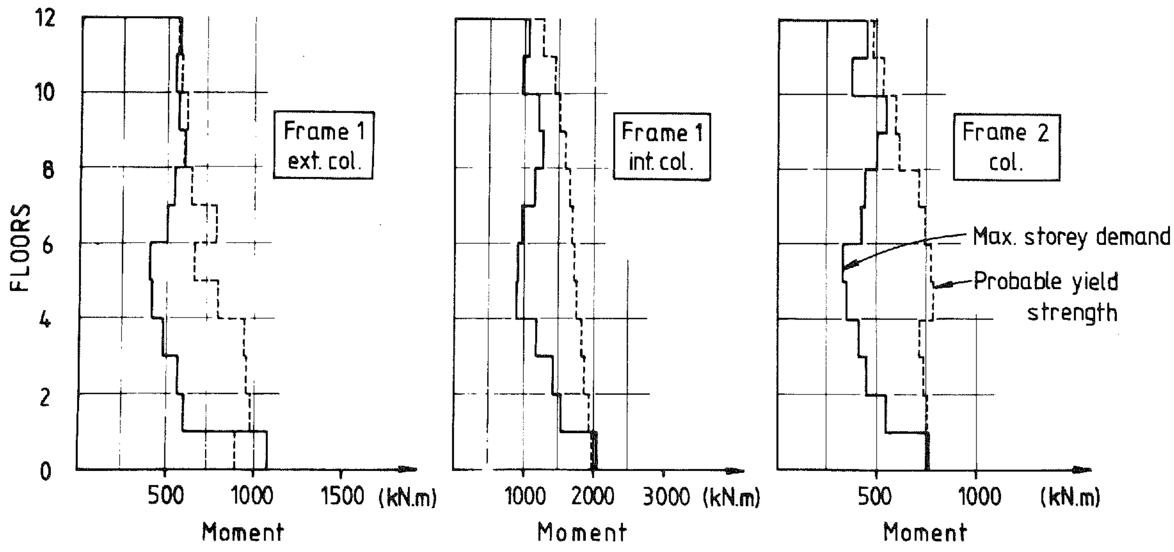


Fig. 14 - Extreme Wall Bending Moment Envelope



(a) El Centro excitation



(b) Pacoima Dam excitation

Fig. 15 - Envelopes of Maximum Recorded Column Bending Moment and Probable Simultaneous Strength Capacity

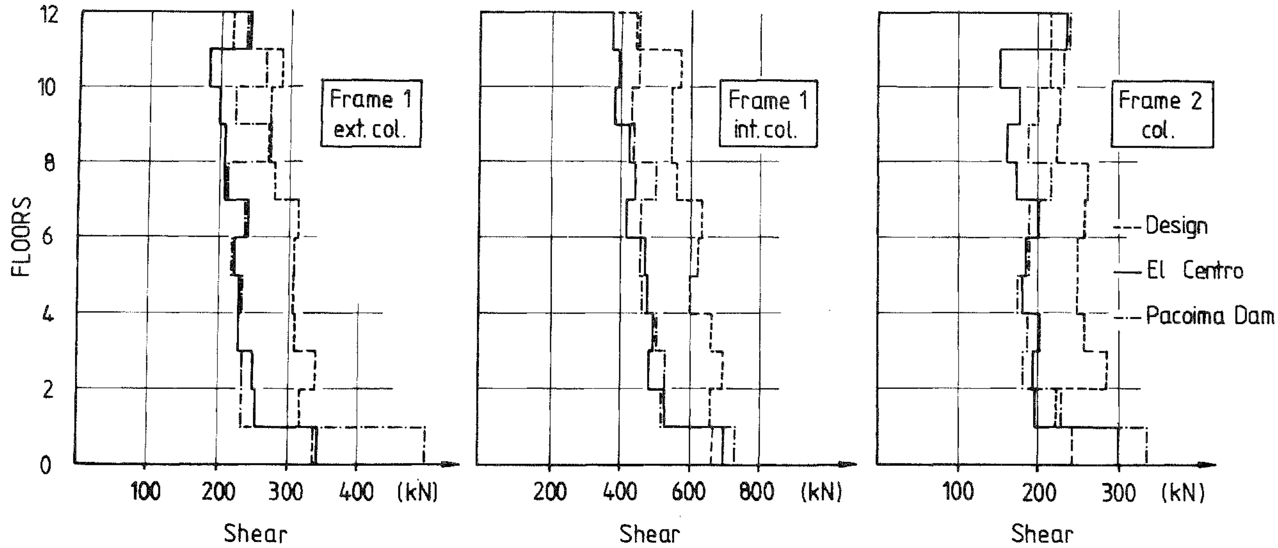


Fig. 16 - Envelopes of Maximum Recorded Column Shear Force and Design (Code) Shear Force, V_{col}

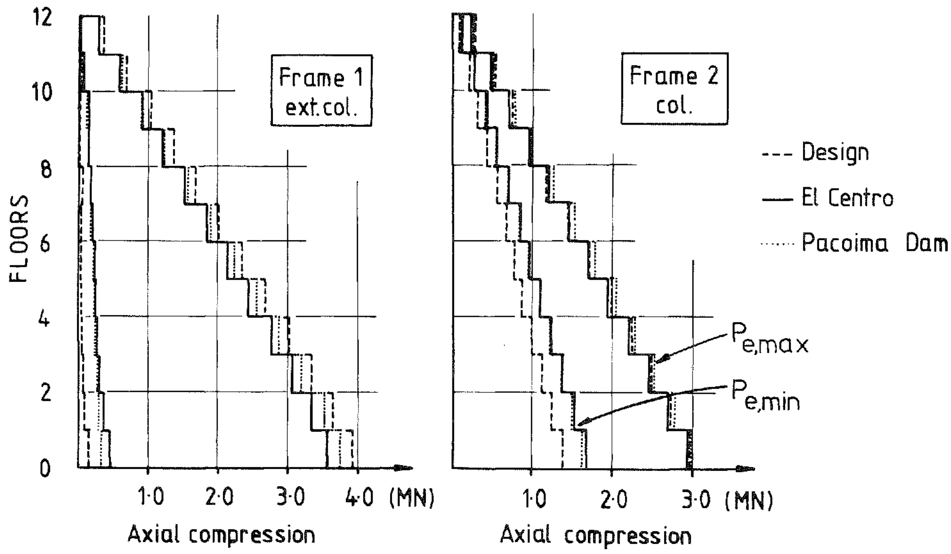


Fig. 17 - Extreme Column Axial Load Envelopes and Design (Code) Axial Forces, P_{max} and P_{min}

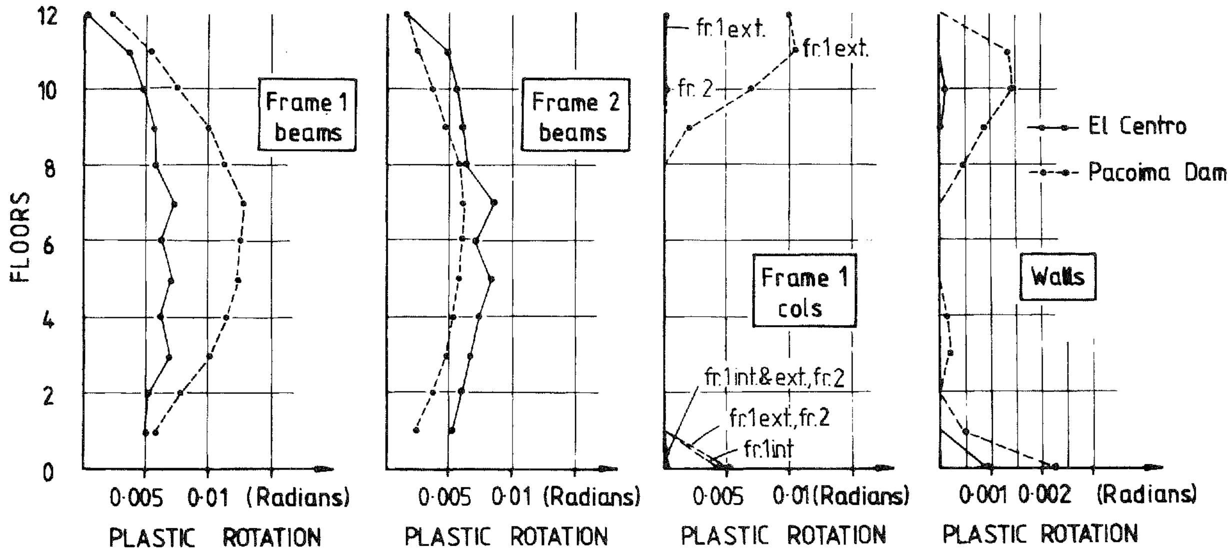


Fig. 18 - Envelopes of Maximum Recorded Plastic Hinge Rotations

4.2.7 Plastic Rotations

Figure 18 shows envelopes of maximum positive or negative hinge rotations observed in beams, columns and walls for the two earthquake records. Beam hinge rotations are considerably below the figure of 0.035 radians, a level of deformation considered to be available for beams detailed according to the provisions of NZS 3101 (1). The apparent discrepancy of El Centro rotations exceeding Pacoima Dam rotations in Figure 18(b) is related to a small difference in the modelling of beam hysteresis used for those two analyses and is not considered to be of any importance. Column hinge rotations during the El Centro excitation were clearly of an acceptably low order, while those observed during the Pacoima Dam event indicate greater but sustainable demands. Wall rotations indicate similar trends with significant yield under El Centro occurring at base level only.

5. ASSESSMENT AND MODIFICATION OF THE DESIGN METHOD

For the purposes of this assessment, emphasis has been placed on the responses to the El Centro accelerogram. It is believed that this excitation is much more credible in the context of New Zealand seismicity than the Pacoima Dam event.

5.1 Wall Shear

Figure 19 shows the relationship between shear ratio (defined in section 3.3) and the effective wall dynamic magnification factor, ω_v^*, defined as follows:

$$\omega_v^* = \frac{V_{wall,\ max}}{\phi_o\ V_{code,\ base}}$$

where $V_{wall,\ max}$ = maximum wall base shear force observed during the dynamic analysis, ϕ_o = wall overstrength factor, taken as 1.45, and $V_{code,\ base}$ = wall base shear force resulting from code specified loading. The "severity contours" indicated in Figure 19 are naturally different for the El Centro and Pacoima Dam events. Figure 20 shows an empirical shear force envelope which is based on trends in the distribution of maximum shear forces for all three structures. The application of this procedure to the three walls studied is seen in Figure 21.

In view of the importance of suppressing non ductile failure of the shear walls, the following simple approach to wall shear design is tentatively suggested: Design shear forces should be based on the envelope shown in Figure 20, with $V_{code,\ base}$ and ϕ_o factor found from the normal static analysis procedures. Effective wall dynamic magnification factor, ω_v^*, may be found from the "design contour" of Figure 19, the value selected being appropriate to the shear ratio of the structure under consideration. The equation of this design contour, ω_v^* = 1 + 0.6 x SHEAR RATIO, may be generalized to a form suitable for walls of any height via the equation $\omega_v^* = 1 + (\omega_v - 1)$ x SHEAR RATIO.

ω_v, the height dependent ω factor for a pure cantilever shear wall, is found in NZS 3101 (1) and is shown in Figure 22.

It may be observed that for a shear ratio of 100% (i.e. all seismic shear taken by the wall), ω_v^* = 1.60, the recommended factor for pure shear walls (1), and ω_v^* = 1.00 for zero values of shear ratio. Thus the scheme merges smoothly with the existing procedure for cantilever shear walls.

Figure 21 shows also the design zone levels calculated using this scheme, and it is evident that the El Centro levels of shear are safely overestimated. Although the scheme cannot cope with base levels of shear likely to occur in a Pacoima Dam type event, this must be viewed in light of the very low likelihood of such an event occurring. Calculations not presented here (11) indicate that quantities of shear reinforcement required near the base are not excessive.

5.2 Wall Moment

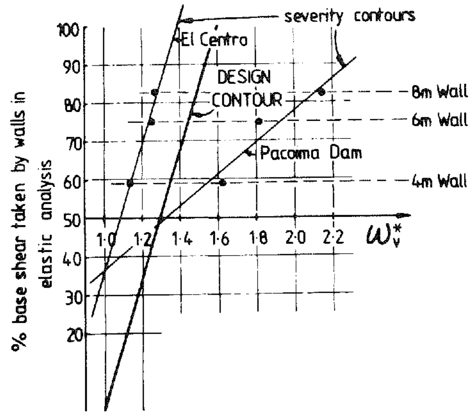
The proposed design scheme, based on the procedure recommended for cantilever shear walls (2) is considered satisfactory in view of the low inelastic demands of the El Centro excitation. However, in an attempt to eliminate all upper level wall yield, it is suggested that a slightly more conservative design envelope be used, as shown in Figure 23.

5.3 Column Moments

Although it has been decided that columns should enjoy a significant measure of protection against yielding, considerations of economy dictate that over-conservatism in selection of design actions should be avoided. Figure 24 shows the degree of column protection available to the frames which, it may be recalled, were designed using arbitrarily basic ω factor of 1.45 (Figure 8). As changing wall size apparently exerts no consistent trend on protection, a ω factor independent of wall shear ratio was deemed appropriate and a basic reduced value of 1.20 is suggested (see Figure 25). Such a factor allows a substantial reduction in column flexural reinforcement requirements and yet offers good protection against yielding for El Centro levels of load (11). Because of the low value of ω, further refinement, taking into account the value of the wall shear ratio, is not justified.

5.4 Column Shear

Column shear force is adequately estimated by the formula 1.3 $\phi_o\ V_{code}$ (1) at all levels by the extreme top and bottom of the structure. It is suggested that 2.0 $\phi_o\ V_{code}$ and 2.5 $\phi_o V_{code}$ respectively should be used for assessing V_{col} at these 2 locations. Again, quantities of transverse steel in the columns implied by these design forces are not great (11) and indeed are often subordinate to the demands of confinement.



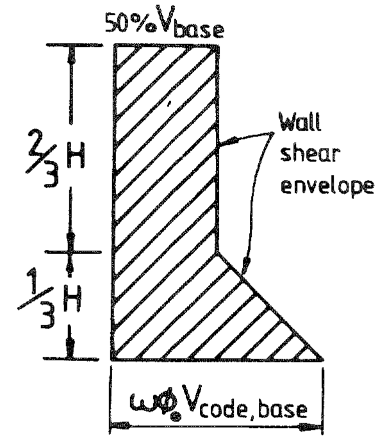


Fig. 19 - Variation of Effective Wall Dynamic Magnification Factor With Shear Ratio

Fig. 20 - Proposed Empirical Wall Shear Force Design Envelope

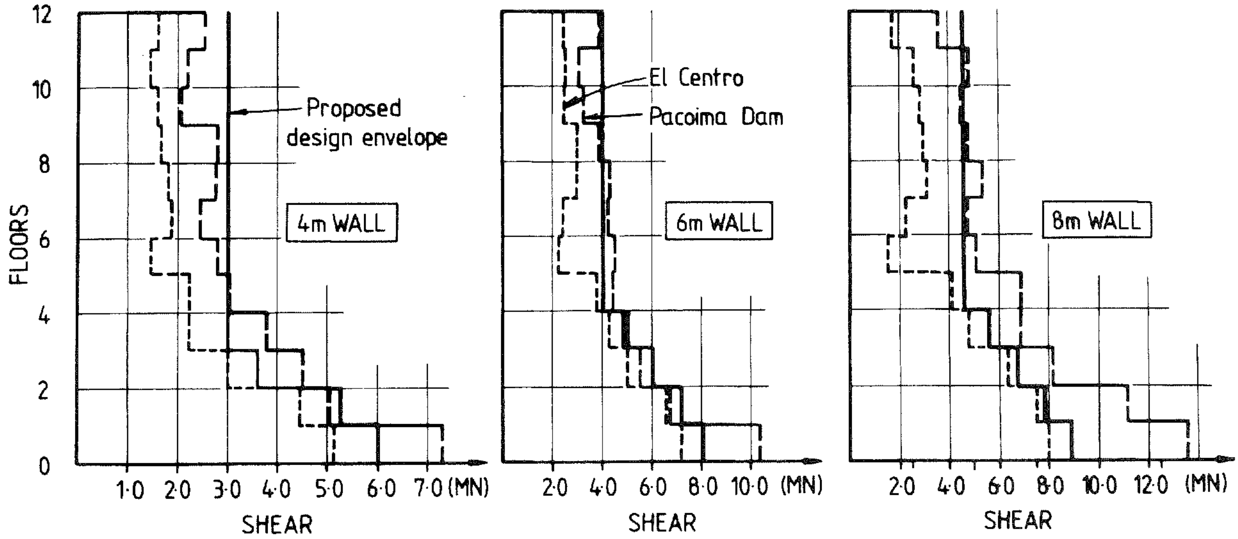
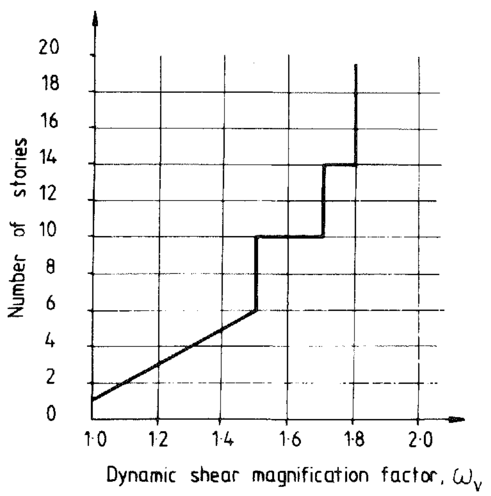
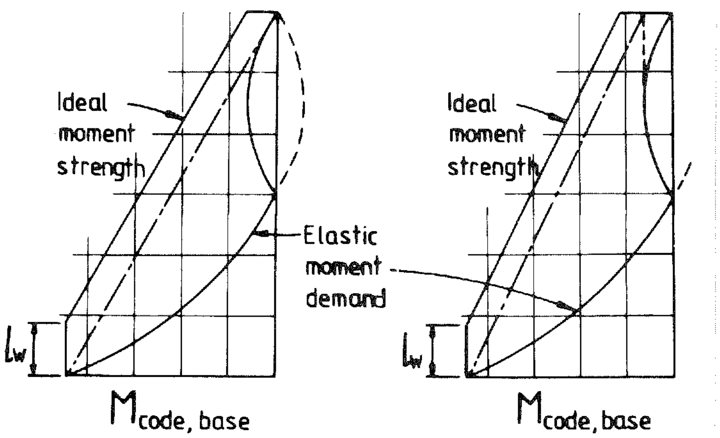


Fig. 21 - Envelopes of Maximum Recorded Wall Shear Force and Proposed Scheme Design Forces





(a) Scheme Based on Free Cantilever Shear Walls

(b) Proposed Empirical Schem

Fig. 22 - Variations of Pure Cantilever ω_v Factor with Number of Storeys

Fig. 23 - Original and Modified Wall Bending Moment Envelopes

5.5 Column Axial Force

As discussed earlier, it is felt that column axial loads for the building component of frame-shear wall structures are adequately assessed by the procedures suggested (1) for pure space frames. Although column ω factors do have an influence on design axial loads this is not great and the proposed changes to column moment dynamic magnification factor ω do not significantly affect axial forces.

5.6 Beam Moment

The adequacy of the beam design scheme, involving moment redistribution in both horizontal and vertical directions is judged by consideration of the inelastic rotations indicated in the analyses. The generally low level and smooth variation of these demands is taken to confirm the suitability of the scheme as proposed in Section 3.4. It also supports the selection of a beam hinging mechanism as a suitable means of energy dissipation. It seems advisable, however, to endeavour to match beam demand as accurately as possible at the top of these multistorey structures. This is to prevent the build-up of excessive beam strength in the upper levels of the structure, thus keeping beam ϕ_o factors to manageable levels and so avoiding large column flexural reinforcement demands in the upper storeys.

6. SUMMARY OF THE PROPOSED DESIGN METHOD

Set out in this section is a step-by-step exposition of the proposed design procedure for coupled frame-shearwall structures. This is based in large part on a recommended method for the evaluation of column actions in ductile multistorey frames (1). The procedure covers the flexural design of beams, columns and walls and the evaluation of design shear forces for columns and walls.

Step 1 Derive the bending moments and shear forces for all members of the frame-shear wall system for the specified lateral earthquake load only, using an appropriate elastic analysis. These actions are subscripted "code".

Step 2 Superimpose the beam bending moments resulting from the lateral load upon the appropriately factored gravity load moments. Subsequently carry out a horizontal moment redistribution in accordance with Section 3.4.3.4 of Ref 1.

Step 3 A vertical moment redistribution allowing a reduction of up to 20% of horizontally redistributed beam moments may be implemented. It is recommended however (Section 5.6) that strength demand at upper floors be matched as closely as possible.

Step 4 Design all critical beam sections so as to provide the required dependable flexural strengths and, determine and detail the reinforcement for all beams of the frame.

Step 5 For both directions of applied lateral load, compute the flexural overstrength of each potential plastic hinge, and determine the moment induced shear forces, V_{oe}, in each beam span.

Step 6 Determine the beam overstrength factor, ϕ_o, at the centreline of each column for both directions of loading, using fixed values of $\phi_o = 1.4$ and 1.1 for ground and roof levels respectively.

Step 7 Derive column design shear forces, V_{col}, at each level according to equation $\alpha\ \phi_o\ V_{code}$ where α has values of 2.5, 1.3 and 2.0 for ground, intermediate and roof levels respectively.

Step 8 Determine at each floor $P_{eq} = R_v \Sigma V_{oe}$, where R_v is an axial load reduction factor found in Table I-2 of Ref. (1).

Step 9 Determine design axial loads $P_{e,max} = D + L_R + P_{eq}$ and $P_{e,min} = 0.9\ D - P_{eq}$ at each floor.

Step10 Calculate magnified column centreline moments above and below beam centrelines as $\phi_o \omega M_{code}$, where the column dynamic magnification factor, ω, takes values of unity at ground and roof levels and 1.2 elsewhere.

Step11 Critical column design moments (at the top or soffit of beams) to be considered with design axial loads (Step 8) are calculated as

$$M_{col} = R_m\ (\phi_o \omega\ M_{code} - 0.3\ h_b\ V_{col})$$

where R_m is an axial load dependent moment reduction factor (1), and V_{col} is found in Step 7.

Step12 Determine design axial loads for the shear walls $D + 1.3\ L_R + E$ and $0.9D - E$.

Step13 Construct the wall bending moment envelope (Figure 23(b)) and together with the design axial loads, determine the required amount and distribution of longitudinal reinforcement over the full height of the wall.

Step14 Calculate the wall base flexural overstrength factor ϕ_o.

Step15 Compute the "shear ratio" for the structure, defined as

$$\frac{V_{wall,base}}{V_{total,base\ code}}$$

Subsequently calculate the wall dynamic shear magnification factor from the equation $\omega_v^* = 1 + (\omega_v - 1)$ x shear ratio, where ω_v is the pure cantilever wall dynamic shear magnification factor (1).

Step16 Calculate wall base design shear force, using Steps 14 and 15 as

$V_{wall,base} = \omega_v^* \phi_o V_{code,base}$

Construct the design shear force envelope as shown in Figure 20 and determine the required transverse reinforcement.

7. CONCLUSION

The study suggests that the coupled frame-shear wall structural system, currently in considerable favour for medium-high rise construction in New Zealand is capable of providing good resistance to seismic attack. The series of dynamic analyses performed on 3 twelve-storey structures indicate levels of member actions that can be estimated with reasonable accuracy on the basis of the traditional simple static lateral load analysis. Satisfactory design of the frame components of these hybrid structures could be achieved by following a capacity design procedure based closely on that for pure frames. The shear wall elements can also be designed in a relatively straight forward manner. Some details of a recommended procedure have been presented. The major features of the procedure are

(a) A wall shear design scheme based on an empirically derived envelope and a dynamic magnification factor which reflects relative wall-frame stiffness as well as wall height.

(b) Column flexural design using a basic dynamic magnification factor of 1.20 and

(c) Beam flexural design allowing both horizontal and vertical moment redistribution.

However, attention is drawn to the fact that the research reported herein constitutes only a first step in an investigation of the seismic behaviour of frame-shear wall structures. Research is currently being undertaken at the University of Canterbury involving an examination of the inelastic response of both 6 and 18 storey structures to simulated seismic attack. In addition, the effect on response of foundation compliance is under study, with preliminary findings suggesting that overall structural response is not unduly sensitive to wall base fixity. It is hoped that at the conclusion of this research programme, a comprehensive and yet simple design procedure for coupled frame-shear wall structures will be obtained.

8. ACKNOWLEDGEMENTS

The work on which this paper is based was carried out at the University of Canterbury, by the first author in the course of his Master of Engineering studies (11), under the guidance of the co-authors.

9. REFERENCES

1. Code of Practice for the Design of Concrete Structures, 1982, NZS 3101, Parts 1 and 2, Standards Association of New Zealand, 283pp.

2. Paulay, T., "The Design of Reinforced Concrete Ductile Shear Walls for Earthquake Resistance", Research Report No. 81-1, University of Canterbury, February 1981, 133pp

3. Carter, B.H.P., "The Seismic Behaviour of Reinforced Concrete Frame-Shear Wall Structures", Master of Engineering Report, University of Canterbury, 1980, 118pp.

4. Jury, R.D., "Seismic Load Demands on Columns of Reinforced Concrete Multi-storey Frames", Master of Engineering Report, University of Canterbury, 1978, 129pp.

5. Paulay, T., "Developments in the Design of Ductile Reinforced Concrete Frames", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 1, March 1979, pp. 35-48.

6. Code of Practice for General Structural Design and Design Loadings for Buildings, 1976, NZS4203, 80pp.

7. New Zealand Reinforced Concrete Handbook", New Zealand Portland Cement Association 1979.

8. Sharpe, R.D. and Carr, A.J., "Inelastic Frame Dynamic Analysis", Computer Program Library, Department of Civil Engineering, University of Canterbury, 1979, 17pp.

9. Sharpe, R.D., "The Seismic Response of Inelastic Structures", Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand, November, 1974, 188pp.

10. Archer, J.A., "Consistent Mass Matrix for Distributed Mass Systems", Journal of the Structural Division ASCE, Vol. 89, No. ST4, August 1963.

11. Goodsir, W.J., "The Response of Coupled Shear Walls and Frames", Master of Engineering Report, University of Canterbury, 1982, 156pp.

10. NOTATION

A_g = Gross area of section

A_s = Area of tension steel

A_{st} = Total steel area in section

b = Width of section

b_w = Web width of section

C = Basic seismic coefficient for the derivation of C_d

C_d = Lateral force coefficient for the equivalent static analysis = CISMR

D = Dead load

E = Earthquake load

f'_c = Specified 28 day concrete strength

f_y = Specified yield strength of steel reinforcement

g = Subscript denoting gross section property

h_b = Overall depth of beam

H = Total height of building

I = Importance factor used in derivation of C_d

l_w = Length of wall section

l_p = Plastic hinge length

L = Live load

L_R = Reduced live load

M = Structural material factor used in derivation of C_d

M_{code} = Column centreline bending moment derived from code specified lateral seismic loading only

$M_{col,red}$ = Column design moment at the face of the beam-column joint

M_i = Ideal flexural strength (= $A_s f_y jd$ for beams)

$P_{e,max}$ = Maximum total design axial load acting on a column during an earthquake = $D + L_R + P_{eq}$

$P_{e,min}$ = Minimum total design axial load acting on a column during an earthquake = $0.9D - P_{eq}$

P_{eq} = Earthquake induced axial column force resulting from beam overstrengths = $R_v \Sigma V_{oe}$

R = Risk factor used in derivation of C_d

R_m = Moment reduction factor dependent on axial load level and ω

R_v = Reduction factor accounting for the likelihood of beam overstrength occurring at all levels of a frame

S = Structural type factor used in derivation of C_d

V_{code} = Shear force in a member due to the design loads

$V_{code,base}$ = Shear force in wall at ground level due to the design loads

V_{col} = Column design shear force = $\omega \phi_o V_{code}$

V_{total} = Code design base shear = $C_d W_t$

W_t = Total seismic weight of a building

Δ = Interstorey drift

ρ = Reinforcement ratio = $A_s/b_w h_b$

ρ_t = Column reinforcement ratio A_{st}/bh

ϕ = Strength reduction factor

ϕ_o = Beam overstrength factor, defined as the ratio of overstrength moment of resistance to moment resulting from code specified loading

ω = General dynamic magnification factor OR dynamic magnification factor for column moment

ω_v = Dynamic magnification factor for isolated cantilever shear walls

ω_v^* = Proposed dynamic magnification factor for wall shear for use in frame shear wall structures

TABLE 1 - Natural Period of First Mode of Vibration (seconds)

Wall size	4 m	6 m	8 m
Approx. method NZS 4203	2.04	1.88	1.71
Modal analysis	1.96	1.78	1.56

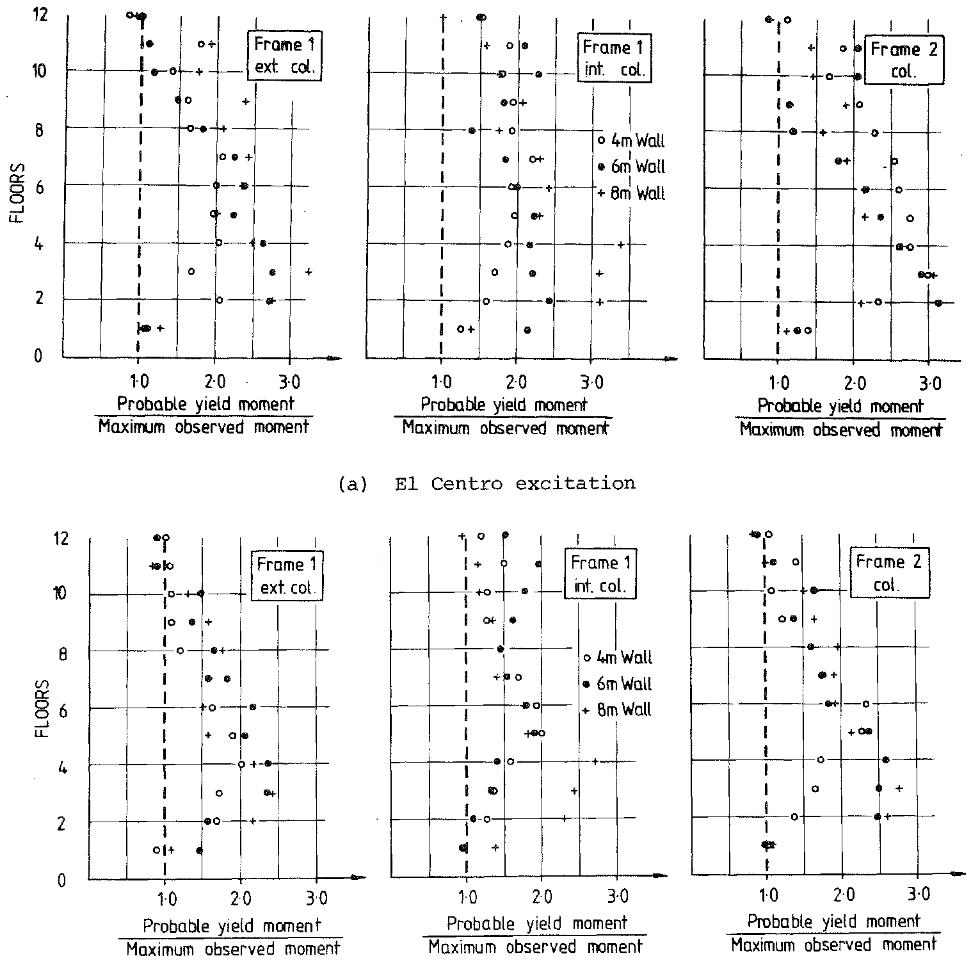
TABLE 2 - Summary of Wall Flexural Reinforcement

Level	4 m Wall	
	End Bars	Distributed Bars
12	8 - D32	2 - D16 @ 350 crs
11	8 - D32	"
10	8 - D32	"
9	8 - D32	2 - D16 @ 300 crs
8	10 - D32	"
7	10 - D32	"
6	12 - D32	2 - D16 @ 300 crs
5	16 - D32	"
4	20 - D32	"
3	24 - D32	2 - D16 @ 250 crs
2	24 - D32	"
1	24 - D32	"

Note: Bar sizes and spacing in mm units
The group of 24 - D32 bars in the end region corresponds with ρ = 4.1% reinforcement content in that region

TABLE 3 - Assumed Member Stiffness Properties for Dynamic Analysis

Property	I	A
Walls	0.6 I_g	0.6 A_g
Columns	0.8 I_g	0.8 A_g
Beams	0.5 I_g	0.5 A_g



(a) El Centro excitation

(b) Pacoima Dam excitation

Fig. 24 - Degree of Flexural Protection Afforded Column Members

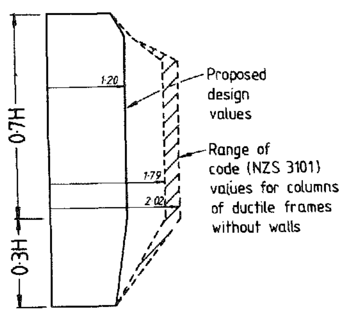


Fig. 25 - Proposed Design Column Dynamic Magnification Factor Distribution, ω