A DESIGN PROCEDURE FOR INTERACTING WALL-FRAME STRUCTURES UNDER SEISMIC ACTIONS

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SUMMARY

Three 12 storey frame 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°W 1970 accelerograms was investigated using a 2-dimensional nonlinear dynamic analysis program. High levels of wall shear forces and generally low levels of column bending moment were encountered. Widespread beam flexural yielding involving moderate levels of inelastic deformations and interstorey drifts, controlled by the walls, indicate the potentially desirable inelastic performance. 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.

DESIGN OF THE MODEL STRUCTURES

General Design Philosophy

The design procedure applied to these framed-wall structures is an extension of a previously postulated (Ref. 1) capacity design philosophy for ductile frames. The principal 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 specially detailed for that purpose. It is a deterministic design philosophy (Ref. 1).

In multistorey framed buildings the desirable hierarchy in plastic hinge formation involves beams rather than columns. Column hinge mechanisms, referred to as soft storeys, are avoided by providing columns with strength in excess of the load input from adjacent hinging beams. The necessary flexural strength of such a column, M_{col}, can be obtained from the simple relationship

$$M_{col} = \omega\phi_o M_{code} - 0.3h_b V_{col} \tag{1}$$

where M_{code} is the moment for the end of the column considered (i.e. centre line of beam) derived by a routine elastic analysis, using a code specified equivalent lateral static load. ϕ_o is the ratio of the beam moment developed with beam plastic hinges at overstrength to the beam moment given by the elastic analysis for the specified lateral load, both being taken at the column centre lines. This factor allows for the actual amount of effective flexural reinforcement provided by the designer, and also for strain hardening of the steel used. The value of ϕ_o is normally in excess of 1.4. The dynamic moment magnification factor ω allows for the fact that during the inelastic

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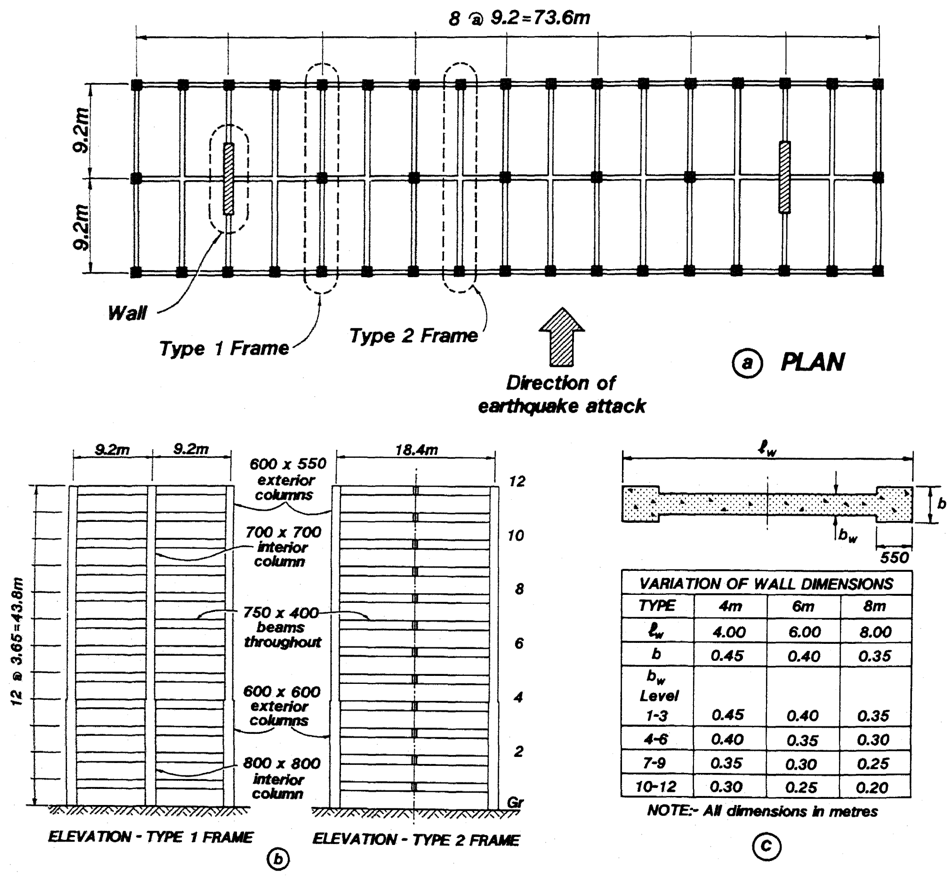
dynamic response of a frame the moment at one end of the column may be significantly larger than the value indicated by the elastic analysis for lateral static loads. This is largely due to response in the higher modes of vibration. Typically $1.4 \leqq \omega \leqq 1.9$ (Ref. 2). The second term of Eq.(1), taking the depth of the adjacent beam h_b and the column design shear force V_{col} into account, reduces the theoretical column moment at the centre line of a beam to the critical column moment at the faces of the beam.



VARIATION OF WALL DIMENSIONS			
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 in metres

Figure 1 - General Dimensions of Buildings Studied

Similar estimates for the maximum or minimum earthquake induced axial loads on columns can be made (Ref. 1). Columns having a corresponding ideal strength, based on the guaranteed yield-strength of reinforcement and compression strength of concrete can then be expected to be protected against the development of plastic hinges in the upper storeys of frames. Accordingly stringent detailing of the reinforcement in the end regions of such columns is not required.

Choice of Structural System for the Study of Buildings

A series of 12 storey somewhat idealized reinforced concrete structures, as shown in Fig. 1, were chosen for this study. The variation of frame and wall stiffness ratio was provided by changing the length of the two walls (ℓ_w = 4, 6 and 8 metres) while the frame components were kept the same.

Details of the Design of the Selected Structures

For computer assisted elastic analysis of unidirectional earthquake attack, standard modelling assumptions for frame members and full base fixity for the walls were made. The equivalent lateral static design load for all these buildings with a fundamental period in excess of 1.2 seconds, was 7.5% of the equivalent mass. The actions so derived were used in the proportioning of members using the "Strength Method" of design with appropriate strength reduction factors, $0.9 \geq \phi \geq 0.7$. To quantify the relative contribution of the walls to total lateral load resistance, the

$$\text{"Shear Ratio"} = V_{wall,base}/V_{total,base,code} \quad (2)$$

was used. For the three types of walls of this study the values of shear ratio were found to be 0.58, 0.75 and 0.83.

In recognition of the predominantly inelastic response of building structures to large ground shaking, significant redistribution of bending moments derived from routine elastic analysis was made use of in determining the final design moments for beams. This involved up to 20% reduction in the peak moment value in any span, redistributed either horizontally to other spans or vertically to beams at other floors. In the process the aggregate moment demand for beams of the 12 floors was not altered. Flexural tension reinforcement ratio, using steel with a yield strength of f_y = 275 MPa, varied from 0.65% to 1.80%.

The prime concern in the design of columns was the selection of the dynamic moment magnification factor ω, described previously in connection with Eq.(1). In view of the control of column deflections and hence their higher mode shapes by the structural walls, it was considered that the allowance for moment increases in columns due to higher mode effects need not be as large as for columns of pure frame structures. Accordingly the maximum value of ω was reduced from 1.9, appropriate for a 12 storey frame (Ref. 2) to 1.45 and this is shown in Fig. 2, where a comparison with other recommended values (Ref. 2) is also made. It is emphasized that the purpose of the magnification of column design moments in accordance with Eq.(1) was to eliminate plastic hinge formation at the ends of columns except at ground and roof levels. Columns so designed with steel having a yield strength of f_y = 380 MPa, had a reinforcement content from a code specified minimum of 0.8% to a maximum of 2.2%.

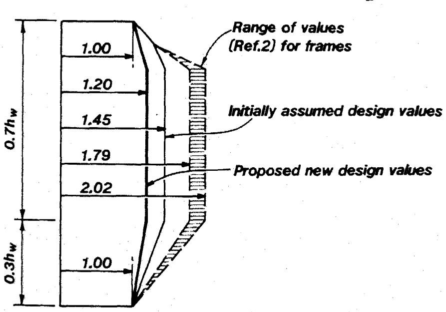


Figure 2 - Variation of Column Dynamic Magnification Factor ω

In the design of walls, first principles based on linear strain distributed along the wall section, with the inclusion of all bars present, were used to determine the necessary vertical reinforcement at the critical base section. The quantity of distributed vertical reinforcement in the web area of the wall section was 0.3% of the concrete area and in the end regions it ranged from 0.5% to

1.34%. To ensure that energy dissipation, when required, will occur only at the base of the wall, the vertical flexural reinforcement is curtailed so as to give flexural resistance that reduces linearly with height rather than that which would follow the bending moment diagram that is obtained from the analysis for the given lateral load (Ref. 2). This enables special detailing for ductility to be provided only in the potential plastic hinge region at the base of the wall. In recognition of the contribution of the higher modes of vibration, the horizontal shear across a wall, derived from the specified lateral load, V_{code}, is magnified so that the design shear force for a free standing cantilever wall becomes:

$$V_{wall} = \omega_v \phi_o V_{code} \tag{3}$$

where the dynamic shear magnification factor is

$$\omega_v = 0.9 + n/10 \qquad \text{when} \quad n \leqq 6 \tag{4a}$$

$$\omega_v = 1.3 + n/30 \leqq 1.8 \quad \text{when} \quad n > 6 \tag{4b}$$

where n is the number of storeys. The ratio of the flexural overstrength of the wall at the base section, M^o, to the moment demand resulting at the same section from the code loading, M_{code}, is defined, by simularity to the case of beams, as the flexural overstrength factor i.e. $\phi_o = M^o/M_{code}$. The aim of this procedure (Ref. 3) is to ensure that a shear failure due to diagonal tension would never occur in a structural wall.

RESPONSE OF THE 4m WALLED STRUCTURE

Space limitations permit the reproduction herein of only a small amount of typical material (relevant to the 4 m walled structure) which is, however, generally representative of the responses of all 3 structures. Emphasis is placed on response to the more credible El Centro event.

Dynamic Analyses

The 2 dimensional inelastic time history program Ruaumoko (Ref. 4) was used to investigate the response of the 3 frame-wall structures to simulated seismic attack. Input data consisted essentially of structural geometry; stiffness and flexural strength data for all members (this latter in the form of simplified hysteretic response curves); lumped nodal weights and rotational inertias; time step of the numerical integration process (a value of 0.01 sec. ensured numerical stability); and a system Rayleigh damping model assigning 5% of critical damping to modes 1 and 10. The two earthquake records selected were the El Centro N-S 1940 and Pacoima Dam S15°W 1970 events, the former being a benchmark for comparison with other analyses, while the severe Pacoima Dam record is considered an upper bound to probable ground motion.

The displacement response, in terms of the horizontal deflections at various floors as shown in Fig. 3, exhibits a primarily first mode oscillation and the development of locked-in plastic displacement. The structure, however, is quite stable. Maximum interstorey drifts were found to be 0.85% (El Centro) and 1.98% (Pacoima Dam) of the storey height.

Wall shear forces of four different levels are compared in Fig. 4. The

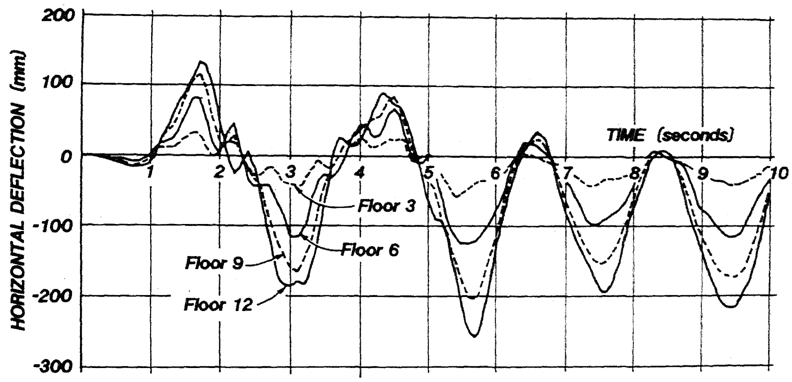


Figure 3 - Horizontal Deflection During El Centro Excitation

probable shear strength, corresponding with the code specified lateral static load and strength design, was assessed as $V_{probable} = 1.33\ V_{code}$, by allowing 13% increase in probable strength of materials (f_y, f'_c) with respect to the specified values and by considering a customary strength reduction factor for shear of $\phi = 0.85$. The design shear for cantilever walls was obtained from Eq.(3) with typical values of $\omega_v = 1.45$ and $\phi_o = 1.45$, so that $V_{wall} = 2.1\ V_{code}$. Finally the shear demands arising from the El Centro and Pacoima Dam records are compared with these values in Fig. 4. The inability of existing procedures, developed for cantilever walls, to adequately predict the necessary shear strength in the storeys, is clearly evident. This demonstrates the serious shortcomings of an elastic analysis for lateral static loads, when an estimate for the strength demand of an inelastic interacting frame-wall structure is to be made.

To overcome this large discrepancy, two aspects had to be considered. Firstly an improved estimate for the absolute maximum of the seismic shear force at the base of the wall was required. As expected, this depends on the stiffness of the wall relative to the frames of the building, as measured by

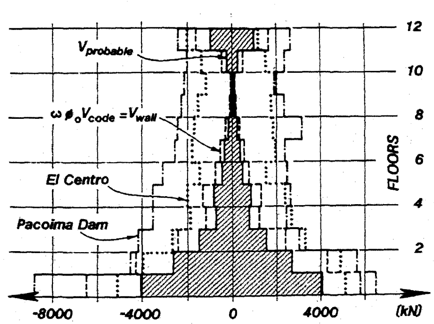


Figure 4 - Envelopes of Wall Shear Force

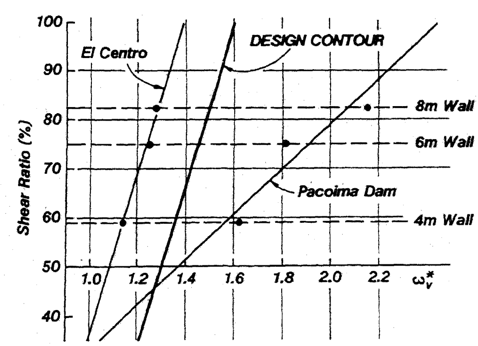


Figure 5 - Variation of Effective Wall Dynamic Magnification Factor

the "Shear Ratio" given in Eq.(2). Secondly an improved envelope for the shear demand along the height of the wall had to be developed.

As Fig. 5 shows, an approximately linear relationship was found between the "Shear Ratio" and the effective dynamic shear magnification factor ω_v^* obtained from the simple relationship

$$\omega_v^* = V_{wall,max}/(\phi_o V_{code,base}) \tag{5}$$

where $V_{wall,max}$ is the maximum base shear force encountered during the dynamic analysis. By relating the severity contours of Fig. 5 to more realistic seismic events, a design contour was proposed (Ref. 5), and this is described by

$$\omega_v^* = 1 + 0.6 \text{ x (Shear Ratio)} \tag{6a}$$

and it may be generalized to a form suitable for a wall of any height as

$$\omega_v^* = 1 + (\omega_v - 1) \text{ x (Shear Ratio)} \tag{6b}$$

where ω_v is given by Eq.(4). Based on the trends of the analysis results, an empirical envelope of the design shear, applicable to all three structures, was proposed (Ref. 5) and this is shown in Fig. 6.

It was observed that the demands for wall moments over the height of the structure were close to those predicted by the linear moment envelope, shown in Fig. 7(a), proposed for free standing cantilevers (Ref. 2) (Ref. 3). Only insignificant yielding in walls was indicated at upper levels.

To cover the larger moment demands in walls interacting with frames, the slightly modified moment envelope shown in Fig. 7(b) is proposed.

Analyses consistently predicted a high degree of protection in columns against yielding in all three structures, when in the presence of concurrent axial forces the maximum column moments developed (Ref. 5). Similarly the procedure recommended for frames (Ref. 1) (Ref. 2) satisfactorily predicted design shear and axial forces in columns in relation to the demands during the

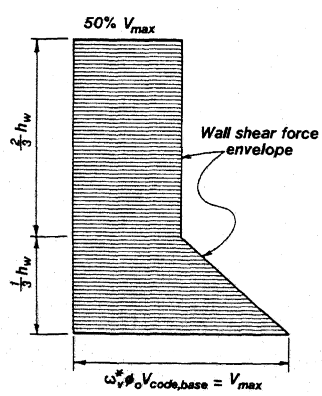


Figure 6 - Proposed Wall Shear Force Design Envelope

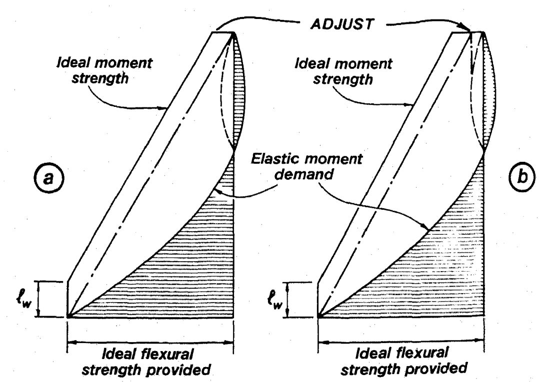


Figure 7 - Wall Bending Moment Design Envelope

inelastic response. However, slight modifications for column design shear forces in the bottom and top storeys are suggested subsequently.

Analytically predicted plastic hinge rotations in the beams were well below 0.035 radians, a level of inelastic deformation considered to be attainable in several displacement cycles in beams detailed in accordance with the requirements of Ref. 2.

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-wall structures. This is based in large part on a recommended method for the evaluation of column actions in ductile multistorey frames (Ref. 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-wall system for the specified lateral static earthquake load only, using an appropriate elastic analysis. These actions are subscripted "code".

Step 2 : Superimpose the beam bending moments so obtained upon the appropriately factored gravity load moments. Subsequently carry out a horizontal and vertical moment redistribution allowing a reduction of up to 30% of beam moments.

Step 3 : Design all critical beam sections so as to provide the required dependable flexural strengths and hence determine and detail the reinforcement for all beams of the frame.

Step 4 : For both directions of applied lateral load, compute the flexural overstrength of each potential plastic beam hinge, and determine the corresponding moment induced shear forces, V_{oe}, in each beam span.

Step 5 : 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 6 : Derive the column design shear forces $V_{col} = \alpha\phi_o V_{code}$ at each level, where the column dynamic shear magnification factor α is 2.5, 1.3 and 2.0 for bottom, intermediate and the top storey respectively.

Step 7 : Estimate in each storey the maximum likely earthquake induced column axial load $P_{eq} = R_v \Sigma V_{oe}$, where $R_v = (1 - n/67) \geqq 0.7$ is a reduction factor that takes the number of storeys n into account (Ref. 2).

Step 8 : Determine the total design axial load on columns $P_{e,max} = P_D + P_{LR} + P_{eq}$ and $P_{e,min} = 0.9P_D - P_{eq}$, where P_D and P_{LR} are forces due to dead and reduced live gravity loads respectively.

Step 9 : The design moments for columns are from Eq.(1)

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

where R_m is an axial load dependent reduction factor applicable to columns subjected to tension or compression stresses not exceeding 10% of the compression strength of the concrete (Ref. 1) and V_{col} is found in Step 6. The value of ω is given in Fig. 2.

Step 10 : Determine the design axial forces due to appropriately factored gravity and for earthquake loads on the walls.

Step 11 : From the maximum earthquake load induced bending moment at the wall base and the above axial loads, determine the necessary vertical wall reinforcement. When curtailing bars with height, follow the linear moment envelope of Fig. 7(b).

Step 12 : Having completed the detailing of the wall flexural reinforcement, determine the flexural overstrength factor ϕ_o with respect to the base moment.

Step 13 : From the elastic analysis using Eq.(2) determine the "Shear Ratio" and hence evaluate from Eq.(6b) and (4) the dynamic shear magnification factor ω_v^* for the walls.

Step 14 : With wall shear force at the base, obtained from the elastic analysis, determine the maximum wall design shear force $V_{wall} = \omega_v^* \phi V_{wall,base,code}$ and from Fig. 6 construct the shear design envelope.

Step 15 : Determine the necessary horizontal wall shear reinforcement in accordance with appropriate code requirements.

CONCLUSIONS

This study suggests that coupled wall-frame structural systems are capable of providing good drift control for moderate and excellent response to severe seismic attack. The analyses performed for 3 prototype structures indicate that member actions, when suitably modified to allow for changes due to inelastic dynamic effects, can be satisfactorily predicted with a traditional simple static lateral load analysis. The major features of the recommendations for modification to static analysis and design procedures are:

(a) A design scheme for wall shear forces incorporating a dynamic magnification factor accounting for both relative wall stiffness and wall height, and an empirically derived shear envelope. This is expected to ensure that when required, energy dissipation in walls will be obtained primarily from flexural yielding in predetermined plastic hinges.

(b) Flexural design of columns based on moment input from beams at flexural overstrength and dynamic magnification of moments so derived by a factor of 1.2. This should ensure that no ductility demand will arise.

(c) A flexural design of beams which allows redistribution of design moments not only horizontally from one span to another, but also vertically among beams of several floors.

The study, the first stage of which is reported here, is continuing at the University of Canterbury.

ACKNOWLEDGEMENT

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