A seismic design strategy for hybrid structures

Thomas Paulay
University of Canterbury, Christchurch, New Zealand

ABSTRACT: To complement existing capacity design procedures adopted in New Zealand for reinforced concrete buildings in which earthquake resistance is provided by ductile frames or by ductile structural walls alone, an analogous methodology is presented for the design of ductile hybrid structures. First the concepts of this deterministic design philosophy are highlighted. Modelling and types of structures in which the mode of wall contribution is different are presented. A step by step description of a capacity design procedure for a structural system in which fixed base ductile frames and walls, both of identical height, interact, is presented. The rationale for each step is outlined and, where necessary, evidence is offered for the selection of particular design parameters and their magnitudes. Brief references are made to the effects of deformable wall foundations. A number of issues which require further study are briefly outlined. It is believed that the methodology is logical, relatively simple and that, when combined with appropriate detailing, it should ensure excellent seismic structural response.

1 INTRODUCTION.

For the majority of ordinary tall buildings, the overriding seismic design criterion will be the prevention of collapse and hence loss of life, as a possible consequence of the largest earthquake which could be expected during the useful life of that building. Structural survival is closely related to the ability of the structure to offer a specific level of resistance against lateral loading, while extreme ground motions may impose significant ductility demands on various parts. Other criteria, such as availability of adequate stiffness and strength, must also be satisfied. These latter criteria relate to damage control in the event of moderate but more frequent earthquakes.

To ensure predictably satisfactory inelastic response of a reinforced concrete building during an extreme seismic event, the designer must rely on viable energy dissipating mechanisms within the structure. These will then provide the necessary hysteretic damping. Subsequently a significant part of the design effort must concentrate on the techniques of detailing potential plastic regions where ductility demands will arise. In the context of design for survival the importance of the accuracy of elastic structural analyses must be de-emphasized. This is because of the inevitable gross approximations involved in the specifications of building codes for using equivalent lateral static design loads (International Association for Earthquake Engineering, 1984) and the uncertainties associated with inelastic dynamic structural response to ground motions, the characteristics of which are yet impossible to predict. Specified static lateral loads or elastic modal analysis techniques should be considered mainly as means to ensure that there is a rational distribution of potential resistance throughout the structure. It should be remembered that for ductile structures the resistance resulting from requirements of lateral code loadings is considerably less than what would be required if buildings were to respond elastically to the design earthquake. Therefore in the design of ductile structures any elastic analysis technique which preserves consistency in the treatment of structural members should be considered satisfactory.

The above considerations led to the development in New Zealand of a simple deterministic seismic design approach, to be reviewed in the following pages. In this the results of traditional elastic analyses for specified lateral static loads are utilized to establish an acceptable hierarchy in the development of energy dissipating mechanisms. This hierarchy must be formulated by the designer in an effort to command the structure "what to do" in the case of an extreme seismic event. Once the choice is made, each member is given appropriate strength or resistance to ensure that, when required, only the chosen plastic mechanisms can develop within the structure. As will be seen, this powerful design tool is very simple. Its simplicity arises from the intent of the designer to command the structure "what it must do", rather than to ask by way of analyses. "what it might do". The aim is thus to ensure desirable and predictable elasto-plastic structural behaviour during an extreme event with unpredictable characteristics.

A justification of the above expectations relies on high quality in the detailing of the potential plastic regions of concrete components. Utilizing experimental research findings of the last 15 years, determined efforts were also made in the relevant code issued by the Standard Association of New Zealand (1982) to unambiguously quantify the "goodness" of detailing. Because of the need for "good" detailing, numerous and quantified recommendations were developed to guide the designer in the treatment of the critical regions of a structure, to which energy dissipation has been assigned. Some of these recommendations were reported by Paulay (1975,1980,1981).

2 THE CONCEPTS OF CAPACITY DESIGN

If a hierarchy in the chain of resistance is to be established, then the designer must rationally choose weak links and strong links. Thus strengths or capacities must be compared. It is for this reason that the term "capacity design" was coined. In the capacity design of earthquake resisting structures, elements of primary load resisting systems are chosen and suitably designed and detailed for energy dissipation under severe inelastic deformations. All other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

When the strength of one element is compared with that of another element, it is necessary to evaluate the likely strengths mobilized during large displacements imposed by severe earthquakes. A common term used in the strength design approach is the nominal or ideal strength S_i. It is obtained from theory predicting the failure geometry while using specified material strengths. The dependable strength or factored resistance is then obtained from $S_r = \phi S_i$, where ϕ is a specified strength reduction factor or it is a combination of materials and member resistance factors. During a large inelastic seismic pulse, material strengths considerably larger than those assumed or specified may be mobilized. For example steel strength at strain hardening may develop. Concrete strength may be enhanced by confinement. Moreover, for practical or other reasons, more reinforcement may have been provided at critical sections than what design equations indicated. All factors taken into account allow the maximum likely strength or overstrength to be estimated as

$$S_o = \rho_o S_i \qquad (1)$$

where ρ_o is the overstrength factor relevant to a particular section. Its value ranges typically from 1.25 to 1.50 depending mainly on the grade of steel used (Park and Paulay 1975).

When comparing the strengths of two adjacent elements, for example beams and columns of a ductile frame, it is convenient to relate these to a code specified seismic load demand. For example the flexural overstrength factor for a beam, appplicable to computed moments at centreline of an exterior column is

$$\phi_o = M^o / M_{e,code} \qquad (2)$$

where M^o = flexural overstrength of the beam as built and derived with the use of equation (1), and $M_{e,code}$ = the moment at the same end, derived from the appropriately factored code specified lateral static seismic load. In this the maximum likely developed strength of a beam is compared with the intended moment demand due to the specified lateral earthquake load only. The strength of the beam, as built, may well have been governed by other load considerations, such as gravity. If it is the designer's intention to make this exterior column stronger than the adjacent beam, a moment input from the beam $\phi_o M_{e,code}$ will need to be considered.

The flexural resistance of a beam section with different amounts of top and bottom reinforcement, depends on the sense of the rotation applied. Therefore the value of the flexural overstrength factor, ϕ_o, depends on the direction of earthquake attack. Conveniently this is identified with the symbols $\overrightarrow{\phi}_o$ and $\overleftarrow{\phi}_o$.

The second important consideration in the establishment of strength hierarchy is the recognition that the pattern of design actions within the structure, such as moments, shear and axial forces, during the inelastic dynamic response of the ductile structure may markedly differ from those derived for a specified lateral static load acting on an elastic structure. To allow for this phenomenon in the estimation of the maximum likely load demand on the strong links of the chain of resistance during a large earthquake, a dynamic magnification factor ω is introduced.

With reference to the simplistic example of a chain shown in Fig. 1, the essence of the capacity design philosophy may thus be expressed by

$$S_s = \omega \phi_o S_w \qquad (3)$$

where S_s = the required ideal strength of a strong link, S_w = the dependable strength assigned to the adjacent weak link with the use of the specified lateral earthquake loading, ϕ_o = an overstrength factor such as given by equation (2) and derived from the maximum likely strength of this weak link, and ω = dynamic magnification factor which estimates the deviation of load patterns during the inelastic dynamic response of the structure from corresponding patterns predicted by elastic analyses for specified lateral static loads. The application of this simple concept is presented in the following sections with respect to hybrid structural systems.

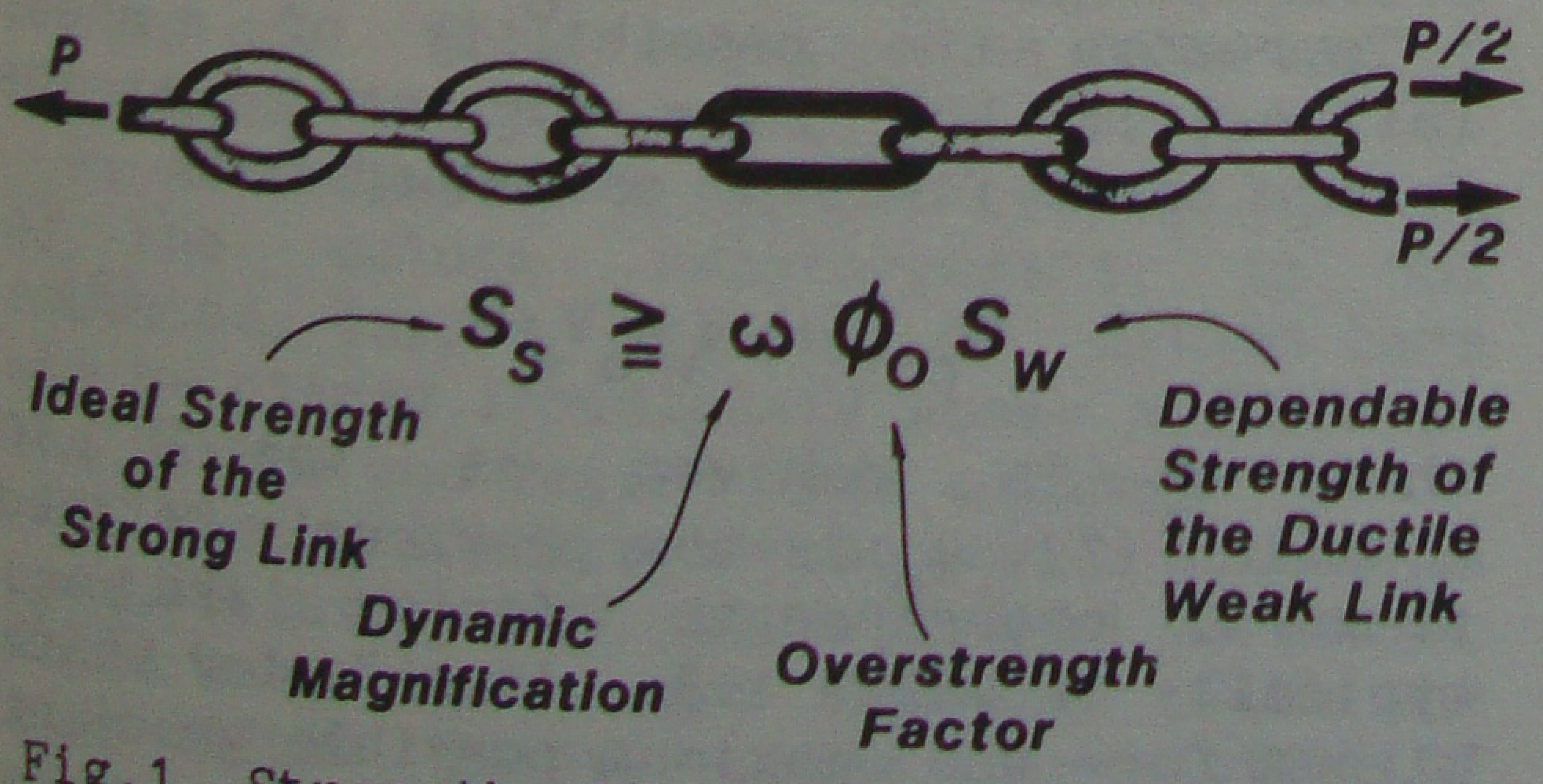


Fig.1 Strength hierarchy of links in a chain

Capacity design procedures for concrete structures have been introduced by the Standards Association of New Zealand (1982) and have also been adopted for specific cases by the Comitte Euro-International du Beton (1983) and the Canadian Standards Association (1984).

As the ideal strength, assigned in this technique by equation (3) to the strong links in the chain of resistance, is an upper bound estimate of load input at a stage when significant damage has already occurred in the weaker links of structure, no further precautions need by applied. Hence the ideal strength S_s of the strong links neeed not be reduced by the customary strength reduction or resistance factors, ϕ. Therefore ϕ = 1 may be used throughout when capacity design procedures are employed to determine the strength of strong links.

3 A REVIEW OF EXISTING CAPACITY DESIGN PROCEDURES

3.1 Cantilevered structural walls

Often the entire lateral load resistance is assigned to a number of strategically positioned fullheight cantilever walls. Assuming, as is generally done, that floor slabs act as infinitely rigid diaphragms, lateral load is readily allocated to each wall. To ensure also satisfactory inelastic response, the following points, illustrated with the simple example of figure 2, should be considered:

1. In recognition of the fact that during a severe earthquake, all walls will need to develop plastic hinges with significant ductility, preferably at the base, some redistribution of resistance between walls may be considered. In this the designer will, among other aspects, need to consider wall flexural strength and foundation resistance to the moments associated with wall hinge formation. For example Wall 3 in figure 2 may carry significantly larger gravity loads than the other two walls. Thereby its potential for flexural resistance is enhanced. Hence the designer may decide to reduce the initial lateral load demand on Walls 1 and 2 and correspondingly assign more earthquake resistance to Wall 3, as shown in figure 2 by the dashed curves. To ensure that no reduction in the total resistane occurs, the condition $\Delta M_1 + \Delta M_2 \leq \Delta M_3$ is to be satisfied. Such load redistribution has negligible effect on the ductility potential of a wall if the reduction of flexural resistance does

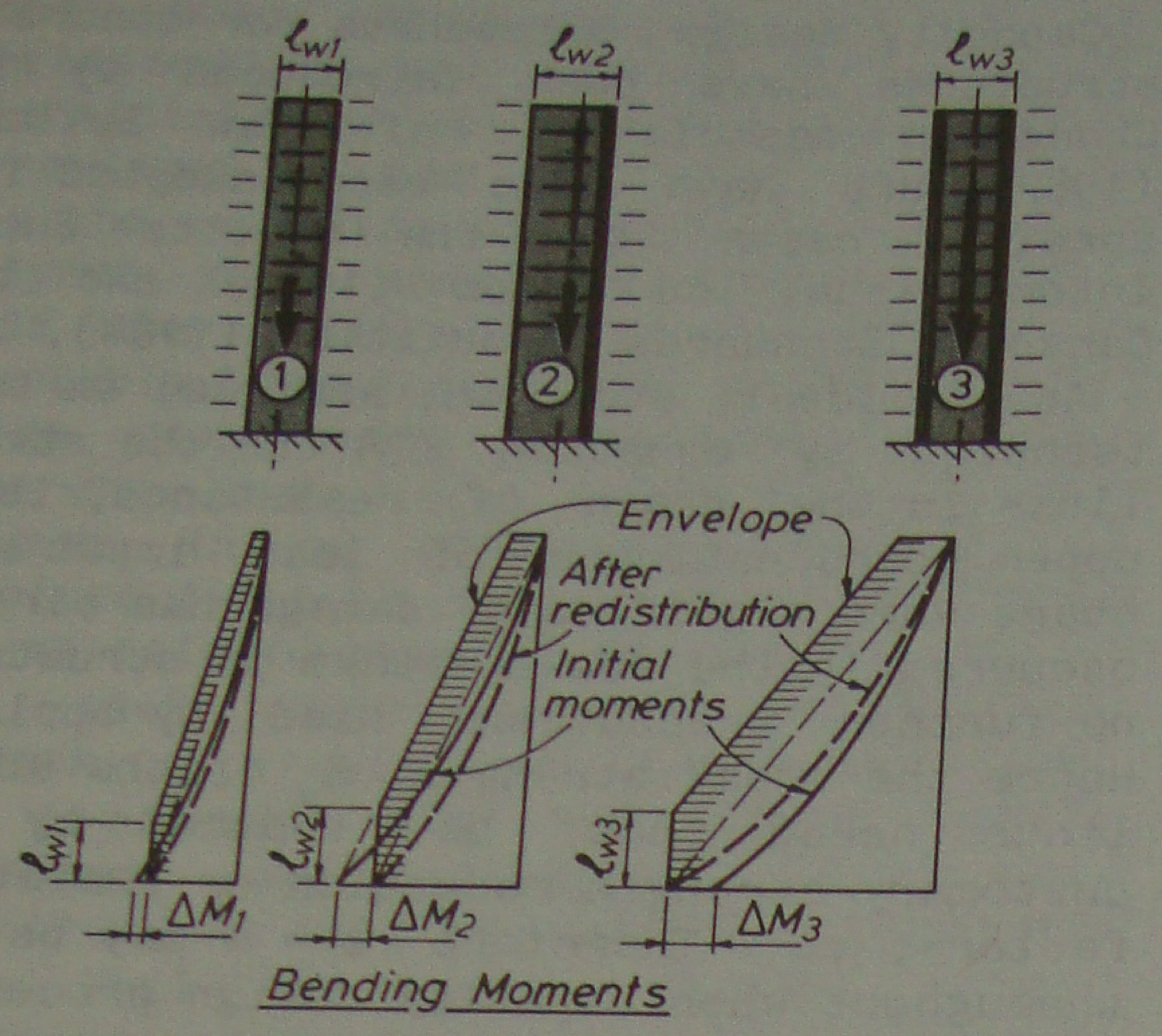


Fig.2 Load distribution between interconnected cantilever walls, and design moment envelopes

not exceed 30% of the initial value.

2. The use of conservative linear resistance envelopes in figure 2 for providing flexural reinforcement in the walls, restricts plasticity to the base. This eliminates the need to detail the wall for ductility, except in this potential plastic hinge zone, which may be assumed to be restricted to a height equal to the length, ℓ_w, of a wall.

3. It is vital to ensure that a shear failure does not interfere with the ductile flexural response. Hence the ideal shear resistance of the potential plastic hinge of a ductile wall, V_{wall}, should not be less than the shear demand during the largest expected seismic event, which is in terms of equation (3)

$$V_{wall} \geq \omega_v \phi_{o,w} V_e \qquad (4)$$

where V_e is the design shear force derived in the "initial" elastic lateral load analysis, ω_v is a dynamic magnification factor recognizing higher mode participation during dynamic response. The flexural overstrength factor $\phi_{o,w}$ is the ratio of the maximum potential flexural strength of the wall base section as constructed, M^o_{wall}, to the moment demand found in the "initial" elastic analysis, $M_{e,wall}$.

The dynamic shear magnification recommended by the Standards Association of New Zealand (1982) is

$$\omega_v = 0.9 + n/10 \text{ when } n \leq 6 \qquad (5)$$

$$\omega_v = 1.3 + n/30 \text{ when } n \geq 6 \qquad (6)$$

where n is the total number of storeys.

These points illustrate the most important features of this deterministic design philosophy as applied to a very simple ductile wall system.

3.2 Ductile frames

With some modifications, the procedure, outlined in the previous section, may be applied to rigid jointed ductile frames. The basic principles of this approach, developed in New Zealand, were summarized previously by Paulay (1980b). In the design of ductile beams, moment redistribution and the shear strength associated with two plastic hinges in each span, are taken into account. Design actions for columns, of sufficient magnitude to prevent significant inelasticity in upper levels, may then be determined with the application of the general relationship.

$$M_{col} = \omega \phi_o M_e \qquad (7)$$

where M_e is the design moment at the end of the column considered and derived from the "initial" elastic analysis for the speciified lateral static load only, ϕ_o is a direction dependent flexural overstrength factor, which in the case of frames is the ratio of the sum of the flexural overstrengths developed by the beams, as built, to the sum of the strengths required for the same beams by the code-specified earthquake loading only, both sets of values being taken at the centreline of the relevant column.

The influence of higher modes on the moment pattern along a column during the dynamic response is recognized by the dynamic moment magnification factor, ω. This is a period dependent parameter.

The effects of the use of equation (7) on the design moments, M_e, of a typical upper storey column are qualitatively shown in figure 3. Common values are $1.3 \leq \omega \leq 1.9$ and $1.4 \leq \phi_o \leq 1.6$.

To remove the possibility of a shear failure, it has been suggested that the design shear force be estimated thus

$$V_{col} = \omega_v \phi_o V_e \qquad (8)$$

where V_e is derived as was M_e in equation (7), ϕ_o is the previously defined flexural overstrength factor and ω_v, typically of value 1.3, is a factor which allows for the maximum possible gradient of the column moments in a storey, as shown in figure 3.

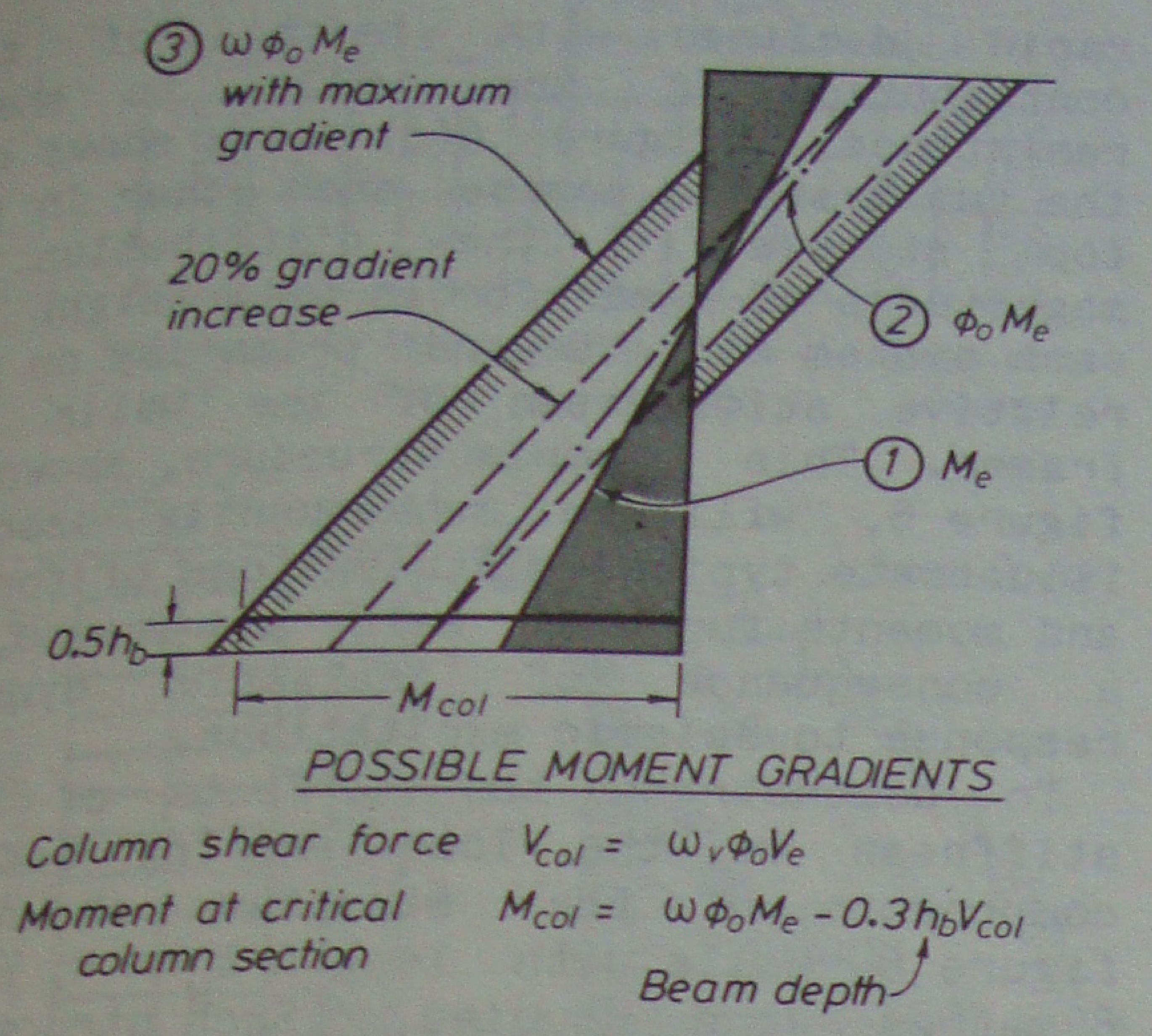


Fig.3 The magnification of design moments for columns

Apart from the primary aim of this approach, the elimination during a large earthquake of simultaneous plastic hinge developments at the top and the bottom of all columns in any storey of a multistorey reinforced concrete frame, other important benefits accrue. These are:

1. The likelihood of plastic hinge developmens at the ends of upper storey columns are eliminated. The exceptional case of some restricted yielding of column reinforcement is not synonymous with plastic hinge formation and consequent significant ductility demand.

2. Because excessive inelastic concrete compression strains at the critical column sections are not expected, the need to confine column cores with significant amounts of transverse hoop reinforcement does not arise.

3. The essentially elastic response of columns during the largest expected earthquake ensures integrity of the concrete and hence its contribution to shear resistance in the end regions can be relied on. This results in the use of less shear reinforcement.

4. As inelastic reversed cyclic loading of the column reinforcement is not expected, lapped splices of column bars may be placed immediately above a floor in the bottom end region of a column.

5. When precast concrete construction is used, columns may be attached to beams, which have been cast together with the beam-column joint, using connections of sufficient strength but limited ductility.

6. Elastic columns improve the performance of beam column joints, which are often critical regions of ductile frames.

4 INTERACTING DUCTILE FRAMES AND WALLS

When lateral load resistance is provided by the combined contributions of ductile multistorey frames and structural walls, the system is often referred to as a "hybrid structure". In North America, the term "dual system" is used. These structures combine the advantages of their constituent components. Because of the large stiffness of walls which are provided with adequate restraints at the foundations, excellent storey drift control may be obtained. Moreover, suitably designed walls can ensure that storey mechanisms (soft storeys) will not develop in any event. Interacting ductile frames on the other hand, while carrying the major part of the gravity load, can provide, when required, significant energy dissipation, praticularly in the upper storeys.

The traditional procedure of designing for earthquake resistance, utilizing elastic analysis techniques and equivalent lateral static loads, is generally used also for hybrid structures. The resulting distribution of lateral load resistance over the height of buildings with ductile frames, or structural walls, is generally accepted as meeting satisfactorily actual earthquake load demands. There is little evidence to indicate that this would be the case also with hybrid structures. One source of concern for possibly drastic differences between "elastic static" and "elasto-plastic dynamic" responses of hybrid structures stems from the recognition of fundamental differences in the behaviour of beam-column frames and structural walls. These differences stem from dissimilar deformation patterns when subjected to the same lateral load, as shown in figure 4. Frames and walls, while sharing in the resistance of shear forces in the lower storeys, oppose each other in the storeys near the top of the building. It was of major interest to examine the load sharing between these two types of interacting elements during inelastic dynamic response to a major seismic event.

A step-by-step design methodology is proposed to meet the intent of the "capacity design" philosophy outlined in section 2. The presentation concentrates

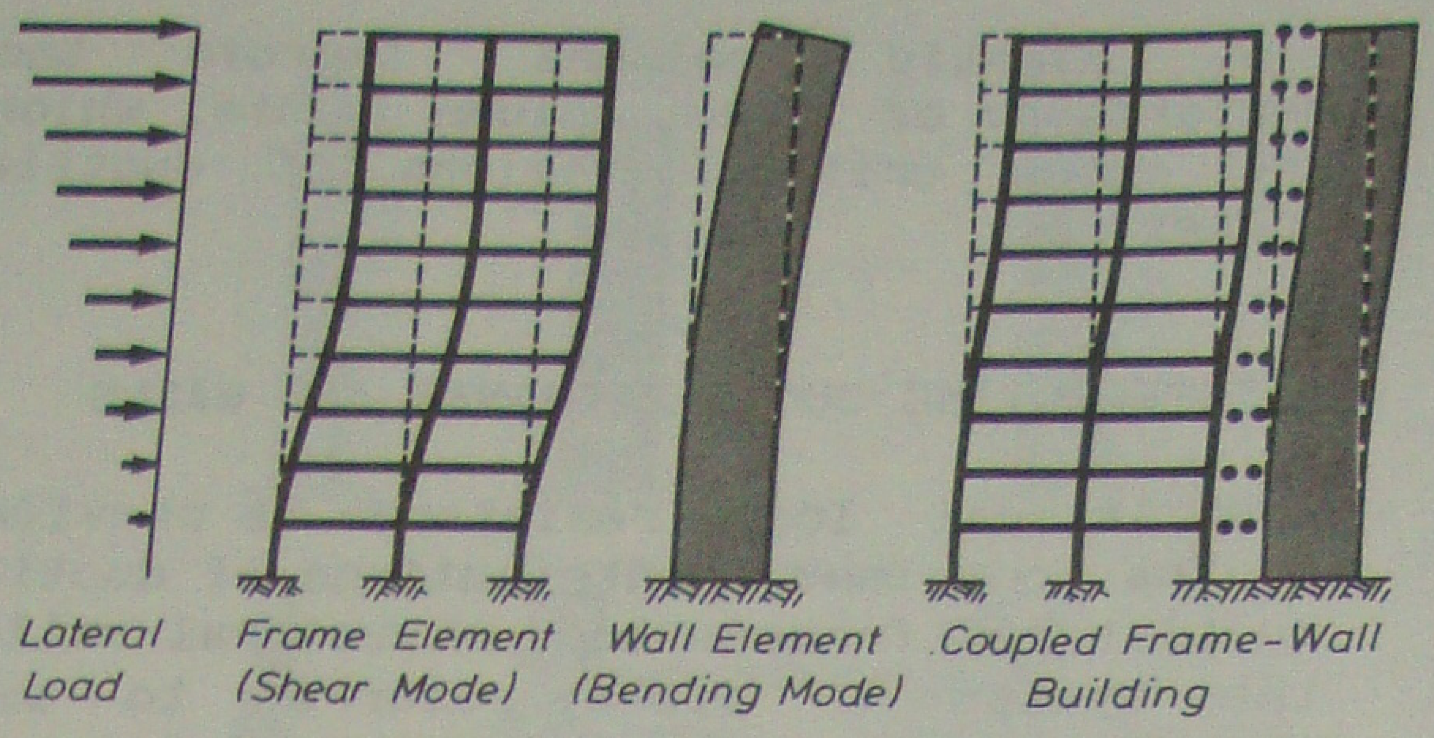


Fig.4 Deformation patterns of laterally loaded frames, walls and coupled wall frame elements

solely on issues relevant to the largest expected seismic event envisaged by building codes. The emphasis is therefore on issues of ductility and the prevention of collapse. Existing procedures, to satisfy design criteria for stiffness and minimum strength, both relevant primarily to damage control, and considered to be equally applicable to hybrid structures, are not referred to in this paper.

4.1 Types of hybrid structures and their modelling

In the majority of reinforced concrete multistorey buildings, lateral load resistance is assigned to both ductile space frames and cantilever strutural walls. Generally these extend over the full height of the building, as shown in figure 5(a). Figure 6(a) shows in plan the somewhat idealized symmetrical disposition of frames and walls in a 12 storey example building. The properties of these two distinct structural elements may be conveniently lumped into a single frame and a single cantilever wall, as shown in figure 6(b). Instead of individual walls, shown in figure 6(a), tubular cores, or coupled structural walls, are also used frequently.

The extensionally infinitely rigid horizontal connection between lumped frames and walls at each floor, shown in figure 6(b), enables the analysis of such laterally loaded elastic structures to be carried out speedily. Typical results are shown in figure 6(c)(d) and (e). Here the sharing between walls and frames of the total storey shear forces is illustrated. The relative participations, which reflect the behaviour of the two different systems, as shown in figure 4, indicate a rapid decline with height of the contribution of the walls to shear resistance. Figure 6(c) also shows how the two systems oppose each other in the top storeys. The distribution of magnitudes of shear forces with height for each system will depend primarily on the relative stiffnesses of the walls and frames. This example structure, shown in figure 6, will be subsequently used to illustrate typical distributions of forces and moments for both walls and frames, as a consequence of inelastic dynamic response to seismic excitations.

To demonstrate the effects of wall stiffness on the load sharing between components of the structure shown in figure 6, walls with length ℓ_w = 4, 6 and 8 metres were studied. Each structure, consisting of two cantilever walls and 7 identical frames, was subjected to the same intensity and pattern of lateral static loading. The contributions of components to each total overturning moment and storey shear forces, are shown in figure 6(d) and (e). Because of the gross incompatibility of deformations in the upper storeys, seen in figure 4, the seven frames are required to resist overturning moments and storey shear forces at these levels, which are larger than the total imposed by the external lateral load. The examples of figure 6 emphasise the fact that cantilever walls of hybrid systems appear to make significant contributions to lateral load resistance, but only in the lower storeys. The contribution of flexible walls diminishes rapidly in the upper storeys.

It is seen that the ratio of lateral load sharing for a particular hybrid system is not a fixed value. The ratio changes from storey to storey. Therefore the requirements of some building codes, whereby a specific fraction of the total lateral load (typically 25%) is to be assigned to the frames of the hybrid

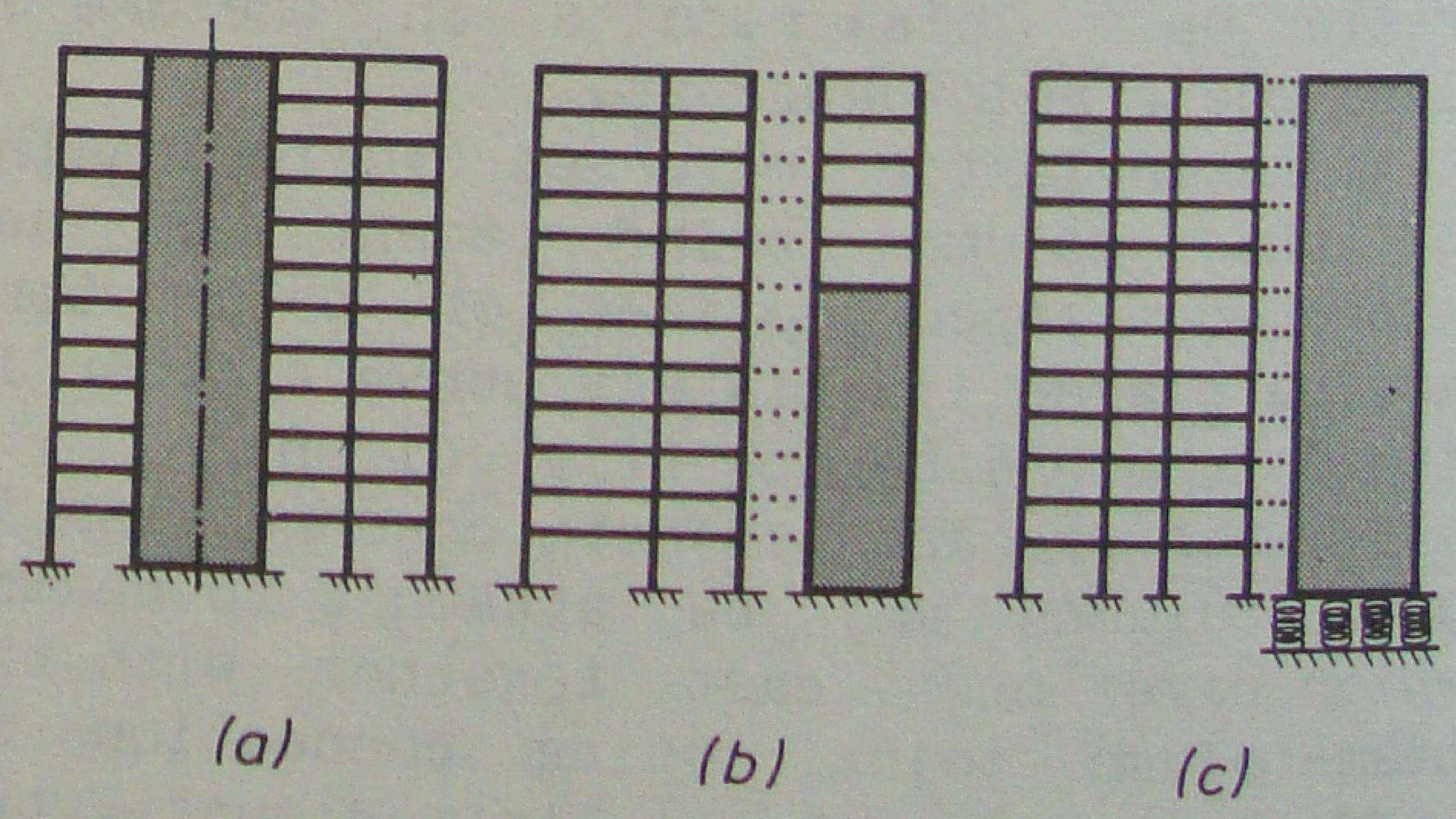


Fig.5 Modelling of different types of hybrid systems

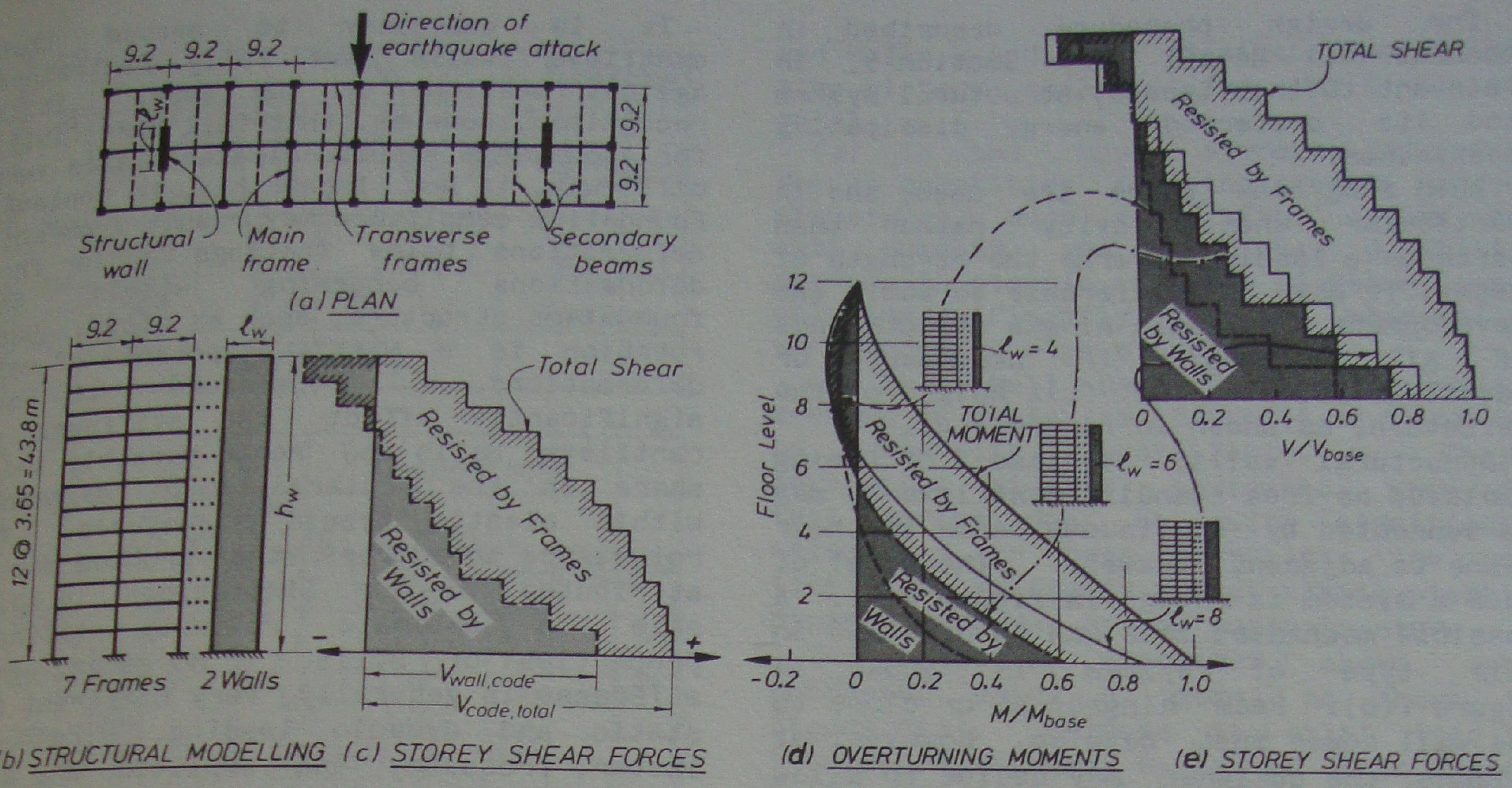


Fig.6 Wall and frame contributions to the resistance of overturning moments and storey shear forces in example structures

system, is ambiguous. No advantages appear to accrue from assigning lateral load resistance to frames, significantly different from that indicated by analysis of the laterally loaded total wall-frame system. For example, as figure 6 suggests, in the upper third of the height, approximately the total storey shear forces should be assigned to columns of the frames, irrespective of the stiffness of the cantilever walls.

As a matter of convenience, the relative contribution of all cantilever walls to the resistance of the total code specified lateral static load on the building may be expressed by the ratio of the sum of the horizontal shear forces assigned to all the walls and the total external shear to be resisted, both values taken at the base of the structures. From figure 6(b) this shear ratio is thus

$$\psi = (\Sigma V_{wall,code}/V_{code,total})_{base} \qquad (9)$$

This wall shear ratio, ψ, will be used subsequently to estimate the maximum likely wall shear demands during the dynamic response of the hybrid system. For the three example structures shown in figure 6(d) and (e) with 4, 6 and 8 m walls, the wall shear ratios are seen to be $\psi = 0.59$, 0.75 and 0.83 respectively.

As the flexural response of walls is intended to control deflections in hybrid structures, the danger of developing "soft storeys" should not arise. The designer may therefore freely choose those members or localities in frames where energy dissipation should take place when required. A preferable and practical mechanism for the frame of figure 6 is shown in figure 7(a). In this frame, plastic hinges, when required during a large expected seismic event, are made to develop in all the beams and at the base of all vertical elements. At roof level, plastic hinges may form in either the beams or the columns. The main advantage of this system is in the detailing of the potential plastic hinges. Generally it is easier to detail beam rather than column ends for plastic rotation. Other advantages were listed in Section 3.2.

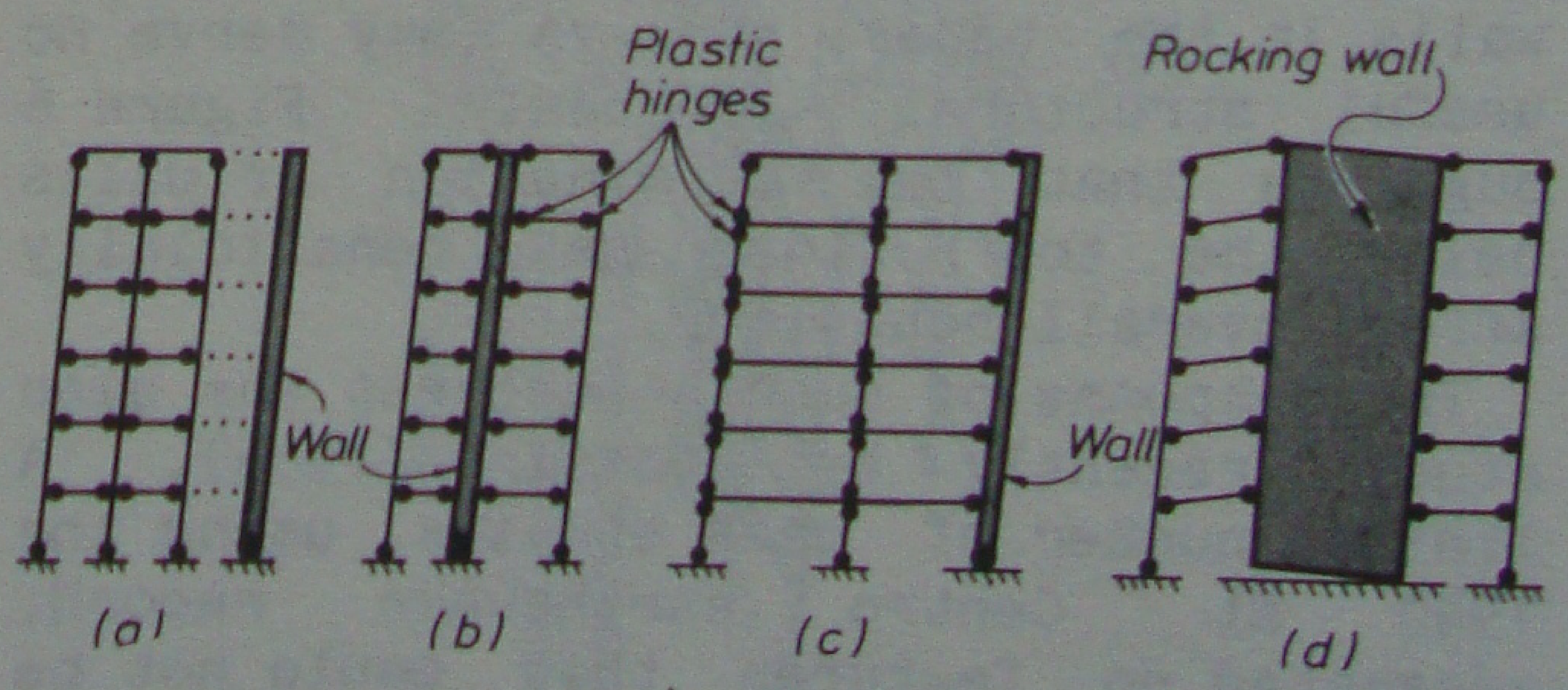


Fig.7 Energy dissipating mechanisms in hybrid structural systems

The design procedure described in considerable detail in Section 5, is relevant to this type of structural system and its preferred energy dissipating mechanisms.

When long span beams are used, and in particular when gravity rather than earthquake loading governs the strength of beams, it may be preferable to admit the development of plastic hinges at both ends of all columns, if necessary or advantageous, over the full height of the structure, as shown in figure 7(c).

Structural walls, instead of being isolated as free standing cantilevers, may be connected by continuous beams in their plane to adjacent frames. The model of such a system is shown in figure 5(a). A possible mechanism that can be utilized in this type of system is shown in figure 7(b). Beam hinges at or close to the wall edges must develop. However, at columns, the designer may decide to allow plastic hinges to form in either the beams or the columns, above and below each floor, as shown in figure 7(c).

4.2 Walls with special features

Although in most buildings structural walls extend over the full height, there are cases when for architectural or other reasons, walls are terminated below the level of the top floor. A model of such a structure is shown in figure 5(b).

Because of the abrupt discontinuity in total stiffnesses at the level where walls terminate, the seismic response of these structures is viewed with some concern. Gross discontinuities are expected to result in possibly critical features of dynamic response which are difficult to predict. It is suspected that the regions of discontinuity may suffer premature damage and that local ductility demands during the largest expected seismic events might exceed the ability of affected components to deform in the plastic range without significant loss of resistance.

On the other hand, elastic analyses for lateral static loads show that structural walls in the upper storeys may serve no useful structural purpose. Figure 6 suggests that the termination of walls below the top floor may beneficially affect overall behaviour.

The response of such structures was also studied recently by Goodsir (1985). A limited number of case studies, using the 1940 El Centro earthquake record, suggested no features that could not be readily accommodated in currently used design procedures.

It is customary to assume that cantilever walls are fully restrained against rotations at the base. It is recognised, however, that full base fixity for such large structural elements is very difficult, if not impossible, to achieve. Foundation compliance may result from soil deformations below footings and/or from deformations occurring within the foundation structure, such as piles. Base rotation is a vital component of wall deformations. Therefore it may significantly affect the stiffness of cantilever walls and hence possibly their share in the lateral load resistance within elastic hybrid structures. The reluctance to address the problem may be attributed to our limitations in being able to estimate reliably stiffness properties of soils. Moreover, soil stiffness is generally very different for static and dynamic loading. For the latter, frequency and amplitude are also parameters which affect soil response.

To gauge the sensitivity of hybrid structures of the type shown in figure 5(c) with respect to foundation compliance of the wall elements only, parametric studies were conducted (Goodsir 1985). The major variables in the structures chosen for analyses were:

1. Variation of wall restraint between the extreme limits of full rotational fixity and a hinge at the base.
2. Variation in the number of storeys in a building. Predominantly 6 and 12 storey structures were studied.
3. The relative contribution of walls to total lateral load resistance within the structure were varied. This was achieved with appropriate variation of wall lengths, ℓ_w, shown in figure 6 and the corresponding variation of the wall shear ratio, ψ, defined by equation (9).
4. Elastic response to code specified lateral static load was compared with the elasto-plastic dynamic response of the structure to the 1940 El Centro earthquake record.

Those aspects of the conclusions of this study, which are particularly relevant to the issues examined in this paper, are as follows:

(a) Above the first floor, the static response of the structure, when, using walls with moderate stiffness, is not significantly affected by the degree of base restraint. As a corollary, the stiffer a wall the more profound is the influence of foundation compliance.

(b) In pinned base walls, as expected, very large and reversed base shear forces are predicted by elastic analyses for

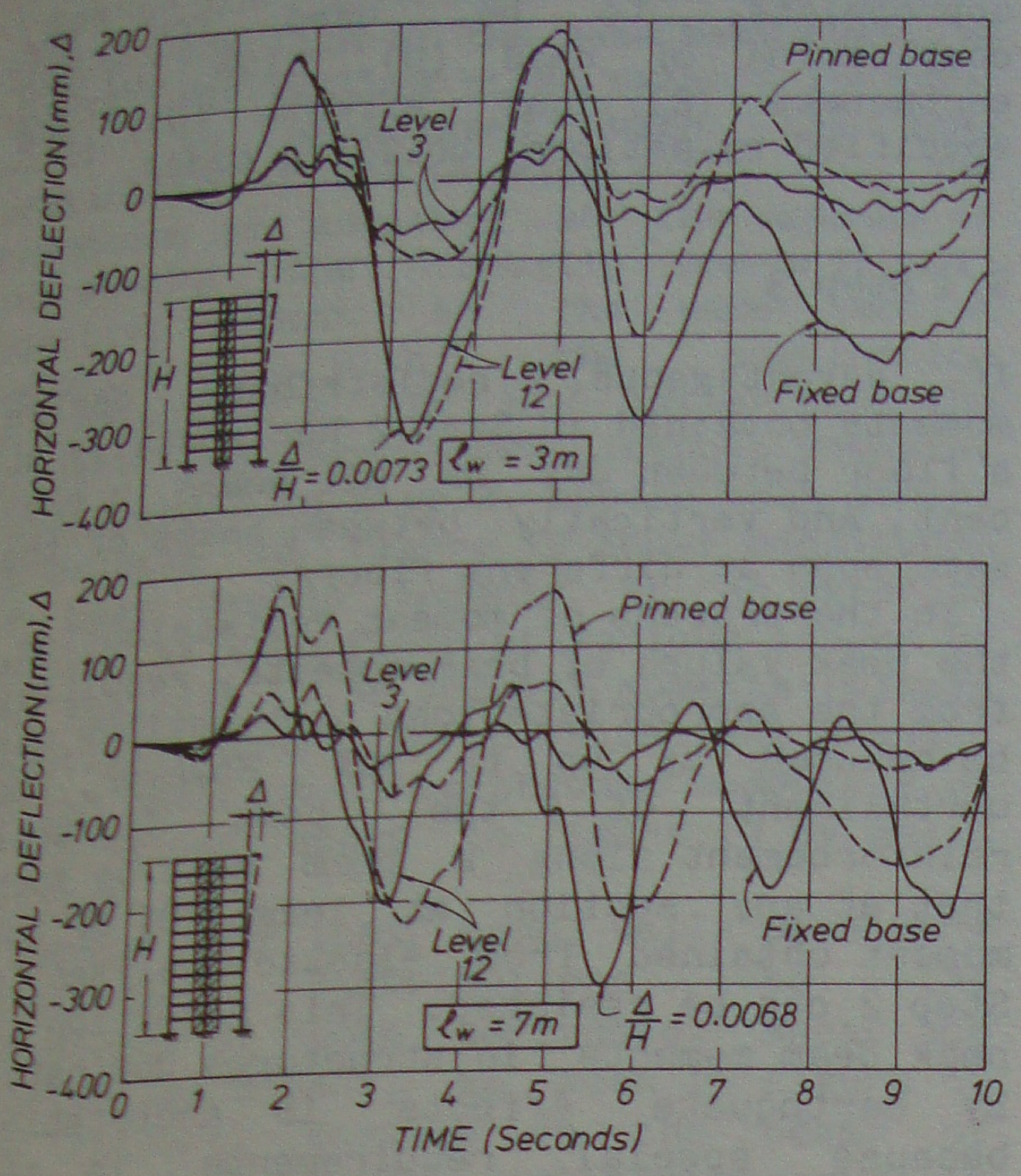


Fig.8 Horizontal deflection histories of 12 storey hybrid structures with fixed and pinned base walls and different stiffness (El Centro Record)

static lateral load. This points to the need for studying the transfer of these large forces to the diaphragm at first floor level. The wall shear reversals in the first storey necessitated dramatic increased in column shear forces in that storey. In the first storey the sum of the column shear forces exceed therefore the total static base shear for the entire structure.

(c) The single most important parameter affecting the seismic dynamic response of such hybrid structures was found to be the period shift brought about by the reduction of wall stiffnesses when complete loss of rotational rstraint at the base was assumed. This is demonstrated by figure 8 where the displacement histories of two example structures are shown. Displacements at the levels of the 3rd and topmost floors are shown only. It is seen that for the first 10 seconds of the record, the differences in displacement responses are negligible when 3 m long walls (ψ = 0.44) are used. The period change is, however, distinctly recognizable in the case when 7 m long walls (ψ = 0.80) were utilized. However, even in this case, the differences in terms of maximum displacements were insignificant.

(d) Extreme levels of shear forces, predicted by elastic analyses for columns and walls, did not eventuate during the time history analysis for the El Centro event.

(e) In the upper storeys important design quantities for the example hybrid structures, such as drifts, column and wall moments, and rotational ductility demands in plastic hinges of beams, were only insignificantly affected when walls were modelled with pinned bases.

(f) Full wall base fixity is normally assumed in design, although it is known to be generally unavailable. These parametric studies indicated, however, that errors due to quite significant relaxation in base restraint, are not likely to seriously affect elasto-plastic dynamic response of the superstructure.

The studies described above were conducted using a model as shown in figure 5(c). Because no axial load is introduced to the walls of this system during simulated earthquake, the set of springs allow for base rotation without any change in the vertical displacements along the axis of the wall. When soil deformations are involved and particularly when uplift can occur, as shown in figure 7(d), significantly increased plastic hinge rotations will be imposed on beams which are monolithically connected to the tension face of a wall. When a rocking wall is part of a two-way framing system, frames at right angles to the wall may well be mobilized. An example of this situation is shown in figure 9, where it is seen that the wall in the B plane will also activate transverse beams in plane 2. Bertero (1985), amongst others, reported on the significant strength enhancement resulting from these three dimensional effects in a test structure.

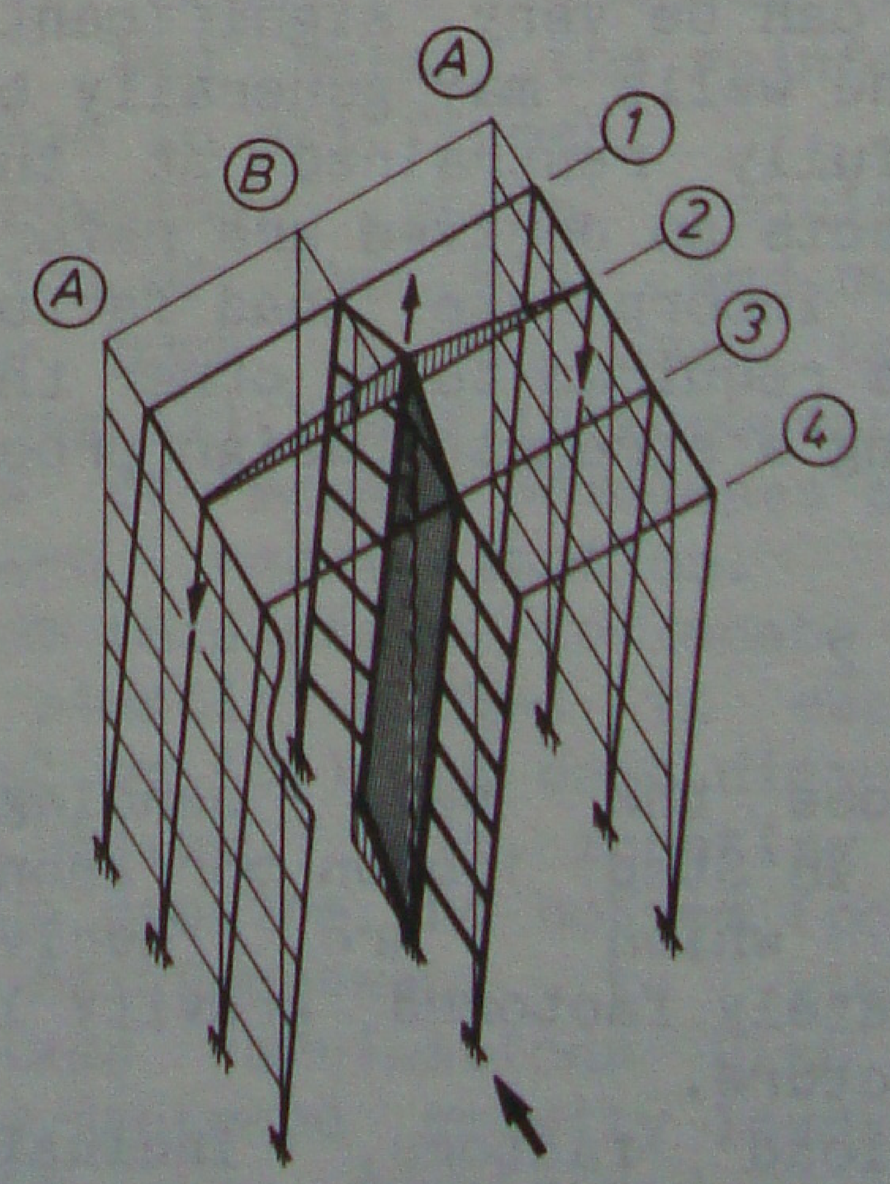


Fig.9 Activation of transverse frames by rocking walls

5 A CAPACITY DESIGN STRATEGY FOR HYBRID STRUCTURES

In the following sections a capacity design approach for hybrid structures is described in a step-by-step manner. The presentation follows the pattern of, and is similar to, the design procedure suggested by Paulay (1980a) and recommended by the Standards Association of New Zealand (1982) for reinforced concrete ductile frames. Where necessary, the presentation of a design step is followed by comments, sometimes extensive. These are relevant to the purpose of and intend to explain the justification for that particular step. Frequent reference is made to figure 6, which shows a prototype frame-wall structure.

The procedure outlined is relevant to the types of structures shown in figures 7(a) and (b). In these structures columns in upper storeys are intended to be protected against significant plastic deformations. Thereby various concessions with respect to their detailing for ductilty, as outlined in Section 3.2, may be utilised.

5.1 Step 1

Derive the bending moments and shear forces for all members of the frame-shear wall system subjected to the code specified equivalent lateral static earthquake load only. These actions are subscripted "code".

In the analysis for the elastically responding structure, due allowance should be made for the effects of cracking on the stiffness of both frame members and walls. Because walls are often lightly reinforced and carry relatively small gravity loads, the reduction of their stiffness due to cracking can be very significant. Both frames and walls may generally be assumed to be fully restrained at their base. Load effects so derived are referred to as E. They incorporate load factors, where these are required to be other than unity, when using a strength design procedure.

5.2 Step 2

Superimpose the beam bending moments obtained in Step 1 upon corresonding beam moments which are derived for appropriately factored gravity loading on the structure.

The load factors, inclusive load combination and importance factors where appropriate, to be used for the combination of dead (D), live (L) and earthquake (E) load effects (U), are specified by national building codes.

5.3 Step 3

If advantageous, redistribute design moments obtained in Step 2 horizontally at a floor between any or all beams in each bent, and vertically between beams of the same span at different floors.

In the process of moment redistribution, the peak values of beam moments, resulting from the appropriate load combinations may be reduced by up to 30% However, the curtailment of the beam flexural reinforcement along a beam must be such that at any section at least 70% of the moment obtained from elastic analyses in Step 2 can be resisted. This reduction in peak beam moments in structures dominated by earthquake effects is acceptable, because special requirements in the detailing for significant ductility also needs to be satisfied. Final moments obtained after redistribution should be checked to ensure that no loss in the total lateral load resistance of the structure results. Also load combinations for gravity load alone must be examined before the proportioning of beams commences.

The principles of redistribution of moments at a level among different spans of beams within frames are well established.

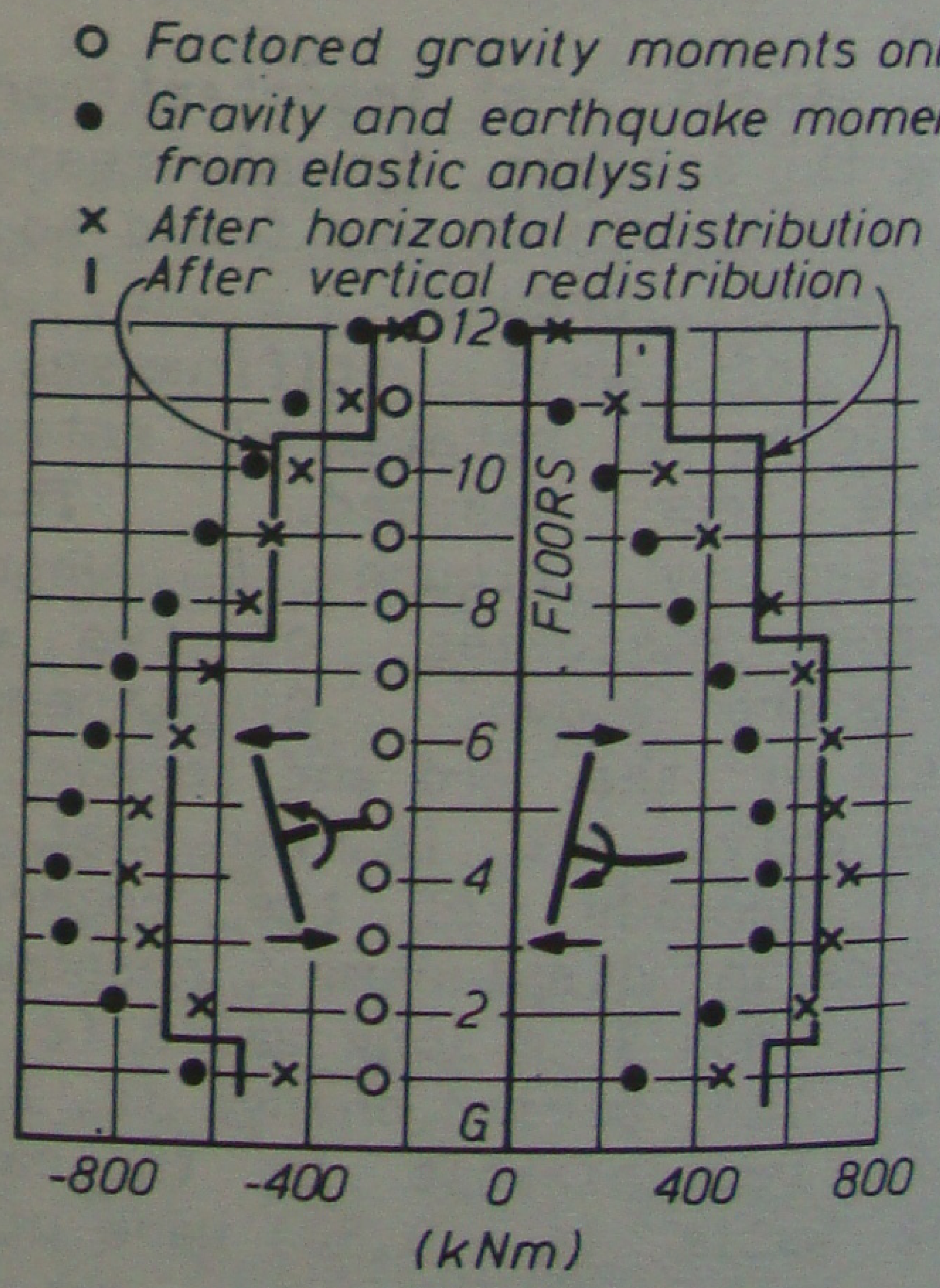


Fig.10 The redistribution of design moments among beams adjacent to an exterior column of a hybrid structure

One of the advantages which may result, is the reduction of the peak beam negative moment, for example at an exterior column, which is associated with the load combination $U = \text{Gravity} + \overleftarrow{E}$. The reduction is achieved at the expense of increasing the (usually non-critical) positive moment at the same section associated with the combination $U = \text{Gravity} + \overrightarrow{E}$. In the latter case the gravity and earthquake induced moments, superimposed in Step 2, oppose each other. An example in figure 10 shows magnitudes of beam design moments at an exterior columns at each floor at various stages of the analysis. The gravity moments (always negative), shown by circles, are changed by the addition of earthquake moments $\overleftarrow{E}$ or $\overrightarrow{E}$, to values shown in figure 10 by solid circles.

Because near their base, walls make very significant contributions to the resistance of both horizontal shear and overturning moment, as seen in figure 6, the flexural demands on the beams of hybrid structures are relatively small in the lower storeys.

In the example frames, the beam moments at the exterior column could be redistributed so as to result in magnitudes shown by crosses in figure 10. It is seen that the negative and positive design moments are now comparable in magnitudes.

To optimize the practicality of beam design, whereby beams of identical strength are preferred over the largest possible number of adjacent floors, some vertical redistribution of beam design moments should also be considered. In the example of figure 10, the design moments shown by crosses may be redistributed up and down the frames so as to result in magnitudes shown by the continuous stepped lines. It is seen that beams of the same flexural strength could be used over 6 floors. The stepped line has been chosen in such a way that the area enclosed by it is approximately the same as that within the curve formed by the crosses. This choice means that the contribution of the frames to the resistance of overturning moments is only insignificantly altered by vertical moment redistribution.

Horizontal redistribution of beam moments at a particular level will change the moment input to individual columns. Hence the shear demand across individual columns will also change with respect to that indicated by the elastic analysis used in Step 1. However, the total shear demand on columns of a bent must not change during the redistribution of design shear forces between columns.

When vertical redistribution of beam moments is carried out, the total moment input to some or all columns at a floor will also change. Hence the total shear demand on columns of a particular storey may decrease (the 5th storey in figure 10), while in other storeys (the 2nd storey in figure 10) it will increase. To ensure that there is no decrease in the total storey shear resistance intended by the code specified lateral loading, there must be a horizontal redistribution of shear forces between the vertical elements of the structure, i.e. the columns and walls. It will be shown subsequently that the upper regions of walls will be provided with sufficient shear and flexural reserve strength to accommmodate additional shear forces shed by upper storey columns. The principles involved in vertical load redistribution discussed here are similar to those used in the design of coupling beams of coupled structural walls (Paulay 1975).

The recommendation that the reduction of peak moments in beams, obtained from elastic analyses in Step 2, should not exceed 30%, results from the concern for the elastic response of the structure during moderate earthquakes. It will often be found that optimal solutions may be obtained with moment or shear redistribution considerably less than that associated with a reduction by 30% of the peak values. No limit need to be placed on redistributable actions which increase the demand on an element assigned to it by elastic analyses.

5.4 Step 4

Design all critical beam sections so as to provide the required factored flexural resistance and detail the reinforcement for all beams in all frames.

These routine steps require the determination of the size and number of reinforcing bars to be used to resist moments along all beams in accordance with the demands of moment envelopes obtained after moment redistribution. It is important at this stage to locate the two potential plastic hinges in each span (figure 7(a) or (b)) for each direction of earthquake attack. In locating plastic hinges which require the bottom (positive) flexural reinforcement to yield in tension, load combinations with minimum and maximum factored gravity loads on the span should be considered, as each combination may indicate a different hinge position. Detailing of the beams should then be carried out in conformity with relevant code provisions.

5.5 Step 5

In each beam determine the flexural overstrength of each of the two potential plastic hinges corresponding with each of the two directions of earthquake attack.

In estimating the maximum likely flexural strength which could be developed at critical beam sections during a large inelastic seismic pulse, the maximum strength of the reinforcement, including contributions due to strain hardening, should be considered without the use of resistance factors. Moreover, it is important to include all reinforcement which could contribute to internal tension forces. This normally involves an estimate of the affective width of T beams in tension. The contribution of slab reinforcement, placed parallel to the beam under consideration, to the enhancement of the flexural resistance can be significant. By necessity the allowance for this slab contribution will be very approximate, for it depends on the plastic rotational demands imposed on the beam during an earthquake. Appropriate recommendations have been formulated by the Standards Association of New Zealand (1982).

The primary aims in the estimate of the maximum moments which may be developed in a beam are, to establish an upper bound estimate of the shear demand in each span, and to allow sufficient reserve flexural resistance in adjacent columns to be provided. Thereby brittle shear failures will be suppressed and the development of a "strong-column weak-beam" system assured.

5.6 Step 6

Determine the lateral displacement induced shear force, V_{oe}, associated with the development of flexural overstrength at the two plastic hinges in each beam span for each direction of earthquake attack.

These shear forces are readily obtained from the flexural overstrengths of potential plastic hinges, determined in Step 5, which were located in Step 4. When combined with gravity induced shear forces, the design shear envelope for each beam span is obtained, and the required shear reinforcement can then be determined. The displacement induced maximum beam shear forces, V_{oe}, are used subsequently to determine the maximum lateral displacement induced axial column load input at each floor.

5.7 Step 7

Determine the beam flexural overstrength factor, ϕ_o, at the centreline of each column at each floor for both directions of earthquake attack. Fixed values of ϕ_o are:

(a) At ground level $\phi_o = 1.4$
(b) At roof level $\phi_o = 1.1$

This factor is subsequently used to estimate the maximum moment which could be introduced to columns by fully plastified beams. The beam overstrength factor, ϕ_o, at a column, is the ratio of the sum of the flexural overstrengths developed by adjacent beams, as detailed, to the sum of the flexural strengths required in the given direction by the code specified lateral earthquake loading alone, both sets of values being taken at the centreline of the relevant column.

The beam moments at column centrelines can be readily obtained graphically from the design bending moment envelopes or otherwise, after the flexural overstrength moments at the exact locations of the two plastic hinges along the beam span have been plotted.

The relevance of the beam flexural overstrength factor, ϕ_o, to column design was briefly reviewed in Section 3.1 and its use was shown in figure 3.

5.8 Step 8

Evaluate the column design shear forces in each storey from

$$V_{col} = \omega_c \phi_o V_{code} \quad (10)$$

where the column dynamic shear magnification factor, ω_c, is 2.5, 1.3 and 2.0 for the bottom, intermediate and top storeys respectively. The design shear force in the bottom storey columns should not be less than

$$V_{col} = \frac{(M^o_{col} + 1.3\phi_o M_{code})}{(\ell_n + 0.5h_b)} \quad (11)$$

where M^o_{col} = the flexural overstrength of the column base section consistent with the axial load and shear which are associated with the directions of earthquake attack.

$M_{code,top}$ = the value of M_{code} for the column at the centreline of the first floor beams derived from the appropriately factored lateral earthquake load in Step 1.

ℓ_n = the clear height of the column.

h_b = the depth of the first floor beam.

The procedure for the evaluation of column design shear forces is very similar to that used in the capacity design of ductile frames. It reflects a higher degree of conservation because of the intent to avoid a column shear failure in any event. Case studies show that in spite of the apparent severity of equations (10) and (11), shear requirements very seldom govern the amount of transverse reinforcement to be used in columns.

Dynamic analyses of example hybrid structures indicated that shear forces induced in the bottom and top storey columns may exceed by a large margin the magnitudes predicted by elastic (Step 1) analyses, V_{code}. It may be noted, however, that in hybrid structures, as figure 6 suggests, the computed static column shear forces, V_{code}, are generally small in these two specific storeys.

Figure 11 compares the column design shear forces so derived with shear demands analytically predicted for the interior columns of the structure shown in figures 6(a) and (b) during the first ten seconds of the 1940 El Centro event. Walls, both with fixed and pinned base, with 3 and 7 metres lengths were considered.

5.9 Step 9

Estimate in each storey the maximum likely lateral displacement induced axial load on each column from

$$P_{eq} = R_v \Sigma V_{oe} \quad (12)$$

where

$$R_v = (1 - n/67) \geq 0.7 \quad (13)$$

is a reduction factor which takes the number of floors, n, above the storey under consideration, into account.

The magnitudes of the maximum lateral displacement induced beam shear forces, V_{oe}, at each floor, were obtained in Step 6. The probability of all beams above a particular level developing simultaneously plastic hinges at flexural overstrength diminishes with the number of floors above that level. The reduction factor, R_v, makes an approximate allowance for this.

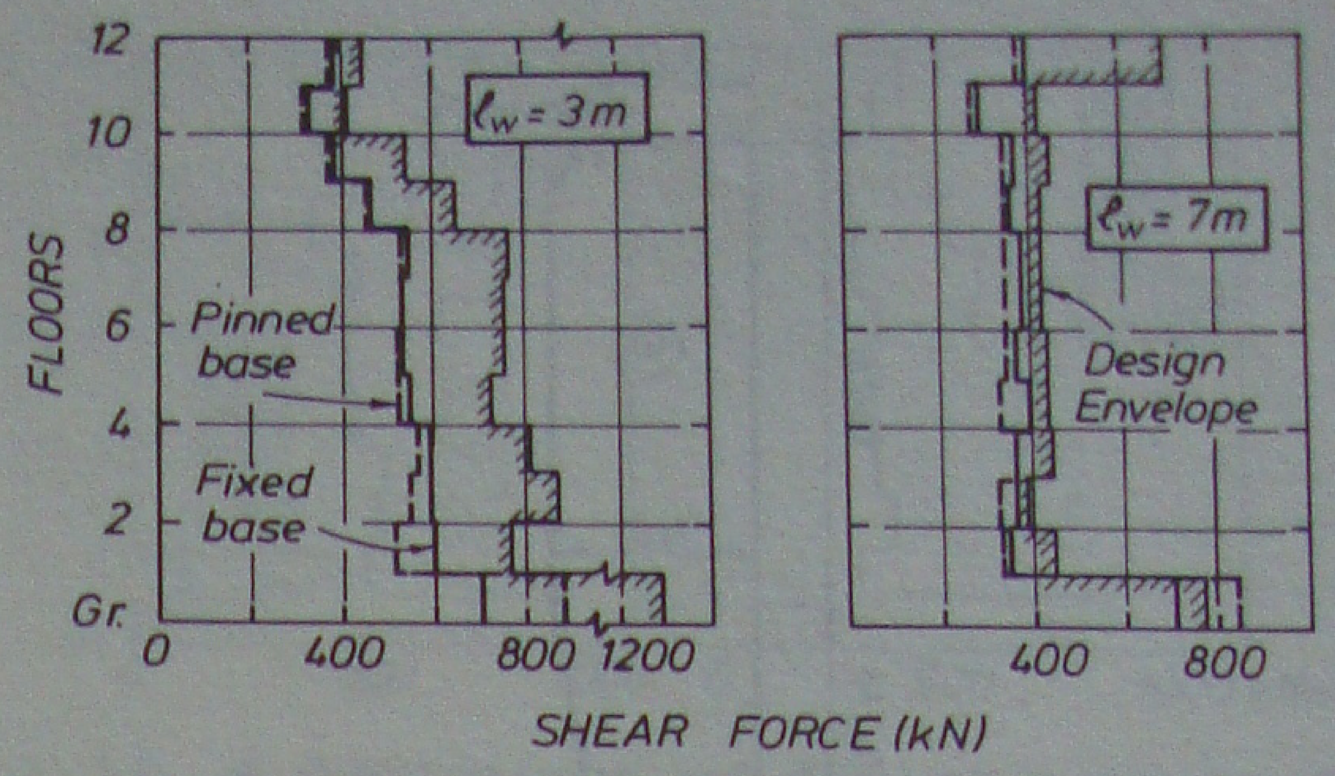


Fig.11 Maximum shear forces encountered across interior columns of a 12 storey hybrid structure during the El Centro event

5.10 Step 10

Determine the total design axial load on each column for each of the two directions of earthquake attack from

$$P_{e,max} = P_D + P_{LR} + P_{eq} \quad (14)$$

and

$$P_{e,min} = 0.9P_D - P_{eq} \quad (15)$$

where P_D and P_{LR} are axial forces due to dead and reduced live loads respectively.

The reduced live load incorporates an allowance for the combination of increasing aggregate tributary floor areas, in accordance with relevant code requirements. Because in capacity design extreme seismic load demands, rather than those derived from the analysis of Step 1, are used, the use of unfactored gravity forces, such as P_D or P_{LR}, are considered to be appropriate when simulating an extreme but realistic seismic event.

5.11 Step 11

Obtain the design moments for columns above and below each floor from

$$M_{col} = R_m(\omega\phi_o M_{code} - 0.3h_b V_{col}) \quad (16)$$

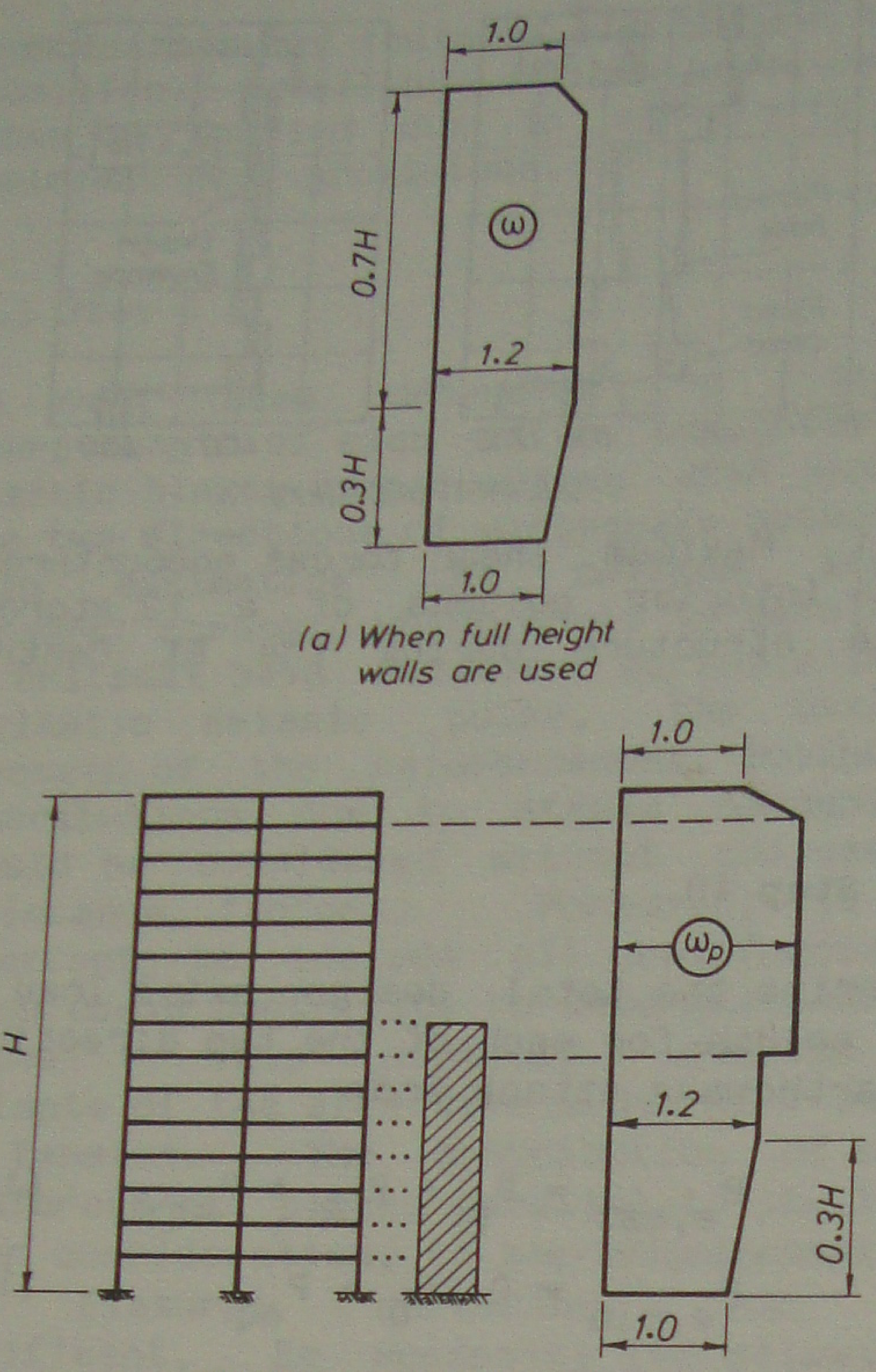
where ω = the dynamic moment magnification factor, the value of which is given in figure 12.

ϕ_o = the beam overstrength factor applicable to the floor and the direction of lateral loading under consideration, determined in Step 7

h_b = the depth of the beam which frames into the column

and

$$R_m = 1 + 0.55(\omega - 1)(10 \frac{P_e}{f'_c A_g} - 1) \leq 1 \quad (17)$$



(a) When full height walls are used

(b) When partial height walls are used

Fig.12 Dynamic moment magnification factors for columns of hybrid structures

is a design moment reduction factor applicable when

$$-0.15 \le \frac{P_e}{f'_c A_g} \le 0.10$$

where P_e (N) is to be taken negative when causing axial tension and where f'_c = specified compression strength of the concrete (MPa) and A_g = gross concrete sectional area of the column (mm^2).

These requirements are very similar to those recommended for columns of ductile frames and summarized in figure 3.

Stages in the derivation of the column design moment, M_{col}, are further illustrated in figure 13. The variation of column moments due to code loading (Step 1), M_{code}, above and below a floor is shown with shaded lines. These moments are magnified throughout the height of the column to $\phi_o M_{code}$, when beams adjacent to the column develop flexural overstrengths at their plastic hinges. It is assumed that the maximum moment input from the two beams, ΣM_{beam}, as shown in figure 13, cannot be exceeded during an earthquake. However, the distribution of this total moment imput between the columns above and below the floor, during the dynamic response, is uncertain. Allowance for disproportionate distribution is made by the dynamic magnification factor $\omega \le 1.2$. For example the estimated maximum moment for the upper column in figure 13, measured at the beam centreline, is thus $\omega\phi_o M_{code}$.

At the top of the beam, at the critical section of this column, the moment is less. The reduction depends on the magnitude of the column shear force generated simultaneously. For this purpose a conservative assumption is made, whereby $V_{min} = 0.6V_{max} = 0.6V_{col}$ (Step 8). Hence the moment reduction at the top of the beam becomes $0.5h_b V_{min} = 0.3h_b V_{col}$ as shown in figure 13.

Because the deformed shape of the building during its response to a large earthquake is largely controlled by the behaviour of the walls, in hybrid systems the higher modes of vibration have relatively little effect on the pattern of column moments. For this reason it was found that a magnification of column design moments by only 20% is sufficient. This is considerably less than values which are applicable to columns of ductile frames, quoted in Section 3.1 and shown in realistic proportions in figure 3.

When the axial load on the column produces small compression, i.e. $P_e \le 0.1f'_c A_g$, or results in net axial tension, some yielding of the column is acceptable. Such columns should exhibit

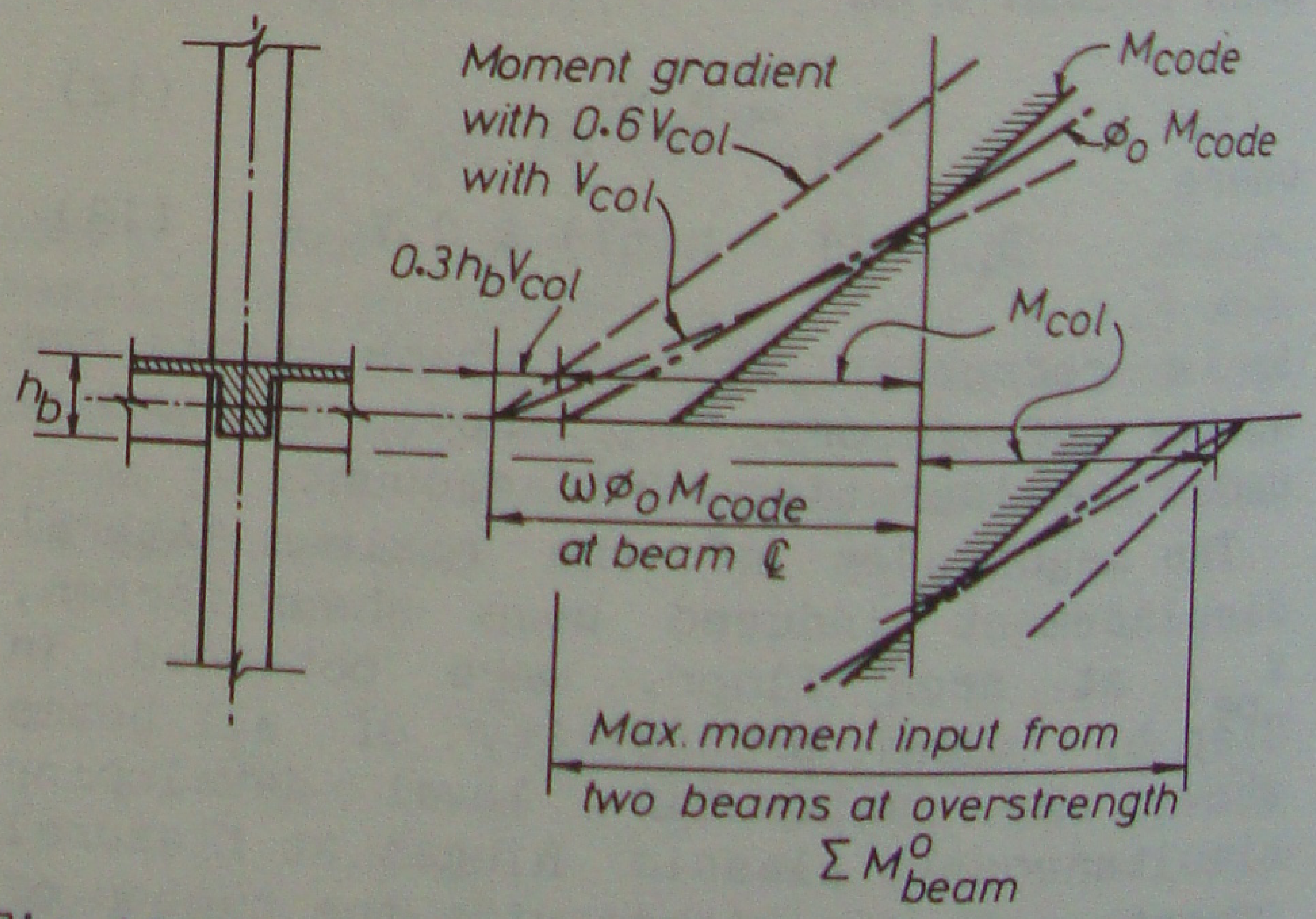


Fig.13 The derivation of design moments for columns of hybrid structures

sufficient ductility even without special confining reinforcement in the end regions. Hence for this situation the design moments may be reduced by the factor R_m given in equation (17). The minimum value of R_m is 0.72. This will enable the amount of required tension reinforcement in exterior columns, where this situation arises, to be reduced. This reduction of column design moments will seldom exceed 20%. To simplify computations, the designer may prefer to use $R_m = 1.0$.

When the reduction factor, R_m, relevant to the given direction of earthquake attack is used in determining the amount of column reinforcement, the corresponding design shear V_{col}, obtained in Step 8, may also be reduced proportionally.

Having obtained the critical design quantities for each column, i.e. M_{col} from Step 11 and V_{col} from Step 8, the required flexural and shear reinforcement at each critical section can be found. Because the design quantities have been derived from beam overstrengths imput, the appropriate strength reduction or resistance factor for these columns is $\phi = 1.0$. End regions of columns need further be checked to ensure that the transverse reinforcement provided satisfies the code requirements for confinement, stability of vertical reinforcing bars and lapped splices.

The design of columns at the base, where the development of a plastic hinge in each column must be expected, is the same as for columns of ductile frames. No dynamic magnification is involved this level.

A study of hybrid structures with partial height walls, of the type shown in Fig. 5(b), has indicated that column moments over the vertical extent of the walls remain essentially the same irrespective of the total height of the walls used. This applies to both moments due to elastic response to lateral static load and due to elasto-plastic dynamic response to the El Centro record. Moreover, it was also found that column moments in the upper storeys, where no walls are present, revert closely to moments in the same storeys predicted for frames when no walls of any kind were present. The conclusions lead to the recommendation of using moment magnification factors, ω, for columns in structures with partial height walls, as shown in the right hand side of figure 12.

In figure 14 the probable flexural strengths of exterior columns, obtained with the procedure outlined in this design step, are compared with moment demands

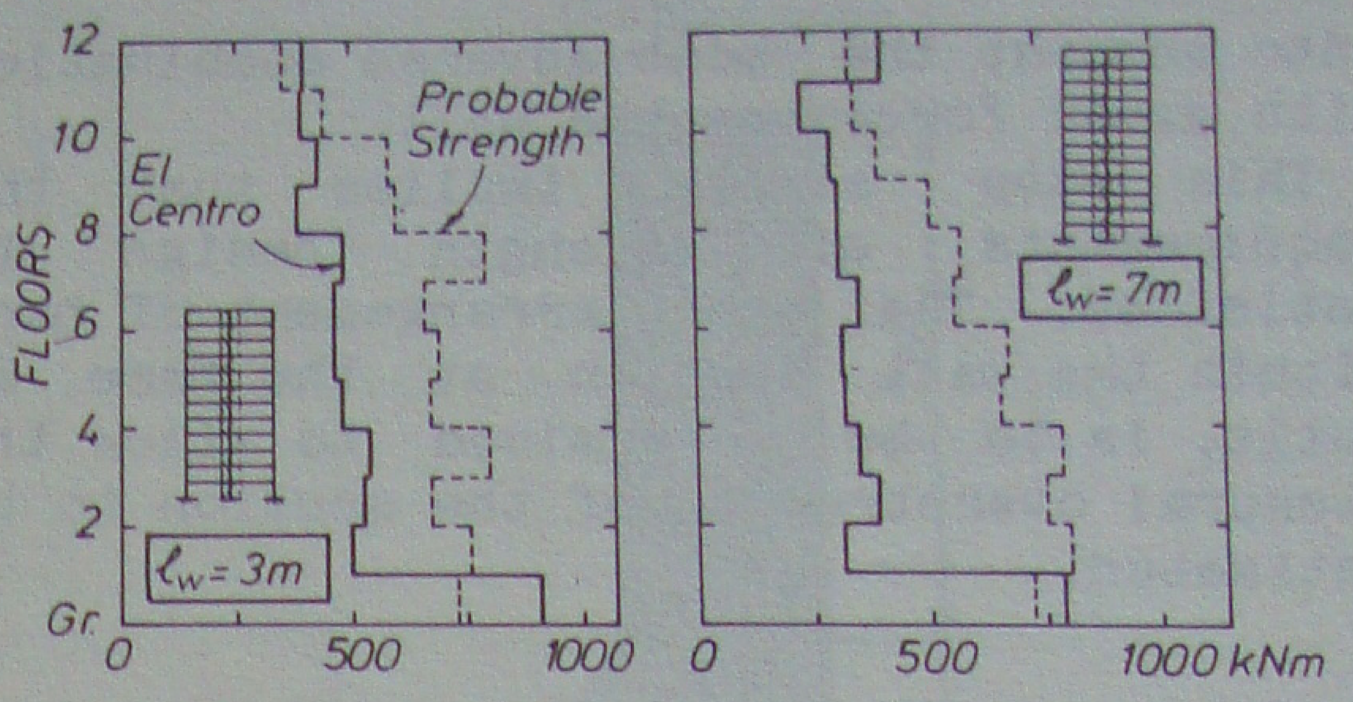


Fig.14 A comparison of design moments for an exterior column with moment demands during the El Centro event

analytically predicted for the El Centro event. It is seen that considerable reserve strength, assuring fully elastic response of these columns, is available in all but the bottom of top storeys, where columns were expected to develop plastic hinges. It is emphasized that the amount of column reinforcement in these cases is generally dictated by minimum code rather than strength requirements.

5.12 Step 12

Determine the appropriate gravity and earthquake induced axial forces on walls.

For the example structure shown in figure 6, it was implicitly assumed that lateral load on the building does not introduce axial forces to the cantilever walls. For this situation the design axial forces on the walls are to be based on the appropriately factored gravity loads only. The compression stresses resulting from these are generally small. For this reason proper load factors must be used to ensure that the beneficial effects of gravity load on the factored flexural resistance of wall sections are not overestimated.

If walls are connected to columns via rigidily connected beams, as shown for example in figure 5(a), the lateral load induced axial forces on the walls are obtained from the initial elastic analysis of the structure (Step 1). Similarly this applies when, instead of cantilever walls, coupled structural walls share with frames in lateral load resistance.

5.13 Step 13

Determine the maximum bending moment at the base of each wall and determine the necessary flexural reinforcement, taking into account the most adverse combination with axial forces on the wall.

This step simply implies that the requirements of strength design be satisfied. The exact arrangement of bars within the wall section at the base, as built, is to be determined to allow the flexural overstrength of the section to be estimated.

5.14 Step 14

When curtailing the vertical reinforcement in the upper storeys of walls, provide flexural resistance not less than given by the moment envelope in figure 15.

The envelope shown is similar to but not the same as that recommended in the New Zealand and Canadian codes for concrete structures for cantilever walls. It specifies slightly larger flexural resistance in the top storeys. Its construction from the initial moment diagram, obtained from the elastic analysis in Step 1, may be readily followed in figure 15. It is important to note that the envelope is related to the ideal flexural strength of a wall at its base, as built, rather than the moment required at that section by the analysis for lateral load. The envelope refers to effective ideal flexural strength. Hence vertical bars in the wall must extend by at least full development length beyond levels indicated by the envelope.

The aim of this apparently conservative approach is to ensure that significant yielding will not occur beyond the assumed height, ℓ_w, of the plastic hinge at the base. Thereby the shear resistance of the wall in the upper storeys is also increased, and hence reduced amounts of horizontal shear reinforcement may be used.

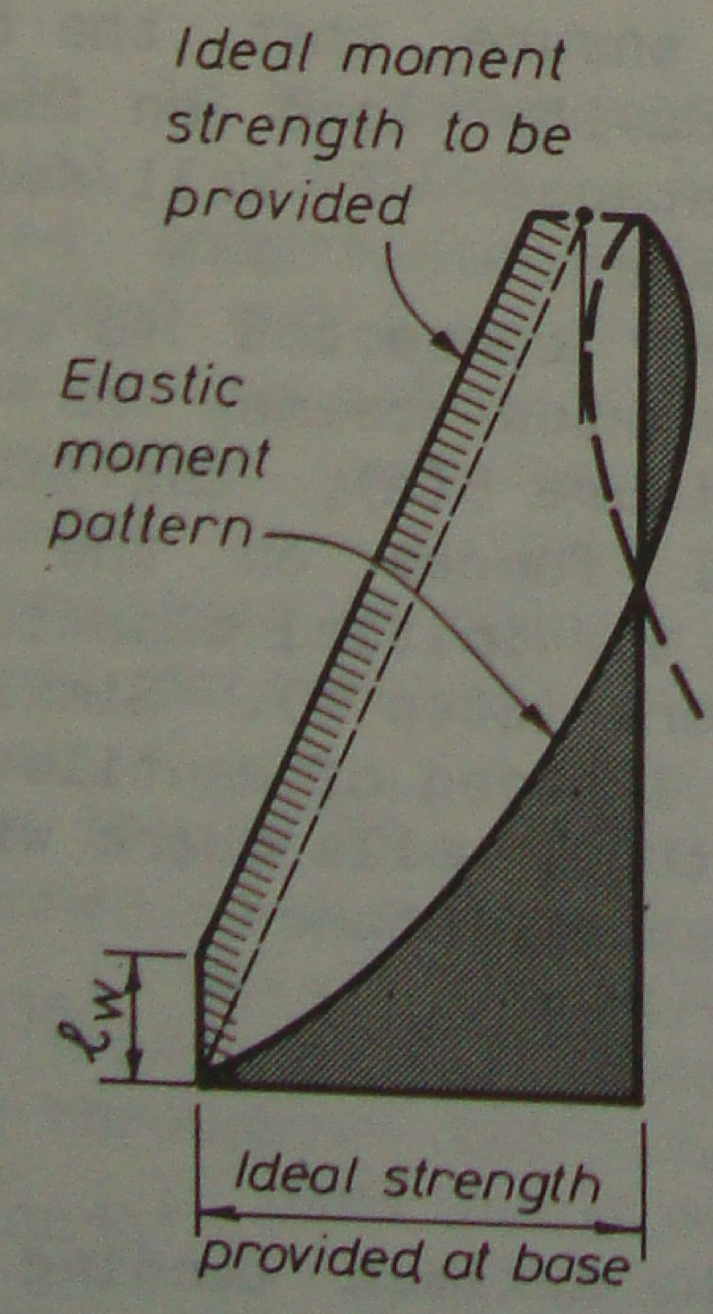


Fig.15 Design moments envelopes for walls of hybrid structures

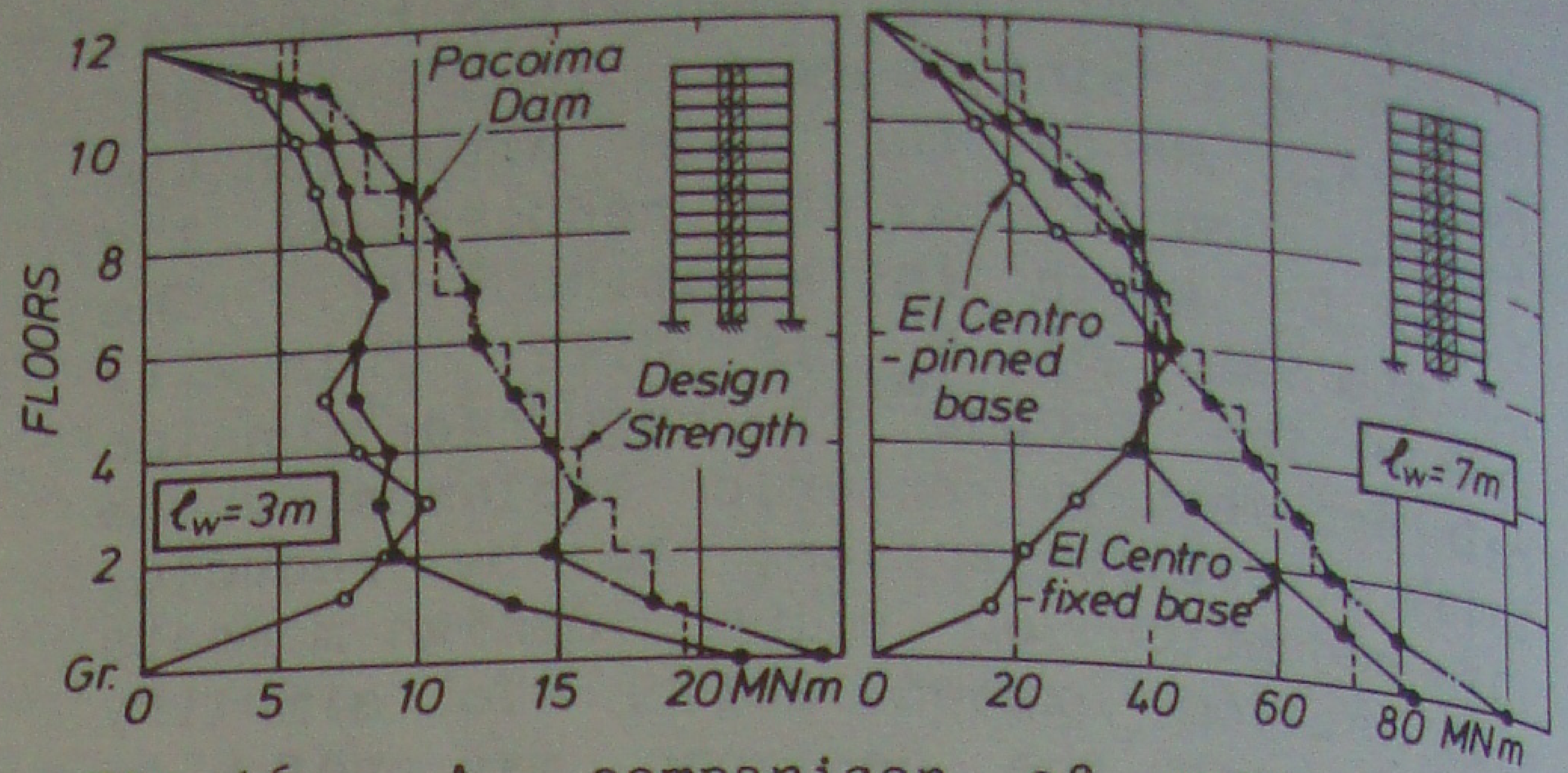


Fig.16 A comparison of wall design moments with those encountered during earthquake records

Figure 16 compares wall moment demands, encountered during the analysis for the 1940 El Centro and the 1971 Pacoima Dam earthquake records, with the moment envelopes given in figure 15. Two 12 storey buildings, in plan as shown in figure 6(a), with walls of 3.0, and 7.0 metres length, were studied. All buildings were designed in accordance with this capacity design procedure. While the envelopes appear to provide considerable reserve flexural strength in the upper storeys during the El Centro record, at various instants of the extreme (and unrealistic) Pacoima Dam event, the analysis predicted the attainment of the ideal flexural strength in most storeys. Analyses showed, however, that curvature ductility demands, even during this extreme event, were very small in the upper storeys. As part of the study of the effects of foundation compliance, discussed in Section 4.2, these structures, with pinned base walls, but otherwise identical with the prototype structures, were also analysed for the El Centro record. It is seen in figure 16, that wall moment demands for the El Centro event in the upper storeys are very similar to those experienced with fixed base walls.

5.15 Step 15

Determine the magnitude of the flexural overstrength factor $\phi_{o,w}$, for each wall. This is the ratio of the flexural overstrength of the wall, M^o, as detailed, to the moment required to resist the code specified lateral loading, M_{code}; both moments taken at the base section of a wall.

The meaning and purpose of this factor, $\phi_{o,w} = (M^o/M_{code,base})$, as discussed in Section 3.1, is the same as that evaluated for beams in Step 7. Strictly, for walls there are two limiting values of overstrength, M^o, which could be considered. These are the moments developed in the presence of two different axial load intensities, i.e. $P_{e,max}$ and $P_{e,min}$. However, it is considered to be sufficient for the intended purpose to evaluate flexural overstrength developed with axial compression on cantilever walls due to unfactored dead load alone.

5.16 Step 16

Compute the wall shear ratio, ψ. This is the ratio of the sum of the shear forces at the base of all walls, $\Sigma V_{wall,code}$, predicted by the analysis for design load, to the total design base shear for the entire structure, $V_{code,total}$.

The relative contribution of all walls to the required total lateral load resistance is expressed, as a matter of convenience, by the shear ratio, discussed in Section 4.1.

$$\psi = \left(\sum_{i=1}^{n} V_{i,wall,code}/V_{code,total}\right)_{base} \qquad (9)$$

It applies strictly to the base of the structure. As figures 6(c) and (e) show, such a shear ratio would rapidly reduce with height, and near the top it could become negative. This indicates that the parameter ψ is not a unique measure to quantify the share of walls in the total lateral load resistance.

5.17 Step 17

Evaluate for each wall the design shear force at the base from

$$V_{wall,base} = \omega^*_v \phi_{o,w} V_{wall,code} \qquad (18)$$

and

$$\omega^*_v = 1 + (\omega_v - 1)\psi \qquad (19)$$

where ω_v is the dynamic shear magnification factor relevant to cantilever walls, given by equations (5) and (6) and discussed in Section 3.1.

The approach developed for the shear design of walls in hybrid structures is an extension of the two stage methodology used for cantilever walls (section 3.1) Goodsir (1985) found that for a given earthquake record, the dynamically induced

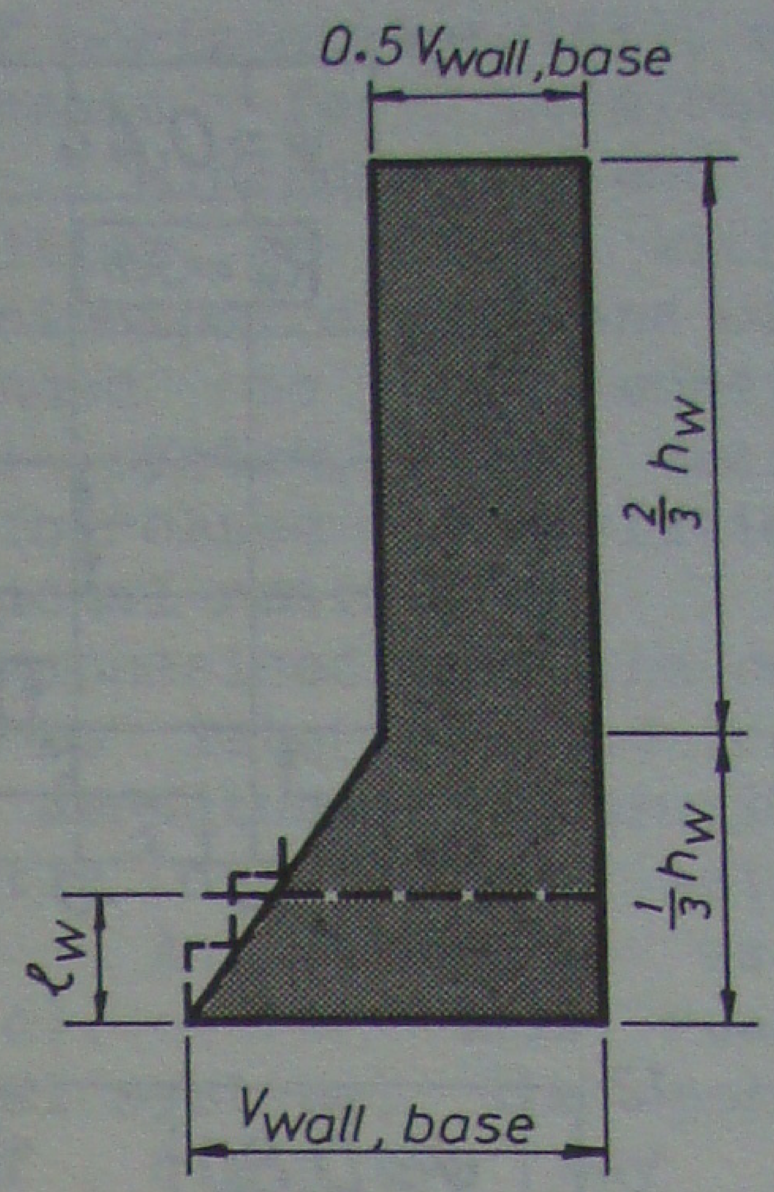


Fig.17 Envelope for design shear forces for walls of hybrid structures

base shear forces in walls of hybrid structures increased with an increased participation of such walls in the resistance of the total base shear for the entire structure. Wall participation is quantified by the "shear ratio", ψ, obtained in Step 16. The effect of the "shear ratio" upon the magnification of the maximum wall shear force is estimated by equation (19). It is seen that when $\psi = 1$, $\omega^*_v = \omega_v$.

Design criteria for shear strength will often be found to be critical. At the base the thickness of walls may need to be increased on account of equation (18), and because of maximum shear stress limitations imposed by codes.

5.18 Step 18

In each storey of each wall, provide shear resistance not less than that given by the shear design envelope of figure 17.

As figures 6(c) and (e) show, demands predicted by analyses for static load may be quite small in the upper half of walls. As can be expected, during the response of the building to vigorous seismic excitations, much larger shear forces may be generated at these upper levels. A linear scaling up of the shear force diagram drawn for static load, in accordance with equation (18), would give erronous predictions of shear demands in the upper storeys. Therefore from case studies, the shear design envelope shown in figure 17 was developed. It is seen that the envelope gives the required shear strength in terms of the base shear for

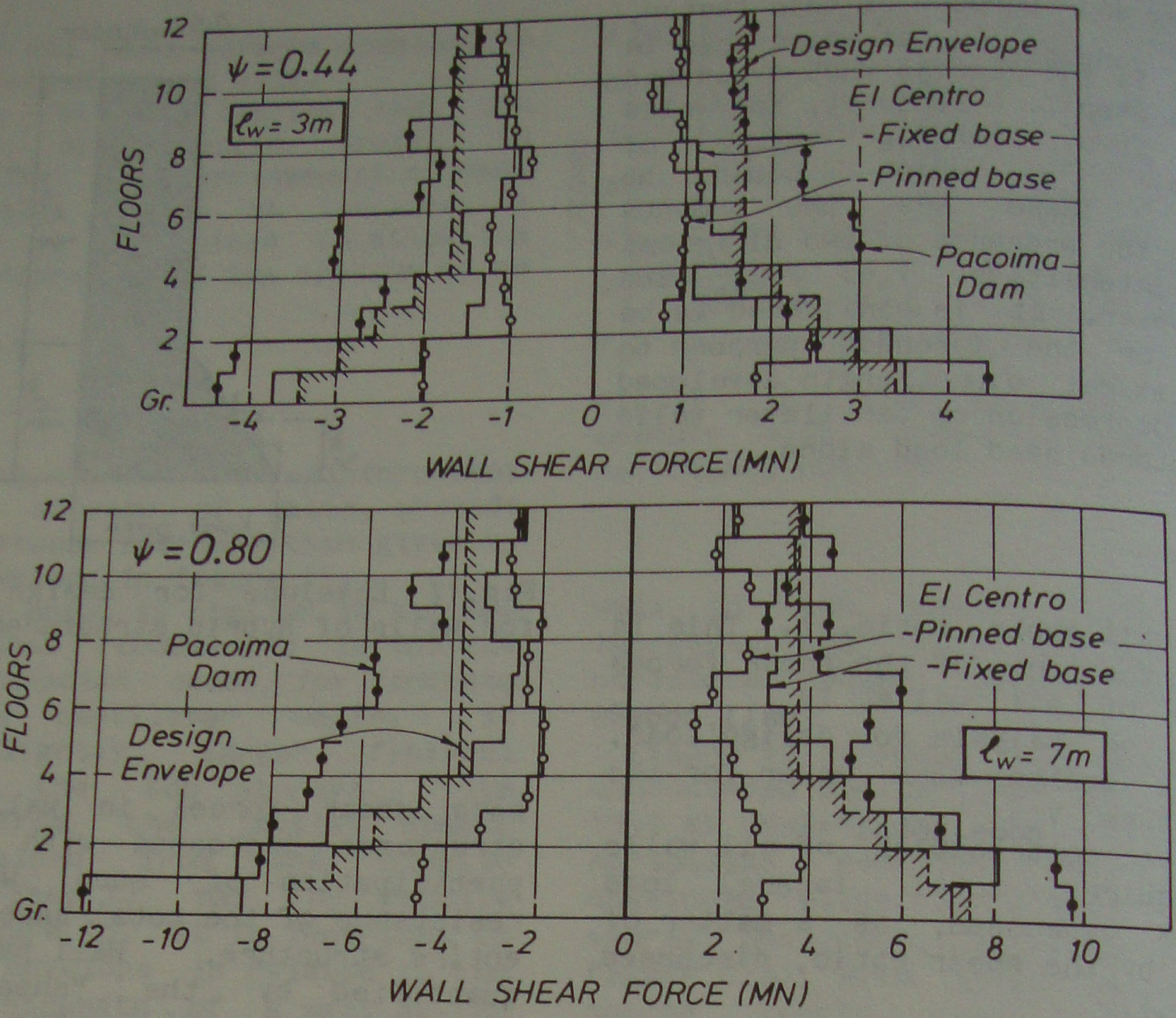


Fig.18 A comparison of the design wall shear envelope with wall shear demands in a 12 storey hybrid structure predicted for two seismic event

the wall, which was obtained in Step 17.

Figure 18 presents some results of the relevant study by Goodsir, Paulay and Carr (1983) of a 12 storey building. It is seen that the shear design envelope is satisfactory when structures with relatively slender walls, with $\psi = 0.44$, were subjected to the El Centro excitation. The shear response of the structure with 7 m walls is less satisfactory in the lower storeys. As may be expected, the predicted demand for shear in pin based walls is less, particularly as the length of the walls, ℓ_w, increases. It should be noted that the shear ratio, ψ, defined by equation (9) is not applicable to pinned base walls. Shear loads predicted for the Pacoima Dam seismic event were found to consistently exceed the suggested design values.

With the aid of the shear design envelope, the required amount of horizontal (shear) wall reinforcement at any level may be readily found. In this, attention must be paid to the different approaches used by codes to estimate the contribution of the concrete to shear strength, v_c, in the potential plastic hinge and the elastic regions of a wall. In the potential plastic hinge region, extending ℓ_w above the base, as shown in figures 15 and 17, the major part of, if not the entire design shear, V_{wall}, will need to be assigned to shear reinforcement. In the upper (elastic) parts of the wall, however, the concrete may be relied on to contribute significantly to shear resistance, allowing considerable reduction in the demand for shear reinforcement.

5.19 Step 19

In the end regions of each wall, over the assumed length of the potential plastic hinge, provide adequate transverse reinforcement to supply the required confinement to parts of the flexural compresssion zone and to prevent premature buckling of vertical bars.

These detailing requirements for ductility are the same as recommended for cantilever or coupled structural walls for example in the New Zealand and Canadian concrete codes.

6 ASPECTS REQUIRING FURTHER STUDY

The proposed capacity design procedure and the accompanying discussion of the behaviour of hybrid structures, presented in the previous section, are by necessity restricted to simple and regular structural systems. The variety of ways in which walls and frames may be combined may present problems to which a satisfactory solution will require, as in many other structures, the application of engineering judgement. This may necessitate some rational adjustments in the outlined step-by-step procedure. In the following, a few situations are mentioned where such judgement in the application of the proposed design methodology will be necessary. Some directions for promising approaches are also suggested.

6.1 Gross irregularities in the lateral load resisting system

It is generally recognised that the larger the departure from symmetry and regularity in the arrangement of lateral load resisting substructures within a building, the less confidence should the designer have in predicting likely seismic response. Examples of irregularity are when wall dimensions change drastically over the height of the building or when walls terminate at different heights, and when setbacks occur. Non-symmetrical positioning of walls in plan may lead to gross eccentricities of applied lateral load with respect to centres of rigidity.

6.2 Torsional effects

Codes make simple and rational provisions for torsional effects. The severity of torsion is commonly quantified by the distance between the centre of rigidity (or stiffness) of the lateral load resisting structural system and the centre of mass. In reasonably regular and symmetrical buildings this distance (horizontal eccentricity), does not significantly change from storey to storey. Errors due to inevitable variations of eccentricity over building height are thought to be compensated for by code specified amplifications of the computed (static) eccentricities. The corresponding assignment of additional lateral load to resisting elements, particularly those situated at greater distances from the centre of rigidity (centre of horizontal twist), are intended to compensate for torsional effects. Because minimum and maximum eccentricities, at least with respect to the two principal directions of earthquake attack, need to be considered, the structural system, as designed, will possess increased translational rather than torsional resistance.

It was emphasised that the contributions of walls to lateral load resistance in hybrid structures usually change dramatically over the height of the building. An example was shown in figure 6(c). For this reason in non-symmetrical systems, the position of the centre of rigidity may also change significantly from floor to floor.

For the purpose of illustrating the variation of eccentricity with height, consider the example structure shown in figure 6(a), but slightly modified. Because of symmetry, torsion due to variation in the position of the centre of rigidity, does not arise. Assume, however, that instead of the two symmetrically positioned walls, shown in figure 6(a), two 6 m long walls are placed side-by-side at 9.2 m from the left hand end of the building, as shown in figure 19, and that the right hand wall is replaced by a standard frame. Because the two walls, when displaced laterally by the same amount as the frames, would in this example structure resist 74% of the total shear in the first storey, the centre of rigidity would be 19.5 m from the centre of the (mass) building. In the 8th storey the two walls become rather ineffective,

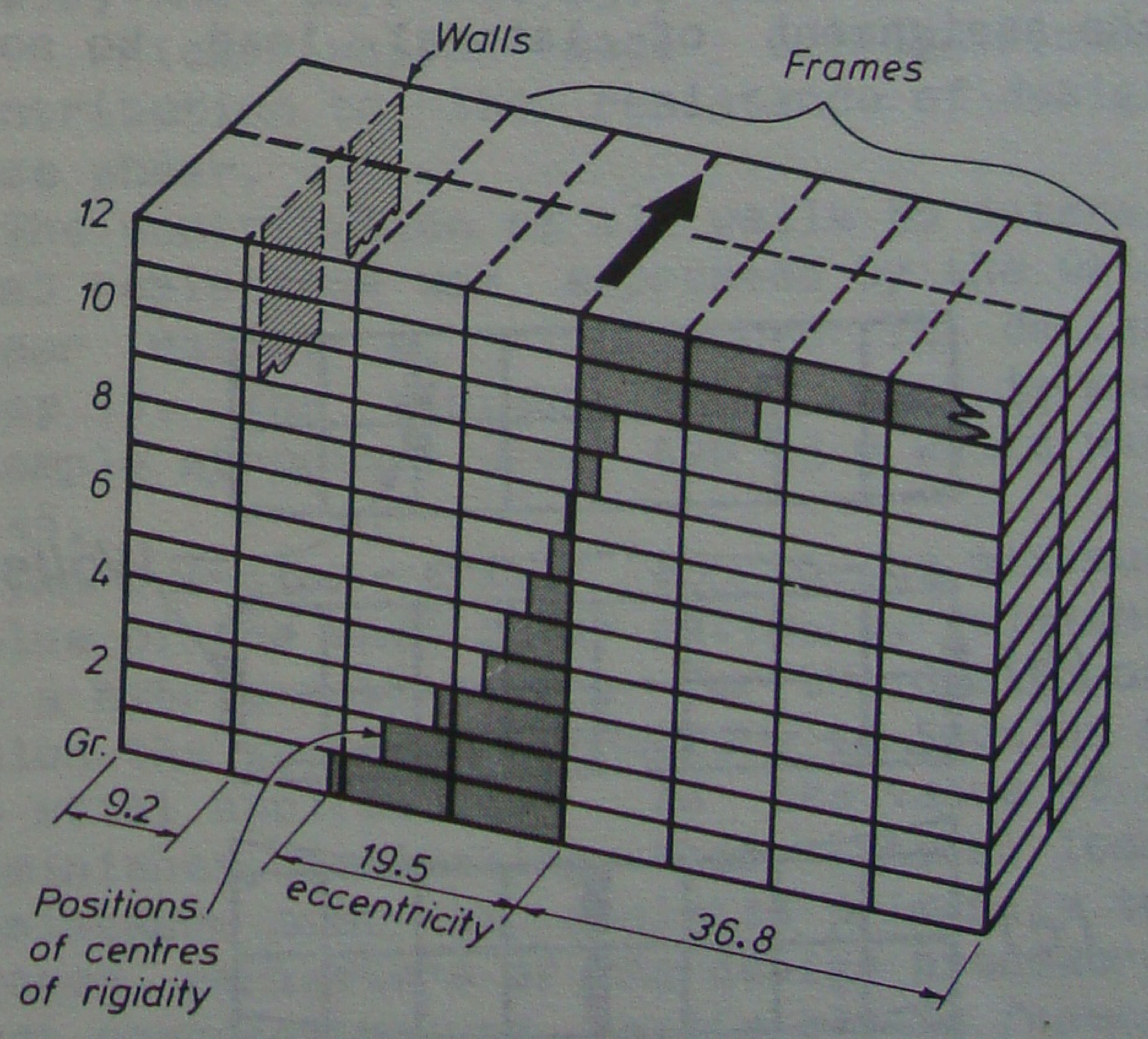


Fig.19 The variation of computed torsional eccentricities in an unsymmetrical 12 storey hybrid structure

as they resist only about 12% of the storey shear i.e. approximately as much as one frame. At this level the eccentricity becomes negligible. As figure 19 shows, the computed static eccentricities would vary considerably in this example building between limits at the bottom and top storey. Note also the different senses. Torsional effects on individual columns and walls will depend on the total torsional resistance of the system, including the periphery frames along the long sides of the building.

6.3 Diaphragm flexibility

For most buildings, floor deformations associated with diaphragm actions are negligible. However, when structural walls resist a major fraction of the seismically induced inertia forces in long and narrow buildings, the effects of inplane floor deformations upon the distribution of resistance to frames and walls may need to be examined.

Figure 20 shows plans of a building with three different positions of identical walls. The building is similar to that shown in figure 6(a). The contribution of the two walls to total lateral load resistance is assumed to be the same in each of these three cases. Diaphragm deformations associated with each case are shown approximately to scale by the dashed lines. Diaphragm deformations in the case of figure 20(a) would be negligibly small in comparison with those of the other two cases. In deciding whether such deformations are significant, the following aspects might be considered:

(a) If elastic response is considered, the assignment of lateral load to some frames (figures 20(b) and (c)) would be clearly underestimated if diaphragms were to be assumed to be infinitely rigid. Inplane deformations of floors, even when derived with crude approximations, should be compared with interstorey drifts predicted by standard elastic analyses. Such a comparison will then indicate the relative importance of diaphragm flexibility.

(b) In ductile structures, significant inelastic storey drifts are to be expected. The larger the inelastic deformations the less important are differential elastic displacements between frames which would result from diaphragm deformations.

(c) As figure 6(c) illustrated, the contribution of walls to lateral load resistance in hybrid structures diminishes with the distance measured from the base. Therefore at upper floors, lateral load will be more evenly distributed among the walls and identical frames. This will greatly reduce diaphragm inplane shear and flexural actions. Hence diaphragm deformations at upper levels would diminish.

(d) Horizontal inertia forces are expected to increase with the distance from the base, while inplane bending and shear effects will diminish because of the decreasing participation of walls at upper floors. Hence it may be concluded that diaphragm flexibility is of lesser importance in hybrid structures of the type shown in figure 20, than in buildings where lateral load resistance is provided entirely by cantilever walls i.e. without the participation of any frames.

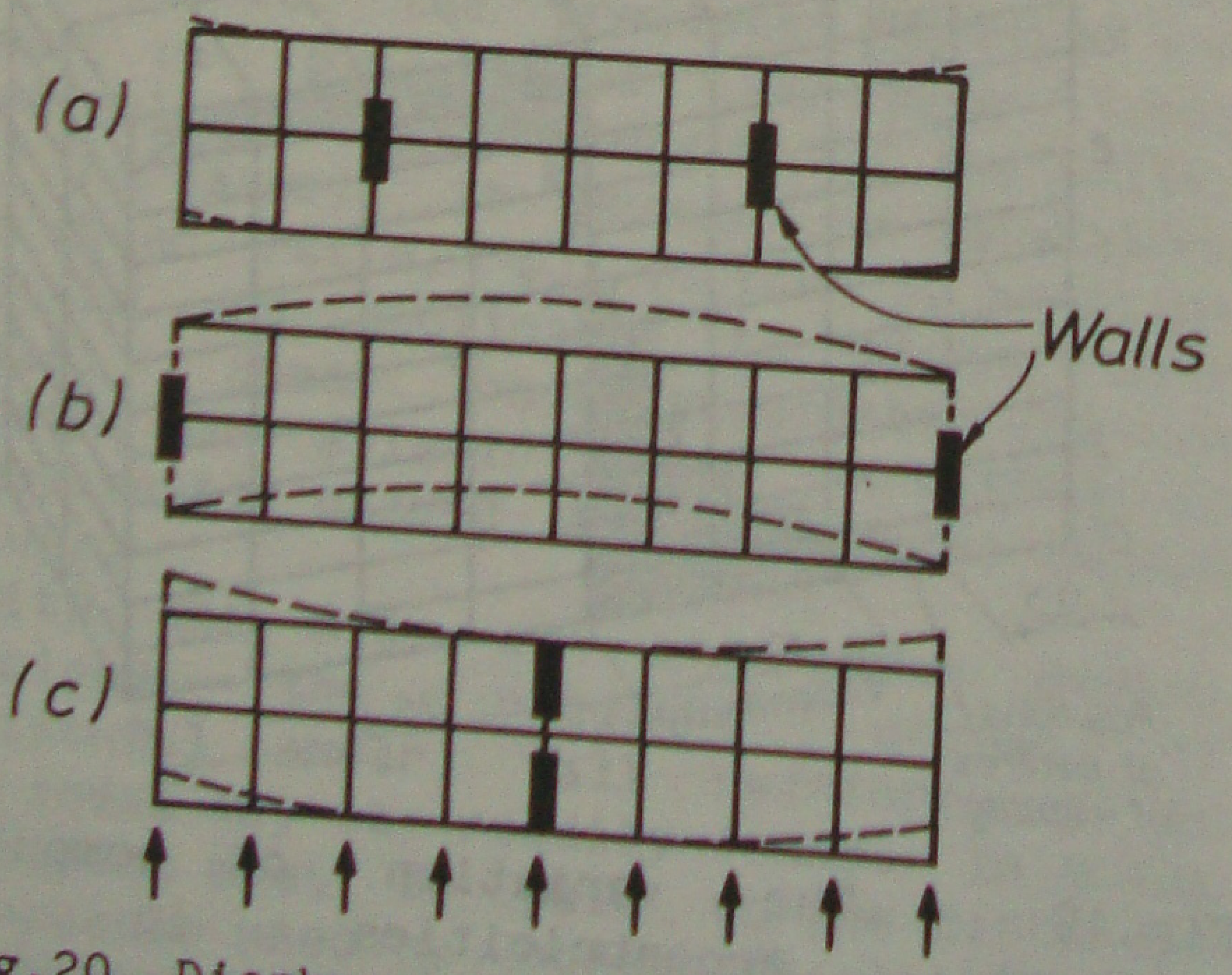


Fig.20 Diaphragm flexibility

6.4 Prediction of shear demand in walls

A number of case studies for structures of the type shown in figure 6, typically with 3.0, 3.6 and 7.0 long walls, have indicated that the capacity design procedure set out in Section 5, led to structures in which:

1. Inelastic deformations during the El Centro event remained within limits envisaged in recent codes. Typically storey drifts did not exceed 1% of storey heights.
2. Plastic hinges in the columns of upper storeys were not predicted.
3. Derived column design shear forces proscribed shear failure without the use of excessive shear reinforcement.
4. Rotational ductility demands at the base of both columns and walls, remained well within the limits readily attained in

appropriately detailed laboratory specimens such as studied by Corley, Fiorato and Oesterle (1981) or by Paulay and Goodsir (1986).

5. Predicted shear demands in the upper storeys of walls were satisfactorily catered for by the envelope shown in figure 17. However, maximum dynamic shear forces at the bases exceeded the design shear level (figure 18).

This latter feature was initially viewed with concern. Therefore further studies of the phenomenon, discussed previously with the description of design Steps 17 and 18, were undertaken. Some of the findings are summarised in the following paragraphs.

(a) Predicted peak shear forces were of very short durations, typcially 0.02 to 0.03 seconds. While there was no experimental evidence to prove it, it was felt that shear failures during real earthquakes could not occur within a few hundredths of a second.

(b) The probable shear strength of a wall, which could be utilized during such an extreme event, is in excess of the ideal strength (equation (18)) used in design.

(c) Some inelastic shear deformation during the very few events of peak shear should be acceptable.

(d) Walls and columns were found not to be subjected simultaneously to peak shear demands. Therefore the danger of shear failure at the base, for the building as a whole, should not arise.

(e) The simultaneous occurrence during an earthquake record of predicted peak shear and peak flexural demands was found to be about the same as the occurrence of peak shear demands. This means that when maximum shear demand occurred, it did generally coincide with maximum flexural demands. Present code provisions in New Zealand were based on this precept.

(f) As a safeguard against premature web-crushing in walls, which has been observed by Corley, Fiorato and Oesterle (1981), when large rotational ductilities were imposed on the plastic hinge region, a limitation on the maximum computed shear stress in this region is warranted. The rationale in this measure is that the contribution of the web region to the carrying of diagonal compression is reduced when the region is also subjected to large reversed cyclic inelastic flexural strains. One way to provide such a safeguard, is to make the maximum allowable shear stress dependable on the expected ductility demand. The limitation recommended in New Zealand is in the form

$$\frac{v_{i,max}}{\sqrt{f'_c}} \leq (0.34\phi_o K + 0.16) < 0.9 \quad (20)$$

where $\phi_{o,w}$ = wall overstrength factor defined in Step 15, K = ductility factor specified by the National Building Code of Canada. Stresses are expressed in MPa. As an example, when f_y = 400 MPa, a typical value of $\phi_{o,w}$ will be of the order of 1.56, so that for a complete ductile system with K = 0.7, the maximum shear stress generated by the design shear force given by equation (18), will be limited to $0.53\sqrt{f'_c}$ MPa

6.5 Variations in the contribution of walls to earthquake resistance

The study of the seismic response of hybrid structures has shown, as was to be expected, that the presence of walls significantly reduced in the upper storeys the dynamic moment demands on columns. This is because the mode shapes of relatively stiff walls do not permit extreme deformation patterns to develop in the inherently more flexible columns. Therefore moment increases in columns above or below beams, due to higher mode effects, as shown in figure 13, are much smaller. This was recognised by the introduction of a smaller dynamic moment magnification factor, $\omega = 1.2$, at intermediate floors, as discussed in design Step 11 and as was shown in figure 12. The applicability of appropriate values for ω was supported with a number of case studies (Goodsir, Paulay and Carr (1983), Goodsir (1985)), in which walls made a significant contribution to the resistance of design base shear.

The contribution of all walls to lateral load resistance was expressed by the wall shear ratio, ψ, introduced in design Step 16. The minimum value used in the example structure with two 3 m walls was 0.44.

The question arises as to the minimum value of the wall shear ratio, ψ, relevant to a hybrid structure, for the design of which the proposed procedure in Section 5 is still applicable. As the value of ψ diminishes, indicating that lateral load resistance must be assigned primarily to frames, parameters of the design procedure must approach values applicable to framed buildings. At a sufficiently low value of this ratio, say $\psi < 0.1$, a designer may decide to ignore the contribution of walls. Walls could then be treated as secondary elements which would need to follow, without distress, displacements dictated by the behaviour of ductile frames.

The minimum value of ψ for which the procedure in Section 5 is applicable has not been established. It is felt that $\psi = 0.33$ might be an appropriate limit. For hybrid structures for which $0.1 < \psi < 0.33$, a linear interpolation of the relevant parameters, applicable to ductile frames and ductile hybrid structures, seems appropriate. These parameters are ω, ω_c, ω^*_v and R_m.

7 SUMMARY

1. The methodology embodied in current capacity design procedures used in New Zealand, relevant to both ductile framed buildings and those in which seismic resistance is provided entirely by structural walls, has been extended to encompass hybrid structures. Appropriate values were suggested for governing design parameters.

2. Regular 6 and 12 storey buildings with varying wall contents were designed using this approach, and subsequently subjected in analytical studies to the El Centro and Pacoima Dam accelerograms. The generally good performance of these buildings during the El Centro excitation suggested that prototype structures should exhibit good seismic performance.

3. As intended, energy dissipation was found to occur primarily in beam and wall base plastic hinge zones.

4. Columns were found to enjoy protection against flexural yielding except at the base and top floor levels, where hinge formation was expected. A dynamic magnification factor for column moments as small as $\omega = 1.2$ proved satisfactory.

5. Column design shear forces were adequately predicted by the design procedure and generally found to be non-critical.

6. The provisions of the linear design wall moment envelopes restricted significant inelastic wall deformations, even during the extreme Pacoima Dam event, to the base.

7. Peak wall base shear forces encountered during analyses were somewhat underestimated by the proposed design procedure. In the context of uncertainties in the analysis, available reserve shear strength, and in particular the predicted very short duration of these shear forces, it was felt that this analytically predicted phenomenon should not be viewed with concern.

8. The proposed envelopes for design wall shear forces adequately estimated shear demands in the upper storeys.

9. It is believed that the methodology proposed is logical and straight-forward. It should provide buildings so designed, and carefully detailed, with excellent seismic performance capability.

10 Using engineering judgement, the approach is capable of being extended to other structural configurations not covered in this paper, but only by consistent application of capacity design principles.

11. The excellent seismic behaviour of well balanced interacting ductile frame-wall structures, particularly in terms of drift control and dispersal of energy dissipating mechanisms throughout the structural system, should encourage their extensive use in reinforced concrete buildings.

8 ACKNOWLEDGEMENTS

This study, being part of a project which involved also considerable experimental work, was generously assisted by grants from the Ministry of Works and Development and the University Gants Committee of New Zealand. The invaluable contribution of Dr. A.J. Carr with respect to computational work and Dr. W.J. Goodsir in the execution and evaluation of case studies is gratefully acknowledged.

9 REFERENCES

Canadian Standards Association. Design of Concrete Structures for Buildings. 1984. A National Standard of Canada. CAN-A23.3-M84:281.

Comite Euro-International due Beton. 1983. Model code for seismic design of concrete structures. Bulletin D'Information. (1-Final Draft Oct. 1983)) 160:117.

Corley, W.J., Fiorato, A.E. and Oesterle, R.G. 1981. Structural walls, Significant Developments in Engineering Practice and Research. American Concrete Institute. Publication SP-72: 77-131.

Goodsir, W.J., Paulay, T. and Carr, A.J. 1983. A study of the inelastic seismic response of reinforced concrete coupled frame - shear wall structures. Bulletin of the New Zealand Society for Earthquake Engineering, Vol 16, No. 3: 184-200.

Goodsir, W.J. 1985. The design of coupled frame-wall structures for seismic actions, Research Report, No. 85-8, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand: 583.

International Association for Earthquake Engineering, 1984. Earthquake resistant regulations - A World List, Tokyo: 904.

Standard Association of New Zealand, New Zealand Standard Code Practice for the Design of Concrete Structures. 1982. NZS 3101:Part 1: 127, Commentary NZS 3101:Part 2: 156, Wellington, New Zealand.

Park, R. and Paulay, T. 1975. Reinforced concrete structures. John Wiley and Sons, New York: 769.

Paulay, T. 1975. Design aspects of shear walls for seismic areas. Canadian Journal of Civil Engineering, Vol. 2, 3: 321-344.

Paulay, T. 1980. Earthquake resisting shear walls - New Zealand Design Trends, Proceedings. Journal of the American Concrete Institute, Vol. 77. 3: 144-152.

Paulay, T. 1980. Deterministic design procedure for ductile frames in seismic areas. American Concrete Institute. Publication SP63. Detroit: 357-381.

Paulay, T. 1981. Developments in the seismic design of reinforced concrete frames in New Zealand. Canadian Journal of Civil Engineering, Vol. 8. 2: 91-113.

Paulay, T. and Goodsir, W.J. 1986. The capacity design of reinforced concrete hybrid structures for multistorey buildings. Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 19.1: 1-17.