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SEISMIC DESIGN OF CONCRETE STRUCTURES THE PRESENT NEEDS OF SOCIETIES

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ABSTRACT

To promote the implementation of existing knowledge in the seismic design of engineering structures and to contribute to more effective disaster mitigation, primarily design practitioners are addressed. To this end, simplicity in the application of a rational deterministic design philosophy, which has been successfully used in New Zealand for a number of years, is advocated. After the review of basic aims, a detailed description of the design strategy relevant to ductile multi-storey frames, buildings with structural walls and dual structural systems is presented. Considerations of elastic response and limited ductility demands are also reviewed briefly. A number of features, generally not embodied in seismic codes, are also included. The presentation concludes with an emphatic illustrated appeal for the recognition of the importance of quantified high quality in the detailing of construction.

KEYWORDS

Buildings, beams, columns, construction details, ductility, foundations, joints, reinforced concrete, structural design, walls.

1 INTRODUCTION

While deliberating on aspects of advancements and applications in one of the most promising areas of our tasks to reduce the effects of natural disasters, *earthquake engineering*, it may be appropriate to reflect on the *immediate needs* of mankind with respect to disaster mitigation. It is over ten years since Frank Press in his address, "The Role of Science and Engineering in Mitigating Natural Hazards", during the Eighth World Conference on Earthquake Engineering in San Francisco, challenged the developed world, and the engineering profession in particular, to initiate an international co-operative program that could lead to tangible relief in the consequences of natural disasters (Press, 1984). Subsequently this initiative has led to a resolution by the United Nations whereby the last ten years of this millennium was declared "The International Decade of Natural Disaster Reduction" (IDNDR).

This address does not offer startling new concepts which might point to new and exciting directions in the developments of earthquake engineering. Rather it attempts to restate *established principles* with proven potential for application to meet immediate needs, particularly in countries where the current state of the art has as yet not been fully translated into effective design and construction practice. Making use of the privilege of a keynote speaker, issues of immediate needs in the application of *structural design*, as perceived by the speaker, with particular reference to the earthquake response of *reinforced concrete buildings* are presented. The presentation is strongly influenced by earthquake engineering as practiced in New Zealand, a small country where some innovative concepts, together with extensive research, have been translated over a short period of time into codified general implementation.

The principal aims of this address were motivated by the recognition that:

(1) The high degree of advancement achieved in earthquake engineering in a few countries, where resources and expertise were available to support this impressive progress, is not even remotely accompanied by corresponding advancements in developing countries.

(2) Population increase, accompanied by corresponding expansion of the building stock, mostly inadequate in terms of seismic performance criteria, is progressively increasing the seismic risk world wide (Bertero, 1992). In spite of exponential technological developments, an ever increasing, indeed staggering, number of people are becoming potential victims of earthquake disasters. There is a compelling need to address during this conference not only researchers but also *practitioners*, who have the vital role in transforming technical developments into effective seismic protection of societies.

(3) The magnitude and complexity of the problems, not only technical but also economic, cultural, political, sociological and environmental, to name a few, must not be allowed to discourage initiative. Modest and affordable endeavours within the acquired expertise of the structural engineering profession, represents promising *immediate* contributions to disaster mitigation.

(4) The study of earthquake damage repeatedly documented clearly *identifiable* faults in structural design. Yet new construction in many countries demonstrates that several *faults*, including those leading to possible collapse, are being *repeated*. Hence a restatement of common misconceptions or ignorance of convincing seismic evidence can only contribute to improved design practice.

(5) There is a compelling humanitarian and moral pressure on the structural engineering profession to act now. Only actions leading to the *construction* of safer buildings will be recognized by earthquakes.

2 ADAPTABLE TECHNOLOGY TRANSFER

One of the aims of a desirable transfer of technology and engineering knowledge is to obtain and share demonstrable results within a relatively short time. This should clearly advance engineering and scientific understanding provided that it can be adapted for use within the country targeted (Housner, 1989). In this, certain constraints imposed on modern seismic design philosophies, such as local building traditions, construction practices and the availability of suitable materials, must be taken into account.

In developing countries, specifically in the Pacific rim, attention should be focused on the performance of relatively simple buildings housing people in rural areas or apartment type blocks in urban regions. For the design of complex industrial construction, it is more likely that special expertise will be obtained.

For most buildings in developing countries greater investments, to provide a high degree of damage control during significant but relatively rare earthquakes, are often preceived to be unaffordable. However, to reduce death and injury, technical prerequisites of a survival limit state must be met. Thus, the design must concentrate on *structural qualities* that will ensure that relatively large earthquake-imposed displacements can be accommodated while the integrity of the building to support gravity loads is maintained.

3 SOME MISCONCEPTIONS IN SEISMIC DESIGN

In many countries the design process intended to provide structural properties that promise satisfactory seismic response of buildings, is synonymous with sophisticated dynamic analyses. Often little attention is given to the *conceptual choice of structural systems* that possess highly desirable seismic properties, such as horizontal and vertical regularity and symmetry. The principle that simplicity of structural form improves predictability of seismic response is seldom appreciated. Traditional techniques of analyses for elastic structures, predicting lateral force, or displacement-induced, so called, "safe stresses" for events during which elastic response is demonstrably not possible, are still enjoying unjustified credibility. A high degree of precision achieved with the use of computers for the derivation of actions due to fictitious lateral design forces is often excessively respected. Precision achieved is taken as a confidence-inspiring assurance that the requirements of the ultimate limit state associated with a severe seismic event, are also satisfied.

The beneficial influence of the *deformation capacity of a structural system*, that is *ductility* in a global sense is now probably widely accepted. However, the *sources* of this highly desirable, indeed life saving, structural property are seldom properly traced, or quantified. The exploitation of ductility for its intended purpose, to provide *reserve* deformation capacity without significant reduction of resistance to lateral forces, is often not fully implemented. Deformation capacity is the most important structural property in areas of high seismic risk and where economic constraints limit the level of seismic resistance that can be afforded.

The perceived dominating role of structural analysis often distracts the designer from paying attention to the end product. This is the structure *as built*. Clearly identified sources of frequently observed structural distress during earthquakes are, inferior quality in construction and materials used, and non-compliance with the designer's instruction. An equally important aspect of catastrophic building response is the designer's neglect of the role of *detailing* in the construction of components of the building system, an aspect to be addressed in Section 15 in greater depth.

Some schools claim that in terms of seismic response the most desirable structural property is *redundancy*. This appears to be based on the *defeatist* notion that if an element or component of the redundant system fails, there will be a second or perhaps a third line of defence to compensate for the loss of that element. On account of inelastic redistribution of internal actions, each reaching its predetermined level, redundancy of systems with only modest ductility capacity, enables *all* components to *fully* contribute to seismic resistance at the ultimated limit state. Irrespective of the degree of redundancy, by proper design and detailing of each and every primary component, its *contribution can and must be relied on.*

4 SIMPLICITY IN DETERMINISTIC DESIGN PROCEDURES

The design strategy for the prevention of collapse under earthquake actions must be well understood to be effective. In routine engineering practice only simple techniques can be expected to be understood. Moreover, the design strategy must be rational so that its application is accompanied by firm conviction in its soundness. Cookbook rules carry the danger of being misinterpreted. Also they may be unintentionally used for irrelevant purposes.

Some postulated ingredients of *a simple, rational and confidence-inspiring seismic design strategy* are:

(1) An admissible unambiguous *load path* for earthquake-induced lateral forces, both in horizontal and vertical planes, down to the foundations must be established.

(2) Simple and well established analysis techniques for elastic structures, satisfying equilibrium criteria, should enable critical regions under the actions of gravity loads and earthquake-induced forces to be

readily identified. Provided that equilibrium criteria are not violated, significant *approximations* in analyses *are admissible*. The inevitable crudeness of our ability in the prediction of the characteristics of ground motions compels us to accept approximations. This is particularly relevant to countries where so far only limited seismic survey, not comparable of that for example in Southern California, has been carried out.

(3) Actions so derived for a structural system must be able to be sustained by viable mechanisms that may be mobilized when earthquake-induced displacements will be significantly larger than those consistent with elastic response. Such *plastic mechanisms* must be complete and kinematically admissible. However, within wide limits they can be freely chosen. The choice of viable energy dissipating systems, providing the intended ductility capacity, is simple and yet one of the most important ingredients of a successful design. A suitable choice relies on a good understanding of and feel for basic structural behaviour and on some experience rather than on analytical skills.

(4) Once the elements of a complete plastic mechanism have been identified, they can be proportioned, using well established techniques, to provide at these localities the desired and affordable *level of resistance*. In building structures these regions are termed plastic hinges. Irrespective of the material used, it is these potential plastic regions that require careful detailing for construction.

(5) Economy is optimized and predicted inelastic response assured if sources of inelastic structural deformation are restricted to preselected regions. *Traditional construction* can be used for all regions within the structural system where inelastic deformations are deliberately *inhibited*. To achieve this the designer must establish a *strength hierarchy*. This implies that potential plastic regions must form the *weak links in the chain of resistance* and all other components, made to be strong links of the system, must be capable of resisting within the elastic domain the maximum actions that may be developed in the weak links. Thereby the strong links, which represent a very great fraction of the total volume of the structure, do *not* require special treatment in enabling them to cope with seismic demands.

(6) The principles of design strategy to be examined in subsequent sections, usually referred to as *capacity design philosophy*, are extremely simple. They have been used in several countries, and particularly in New Zealand, in the last decade (Park and Paulay, 1975, Paulay and Priestley, 1992). The procedure is a *design* rather than an analytical *tool*. With its use the designer can determine in the event of a major earthquake the destiny of the building. The less sophisticated the structural system the simpler the application. It holds the best promise for the prevention of collapse in conventionally constructed structures with an affordable low level of lateral force resistance.

Unfortunately the understanding of the concepts of capacity design philosophy, that are gradually being incorporated in codes of developed countries, is not well established in earthquake affected countries of the Pacific rim. Its introduction to design practice and educational institutions of this region could be a promising feature of knowledge transfer within the aims of a World Seismic Safety Initiative, a long term project initiated within the International Association for Earthquake Engineering in response to the challenge of the IDNDR.

The sequence of design steps outlined lends itself also to the conduct of seismic diagnosis of existing buildings that are suspected to be inadequate in terms of strength, or ductility capacity, or construction details, and thus may require retrofitting (Priestley and Calvi, 1991).

5 BASIC AIMS IN SEISMIC DESIGN

The widely accepted aims of the seismic design of structural systems are best defined by recalling the *limit states* that are to be satisfied. Boundaries between limit states for seimic design cannot be defined precisely. However, a much larger degree of uncertainty is involved in the recommendations of building codes of various countries to determine the intensities of lateral seismic design forces. The simple design strategy described subsequently is intended to *accomodate* such *uncertainties*.

5.1 Serviceability Limit State

Relatively frequent earthquakes associated with minor intensity of ground shaking should not interfere with functionality, such as the normal operation of a building and the use of its content. *No damage* needing repair is expected. Essentially elastic response with predictable deformations is intended. Therefore the controlling structural property will be *stiffness*.

For meaningful predictions of the elastic response of reinforced concrete structures, realistic approximations for the evaluation of the effective stiffness of structural members need to be made. Within the domain of elastic response this will be affected primarily by the degree of cracking of the concrete and the restraint offered by joints where members are connected to other members. Many codes do not draw designer's attention to gross errors that result from a widely used practice, whereby stiffnesses are based on the gross uncracked sections of reinforced concrete members.

Figure 1 shows typical load-displacement relationships for reinforced concrete flexural members. It suggests non-linear response, as a consequence of progressive cracking, well before inelastic strains develop. It also shows, what is encountered in laboratory experiments, that after repeated loading not exceeding approximately 75% of the nominal strength of the member, S_n, approximately linear response is obtained. This then can be used as a realistic estimate of flexural stiffness when examining service performance. The extension of this assumed linear response may also be utilized to define Δ_y, a *reference yield displacement* used subsequently to quantify conveniently the displacement ductility to which the member on the system may be subjected to.

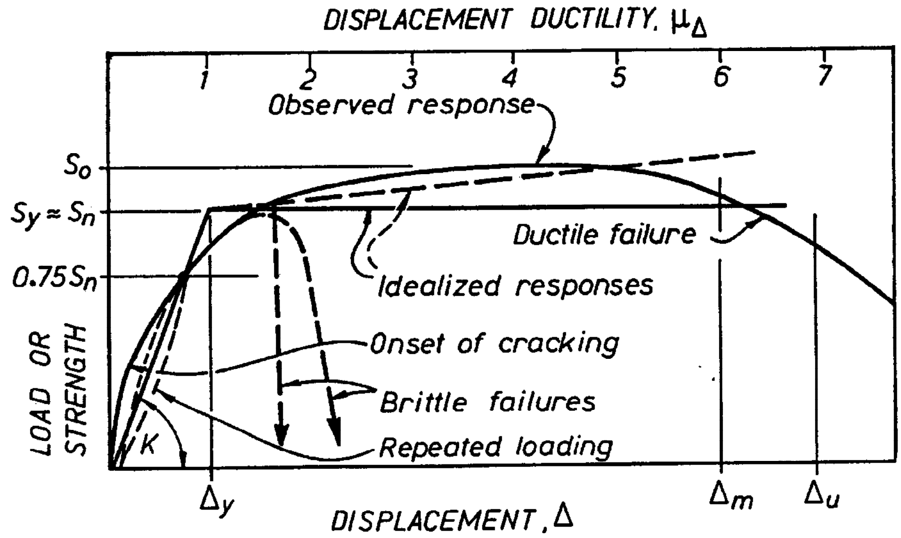


Fig 1. Typical load-displacement relationship for a reinforced concrete member.

To assist designers, assumptions for the effective second moment of area, I_e, and cross sectional area, A_e, of prismatic members, in terms of the values based on those relevant to the gross uncracked section, I_g and A_g, have been recommended (Standards New Zealand, 1995). These are reproduced in Table 1. The coefficients depend on the extent of cracking anticipated to result from the intensity of lateral design forces applied to the elastically responding structure satisfying criteria for the servicability or the ultimate limit states. The intensity of earthquake design forces associated with the serviceability limit state is based on the recommendation that they be taken one sixth of the forces corresponding with the elastic response of the system, in accordance with the chosen response spectrum. Thus for a structure designed to accomodate at the ultimate limit state a ductility demand, $\mu_\Delta = 6$, the intensity of the design forces for both limit states are expected to be similar. Hence the extent of cracking affecting the stiffness of the elastically responding structure, as columns (1) and (4) of Table 1 show, may be assumed to be the same for both limit states. On the other hand, for a system with $\mu_\Delta = 3$, the earthquake design forces corresponding with the ultimate limit state are expected to be approximately twice as large as those satisfying the serviceability limit state. Hence for the latter case, with reduced extent of cracking, the use of larger stiffnesses, as recorded in column (3) of Table 1, is appropriate.

Table 1 - Effective concrete section properties, I_e, A_e

Type of Member	(1) Ultimate limit state	(2) Serviceability limit state	(3)	(4)
		$\mu_\Delta = 1.25$	$\mu_\Delta = 3$	$\mu_\Delta = 6$
1. Beams*				
(a) Rectangular beams	$0.40I_g$	I_g	$0.70I_g$	$0.40I_g$
(b) T-, L- beams	$0.35I_g$	I_g	$0.60I_g$	$0.35I_g$
2. Columns***				
(a) $P_u/f'_cA_g > 0.5$	$0.80I_g$	I_g	$0.90I_g$	$0.80I_g$
(b) $P_u/f'_cA_g = 0.2$	$0.60I_g$	I_g	$0.80I_g$	$0.60I_g$
(c) $P_u/f'_cA_g = -0.05$	$0.40I_g$	I_g	$0.70I_g$	$0.40I_g$
3. Walls***				
(a) $P_u/f'_cA_g = 0.2$	$0.45I_g$, $0.80A_g$	I_g, A_g	$0.70I_g$, $0.90A_g$	$0.45I_g$, $0.80A_g$
(b) $P_u/f'_cA_g = 0.0$	$0.25I_g$, $0.50A_g$	I_g, A_g	$0.50I_g$, $0.75A_g$	$0.25I_g$, $0.50A_g$
(c) $P_u/f'_cA_g = -0.1$	$0.15I_g$, $0.30A_g$	I_g, A_g	$0.40I_g$, $0.65A_g$	$0.15I_g$, $0.50A_g$
4. Coupling beams**				
(a) Diagonally reinforced***	$\dfrac{0.40I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$	$\dfrac{I_g}{1.7+1.3\left(\frac{h}{L}\right)^2}$	$\dfrac{0.70I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$	$\dfrac{0.40I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$
(b) Conventionally reinforced***	$\dfrac{0.40I_g}{1+8\left(\frac{h}{L}\right)^2}$	$\dfrac{I_g}{1+5\left(\frac{h}{L}\right)^2}$	$\dfrac{0.70I_g}{1+8\left(\frac{h}{L}\right)^2}$	$\dfrac{0.40I_g}{1+8\left(\frac{h}{L}\right)^2}$

* Allowance must be made for haunches when used.

** The effects of shear deformations and strain penetration into walls along beam bars have been included.

*** Definition of symbols: P_u = axial load on column or wall at the ultimate limit state, h = overall depth of beam section, L = clear span of a coupling beam.

5.2 Damage Control Limit State

For ground shaking of intensity greater than that corresponding to the serviceability limit state, some damage may be expected. However, the damage with a low probability of occurrence during the expected life of the system should be economically repairable and such as to enable reinstatements of the system to full service.

If a concrete structure is to be protected against damage during a selected or specified seismic event, inelastic excursions of significance during its dynamic response should be prevented. This means that the structure should have *adequate strength* to resist internal actions generated during the elastic dynamic response of the system. This level of resistance is defined by the *ideal* or *nominal strength*, S_n, shown in Fig. 1.

5.3 Ultimate Limit State

The single most important design criterion is the *preservation of life*. Because of the severe but rare seismic events that are considered, this is also referred to as a *survival limit state*. With judicious detailing of critical regions (Section 15), structural damage to members will be minimized even after exceptional seismic events. However, large residual plastic deformations that could occur may make repair impractical. The principal aim is to ensure that collapse will not occur. Therefore the designer will need to address structural qualities which will ensure that for the expected duration of such a rare earthquake, relatively large displacements can be accommodated without significant loss of lateral force resistance, and that the integrity of the structure to support gravity loads is maintained.

The most important property associated with this survival limit state is *ductility*, that is, ability to accommodate large inelastic deformations without significant loss of resistance. The exploitation of this property is a relatively recent and challenging feature in the evolution of structural engineering. For this and other reasons great emphasis in this presentation is placed on *inelastic structural response*.

The *ductility capacity* of a reinforced concrete building system is conveniently quantified by the ratio of the lateral displacement at a suitable level, such as the roof, to the yield displacement at the same level; that is, the *displacement ductility factor*. Because the transition from elastic to inelastic response is non-linear, acceptable simplifications need to be made, particularly with respect to the definition of the displacement at first yield (Section 5.1) and such are shown in Fig. 1. While such global ductility is indicative of inelastic response of the entire system, the designer must also pay attention to repeated ductility demands that arise in critical potential plastic regions of the structure.

6 DUCTILITY

6.1 Definitions

Because ductility is a vital structural property if survival of large earthquakes is to be assured, a brief review of its well established features is warranted. The term is often misused and its quantification misinterpreted. Ductility in general is the ability to sustain large deformations, beyond the onset of yielding, without significant loss of resistance. Ductility of that kind, idealized in Fig. 1 by the bi-linear load-displacement relationship, can then be used as a measure of energy dissipation, utilized for hysteretic damping. Contrasting relationships associated with brittle failures at or shortly following the attainment of maximum resistance, or nominal strength, S_n, are also seen in Fig. 1. The primary aim in satisfying the intents of the survival limit state criteria is to ensure, for example by proper detailing, that such brittle failure will not occur.

Ductility expressing strain ratios defines the properties of the materials. It is evident that steel in tension is generally very ductile in comparison with concrete in compression, unless the *strain ductility* of concrete is enhanced by confinement.

The origin of global or *system ductility* in building construction is, in general, the inelastic rotation of plastic hinges. This is best quantified at a critical section of the plastic hinge by the ratio of the curvature at the ultimate state to that at the onset of yielding, termed the *curvature ductility* ratio, μ_ϕ. This is the property over which the designer has some control. Therefore a good understanding of factors which affect it, is extremely important.

The summation of curvatures over the effective length of the potential plastic region, the so called *plastic hinge*, quantifies the total plastic rotation of the region in terms of *member ductility*.

Recognition of member ductility, without the need to evaluate it in routine design, is important when assessing its relation to the global or *displacement ductility of the structural system,* μ_Δ.

The relationships between these types of ductilities are well established (Park and Paulay, 1975, Paulay and Priestley, 1992). The value of the displacement ductility ratio, μ_Δ, utilized in the design, is chosen by the designer, based for example on recommendations of codes. The curvature ductility ratio, μ_ϕ, estimated via the ductility demand imposed on the relevant member, governs the nature of the detailing of the potential plastic regions.

6.2 Demand and capacity

One of the difficult tasks in the estimation of the inelastic response of a structure with given resistance in terms of lateral forces, is the gauging of the ductility demand that might be imposed on it by the design earthquake. The predicament is bypassed in building codes by the selection of elastic design spectra considered appropriate for the locality, and subsequent specification for the reductions of response in recognition of an assumed ductility capacity of the structural system.

Typical smoothed spectra *cannot guarantee*, however, that transient ductility demands during a large earthquake will not exceed that assumed. While such design spectra, widely utilized in countries affected by seismicity (IAEE, 1992), may be used when determining the required strength with respect to lateral design forces, S_E, they should not be considered as being reliable predictors of maximum ductility demands. The *inherent crudeness* of the recommendations of current seismic design procedures for the level of ductility capacities of different structural systems, to be assumed in establishing the intensity of lateral design forces, must also be recognized. Fortunately with the implementation of thoughtful detailing of potential plastic regions of the structure, in most situations it is relatively easy to provide *reserves in ductility capacities* without incurring significant economic penalties. Moreover, in a well detailed structure the development of intended maximum ductility will always generate resistance somewhat greater than the required strength. By providing reserve ductility capacity, structures can be constructed that are very *tolerant* with respect to our inability to predict ductility demands more accurately. This concept is an *essential ingredient* of the seismic design strategy advocated in this presentation.

An example for the possibility to provide reserve ductility capacity is illustrated in Fig. 2. This shows the hysteretic response of an isolated reinforced concrete interior beam-column assembly of a typical two-way ductile multistorey frame. The near full size test specimens (Cheung *et al*., 1993), consisting of two beams at right angles, monolythic with a 130 mm thick floor slab, was subjected to different types of cyclic displacements with stepwise increases of amplitudes in one or both principal directions of the framing system (ACI, 1991). These displacements are expressed also as a percentage of the storey height. The outline of the specimen and the loading pattern in the East-West direction only is also seen in Fig. 2. The displacement patterns imposed with larger ductilities, such as associated with the shaded area in Fig. 2, were particularly severe because, as a result of skew loading, plastic hinges were developed simultaneously in all four beams meeting at the joint.

The excellent hysteretic response of the unit is evident. It is seen that with progressively increasing inelastic displacements, stiffness degradation, as expected, was inevitable. However, strength reduction at peak displacements was found to be negligible. The normalized strength at unity in terms of the horizontal column shear force, V_i, corresponds to the ideal strength of the critical beam sections, including some contribution of flanges in tension, based on measured rather than specified material strength properties. It is also seen that 15 to 20% excess strength was achieved. This performance resulted from the judicious detailing of all plastic hinges and particularly the beam-column joint, aspects of which are examined in Section 8.4. Subsequently some relaxations for joint reinforcement were considered.

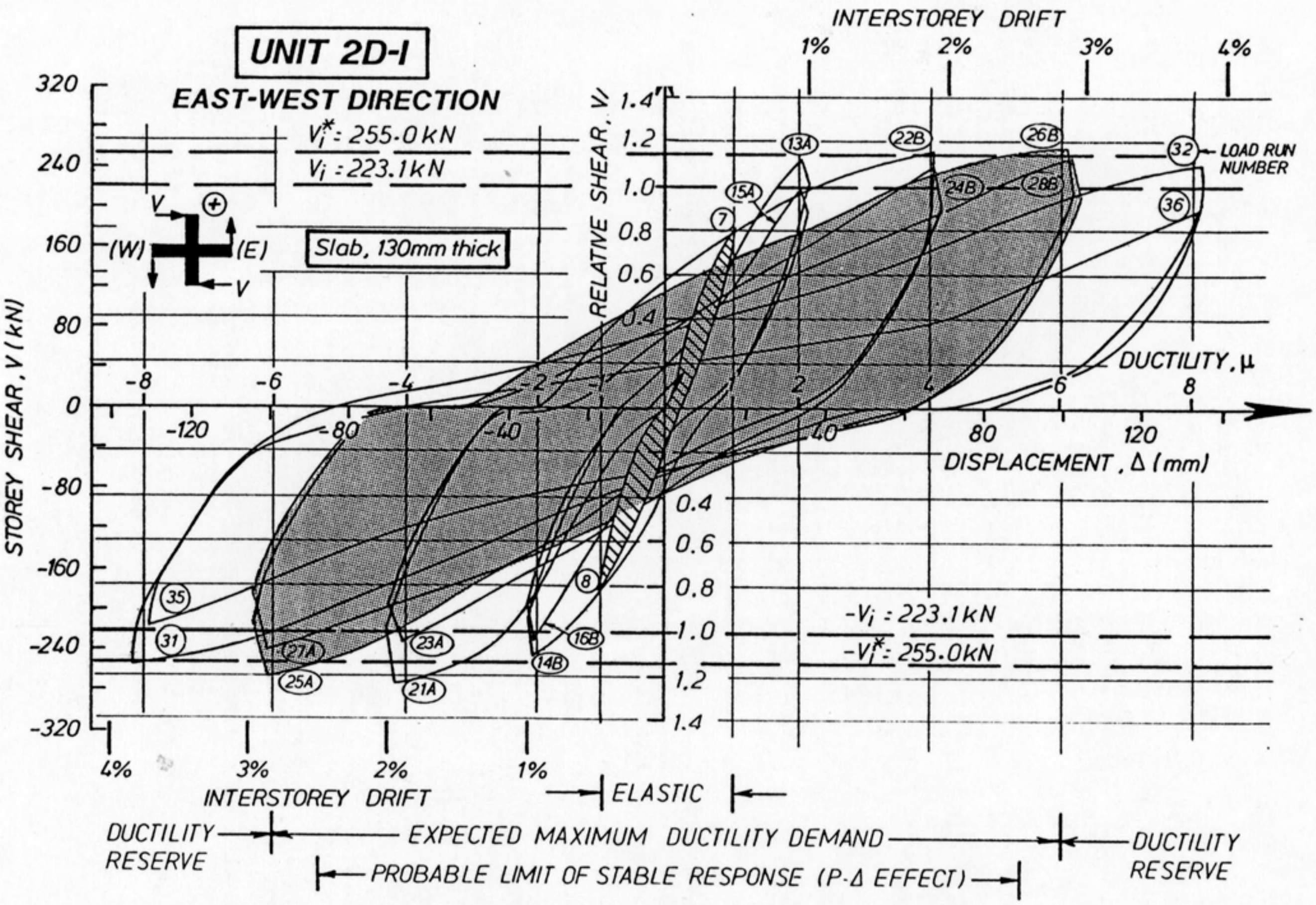


Fig. 2. Hysteretic response of a two-way frame sub-assemblage.

At the base of Fig. 2 displacement ranges corresponding with elastic response, expected maximum ductility demands and *ductility reserves* are compared. It is considered that when interstorey drift exceeds about 2.5%, significant reduction of frame resistance with respect to lateral forces must be expected due to P-delta effects.

7 A DETERMINISTIC LIMIT STATE DESIGN STRATEGY

7.1 Concepts of a Philosophy

As stated, it must be recognized that current estimates at any given location of seismic activity are extremely crude. For common situations, the strength of the structure with respect to the resistance of lateral forces is chosen to be a fraction of the strength that corresponds to elastic dynamic response to a crudely predicted seismic event. Therefore building codes make approximations in estimating the reduced intensity of acceptable lateral design forces. Consideration in design of *elastic dynamic response* characteristics, such as the contribution of the higher mode shapes of vibration to internal structural actions, are often emphasized and recommended. However, in the ultimate state, *inelastic response*, relying on ductility and ability to dissipate seismic input energy, will primarily govern structural response. The crudeness resulting from predictions of ground motions, the results of the analyses of models of elastic structures and code assessments relevant to ductilites, do not justify the often perceived accuracy aimed at, and claimed, in the design for an ultimate limit state.

These considerations suggest that gross approximations, particularly when they simplify routine design processes, are both attractive and justified. This is particularly the case when a structural system is rationally and deterministically *chosen* so as to be able to mobilize energy dissipating regions, examined in Section 7.2, which will have ample reserve deformation capacity to accommodate significant departures from displacements associated with initial estimates for ductility. Such an approach avoids the need for sophisticated techniques of analysis to evaluate the development of numerous possible plastic mechanisms in a complex structural system. Instead, the strategy invites the designer to "*tell the structure*" where plastic hinges are desirable or convenient and practicable at the ultimate limit state, and to *proscribe plastification* in all other regions. The strategy leads to the establishment of a suitable strength or capacity hierarchy between components of the total system, as suggested in Section 4.

In the *capacity design* of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. All other elements are then to be protected against actions that could cause failure by providing them with strength greater than that corresponding to the maximum feasible strength in the potential plastic hinge regions.

Capacity design requires the maximum probable values of displacement-induced forces to be estimated. Such forces are associated with the development of the *overstrength* of potential plastic hinges. This is examined in Section 7.3. For this, the strength properties of components *as built*, including strength enhancement of both steel and concrete under large imposed strains, need to be evaluated. The contribution to internal tension forces of all reinforcement, irrespective of its intended purpose, such as temperature or shrinkage control or to satisfy code and construction requirements, including those in floor slabs, must be included wherever such bars can be subjected to earthquake-induced tensile strains. When, in critical regions, excess reinforcement is provided, this must not be interpreted as a feature resulting in increased safety. In such cases, which are not uncommon, excessive overstrength may be developed and as a consequence all elements of the system intended to remain elastic must be designed for correspondingly-increased resistance.

Many concepts of this philosophy have been incorporated in building code proposals in Europe (European Commission for Standardization, 1993) and Japan (Architectural Institute of Japan, 1994), and also for bridges (Applied Technology Council, 1993).

7.2 The Choice of Plastic Mechanisms

In conformity with widely accepted principles, with very few exceptions, plastic mechanisms in reinforced concrete structures must rely on flexure as the source of energy dissipation. Therefore, with few exceptions, mechanisms associated with inelastic deformations originating from shear, transfer of bond between reinforcement and concrete, sliding and instability of members, must be *definitively suppressed*. The choice of the designer involves thus the selection of plastic hinges in beams, columns or walls that enable a kinematically admissible complete mechanism to be developed in the given structural system. An important aim in this selection is that for a given global or system displacement ductility demand, μ_Δ, the associated curvature ductilities at plastic hinges remain within proven limits.

The above considerations are illustrated in a series of diagrams in Fig. 3, which show desirable or acceptable mechanisms, and those that are to be avoided. The same ultimate displacement, Δ_u, has been assumed for all the example systems shown. The advantages of *strong column - weak beam mechanisms* in ductile multistorey frames are well established. When columns are provided with sufficient strength, plastic hinge formation in them can be avoided in all storeys above level 2, as shown in Fig. 3(a). When columns are adequately detailed for hinge formation, the widely used system shown in Fig. 3(b) may also be adopted. It is to be noted, however, that the possibility of simultaneous hinge formation at both ends of columns in any story, such as seen in Fig. 3(c), generally termed a *soft storey*, must not be permitted. It is evident that rotational ductility demands in the hinging columns, such as shown in Fig. 3(c), may become excessive.

The frame plastic mechanism shown in Fig. 3(b) is implied in the seismic design philosophy practiced and codified in the United States and in many other countries where similar recommendations have been adopted. In terms of the detailing of the columns for ductile response, this mechanism requires in the end regions of columns, transverse reinforcement in the form of stirrups, hoops, ties or spirals, in addition to those used in similar columns constructed in regions not affected by seismic considerations. This is to ensure that the rotational ductility capacity of a potential plastic hinge at the bottom *or* the top end of the column in a story is adequate. Moreover, the location of lapped splices of the principal column reinforcement is required to be in the region of midstorey height. Under reversing cyclic inelastic straining of the reinforcement, the capacity of lapped splices is known to deteriorate rapidly, unless heavy transverse reinforcement providing adequate clamping of each pair of spliced bars, is provided (Section 15.2.3). Another reason for avoiding lapped splices in potential plastic hinge regions, even if adequately reinforced, is the drastic reduction of the length over which reinforcing bars can yield. Thereby for a given hinge rotation, correspondingly increased tensile steel strains will be developed.

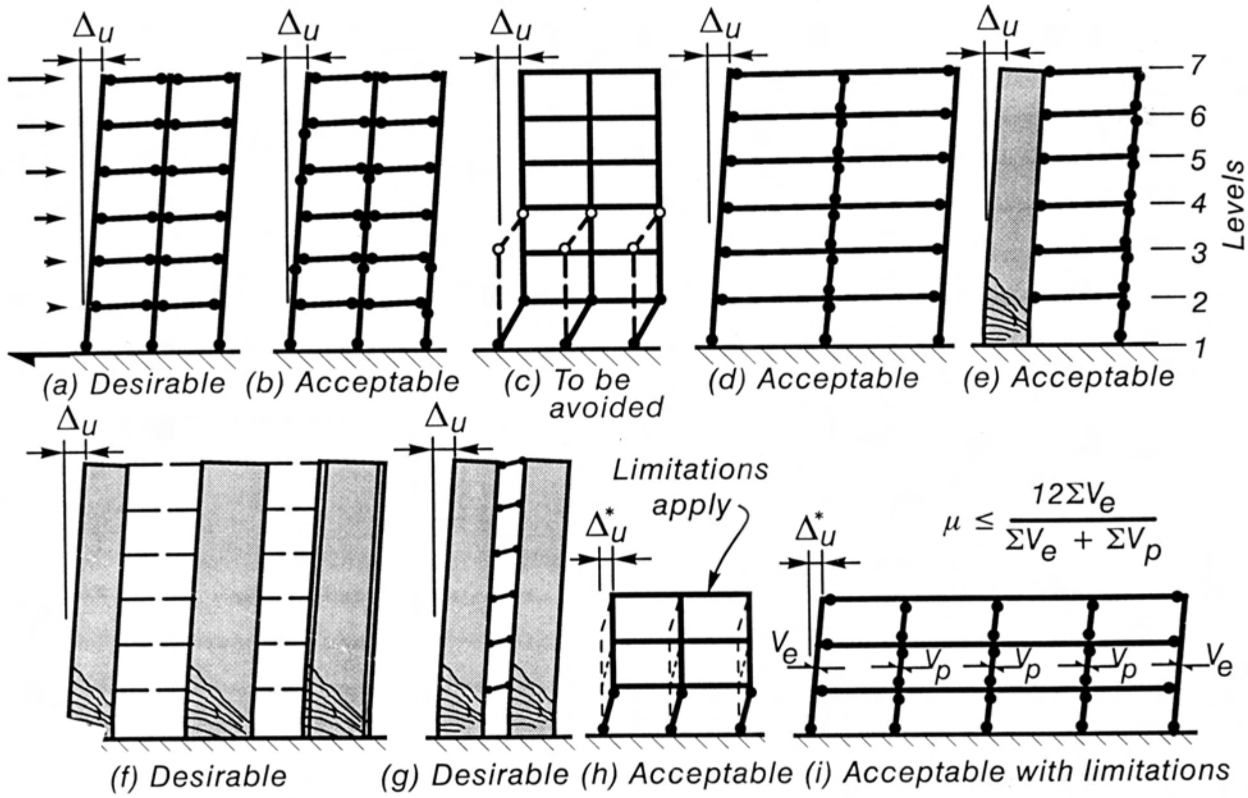


Fig. 3. Plastic mechanisms in multistorey buildings.

The phenomenon may lead to a concentration of damage over a short length of the column and perhaps even to premature fracture of reinforcing bars. The purpose of transverse reinforcement in potential plastic hinge regions , such as at the base of columns in multistorey buildings, is briefly reviewed in Section 15.2 where the suitable detailing of such reinforcement is discussed.

The system illustrated in Fig. 3(a) allows both the reduction of the amount of transverse reinforcement in the end regions of columns above level 2 and the placing of lapped splices immediately above the top of a floor. This convenient concession is justified because, with the precautions summarized in

Section 8.3, the formation of plastic hinges with significant curvature ductility demands need not be expected in such columns.

When the exterior columns of a frame, absorbing actions from one beam only, are made strong enough to ensure that a *soft storey* cannot develop, the formation of simultaneous plastic hinges at both ends of all interior columns (Fig. 3(d)) is acceptable, provided that all potential plastic hinges in these columns are detailed accordingly. A similar situation, usually encountered when long span beams need to be used, arises when walls (Fig. 3(e)) ensure that a soft storey cannot develop. Frames relying mainly on energy dissipation by columns, as shown in Fig. 3(i), are considered in Section 9.2.

When structural walls, which are interconnected by adequately designed diaphragms, are used to provide the necessary lateral force resistance, it is preferable to restrict the formation of plastic hinges to the base of the walls, acting as cantilevers, as shown in Fig. 3(f). This can be readily achieved when the walls above the plastic zone are provided with sufficient reserve flexural capacity. The precaution should ensure that, irrespective of the ground motions, essentially elastic response of such walls will occur at levels above the potential plastic region (Section 15.3.1). This will allow, in the elastic regions of the walls, to dispense with the more onerous requirements of the detailing for ductility.

Figure 3(g) shows the desirable and readily achievable plastic mechanism in a pair of structural walls interconnected by coupling beams which are specially reinforced to enable large member ductilities to be developed (Section 15.3.3). This system is capable of delivering excellent energy dissipating capacity in a very stable manner.

Frames in which *soft storeys*, as shown in Fig. 3(h), can develop should only be used when the displacement ductility demand assigned to them is restricted. Moreover, for an assumed overall displacement ductility demand, μ_Δ, it is necessary to evaluate the rotational ductility demand that may be imposed on the plastic hinges that are to be expected to form at both ends of all columns in any of the "soft storeys". The plastic hinges of columns of such frames, designed for severely restricted ductility demands, are likely to be required to be detailed for large curvature ductility demands. The structure shown in Fig. 3(h) is an example to illustrate the need for the evaluation of *local ductility demand* as a function of the overall displacement ductility associated with the deflection, Δ_u, at the ultimate limit state. Such estimates, satisfying elementary geometric relationships within the plastic mechansim, as illustrated in Fig. 18 and briefly discussed in Section 11.4.2, are very simple to make.

When long span beams need to be provided, the gravity load requirements for these may be more severe than those associated with seismic demands. Hence it may be difficult and even irrational to design interior columns under seismic attack to be stronger than the beams. Fig.3(i) depicts a typical *gravity load dominated frame* of this type. As in the example frame in Fig. 3(d), the prevention of the formation of a soft storey is assigned to the exterior columns. Usually the displacement capacity of such a system needs to be restricted. Aspects of the design of such frames, together with other issues relevant to the dominance of gravity load requirements, are examined in some detail in Section 9.2.

When admissible plastic mechanisms, such as the examples shown in Fig. 3, are chosen, it becomes evident *which members*, or parts of members, are to remain elastic at all times in accordance with the intent of capacity design. All that needs to be done, is to evaluate the *overstrength* of the selected plastic hinges, *as detailed and hence constructed*. This is reviewed in the next section. The resulting actions due to the development of ductility, when combined with those due to gravity, lead then to the design actions for members or regions to be protected against inelastic deformations.

7.3 Overstrength

The term overstrength was introduced in the late 1970s with the development in New Zealand of the philosophy of capacity design. It was necessitated by the realization of the need to know the probable maximum actions that could be developed in materials, structural members and the entire structural system, as a result of large ductility demands imposed by earthquakes. This enables then the desired hierarchy of strengths within the system to be quantified. For the benefit of designers not familiar with this philosophy, the basic definitions of relevant terms are briefly reviewed here.

7.3.1 Material overstrength. The strength of materials, such as concrete and steel, developed with large strains will be greater than that specified and used when proportioning structural members. In particular, strength enhancement of steel due to strain hardening and that of concrete in compression due to confinement must be recognized. The overstrength of constituent materials, S_o, can be defined with the aid of a materials overstrength factor, λ_o, so that

$$S_o = \lambda_o S_n \tag{1}$$

where S_n is the nominal or ideal strength, based on specified material properties.

7.3.2 The overstrength of members. Apart from strength enhancement of materials, the probable maximum strength of members, governed by flexure, will depend on a few additional well established parameters. The enhancement of the moment of resistance of the critical section within a plastic hinge can be expressed by the *flexural overstrength factor*, ϕ_o, which is the ratio of the enhanced moment, M_o, and the moment M_E, indicated by the analysis of the elastic structure subjected to only the specified lateral design forces at the ultimate limit state.

The difference between the materials and flexural overstrength factors, λ_o and ϕ_o, respectively, becomes apparent when it is realized that the magnitude of the flexural overstrength, M_o, is also affected by:

(a) The value of the strength reduction factor ϕ, such as specified for example by the ACI 381-89 (American Concrete Institute, 1989), or its equivalent used for example in European codes. This factor relates the required flexural strength, M_u, to the nominal strength, M_n, based on the specified material properties thus

$$M_u \leq \phi M_n \tag{2}$$

(b) The requirements for the resistance of gravity loads with or without earthquake design forces or wind forces, may be more severe than those due to the design earthquake forces.
(c) The moments used for the proportioning of flexural members may have differed from those derived by the initial analysis of the elastic structure, because some inelastic redistribution of design moments was utilized. Such redistribution of design moments along beams of continuous frames or among cantilever walls offers several advantages and may lead to a reduction of the overstrength of the structural system which otherwise might become excessive. It should be noted that of concern is only the overstrength of the relatively few weak inelastic links within the chain of resistance. Excessive overstrength leads to the boosting of the required strength of the remaining numerous elastic links of the chain. Moreover, the accumulation of excessive overstrength, for example from beams at a large number of levels in multistorey frames, may impose unnessarily heavy demands of the foundation system.
(d) For reasons of practicality in construction, the flexural reinforcement provided may not exactly match that required.

The flexural overstrength factor, ϕ_o, is subsequently used to appropriately magnify the end moments of columns to equilibrate beam moments at beam-column joints, that is, at the node points of the frame model. As stated earlier, the materials factor, λ_o, relates to the critical section within a plastic hinge. In beams this may be at the column face or within the span, for example as seen in Figs. 13 and 19. However, the factor ϕ_o is applicable to moments at node points. Accordingly, based on equilibrium criteria, its value at these node points must be determined using the values of overstrength moments, M_o, at the locations of plastic hinges with gravity loads sustained simultaneously.

Figure 4 assists in clarifying these relationships, not encountered in gravity loaded structures. The moments in the vicinity of an interior joint, due to seismic design action only, are those shown in Fig. 4(a). Because beam and column moments are in equilibrium at the node point, they are used as reference quantities to assist in the preservation of equilibrium during susbsequent changes while the intended frame mechanism, such as shown in Fig. 3(a), develops. The beam moments resulting from the overstrength of each beam on either side of the column, are shown in Fig. 4(b). The flexural overstrength factor in this case must be based on the sum of the two beam moments, $\Sigma M_{b,E}$, due to the design earthquake forces. To maintain joint equilibrium, the sum of the corresponding column moments, $\phi_o \Sigma M_{c,E}$, must be equal and opposite. In this figure it has been assumed that the moments in the columns above and below the beam bear the same relation to each other as in Fig. 4(a). This relationship will, however, significantly change during the inelastic dynamic response of the frame, *without any increase* in the value of $\phi_o \Sigma M_{c,E}$. These changes are taken into account when determining the final design moments applicable to the critical end-sections of the column, as described in Section 8.3.

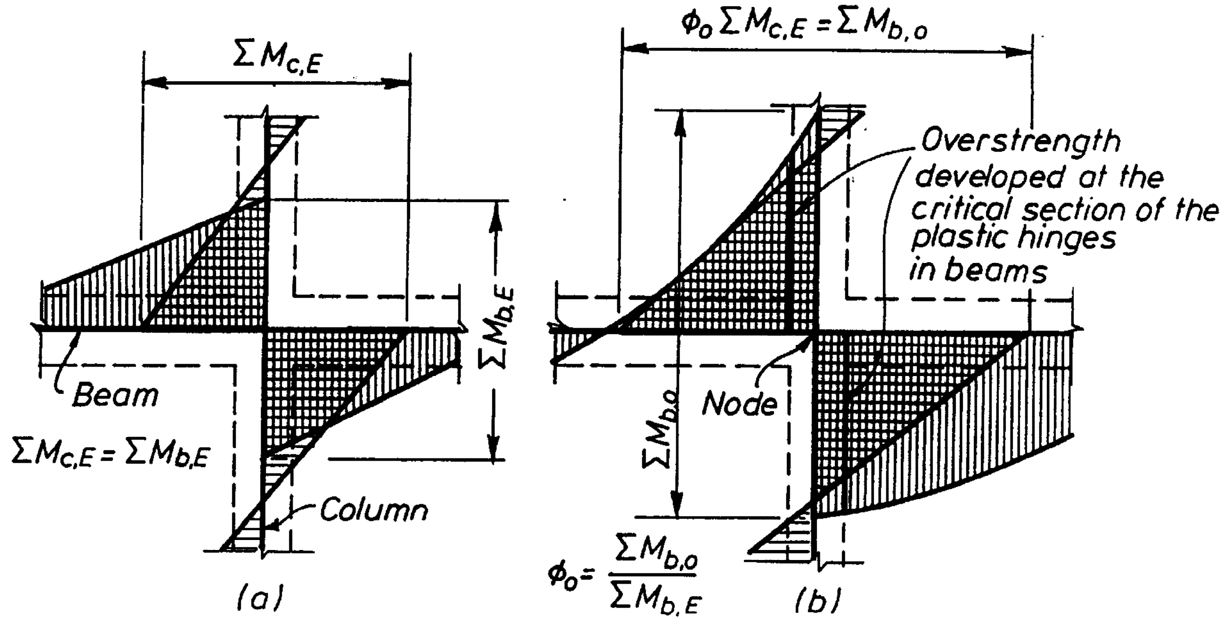


Fig. 4. The relationship between beam and column moments at a node.

It is emphasized that the moments in Fig. 4(b) are induced by large lateral seismic displacements. They are based on the *properties of the beams as designed and detailed* and therefore include whatever considerations have been given to gravity loads or other effects.

7.3.3 The system overstrength factor. In certain situations it is important to compare the sum of the overstrengths of a number of interrelated members at corresponding nodes with the total demand made on the same members by the specified earthquake forces alone.

As an example, consider a multistorey building in which the entire seismic resistance in a particular direction is provided by n reinforced concrete cantilever walls. The relevant material overstrength and strength reduction factors are in this example $\lambda_o = 1.3$ and $\phi = 0.9$, respectively. A design matching exactly the specified flexural strength at the base of the walls would then result in $\phi_o = \lambda_o/\phi = 1.44$ for each wall. Deviations from this value, almost certainly encountered in practice, would indicate that some walls have been designed for larger or smaller moments than those required for the prescribed seismic resistance (i.e. M_n differs from M_E/ϕ). If the system overstrength factor, defined as

$$\psi_o = \sum_i^n M_o \Big/ \sum_i^n M_E = \sum_i^n (\phi_o M_E) \Big/ \sum_i^n M_E \tag{3}$$

is less than 1.44, this would indicate that the strength requirements for the structure as a whole, to resist seismic design forces, has been violated. On the other hand a value much larger than 1.44 in this example, will warn the designer that for reasons which should be identified, excess strength has been provided. This is particularly important when the design of the foundation structure, restraining these walls, is to be considered.

8 THE APPLICATION OF CAPACITY DESIGN TO MULTISTOREY DUCTILE FRAMES

In multistorey ductile frames with plastic mechanisms shown in Fig. 3(a), columns should be made stronger than beams. The question to be answered is thus : *"How much stronger than the chosen weak links, such as the beams, should all those regions or members, such as columns, be, to ensure that the chosen strength hierarchy will achieve its intended purpose."*

Because of the extensive coverage of the subject in recent publications (Paulay and Priestley, 1992, Paulay, 1993), only the highlights of the application of capacity design of ductile frames, with the majority of details omitted, are briefly reviewed here. The critical regions of frames that are affected during the seismic response, shown in Fig. 5, are addressed.

8.1 The Design of Beams for Flexural Resistance

After the analysis of the structure for both gravity loads and seismic design forces, the critical bending moment patterns for all beams are established. As stated earlier, at this stage a rational and advantageous redistribution of the design moments along continuous beams should be considered. This then enables beams to be sized. Particular attention is subsequently to be paid to the detailing of the potential plastic hinges shown at (1) in Fig. 5.

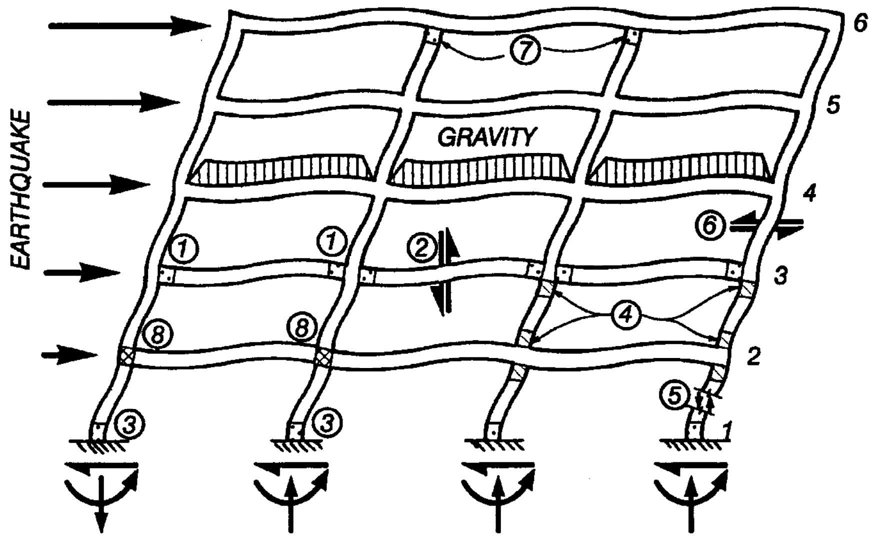


Fig. 5. Critical regions of a ductile frame designed for seismic resistance.

In the assessment of the flexural strength relying on the tensile strength of the top beam reinforcement, realistic allowance should be made to the contribution of bars placed in the floor slab when this can act as a tension flange. This is of considerable importance when assessing the required shear strength of the beam and the flexural strength of the supporting columns.

8.2 The Shear Strength of Beams

In order to avoid under any circumstances the possibility of encountering during an earthquake a shear failure, the shear force associated with the development of two plastic hinges in each span (Fig. 42) with overstrength, needs to be evaluated (see (2) in Fig. 5). This simple example of the concept of capacity design is now incorporated in several codes. To the earthquake induced shear force those due to the simultaneously acting gravity loads must be added. When determining the necessary shear reinforcement, conservative assumptions with respect to shear transfer mechanism in the plastic regions of the beam should be made in recognition of the expected deterioration of the shear resistance in these regions resulting from imposed cyclic reversing inelastic deformations (Section 15.1.4).

8.3 Column Design

8.3.1 At level 1. The initial analysis furnished all the data required to design each column at the base of the frame. A redistribution of design moments from columns subjected to earthquake-induced tension to those under compression offers distinct advantages. At this level the formation of plastic hinges in all columns is to be expected (see (3) in Fig. F5). Some aspects of the detailing of this region is considered in Section 15.2.1.

As a rule, the development of a plastic hinge in first storey columns with uniform reinforcement over the storey height is not expected at the top end of these columns. However, under large ductility demands on the beams, significant elongation of those must be expected, as discussed in Section 10 and shown in Fig. 15. Therefore the detailing at the top end of these columns should be the same as at the base.

8.3.2 Above level 2. For the purpose of the design of columns in the upper storeys, the results of the initial analysis for gravity loads and earthquake design forces become *largely irrelevant*. The moments, M_E, derived for the earthquake design forces only, are used only as reference values that are subsequently magnified.

In accordance with aims of capacity design, the strength of the columns must be in excess of that of the beams, chosen to be the *weak links* of the chain of resistance. The necessary strength enhancement, illustrated in Fig. 6, is achieved in two steps. This significant increase of the flexural strength of columns is recommended to ensure that *plastic hinges will not form* in the columns above level 2 of the frame.

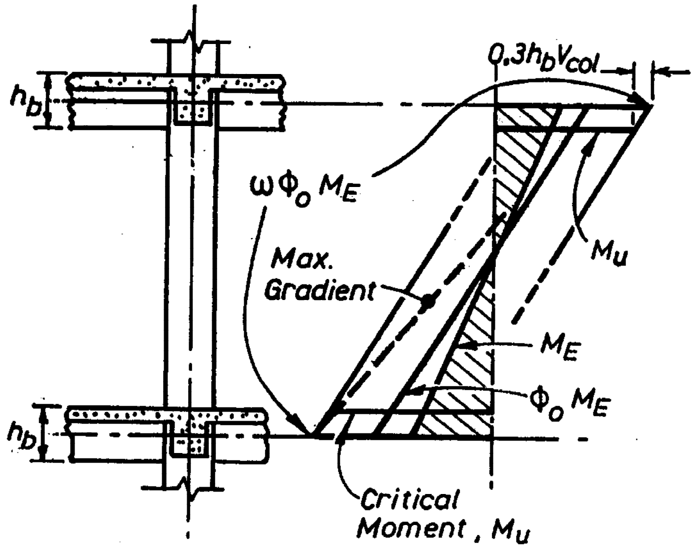


Fig. 6. Moment magnification for columns in the upper storeys of a frame.

Firstly the column must be capable of absorbing the moment input from adjacent beams at overstrength. This is achieved with the use of beam overstrength factor, ϕ_o, as reviewed in Section 7.3.2. It corresponds to the proportional magnifications in Fig. 6 of the moments M_E, derived from the application of the earthquake design forces only.

Secondly, the moments at each end of a column must be increased further independently by a dynamic magnification factor, ω. This is in recognition of the fact that during the elasto-plastic dynamic response of a framed building, the column moment patterns, variable with time, will be very different from those derived for the elastic structure. In more familiar terms this manifests itself in movements over the height of the location of the point of contraflexure, or by the varying distribution between the columns above and below a beam of the total moment input from those beams. The final moment, to be considered in determining the nominal flexural strength, suitably adjusted to correspond with the critical section of the column, as shown in Fig. 6, is thus $\omega\phi_o M_E$. Such locations are shown at (4) in Fig. 5.

8.3.3 Design axial forces. For the determination of the necessary reinforcement, the knowledge of the maximum probable earthquake and gravity induced axial loads, shown at (5) in Fig. 5, are also required in addition to the magnitudes of the bending moments derived in the previous step. The estimation of the maximum likely earthquake induced axial forces in compression or tension, must be based on the summation of the corresponding shear forces at all levels above the level under consideration, developed in the beams at overstrength. As the number of storeys increases, some allowance may be made for the reduced probability that plastic hinges in beams with overstrength will develop simultaneously at all levels. The proportioning of the column sections may now proceed.

8.3.4 Design shear forces. As Fig. 6 records, the magnified large design moments at the ends of the column will not occur simultaneously. Therefore the estimation of the column design shear force, shown at (6) in Fig. 5 is best based on a conservatively assumed moment gradient, as shown in Fig. 6. Typically in one-way frames the nominal shear strength of the column, taking the sense of axial load into account, is recommended (Standards New Zealand, 1995) to be $V_{col} = 1.3\phi_o V_E$.

The design shear force for columns in the first storey should be based on the flexural overstrengths that could be developed simultaneously at both ends of these columns.

8.3.5 Top storey columns. As (7) in Fig. 5 implies, the formation of plastic hinges in these columns, when convenient and appropriately detailed, is not objectionable.

8.4 Beam-Column Joints

The final task in the capacity design of ductile frames is the design of the connections between beams and exterior or interior columns shown at (8) in Fig. 5. Because for a long time, the criticality of the role of beam-column joints has not been recognized in codes and routine design practice, or because of lack of evidence for joint distress during earlier major earthquakes, the importance of the issue was dismissed, this section is devoted to this topic. By necessity the review is restricted to the behaviour and design of interior beam-column joints.

In spite of determined efforts in synthesizing design approaches, while extending some pioneering work (Hanson and Conner, 1967) there are still differing approaches advocated in countries, such as United States, Japan and New Zealand, where the major relevant early research contributions originated.

8.4.1 Design criteria for joints. Only in exceptional situations do joints become critical structural elements under the actions of gravity loads. This contrasts with conditions prevailing under the actions of earthquake forces. To ensure the satisfactory performance of beam-column joints, the following design criteria, summarised in Fig. 7, should be considered:

(a) The strength of the joint should be adequate to meet the maximum demand corresponding to the chosen plastic mechanism for the frame. A joint failure, such as shown in Fig. 7(a), should be precluded.
(b) The capacity of the column, of which the joint is an integral part, should not be jeopardized by possible strength degradation within the joint core (Fig. 7b)).
(c) Because both shear and bond mechanisms represent *inferior energy dissipating systems,* the response of joints should remain predominantly within the elastic domain.
(d) Joint deformations should not significantly increase storey drift. As Fig. 7(c) suggests, significant interstorey deflection may occur because of bond-slip of large diameter beam bars within the joint core, well before the nominal strength of the beams is approached. This will jeopardize the requirements of serviceability criteria.
(e) The reinforcement necessary to ensure adequate joint strength should not cause undue construction difficulties. (Fig. 7(d)).

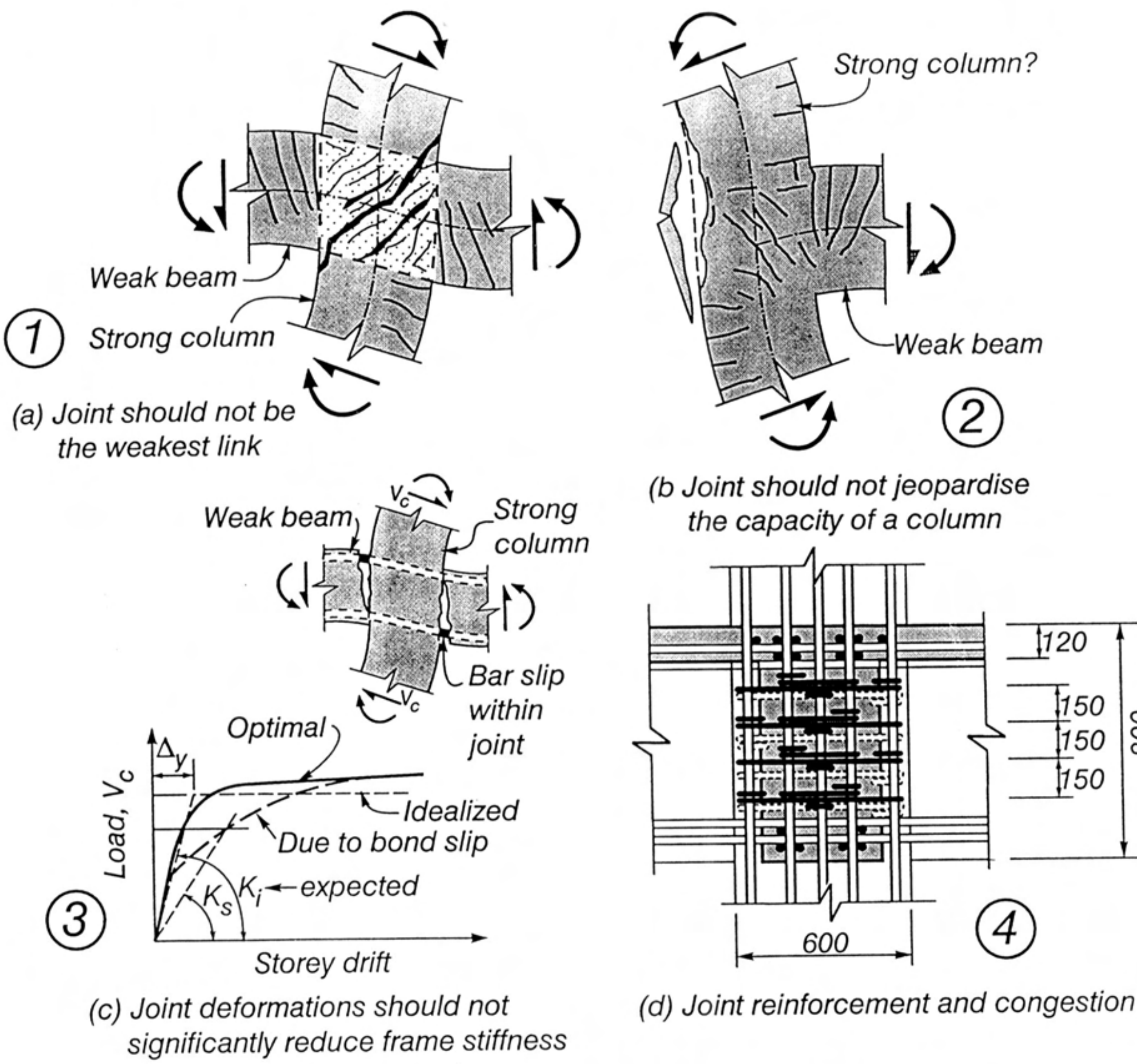


Fig. 7. Illustrations of the criteria for the design of beam-column joints.

There are two major actions that need to be considered in joint design, the *shear strength of the joint core* and the *anchorage by bond of the beam* and *column reinforcement* passing through the joint.

The recommendations given in the following sections are relevant to frames designed for expected ductility demands in the range of $3 < \mu_\Delta < 6$. For structures of restricted ductility (Section 11) certain relaxations in the joint design requirements may be considered.

8.4.2 Joint shear strength. Actions in equilibrium introduced to an interior joint by means of plastic hinges in adjacent beams are modelled in Fig. 8(a). The tensile forces introduced by the two beams, T_1 and T_2, may be associated with strain hardening. The relative values of the corresponding compression forces in the beams introduced by concrete, C_{c1} and C_{c2}, and by compression reinforcement, C_{s1} and C_{s2}, will depend on the efficiency of the anchorage of the beam reinforcement within the joint core. The traditional assumption that concrete and steel strains in fibres of a beam section are identical, is generally violated to various degrees. Because the column shear force, V_{col}, is known, the horizontal joint shear force is readily found to be

$$V_{jh} = T_1 + C_{c2} + C_{s2} - V_{col} = T_1 + T_2 - V_{col} \tag{4}$$

Similar considerations allow the vertical joint shear force, V_{jv}, also to be determined. The severity of the horizontal joint shear force, V_{jh}, will be appreciated if it is realized that it is 4 to 5 times as large as the column shear force, V_{col}.

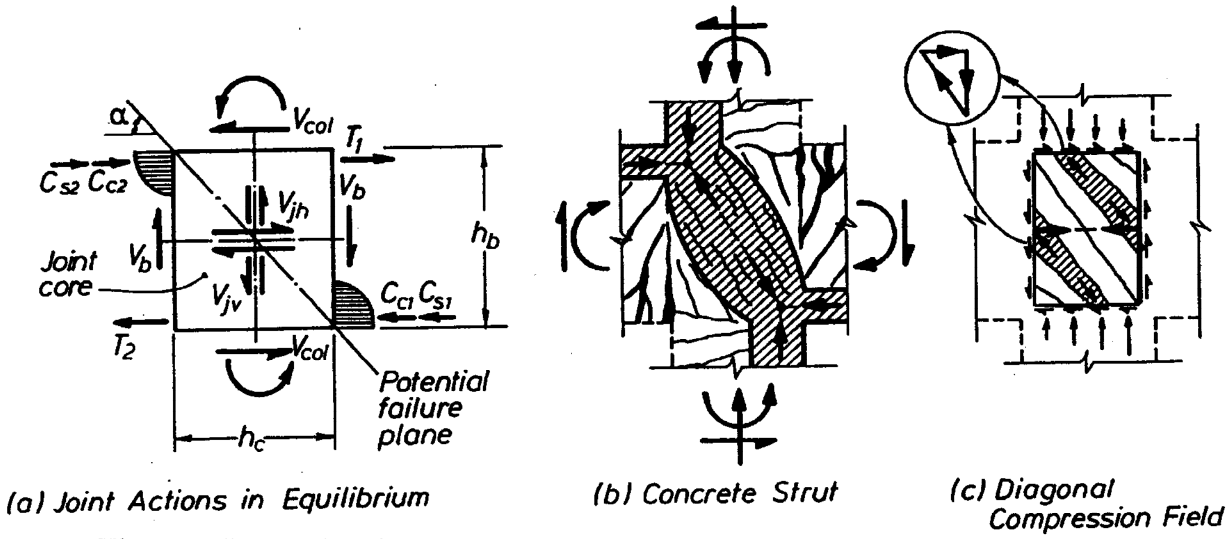


Fig. 8. Internal actions in equilibrium at an interior beam-column joint and joint shear resisting mechanisms.

Two simple models of shear resistance have been suggested (Park and Paulay, 1975) and these are shown in Fig. 8. In one model (Fig. 8(b)) all concrete compression forces and some forces transmitted by bond from the beam and column reinforcement in the shaded area of the concrete core, close to the diagonally opposite corners of the joint, together with the beam and column shear forces, may be equilibrated by means of a single robust diagonal concrete strut. Theoretically no reinforcement in the joint core is required to sustain this mechanism. The remaining horizontal forces introduced to the joint originate from bond transfer from beam and column bars to the core concrete. This shear flow can be sustained by a variable angle diagonal compression field shown in Figure 8(c). For this mechanism both horizontal and vertical joint shear reinforcement is necessary. In the presence of significant axial compression load on a column, the role of the vertical joint shear reinforcement may, at least in parts, be replaced by vertical concrete compression stresses at the horizontal boundaries of the joint core. The share of each of these mechanisms (Fig. 8) in the total joint shear resistance represented a *major challenge* to researchers for the past 20 years. In some codified empirical approaches criticality is expressed by nominal joint shear stresses, for which upper limits are set. Irrespective of the magnitudes of joint shear forces, the required horizontal joint reinforcement is related to that specified for the end regions of the columns above and below the joint, that is, beyond the boundaries of the joint. While the average joint shear stress is a useful index, it is suggested that a design approach based on mechanisms, such as shown in Fig. 8, and relevance to the magnitude of the joint shear force, is more rational and transparent. Moreover, this approach assists in developing innovative solutions for unusual situations not catered for in codes.

8.4.3 Bond behaviour. As Fig. 8(a) implies, the total force from the top beam reinforcement, $T_1 + C_{s2}$, needs to be transferred by means of bond to the joint core. When this is expressed in terms of traditionally used average bond stress over the depth of the column, h_c, it is found that the resulting stress intensity is well beyond values implied by generally accepted code specifications. It is evident that the control of bond in beam-column joints is a complex and difficult issue. Because of the large number of variables, the majority of which are significantly affected by inelastic excitations imposed by an earthquake, no experimentally verifiable analytical techniques are known to exist that could reliably predict bond response. Based largely on observations made during numerous tests with beam-column joints satisfying specific structural seismic performance criteria, and the study of bond response of deformed bars under reversing cyclic loading (Eligehausen *at al.*, 1983, Fillippou *at al.*, 1983), "acceptable" anchorage behaviour was found to be assured if the diameter of bars, d_b, passing through a joint, as in Fig. 8, is limited in this form

$$\frac{d_b}{h_c} < 5.4\,\frac{\xi_p\,\xi_t\,\xi_f}{\xi_m\,\lambda_o}\,\frac{\sqrt{f_c'}}{f_y} \qquad (5)$$

where the factors, ξ, make allowances for the following effects:

(a) The beneficial effect on bond of concrete compression stress, acting transversely to an embedded bar may be estimated from

$$\xi_p = \frac{P_u}{2f_a' A_g} + 0.95 \qquad (6)$$

with the limitation that $1.0 \le \xi_p \le 1.25$, and where P_u is the design axial compression load on the column above the joint at the ultimate limit state and A_g is the gross area of the column section.
(b) The detrimental effect on bond by sedimentation of and water gain in fresh concrete is recognized by assuming $\xi = 0.85$ for a top beam bar where more than 300mm of fresh concrete is cast below the bar. For all other cases $\xi = 1.0$.
(c) When beam bars pass through an interior joint of a two-way frame, the concrete around the beam bars may be subjected to large tensile strains transverse to the bars considered. Therefore, for two-way frames $\xi_f = 0.85$ should be assumed, while for one way frames $\xi_f = 1.0$.
(d) To allow for more severe bond conditions for the smaller of the bottom or top beam reinforcement with area A_s', passing through the joint and subjected to compression at a section

$$\xi_m = 2.55 - \frac{A_s'}{A_s} \qquad (7)$$

with the limitation of $0.75 \le A_s'/A_s \le 1.0$, where A_s is the area of the total tension reinforcement at the section. For beam bars that are part of A_s, $\xi_m = 1.55$. This parameter estimates the magnitudes of maximum steel stresses that are associated with the tension and compression forces T_1 and C_{s2}, respectively, shown in Fig. 8(a).
(e) The steel tension stress developed at overstrength is gauged by the factor λ_o (Section 7.3.1), which is usually of the order of 1.25. When plastic hinges are located away from the face of a column, so that this section remains elastic (Fig. 19), $\lambda_o \le 1.0$ may be assumed.

Equation (5) was based on average bond stresses over the length h_c of $u = 1.35f_c'$ (MPa), with local maximum shown in Fig. 9(c) approaching the experimentally found peak value of $2.5f_c'$ (MPa).

Gross violation of this recommendation can be expected to lead to excessive bond-slip within the joint. This will reduce frame stiffness well before first yield is encountered (Fig. 7(c)). Hence essential serviceability requirements are likely to be violated. It should also be noted that the restoration of good bond in joints by repair is generally beyond economic constraints.

It is suggested that acceptable anchorage behaviour will ensure that frame deformations due to bond-slip are negligible during elastic frame response, and that no significant loss in energy dissipation due to excessive bond-slip will occur till the expected maximum local ductility demand has been exceeded. Such bond performance was demonstrated in a test unit, the hysteretic response of which is shown in

Fig. 2. Eventual bond breakdown was encountered in this test, but only after a second displacement cycle corresponding to a drift index of 4%.

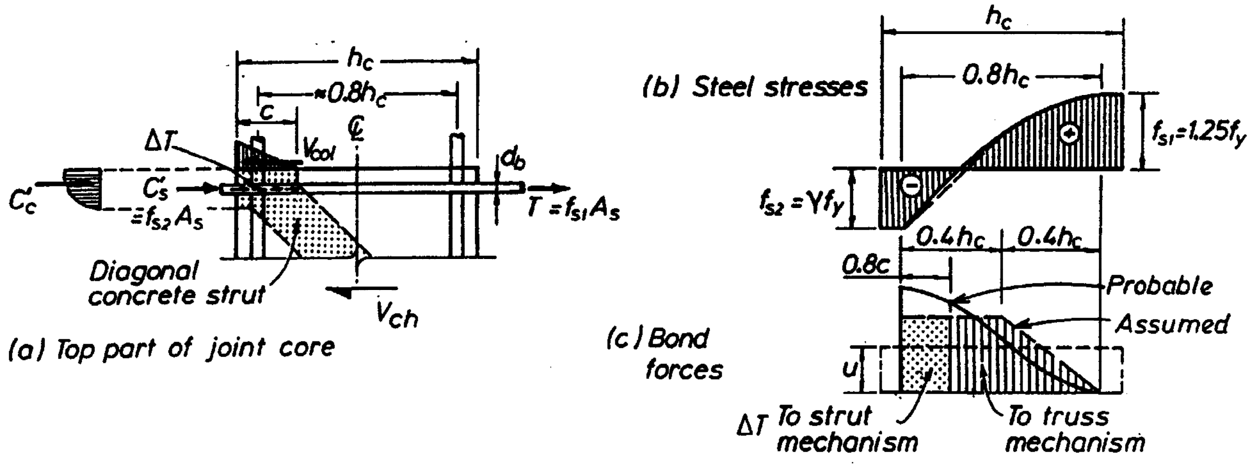


Fig. 9 Internal actions associated with the strut and truss mechanisms in an internal beam-column joint.

8.4.4 Joint shear reinforcement. Having determined the total horizontal joint shear force, and by estimating the joint shear force transmitted by the strut mechanism shown in Fig. 8(b), the joint shear resistance assigned to the truss mechanism (Fig. 8(c)), requiring tension reinforcement, is established. The variable distribution of bond forces, and consequent concrete shear stresses introduced to the core, along the beam bars passing through the joint influence the relative contribution to shear resistance of these two mechanisms. As Fig. 9(c) implies, horizontal shear stress distribution should not be assumed to be uniform. Figure 9, depicting internal actions at the ultimate state, taking into account also probable effects of load reversals and local bond deterioration, summarizes the assumptions on which the design approach is based. Figs. 9(b) and (c) show typical distributions of steel stresses and bond forces along the top beam bars, after a number of moment reversals have occurred in adjacent beam plastic hinges. It is assumed that in the top part of the joint (Fig. 9(a)) the sum of the concrete compression stresses, C_c', the column shear force, V_{col}, and a fraction, ΔT, of the total bond force $T + C_s' = (1.25 + \gamma)f_y A_s$, make up the horizontal componet, V_{ch} of the compression force in the diagonal concrete strut (Fig. 8(b)). The remainder of the bond force, i.e. $V_{sh} = T + C_s' - \Delta T$, is assigned to the truss mechanism. This approach allows then the area of the required *horizontal joint reinforcement* to be calculated as a fraction of the area of the total beam flexural reinforcement, A_s, (Fig. 9(a)) thus

$$A_{jh} = \frac{V_{sh}}{f_{yh}} = \left(\frac{6v_{jh}}{f_c'}\right) \alpha_j \frac{f_y}{f_{yh}} A_s \tag{8}$$

where the beneficial effect of any axial compression load, P_u, on the column of a one-way frame is taken into account by the factor $\alpha_j = 1.4 - 1.6 \{(P_u/(f_c'A_g)\}$. A limitation for the joint shear force to be resisted by the horizontal joint shear reinforcement with area A_{jh}, is that $0.4 < V_{sh}/V_{jh} < 0.8$. The role of the shear stress, v_{jh}, in Eq. (8) is reviewed in the next section.

Similar considerations lead to the requirements of *vertical joint shear reinforcement* with total area

$$A_{jv} = \alpha_v \frac{h_b}{h_c} f_c' A_{jh} \frac{f_{yh}}{f_{yv}} \tag{9}$$

where $\alpha_v = 0.7/\{1 + P_u/(f_c' A_g)\}$ (10)

When intermediate column bars, as seen in Fig. 47, pass through the joint, this reinforcement, not being highly stressed when columns are made significantly stronger than beams, can be considered to be part of A_{jv}. Hence in such common cases very little, if any, additional vertical joint shear reinforcement is required.

8.4.5 Joint shear stresses. To gauge the relative severity of joint shear demands, it is convenient to evaluate the average horizontal joint shear stress, without attaching any physical meaning to it, from

$$v_{jh} = \frac{V_{jh}}{b_j h_j} \tag{11}$$

where b_j and h_j define, as Fig. 10 shows, the joint area to be considered effective.

This shear stress is a measure of the diagonal concrete compression stresses generated in the joint core. Because the concrete in this region, extensively cracked in both diagonal directions, is subjected to significant transverse tensile strains, its compression strength must be expected to be significantly less than f_c'. To avoid a premature diagonal compression failure, it is recommended that in fully ductile systems the limit $v_{jh} \le 0.2f_c'$ be observed.

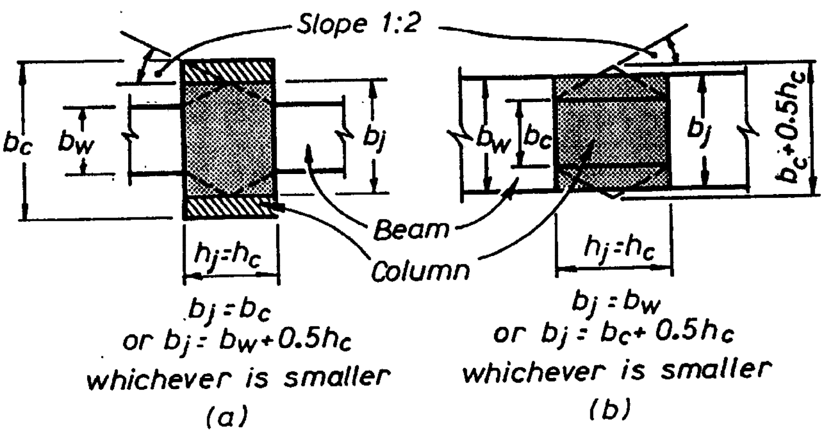


Fig. 10 Assumptions for effective joint dimensions.

Equation (8) was developed assuming that $v_{jh} = f_c'/6$. A restriction or a relaxation, respectively, in the control of transverse tensile strains was considered prudent for nominal shear stresses larger or smaller than $f_c'/6$. This intent is quantified by the term $6v_{jh}/f_c'$ in Eq. (8).

It is emphasized that the primary role of transverse reinforcement in beam-column joint is to *resist shear* by means of sustaining a diagonal compression field as implied by Fig. 8(c). There is a widespread unsubstantiated notion that joints require confinement as do the end regions of columns, where the formation of plastic hinges with significant curvature ductility demands may impose large compression strains on the concrete. With the intention to restrict joint response to the elastic domain and the joint shear limitations stated above, concrete compression stresses will seldom exceed $0.4f_c'$. Consequently compression strains will be moderate and the need to confine the concrete core does not arise.

9 THE EFFECT ON SEISMIC DESIGN OF GRAVITY LOADS

The choice of plastic mechanisms associated with ductile response of reinforced concrete frames, such as shown in Fig. 3, is also affected by the relation of the actions resulting from earthquake design forces to gravity load induced actions.

9.1 Earthquke Dominance

Most publications address frames in which the dimensioning of beams is governed by the specified combination of the effects of factored gravity loads and prescribed earthquake design forces, rather than those due to appropriately factored gravity loads only. Such frames may be defined as being *earthquake dominated*. Maximum bending moments in continuous beams are usually encountered at the faces of the columns. Hence the familiar mechanisms, such as seen in Figs. 3(a) and (b), represent rational choices.

In long span beams, where gravity load induced moments may be relatively large, it may be difficult or even impossible to develop a plastic hinge at the face of the column with the bottom beam reinforcement yielding in tension. A critical section with large combined positive moments may necessitate the formation of the plastic hinge in the span, as shown in Fig. 13(a). Two aspects should be considered when this situation is encountered:

(a) Whereas plastic hinge rotations at both ends of earthquake dominated beams with relatively short spans are of the same order as those due to interstorey deflections, they are magnified when one or both plastic hinges develop in the span away from the columns (Fig. 19). Section 11.4.1 describes how this phenomenon should be accounted for.
(b) When storey displacements, opposite to those leading to the shaded moment diagram shown in Fig. 13(a), are imposed on a frame, the two plastic hinges which will develop in such beams will be at locations different from those shown by the full circles in Fig. 13(b). Under reversing ductility demands, inelastic hinge rotations at four locations will then accumulate leading to increased beam deformations, as shown in Fig. 13(c). Further aspects of this phenomenon are discussed in Section 10.

9.2 Gravity Load Dominance

In medium to low rise frames, particularly with relatively long span beams, the strength of the latter is often governed by the factored gravity loads only. A typical frame of this type, together with representative beam bending moments, is shown in Fig. 11.

In such *gravity load dominated frames*, the enforcement of beam mechanisms, such as seen in Figs.3(a) and (b), would require columns with excessively large flexural strength. Moreover, at the development of two plastic hinges in each beam span, the lateral force resistance of such a frame might be much greater than that intended. In such cases a *mixed sidesway mechanism* may be chosen, provided that with few exceptions precautions are taken to prevent the development of a "soft storey" where plastic hinges could develop simultaneously at both ends of all columns in a storey.

In such situations concepts relevant to the seismic response of frame-wall systems, such as shown in Fig. 3(e), can be exploited. With adequately designed walls, plastic hinge formation can be restricted to the wall base, with regions of the walls above the first storey designed and detailed to remain elastic. Walls so designed will then ensure that a "soft storey" cannot develop in the system. With this assurance it becomes immaterial whether the plastic mechanisms in the frames develop only in beams, only in columns, or in both beams and columns. The detailing of plastic hinges in beams is, however, a little easier than those in columns. Relatively stiff walls will ensure that storey drifts and hence plastic rotations in frame members remain moderate and within the curvature ductility capacities achieved by the appropriate detailing of potential plastic hinge regions.

Suitable plastic mechanisms for gravity load dominated frames are shown in Figs. 11(a) and (c). Plastic hinges are accepted at both ends of all *interior columns* because it would be difficult to design these columns above and below floor level to be capable of absorbing moments at overstrength from two beams, the design strength of which was strongly dominated by gravity load demands. At an exterior beam-column joint, two column sections are available to resist the moment input from only one beam hinge. Therefore, it is feasible to provide exterior columns with sufficient strength to ensure that plastic hinges will not form in them. If necessary, the flexural capacity of beam ends at exterior columns can be deliberately reduced by utilizing some inelastic moment redistribution in the beams.

Preventing a soft storey mechanism in a frame (Fig. 3(c)) may thus be achieved by using two exterior columns designed to remain elastic. Details of this approach are briefly reviewed here with the aid of Fig. 11. The reliability of the controlling role of these columns in a given storey will depend on their flexural strength in relation to the overstrength of all interior column plastic hinges in the same storey. This strength comparison can be conveniently quantified by a relation between the sum of the storey shear forces sustained by the elastic columns, ΣV_e, and those of the interior columns with plastic hinges, ΣV_p. A suitable ratio is that of the storey shear capacities of exterior columns to those of all the columns thus

$$r_v = \frac{\Sigma V_e}{\Sigma V_e + \Sigma V_p} \tag{12}$$

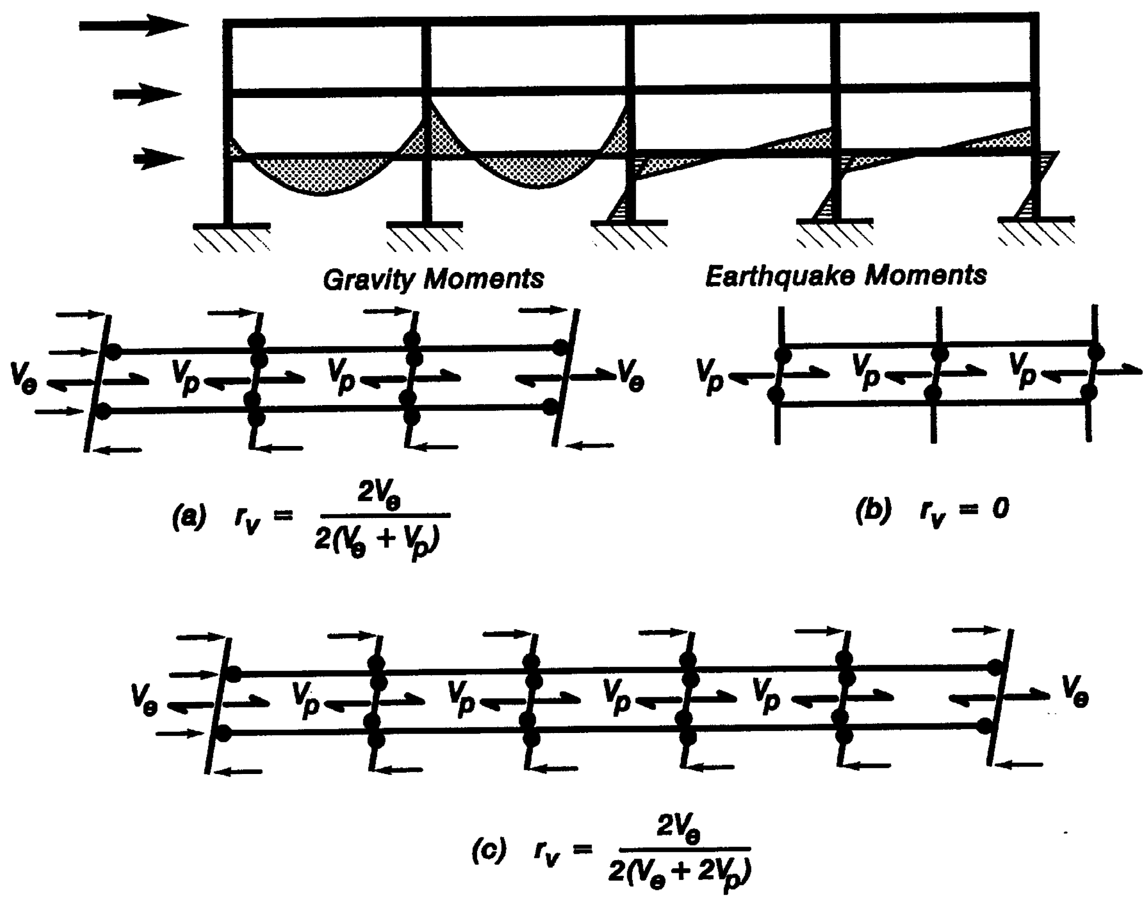


Fig. 11. Ductile frames with hinging columns.

The larger the expected displacement ductility demand, μ_Δ, in a given storey, the higher the flexural overstrength of plastic hinges of interior columns and the more important the reliability of the exterior columns in remaining elastic.

It has been suggested (Standards New Zealand, 1995) that the ductility capacity of a system with hinging columns be estimated from

$$\mu_\Delta \le 12 r_v \le 6 \tag{13}$$

Thus with $r_v = 0.5$ a ductility capacity of 6, generally assumed as a maximum in regions of severe seismicity, can be used. A much smaller ratio, such as $r_v \approx 0.21$ may be accepted for systems expected to be exposed to restricted ductility demands, such as $\mu_\Delta = 2.5$. Such systems are examined in some detail in Section 11.

In practice the overstrength of the interior columns, V_p, forming the weak links of the chain of resistance and hence designed first for the specified earthquake induced storey shear force, will be known. The sum of the minimum strength of the exterior columns, quantified by ΣV_e, needed to ensure that under the assumed ductility demand, μ_Δ, the intended mechanism is maintained, is then to be found. For this purpose Eq. (12) may be inverted, and combined with Eq. (13), giving

$$\Sigma V_e \ge \frac{\mu_\Delta}{12-\mu_\Delta} \Sigma V_p \tag{14}$$

For a frame with five bays, as shown in Fig. 11(c), a ductility capacity of only $\mu_\Delta = 4$ should be relied on if the strength of the exterior elastic column is $V_e = V_p$. A column strength increase of $V_e = 2\,V_p$ would need to be satisfied for $\mu_\Delta = 6$. This suggests the beneficial role of wall-like elements interacting with such frames. When only a limited ductility, corresponding to $\mu_\Delta = 3$ is required, it is found from Eq.(14) that for the five bay frames $V_e \ge 0.57 V_p$ is sufficient.

The overall strength requirement must of course be satisfied, whereby the sum of all column shear forces in a storey, $\Sigma V_e + \Sigma V_p$, must not be less than the total design storey shear force amplified by the overstrength factor, ϕ_o. In optimal design for seismic actions, the value of ϕ_o is of the order of 1.5. If it is larger, because of unavoidable built-in reserve strength of the system, a reduction in the anticipated ductility demand for the seismic event can be expected, and hence allowed for. This aspect is further considered in Section 11. Moreover, an elastic exterior column must always be capable of absorbing the moment input from the plastic hinge in the adjacent beam in accordance with Section 8.3.2. As Fig. 11(b) indicates that for a "soft storey", which is to be avoided, $r_v = 0$.

The examples in Fig. 11 illustrate in each case one storey in a single multi-storey frame. However, the expressions given above, to assess the required strength of elastic columns, V_e, are also valid for a complete framing system. For example the two exterior frames in the building shown in Fig. 38, being earthquake dominated, could be conveniently designed to develop plastic hinges in beams only with all columns at level 2 and above remaining elastic. The strength of the interior frames may be dominated by gravity loads and hence all columns of those frames may be chosen to develop simultaneous plastic hinges at both ends. Thus in this example system there will be 10 elastic and 9 hinging columns, the latter with appropriately evaluated overstrengths corresponding to ΣV_p. Equations (12) and (14) will indicate that with the relatively large number of elastic columns, large displacement ductility capacity could be relied on while accepting column mechanisms for all interior frames. Systems, such as shown in Fig. 11, should be particularly attractive in regions of moderate seismicity.

It should be noted that all potential plastic hinges in columns have to be carefully detailed and provided with adequate transverse reinforcement the same way as for columns at foundation level, shown in Figs. 3(a), (b) and (d).

Reinforced concrete members, particularly beams and walls, increase their length after the onset of extensive cracking. The phenomenon has negligible effect on elastic response. However, after the formation of plastic hinges, such as shown in Fig. 12, significant elongations may occur. Related phenomena are not known to have been addressed in codes.

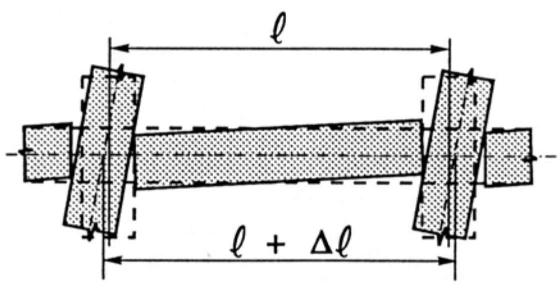


Fig. 12. Beam elongation associated with the formation of plastic hinges.

The magnitude of longitudinal expansion in the plastic hinge regions depends primarily on the curvature ductility demand. Significant fractions of the inelastic steel tensile strains are not recoverable because of imperfect crack closure [Fenwick *et al.*, 1991]. Some elongation follows from the idealized geometrical conditions illustrated in Fig. 12, which suggests that, for a given inelastic interstorey drift, elongation from this source is approximately proportional the member depth. Moreover, up to $\mu_\Delta \approx 6$, elongations due to non-recoverable steel strains increase with repeated reversible plastic hinge rotations, that is, as a result of cumulative ductility demands.

Beam deflections and consequent elongations may be more severe when plastic hinge rotations are not reversible during the seismic response. This can occur, as mentioned earlier, when gravity loads are significant and hence plastic hinges need to develop within a span, as shown in Fig. 13(a). Upon load reversals moment demands in these regions become small and hence these zones remain elastic. Therefore there is negligible inelastic strain recovery (Fig. 13(b)). Because inelastic strains are unidirectionally cumulative under reversing seismic response, plastic hinge rotations and hence beam deflections (Fig. 13(c)) as well as elongations (Fig. 13(d)) continue to increase with progressive ductility demand for the duration of the inelastic seismic response.

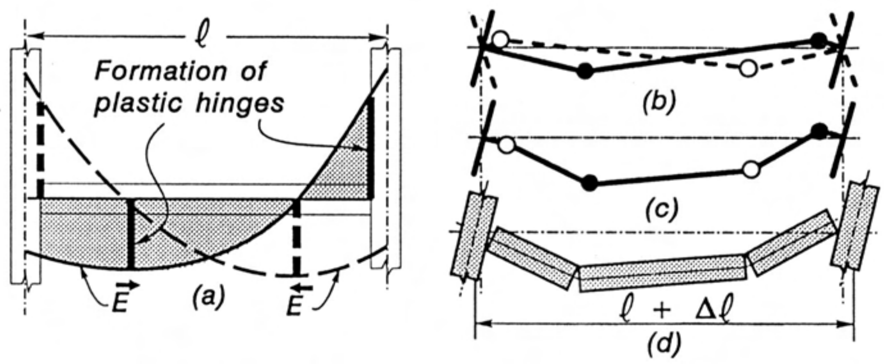


Fig. 13. Cumulative inelastic deformations with non-reversible plastic hinges.

The phenomenon has been studied and experimentally verified (Fenwick *et al.*, 1991, Fenwick and Megget, 1992, Booth, 1994) and some results of interest are shown in Fig. 14. It compares strains measured over the top and bottom layer of reinforcement in a 500 mm (19.7 in) deep beam. The strains within the plastic hinge, measured over a distance 84% of the beam depth, indicate elongations over this length. A comparison of the response of plastic hinges with unidirectional and reversing rotations show comparable elongations at large imposed ductilites, which may be as large as 2.5% of the beam depth. It may be noted that for a fully ductile frame with an estimated displacement ductility demand of $\mu_\Delta \approx 6$, the member ductility, μ, in several beams may well be 8.

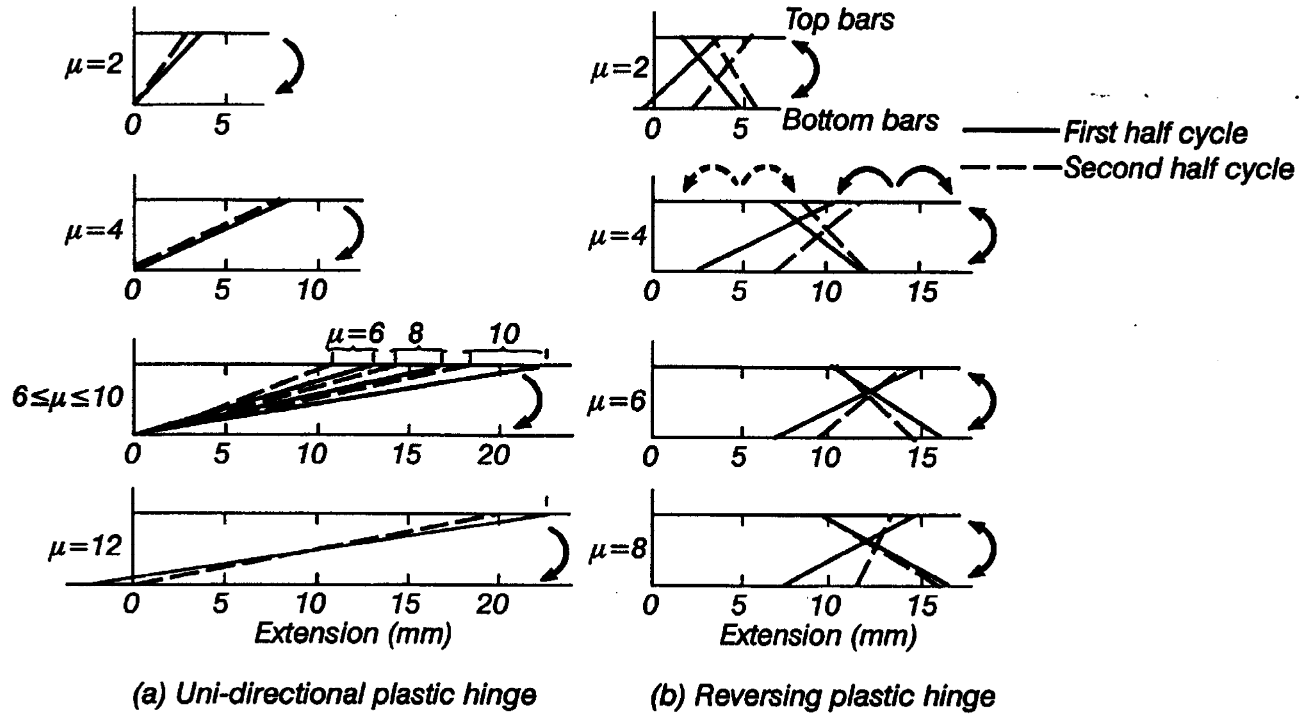


Fig. 14. Longitudinal strains in unidirectional and reversing plastic hinges leading to beam elongations.

Figure 15 shows the consequences of beam elongations. Typical locations of plastic hinges in "strong column-weak beam" frames are shown in Fig. 15(a). Figure 15(b) shows plastic hinges that could be expected in the same frames after a number of inelastic displacement reversals with due allowance for beam elongations. The most severe effect occurs in the first storey columns of a multistorey frame. The total elongation of the continuous first floor beam is proportional to the number of plastic hinges with two hinges in each span. Therefore, the displacement of the top end of columns relative to the base changes with the distance of the column measured from approximately the centre of the frame. For design purposes the following features are relevant:

(1) Additional plastic rotations are imposed at the base of some of the columns, while such rotations are relieved in other columns.
(2) The total maximum seismic shear force across the first storey depends on the overstrength of the associated complete plastic mechanisms, normally consisting of two plastic hinges in each beam span and a plastic hinge at the base of all columns, as shown in Fig. 15(a). Additional column deformations due to beam elongation, such as seen in Fig. 15(b), have negligible influence on the magnitude of the storey shear force.
(3) With different inelastic deformation patterns in the columns of the first storey, a marked redistribution of moments and hence shear forces in these columns will occur without the total shear force being changed. For example a significant increase of shear forces must occur in columns (3) and (4) of the frame shown in Fig. 15(b). A corresponding reduction in shear force in columns (1) and (2) is to be expected.
(4) The formation of plastic hinges at the top end of some columns (Fig. 15(b)) must be anticipated. Consequently such regions should be detailed for curvature ductility demands similar to those at the column base.

(5) The shear demand on the most severely affected columns could increase to the maximum possible value, which results when a plastic hinge at overstrength develops simultaneously at each end of that column.
(6) In capacity designed ductile frames with strong columns and weak beams, a soft first storey with all columns simultaneously hinging at the top and the bottom will not arise as a consequence of significant beam elongations.
(7) When prefabricated floor systems are used, enlarged seatings and suitable detailing for simply supported floor panels should be provided to prevent loss of support at panel ends, leading to possible collapse of the floor system.

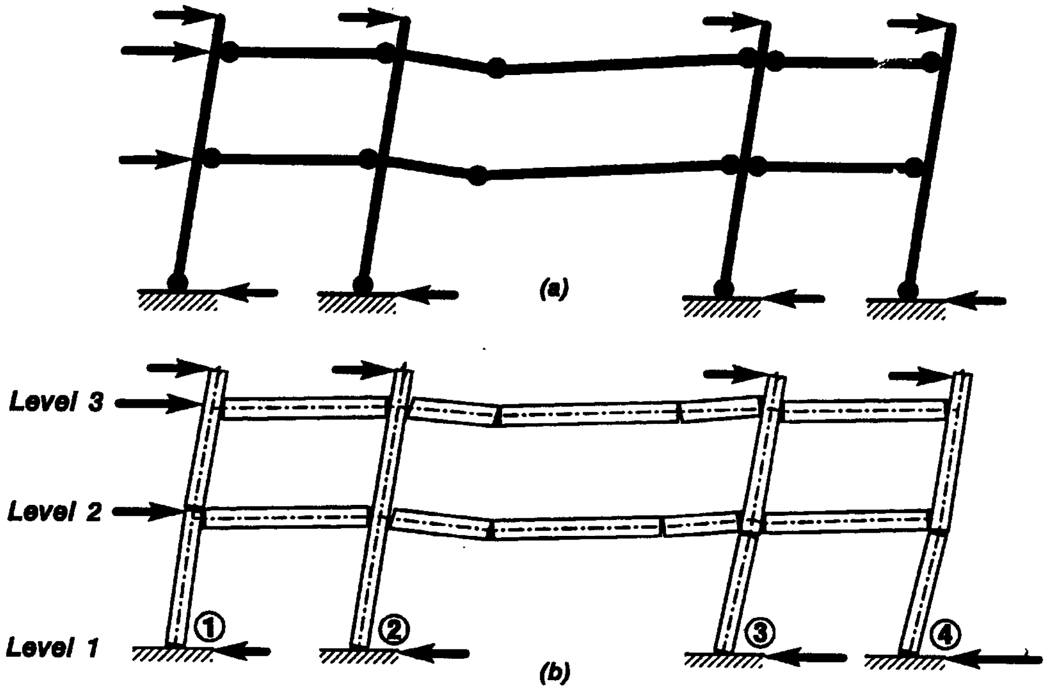


Fig. 15. The effects of inelastic beam elongations on first storey columns.

11 STRUCTURES WITH RESTRICTED DUCTILITY

11.1 Conditions For Restricted Ductility Demands

In many situations earthquakes are unlikely to impose large ductility demands on structures. The range of limit states in terms of ductility that is generally considered in seismic design, is illustrated in Fig. 16. It shows the approximate bands that may be used to define fully ductile, of restricted ductility and essentially elastic response corresponding with displacement ductility factors of the order of 6, 2.5 and 1.25. In this example a single structure with a given stiffness, to define yield displacement, Δ_{yi}, was used. The dashed lines in Figure 16 indicate the likely non-linear response.

Restricted ductility demands may arise in the following situations (Fig. 17):

(1) Inherently a structure may possess strength considerably in excess of the strength corresponding to fully-ductile seismic response. The causes of this may be moderate seismicity in the region, or that other design requirements for strength, such as gravity loading and wind forces, are more critical. The tall structure shown in Fig. 17(e) may be one for which design actions resulting from wind forces would

have been greater than those derived for code specified seismic forces corresponding with full ductility demand. Because earthquake induced displacement in the structure shown in Fig. 17(c) will be controlled by the perforated wall, ductility demands on members of the attached frames will be limited.

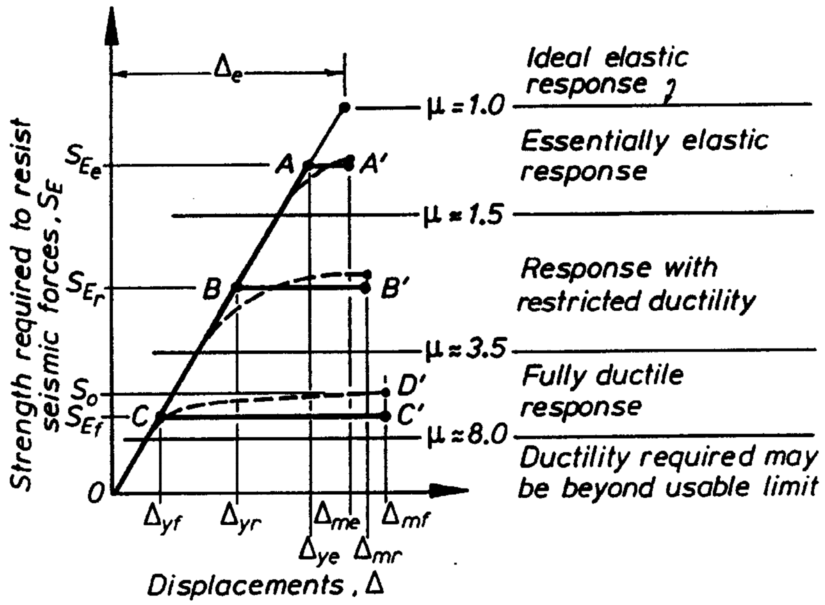


Fig. 16. Relationships between strength and ductility.

(2) In certain circumstances detailing for full ductility may be found to be difficult or too costly. The adoption of larger seismic design forces, to reduce ductility demands during earthquakes, may be a more attractive proposition. Greater economic benefits may well be derived from consequent simplifications in the detailing for reduced ductility and perhaps from adopting less than optimal locations for plastic hinges (Fig. 17(d)).

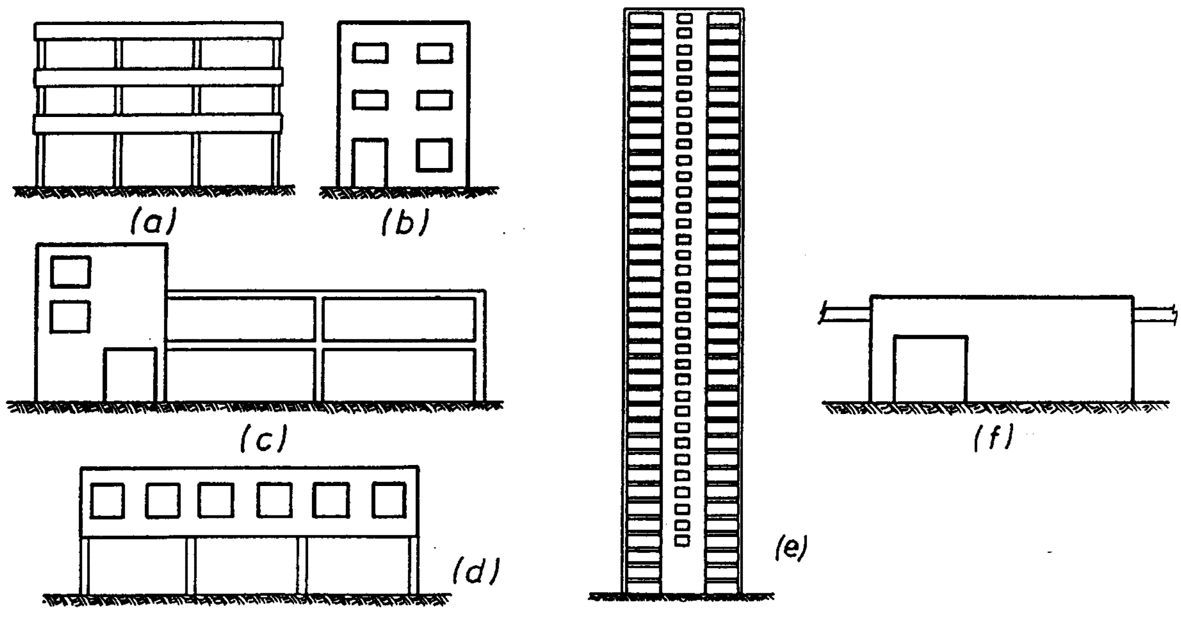


Fig. 17. Examples of structures with restricted ductility.

(3) There are moderate sized structures, the configuration of which do not readily permit a clear classification in terms of structural types. The precise modelling of such structures may often be

difficult. Consequently, the prediction of their inelastic seismic response is likely to be rather crude. However, without incurring economic penalty, a more conservative design approach, relying on increased lateral force resistance and consequent reduction in ductility demands, may be more promising. Examples of such structures are seen in Figs.17(b) and (c).

(4) The overall ductility demand on certain structural systems may need to be reduced in order to protect certain elements from excessive local ductility demands. Examples are those shown in Figs. 17(a) and 18.

11.2 Design Strategy

In terms of the *choice of seismic design procedures*, there are only two classes of structures, those responding elastically and ductile systems. In essentially elastic structures, shown in Fig. 16, inelastic deformations of any significance are not expected. Hence no special detailing for any region, other than that adapted for gravity load conditions, may be required. In ductile systems, as Fig. 16 implies, the magnitudes of ductility demands and hence the need for improved detailing, may vary.

Structures, typically used for buildings, the design of which is intentionally based on *elastic response*, may in fact enter the inelastic range when the seismic event is larger than anticipated. Therefore structures, even in this class, should have at least some reserve ductility capacity. Some details of this aspect of seismic design are discussed in Section 13.

In structures of restricted ductility the development of plastic regions within the structural system *must* be allowed for, as in fully-ductile structures. Therefore, the establishment of a kinematically admissible plastic mechanism in structural systems exposed to limited displacement ductility demands, is essential. The location of these regions should be clearly *identified* to enable the necessary detailing to be provided. It has been suggested that structures with limited ductility should be considered as a separate group of structural system for which simplification in both analysis and detailing of the reinforcement should be adopted. The justification for this is claimed to be that in terms of seismic effects, such structures are seldom critical and hence a high degree of sophistication in their design is not warranted. When the design process for ductile systems with a transparent rationale is made simple, it can be applied with equal ease to this group of structures. The only difference in the design process, that is suggested, is a quantified relaxation of the requirement established for the detailing of fully ductile reinforced concrete structures. The application of capacity design principles will ensure that no serious underestimates, resulting from oversimplified and often arbitrary rules of thumb, will be made of likely local ductility demands in systems which are expected to be exposed only restricted overall ductility.

11.3 Specific Cases

Figure 18 shows mechanisms where "soft storeys" may need to be accepted. This may be based on a conclusion that for such a structure the overall ductility demand, μ_Δ, expressed by the lateral deflection at roof level, Δ_u, will be restricted. It is seen that restricted ductility demands for the mechanism in Fig. 18(a) will indeed result in corresponding reduced plastic hinge rotations. Therefore, if such a structure with a *well conditioned plastic mechanism*, consisting of strong columns and weak beams, is chosen to meet restricted ductility demands, $\mu_\Delta = \Delta_u / \Delta_y$, the rules applicable to the detailing of potential plastic regions for full ductility, may be relaxed. Examples for this are shown subsequently. However, for the same inelastic roof deflection and hence overall ductility demand, much larger plastic hinge rotation will occur in the columns in any of the mechanisms seen in Figs. 18(b) to (d). It may be readily shown that with the notation used in this figure, the column displacement ductility demand, μ_c, is related to the overall ductility, μ_Δ, thus

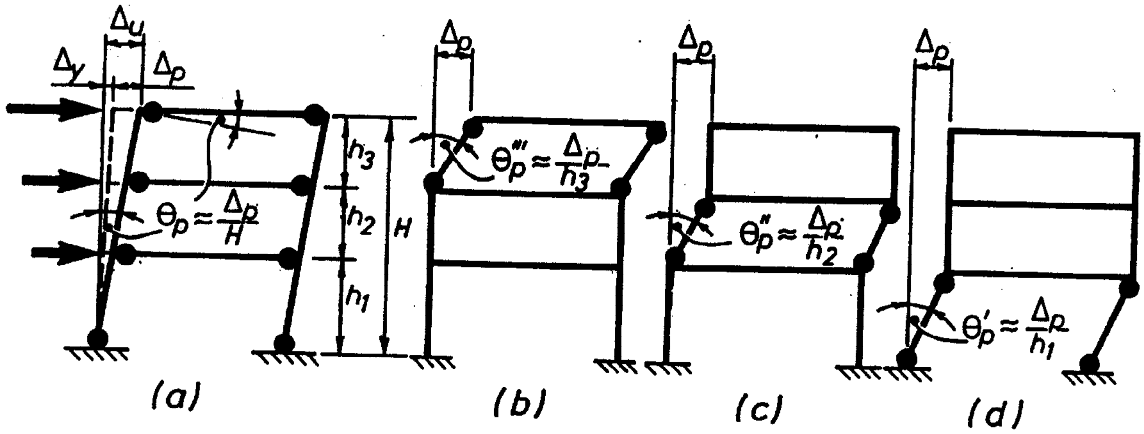


Fig. 18. Sway mechanisms in frames with restricted ductility.

$$\mu_c = 1 + (H/h_i)(\mu_\Delta - 1) \tag{15}$$

For example if a three storey frame with equal storey heights, h = H/3, is to be designed for restricted ductility (Fig. 16) with μ_Δ = 2.5 originating from storey sway mechanisms, the column displacement ductility demand will be μ_c = 5.5. This would then require the detailing of the column plastic hinges as those in fully-ductile systems. The example emphasizes the need for a careful study of possible mechanisms, even in structures designed for significantly reduced ductility demands. This, however, is a relatively simple task.

The principles of capacity design, as outlined for fully-ductile structures, should still be employed to ensure that parts of the structure intended to remain elastic are sufficiently *protected* against possible overload from adjacent plastic regions, and hence against a possible brittle failure. However, significant simplifications may be made in the evaluation of the design actions to be used for the analysis of the potentially brittle elements. Often engineering judgement will be sufficient to estimate the desirable reserve strength to be assigned to such members.

Finally, as a result of the expected reduction in ductility demands on plastic hinges, a relaxation of the detailing requirements for such potential regions is justified. Because of the paucity of actual information, it has been suggested (Paulay and Priestley, 1992) that interpolation, tempered with rational judgement, should be used when finalizing the detailing of the potential plastic regions of structures with restricted ductility. The interpolation should be for the quality of detailing shown between that accepted for elastic structures (μ_Δ < 1.25), that is for gravity load situations, and that recommended in Section 15 for fully-ductile systems (μ_Δ < 6).

Some structures with large irregularities will require gross approximations in analysis to be made. It should be appreciated, for example, that there is a gradual transition from a deep-membered frame to a wall with openings. The size of openings will suggest whether dominant wall or frame behaviour is to be expected. An approach relevant to walls with openings, usually designed to absorb limited ductility demands, is presented in Section 12.5. Once more it is emphasized that these structures are not sensitive to accuracy involved in overall analysis.

Particular difficulties with analyses may arise in irregular dual systems. In such cases it may be convenient to reduce the structure to a primary lateral-force-resisting system, consisting of potentially effective members only. By conceptually excluding in the design certain elements from participation in lateral force resistance, a complex structure may be reduced to a simple system or subsystems of restricted ductility. Elements excluded from the primary system must then be considered as secondary elements, capable of carrying appropriate gravity loads. While lateral force effects on such elements may be ignored, critical regions should be detailed for limited ductility to enable the secondary system to maintain its role of supporting gravity loads when subjected to restricted lateral displacements, which will be controlled by the chosen primary system. Fig. 17(c) shows an example structure for which this approach would be applicable.

11.4 Frames With Limited Ductility Demands

Some examples of frame response influenced by restrictions on both ductility demands and capacities have been considered previously. One class of frames, considered in Section 9.2, is that dominated by gravity loads. Another group, reviewed in Section 11.3, is that where, in spite of moderate system ductility demands, exceptionally large local ductility demands may arise due to the development of a "soft storey".

11.4.1 Beams with plastic hinges in the span. A feature, common when gravity loads on beams are significant, is that plastic hinges may not form at column faces only (Fig. 13). In such cases the reduction in the overall ductility demand, μ, assigned to the structural system should be considered even when the mobilization of a strong column-weak beam mechanism is assured. The anticipated rotation of such plastic hinges, θ_b, shown in Fig. 19, may be estimated from the rotation of the beam-column joint, θ_c, when it is amplified by the factor $m_\ell = \ell/\ell^*$. The overall ductility of the frame may be approximated, rather crudely, by the angle θ_c. Hence the local ductility demand, μ_m, relevant to beams, such as shown in Fig. 19, may be taken as

$$\mu_m \approx m_\ell \mu \tag{16}$$

The implications are that :

(a) When $\mu_m = 6$, the maximum usually envisaged for fully ductile frames, the assumed system ductility should be limited to

$$\mu \leq \frac{6}{m_\ell} \leq 6 \tag{17}$$

This may necessitate increased earthquake design forces to be used.

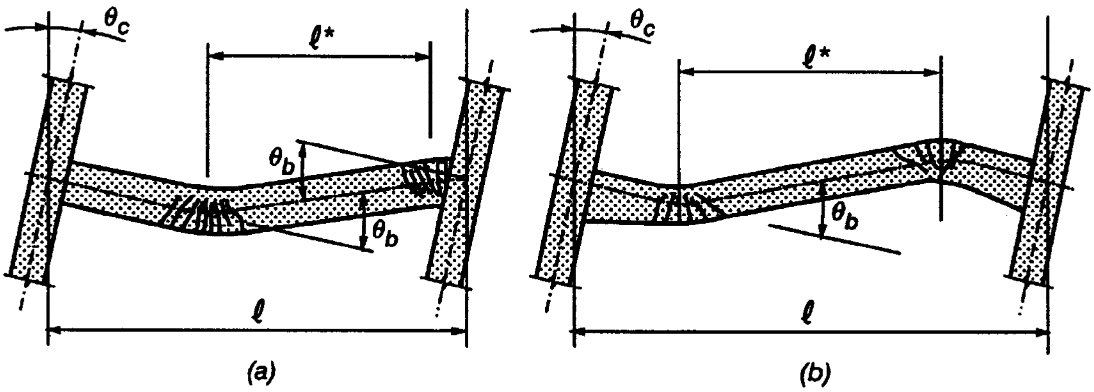


Fig. 19. Locations of plastic hinges in beams.

(b) When a structure comprising "strong column-weak beam" frames, is chosen to be designed as a system with restricted ductility with $\mu \leq 3$, the beams with plastic hinges, as in Fig. 19, should be detailed for full ductility demands whenever

$$\mu_m = m_\ell \mu > 3 \tag{18}$$

11.4.2 Frames with potential soft storeys. Phenomena associated with the development of a sway mechanism relying on the simultaneous formation of plastic hinges in all columns of that storey, have been discussed in Section 11.3. While such mechanisms should be suppressed in medium to high rise frames, they may be admissible in low rise buildings provided that certain limitation, as implied by Eq. (15), are imposed. This is an example of a structure to be designed for restricted system ductility demands while potential plastic hinges in columns may need to be detailed for full ductility. By inverting Eq. (15) it is seen that for a chosen value of column ductility capacity, $\mu_c \leq 6$, the tolerable system ductility capacity becomes

$$\mu_\Delta = 1 + \frac{h_i}{H} (\mu_c - 1) \tag{19}$$

The earthquake design lateral forces must then be chosen to correspond with this reduced system ductility capacity.

The above approach is simple and probably conservative. It does not appear to impose undue economic disadvantages on medium rise buildings in which collapse due to this mechanism has been repeatedly observed during significant earthquakes. Dynamic time-history studies may well indicate that for moderate earthquake records the expectation of the formation of soft storeys is unnecessarily conservative.

11.4.3 Frames developing plastic hinges in columns. In Section 9.2 the influence of gravity load dominance, necessitating plastic mechanisms relying predominantly on the formation of plastic hinges at both ends of interior columns, was examined in some detail. The mechanisms depicted in Fig. 11 and the concern for the reliability of a few "elastic" columns controlling column storey sway mechanisms, lead to the recommendation that in most cases such structures should be considered to have restricted ductility capacity.

12 STRUCTURAL WALLS

12.1 Common Design Practice

The particularly useful role of structural walls in controlling the seismic response of buildings has convincingly proved itself. For many years engineers used walls with some reluctance because of the overwhelming evidence of shear failures observed during earthquakes. Moreover, it was thought that walls cannot be designed for adequate ductility capacity. Therefore, with the use significantly increased seismic factors, conservative assumptions with respect to earthquake design forces were prescribed by relevant codes. This penalized buildings with walls in comparison with those utilizing frames.

Research, over the last three decades, combined with a re-examination of elementary principles, relevant to the design of reinforced concrete beams and columns, revealed that curvature ductility capacities at critical wall sections, to provide a level of system ductility approaching that in framed structures, can be achieved with ease. Also studies of the elasto-plastic dynamic response of cantilever walls, such as shown in Fig. 20, combined with the application of capacity design principles enable the shear strength of walls to be made larger than the shear demand that is associated with its ductile flexural response. Thereby in well detailed walls the occurance of a potential shear failure, leading to dramatic strength degradation, can be eliminated.

A certain mystique surrounding structural walls may have prompted the introduction of unsubstantiated rules and irrational design methods into codes of some countries. Some of these design guides attempted to combine traditional concepts of working stress design with the needs to meet criteria of the ultimate limit state. Some of these trends appear to have been influenced by analogies with the behaviour of infilled frames in which shear strength was assigned to the web panel, while the necessary flexural strength and gravity load resistance was assumed to be provided by boundary elements only. Other schools promoted the use of massive boundary elements, both vertically and horizontally, assumed partly behaving as flexural members generating distributed transverse forces which are expected to provide confinement to the web panel of the wall.

Some researchers perceive that, perhaps because of the relative importance in seismic resistance of walls, or their size, traditional approaches based on macromodels, using simulation as in the design of reinforced concrete beams and columns, are not sufficiently accurate. Therefore finite element analysis techniques are sometimes advocated as being more promising.

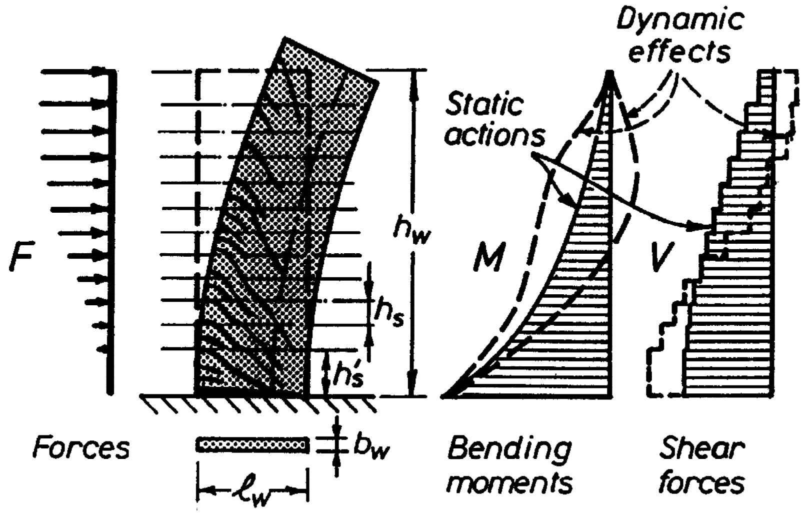


Fig. 20. A cantilever wall subjected to lateral forces.

In the following sections several features of the state-of-the-art, as seen 10 years ago (Paulay, 1986), are updated with some of the important principles restated. In this the use of established and familiar techniques, based on macromodels, is advocated. Some relatively recently recognized design aspects reported here have already been codified (Standards New Zealand, 1995). Because of the great potential of structural walls meeting most satisfactorily seismic design criteria, for both the serviceability and ultimate limit states, selected issues are more extensively reviewed here. To restate the motivation presented in the introduction, this review is provided with a conviction that *simplicity and transparency in design* can sustain our confidence in the reliable performance of structural walls when extreme seismic demands are imposed on them.

12.2 The Flexural Response of Wall Sections

12.2.1 Strength considerations. The first task of the designer is to find the necessary amount of vertical reinforcement at the critical section of a wall to resist the required bending moments, such as implied in Fig. 20. This can be done by a rapidly converging trial and error process or with the use of standard software. In contrast to the conventional form of reinforcing beams, in wall sections there is greater freedom of choice with respect to the arrangement of the flexural reinforcement.

A notion still persists that efficient flexural resistance relies on concentrated reinforcement, much like in doubly reinforced beams, placed in the boundary regions of the section. Erroneously, some codes have even suggested that the contribution to flexural resistance of uniformly distributed reinforcement, usually satisfying code requirements for a minimum quantity, be ignored, and that reliance be placed only on reinforcement situated in the boundary regions or in specially formed boundary elements. This practice, apart from being wasteful, underestimates flexural strength and hence the seismic shear demand associated with it.

12.2.2 Ductility considerations Another task of the designer is to gauge the ductility capacity of the wall, acting as a cantilever (Fig. 20). This issue needs to be considered in terms of both, curvature ductility $\mu_\phi = \phi_u/\phi_y$, where ϕ_y and ϕ_u is the curvature at first yield and at the ultimate limit, respectively, and displacement ductility, μ_Δ, as defined in Fig. 1. Within the usual range of wall responses, extremely simple but conservative checks with respect to the adequacy of curvature ductility capacity can be made. When these simple criteria, such as Eq. (22), are not satisfied, the designer may either change relevant properties of the wall so as to satisfy the criterion, or undertake a more refined analysis to demonstrate that the chosen properties do in fact satisfy curvature ductility requirements.

When the principal parameters, such as the aspect ratio of a wall $A_r = h_w/\ell_w$ (Fig. 20) and the length of the plastic hinge, ℓ_p, at the base of a cantilever wall, estimated from

$$\ell_p = (0.20 + 0.044A_r)\ell_w \tag{20}$$

are taken into account, the relationship between the assumed displacement ductility demand, μ_Δ, and the required curvature ductility capacity, μ_ϕ, can be established from first principles. The results of such an analysis are shown in Fig. 21.

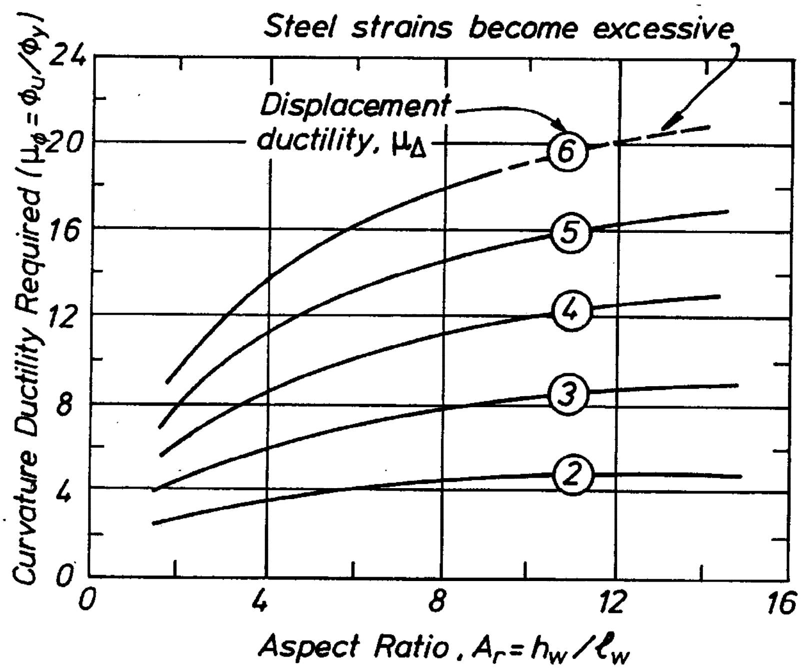


Fig. 21. Required curvature ductility capacity of sections of cantilever walls as a function of the displacement ductility demand and the aspect ratio of walls.

Figure 22 (Corley *et al.*, 1981) shows the influence of certain parameters on the development of ultimate curvature for a rectangular wall section subjected to flexure only. Curvatures associated with uniformly distributed vertical reinforcement and those developed when some additional reinforcement is placed in the boundary region, are compared. In these examples the amount of uniformly distributed reinforcement was chosen to be that considered by most codes as a minimum, i.e. of the order of 0.25% of the gross concrete area of the section.

Alternative arrangements of reinforcement are shown in Figs. 22(a) and (b), where the relevant notation is also recorded. Fig. 22(c) shows that the curvature attainable with the maximum concrete compression strain, ε_{cu} = 0.003, developed, is

$$\phi_u = \frac{\varepsilon_{cu} + \varepsilon_{su}}{\ell_w} = \frac{\varepsilon_{cu}}{c} \tag{21}$$

The evaluation of the neutral axis depth, c, is a byproduct of the rountine analysis for flexural strength. Hence with a known ratio c/ℓ_w, *the single most important parameter*, the curvature ductility capacity of wall sections can be readily estimated. It should be noted that the designer's prime interest is only to be satisfied that the ratio c/ℓ_w is not larger than that which corresponds to the estimated curvature ductility demand.

Figure 22 demonstrates that:

(a) The ultimate curvature capacity decreases with increased total reinforcement content, ρ_t. As a reference, the curvature, associated with uniformly distributed reinforcement with ρ_t = 0.25%, developing unit moment, has been taken as 100%.

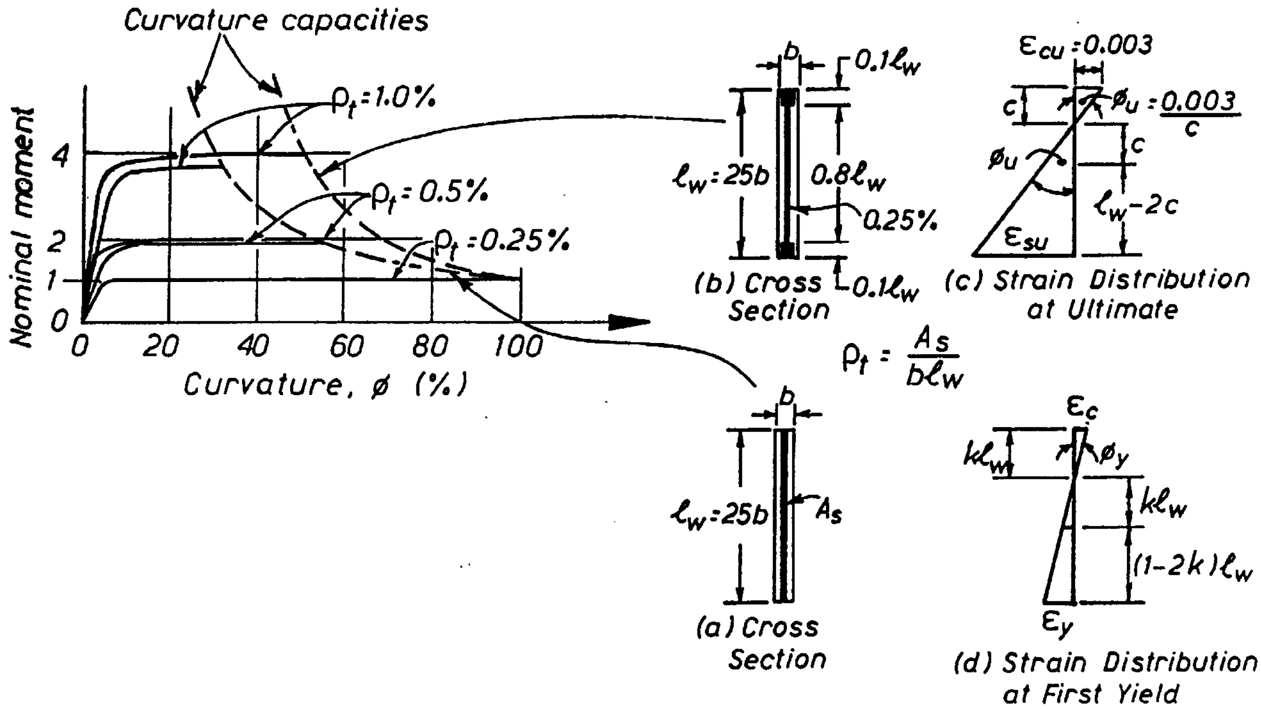


Fig. 22. Effects of the amount and the distribution of vertical reinforcement on the curvature ductility capacity of rectangular wall sections.

(b) The curvature ductility capacity of the wall section may be increased if the vertical reinforcement in excess of that corresponding to ρ_t = 0.25%, is placed in the boundary regions, as shown in Fig. 22(b).
(c) For commonly used walls, where ρ_t < 1.0%, the increase of flexural strength with the excess reinforcement placed at the boundaries, is negligible.

The results of curvature analyses for rectangular wall sections with only uniformly distributed vertical reinforcement, as shown in Fig. 22(a), corresponding to different values of reinforcement ratios, ρ_t, are presented in Fig. 23. It furnishes useful information with respect to two quantities of primary interest to the designer, c/ℓ_w and μ_ϕ. In the analysis the following parameters, additional to those referred to previously, had to be considered: the ratio of the yield strength of the steel to the compression strength of the concrete, f_y/f'_c, the ratio of the assumed maximum allowable concrete compression strain in the extreme compression fiber to the yield strain of the steel, $\varepsilon_{cu}/\varepsilon_y$, and the neutral axis ratio at the onset of yielding, k, defined in Fig. 22(d) as a function of the ratio of the moduli of elasticity of the two materials, $n = E_s/E_c$. The shaded area in Fig. 23 shows the range of the mechanical reinforcement ratio, $m = \rho_t f_y/f'_c$ commonly encountered in practice.

The conclusions of importance to structural design that may be drawn from the relationship shown in Fig. 23 are:

(i) When the mechanical reinforcement ratio, m, exceeds approximately 0.12, the curvature ductility capacity, μ_ϕ, of wall sections with uniformly distributed reinforcement becomes restricted. This value of m corresponds to relatively large reinforcement ratios, ρ_t, in the range of 0.6 to 1.0%. It is therefore recommended that uniformly distributed reinforcement in rectangular wall sections in excess of ρ_t = 0.5% should not be used.

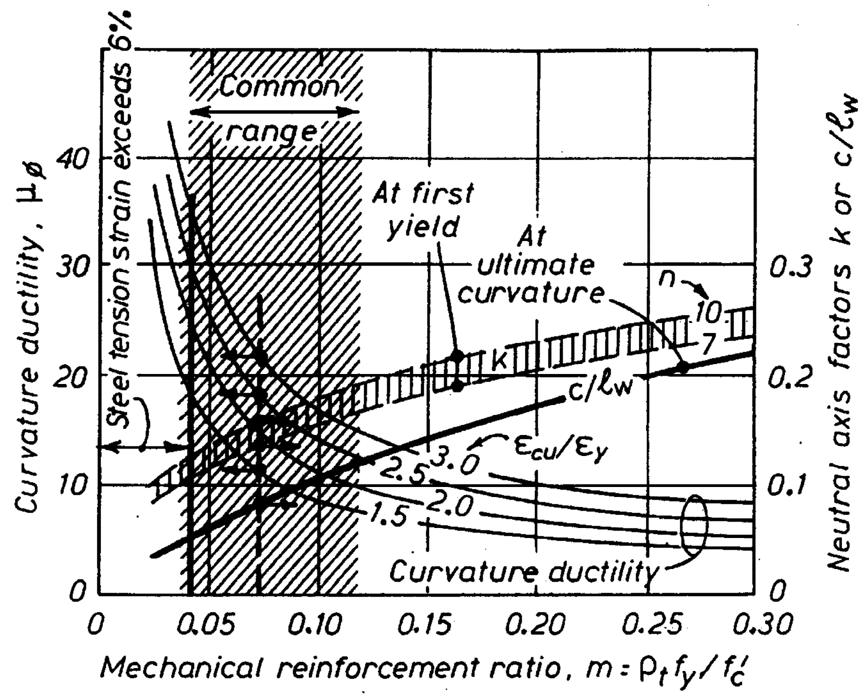


Fig. 23. Neutral axis-wall length ratios and curvature ductility capacities of rectangular wall sections with uniformly distributed vertical reinforcement.

(ii) When the value of m is less than 0.10 representing a situation likely to be encountered in the great majority of structural walls, the curvature ductility capacity increases rapidly.
(iii) When m reduces to approximately 0.04 ($\rho_t \approx 0.2\%$), the large curvature ductilities that could be developed would be associated with excessive steel strains in the extreme tension fibre ($\varepsilon_{su} \approx 6\%$).
(iv) The ratio of the neutral axis depth at ultimate to the wall length, c/ℓ_w, a parameter readily available during the design process, indicates the two limits, $0.05 < c/\ell_w < 0.12$, that have been considered above. Properties leading to $c/\ell_w < 0.06$ imply large ductility capacities at the expense of very large tensile strains at ultimate curvature. However, it should be noted that large curvature ductility capacity is not likely to be utilized, even under extreme seismic demands. Therefore in lightly reinforced wall sections the imposed curvature ductility demands may generate concrete compression strains that are less than critical, i.e. $\varepsilon_{cu} < 0.004$, often assumed for seismic criteria.
(v) For a given value of the mechanical reinforcement ratio, m, curvature ductility capacity will significantly increase with the strain ratio $\varepsilon_{cu}/\varepsilon_y$. The commonly used value of this ratio will be of the order of 0.003/0.002 = 1.5. It is seen that at the upper limit of the practical range of m, a very significant increase in ductility capacity can be achieved if the extreme fibre concrete compression strain is increased to ε_{cu} = 0.006, resulting in $\varepsilon_{cu}/\varepsilon_y \approx 3.0$. This may be readily achieved if in a part of the flexural compression zone the compressed concrete is lightly confined. This issue is examined in Section 12.3.

12.2.3 <u>The effects of axial compression load on walls</u>. Axial compression load may significantly reduce the curvature ductility capacity of reinforced concrete rectangular sections, unless the strain capacity of the concrete in compression is boosted by confinement. Relevant simple principles, including acceptable approximations, which may also be utilized in routine design, are illustrated in Fig. 24.

Figure 24(a) shows the position of internal tension, T_1, and compression, C_1, forces at ultimate and the mode of flexural resistance sustained by uniformly distributed wall reinforcement. Axial compression load, P_i, will require an additional region of the section, as seen in Fig. 24(b), to carry compression stresses of the order $0.85f'_c$. The eccentric position of this force, P_i, allows a significant moment with respect to the centroid of the gross concrete section to be resisted. Concentrated reinforcement placed in the end regions, if required, developing equal tension, T_3, and compression, C_3, forces, as shown in Fig. 24(c) provides the third component of the moment of resistance. The designer has little control over the length, a_2, of compression. However, with suitable choices of the magnitudes of the forces T_1 and T_3, the length a_1 may be changed to suit curvature ductility requirements.

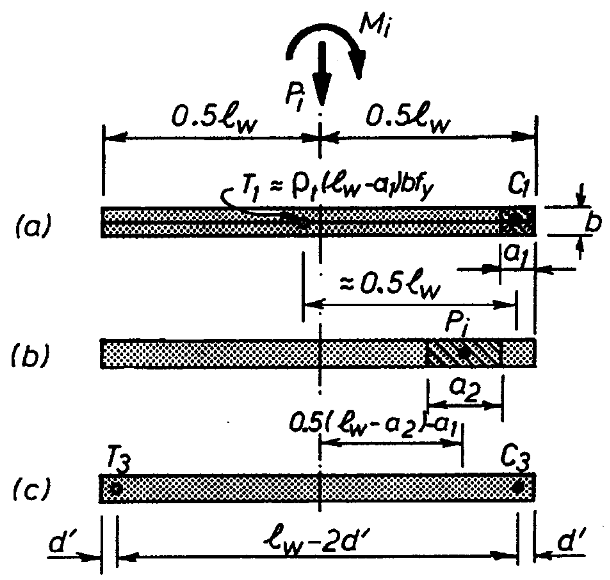


Fig. 24. Approximations in the location of stress resultants in rectangular wall sections.

In the evaluation of the neutral axis depth to wall length ratio, c/ℓ_w, as it affects curvature ductility, considered in Fig. 23, the enlarged value of $c \approx (a_1 + a_2)/0.85$ (Figs. 24(a) and (b)) need now be considered.

11.2.4 Specific design features. It has been repeatedly emphasised in the preceding sections that in the selection of the flexural reinforcement the aim should be to develop in the wall section the largest possible flexural compression which is compatible with the curvature ductility capacity to be provided. The reason for this is the concern for shear transfer from the wall to its foundations across the critical section in the plastic hinge. As Fig. 20 suggests, the predominant fraction of the base shear force needs to be transferred within the flexural compression region of the wall section. After some inelastic displacement reversals the mechanism to be relied on is that of *shear friction*. It must be appreciated that at this level conventionally computed average shear stresses, assumed as being spread over the length of the wall, are meaningless. To lower the intensity of shear friction stresses, the effective area, approximately equal to cb (Fig. 22), should be made as large as ductility requirements permit. When the average compression stress over a rectangular wall section due to the axial force, P_i, approaches $0.08f'_c$, the limit of the curvature ductility capacity of that wall, designed for a system ductility demand of $\mu_\Delta \approx 5$, will be reached.

Analysis procedures implied by Fig. 24 are approximate but very transparent, and should be considered adequate for seismic design. Small steel stresses implied by small steel strains in the vicinity of the neutral axis of the section (Fig. 22(c)) may well be replaced by yield strength. After just one steel stress reversal associated with significant curvature ductility demand, all bars in this region will be yielding.

Thereby the use of elaborate section analysis for sections with distributed reinforcement, sometimes advocated, may be safely replaced by the extremely simple procedure illustrated in Fig. 24.

Adequate curvature ductility capacity for cantilever walls with aspect ratio, A_r, not exceeding 8 and designed for displacement ductility demands, μ_Δ, not exceeding 5, can be expected to have been provided (Standards New Zealand, 1995) when at the development of the flexural overstrength the neutral axis depth, c, is not larger than the critical value given by

$$c_c \leq (0.3\phi_o/\mu_\Delta)\ell_w \tag{22}$$

For walls exceeding only slightly the required flexural strength, the value of ϕ_o is typically of the order of 1.5. Hence when $\mu_\Delta = 5$, the c_c/ℓ_w ratio becomes 0.09. When the computed value of c, including the influence of axial compression, does not exceed this value, *no further check* on ductility capacity needs to be made. Equation (22) also implies that a displacement ductility capacity of only $\mu_\Delta = 3$, necessitating reduced curvature ductility can be achieved provided that $c/\ell_w < 0.15$. In the derivation of Eq. (22), based on the relationship presented in Fig. 21, the conservative assumption was made that $A_r \approx 8$. The yield curvature, in terms of the definitions given in Fig. 1, was taken as $\phi_y = 0.0038/\ell_w$. When for walls with $A_r < 8$ the neutral axis depth, c, is found to exceed the value given by Eq. (22), c_c, being inversely proportional to μ_ϕ, may be simply re-evaluated from Fig. 21.

When, in exceptional circumstances, the axial load on rectangular walls is large or when unsymmetrical wall sections, such as one with a tension flange at one end (Fig. 26) is used, the limitation of Eq. (22) may not be able to be satisfied. The required curvature ductility capacity, when $c > c_c$, can then only be attained with the generation of increased concrete strains at the extreme compression fibre. Such strains can be readily sustained, but only if the concrete in the affected region is suitably confined. This issue is reviewed in Section 12.3.

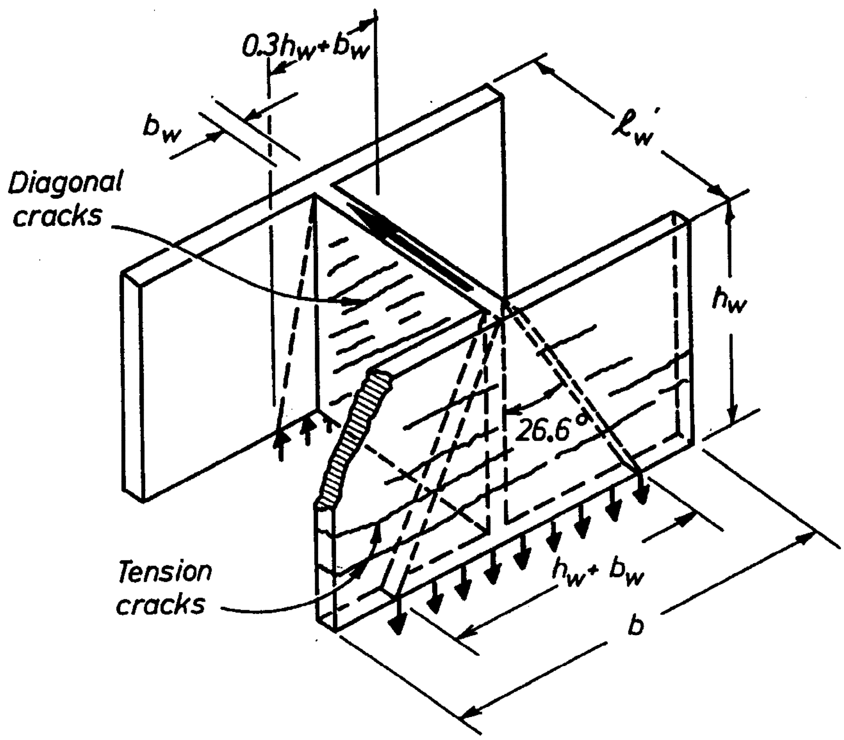


Fig. 25. Estimation of flange width effective for strength of structural walls.

12.2.5 Articulated wall sections. The discussion of the behaviour of rectangular wall sections needs to be supplemented with a few notes applicable to articulated wall sections. Wall shapes are often dictated by functional requirements for a particular building. Clearly flanges or enlarged boundary elements, such as shown in Fig. 30(b), offer advantages that should be utilized.

Enlarged boundary elements may provide stability with respect to out-of-plane buckling in the plastic regions of walls, reviewed in Section 12.4. Increased thickness at the boundaries allow shallower flexural compression zones to be developed within the plastic hinge. Thereby the curvature ductility capacity may be significantly increased. Boundary elements in this respect are particularly beneficial when significant axial compression loads, such as occur in coupled wall systems, need also be sustained.

One aspect, about which designers are often concerned, is the effective widths of flanges. While codes make recommendations for beams, similar rules for walls are seldom to be found. Suggestions with respect to ductile flanged cantilever walls are incorporated in Fig. 25. For the spreading of stresses in compression flanges, a conservative assumption with respect to the effective flange width $b_{eff} = b_w + 0.3h_w$ may be made without for design purposes affecting the accuracy of the flexural analysis of the section.

The assessment of the contribution of vertical reinforcement placed in a tension flange is more critical. An apparent conservative estimate, similar to that made for compression flanges, may grossly underestimate the contribution of a tension flange. In the context of capacity design, this would result in underestimation of the flexural overstrength in the potential plastic hinge, and consequently the shear demand imposed on the wall. For this reason the contribution at full strength ($\lambda_o f_y$) of all vertical bars within an effective width of $b_{eff} = b_w + h_w$, as shown in Fig. 25, should be considered. This implies that for walls with flange width, b, equal or larger than the wall height, h_w, the entire flange should be considered as being effective in tension. Tensile strains in vertical bars near the edges of flanges may be smaller than those at the web-flange junction, but at the imposition of significant curvature at the ultimate limit state, all bars within the recommended effective width will develop yield strength. An aspect to be considered in the assessment of the contribution of tension flanges should be the *ability of the foundation structure* to effectively absorb such uplift forces.

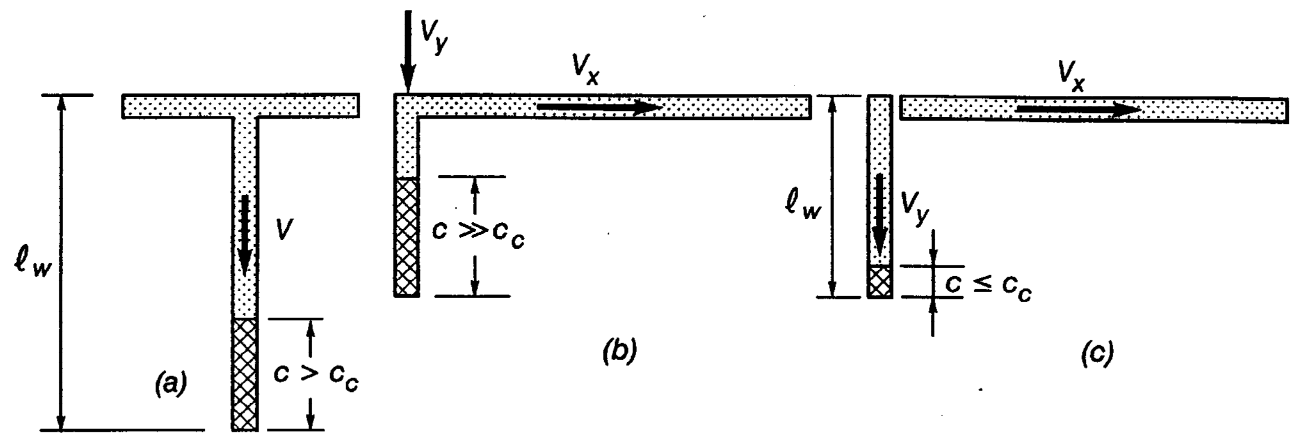


Fig. 26. Articulated walls sections

Another feature of articulated wall sections is the effect on curvature ductility capacity of unsymmetrically arranged flanges. Figure 26 shows some examples. The fully effective tension flange, when combined with the influence of axial compression load on the wall, may require, as Fig. 26(a) suggests, a very large flexural compression zone at the critical section of the plastic hinge. This will greatly restrict the curvature ductility capacity of the section. Significant amounts of transverse confining reinforcement, as examined in Section 12.3.2, may be required to boost the ultimate curvature, ϕ_u, to the level required.

Figure 26(b) shows an angle shaped wall commonly encountered in practice. While, with some confinement at the end of the long leg, adequate ultimate curvature, resulting from the shear force V_x, could be developed, this is not the case for seismic actions in the other principal direction. The very large flexural compression zone, being much larger than the critical value c_c, defined by Eq. (22), will prohibit the use of such a wall in fully ductile ($\mu_\Delta \approx 5$) systems. Reduced ductility capacity, on the other hand, would penalize the system by having to resist much larger seismic design forces. The separation of such angle shaped walls into two independent rectangular sections, as shown in Fig. 26(e), would eliminate these difficulties.

12.3 Confinement in Walls

The need in certain circumstances to confine regions of a wall is seldom recognized. However, some codes suggest that boundary elements, designed to accomodate the flexural reinforcement, should be fully confined in the potential plastic region, irrespective of ductility considerations, the same way as are columns. With an appreciation of the factors that control the development of curvature in the potential plastic hinge region of a wall, reviewed in Section 12.2, it is, however, easy to establish the relevant design criteria.

12.3.1 Regions of confinement. Figure 27 shows strain patterns in the critical section of a wall, to be considered. Strain profile (1), based on a concrete compression strain of 0.004, sustainable by unconfined concrete, shows the critical neutral axis depth, c_c, associated with the given ultimate curvature demand, ϕ_u, such as given by Eq. (22).

When this neutral axis depth increases to c, the curvature capacity reduces to ϕ_u^*, unless the concrete compression strain increases. This is shown by the strain profile (2) in Fig. 27. The reasons for this increase of the flexural compression region of the wall section are: significant axial compression load needs to be carried, or a large amount of tension reinforcement is required in an unsymmetrical section, such as illustrated in Fig. 26(a), or the combination of both of these sources.

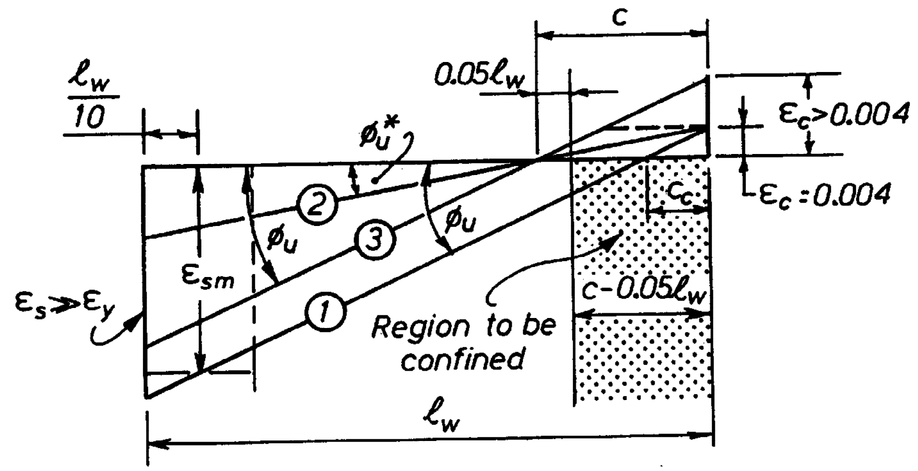


Fig. 27. Strain patterns for wall sections.

It is thus seen that in order to allow the desired ultimate curvature, ϕ_u, to develop, as shown by strain profile (3), concrete compression strains over a length of c-c_c will exceed 0.004. In such relatively rare cases, it is recommended that transverse reinforcement over the length ($c - 0.05\ell_w$), as shown shaded in Fig. 27, be provided to confine the compressed concrete.

12.3.2 The amount of transverse confining reinforcement. The considerations of the degree of confinement of compressed concrete by transverse passive pressure, provided by suitably arranged transverse reinforcement, are the same as those applicable to columns. Structural walls are in fact columns with an exceptional elongated section. Because the separate confinement of the flexural compression regions of column sections, developed at opposite faces under reversing moments, is impractical, hoops and ties traverse the entire section (Fig. 46). Under earthquake actions the extensive centre portion of wall sections in a plastic hinge region will never be subjected to compression strains. Hence the need for confinement will arise only in the critical boundary regions, defined in Fig. 27.

The capacity of the transverse confining reinforcement with area A_{sh}, yield strength f_{yh} and vertical spacing s_h will be proportional to the width to be confined, h", the ratio of the relevant gross concrete area, A_g^*, to that of the confined core A_c^*, the strength of the concrete, f_c', the displacement ductility demand, μ_Δ, and the previously discussed parameters quantifying the curvature capacity of the section. This consideration led to the formulation of the requirement (Standards New Zealand, 1995) for the quantity of transverse reinforcement, A_{sh}, with vertical spacing s_h.

$$A_{sh} = \left(\frac{\mu_\Delta}{40} + 0.1\right) s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{\ell_w} - 0.07\right) \tag{23}$$

Figure 28 illustrates the interpretation of Eq. (23). It shows that $A_g^* = h'b$, where $h' \geq (c - 0.05\ell_w)$ $A_c^* = h''b''$, and the area of one tie is $A_{te} \geq (s_v/h')A_{sh}$.

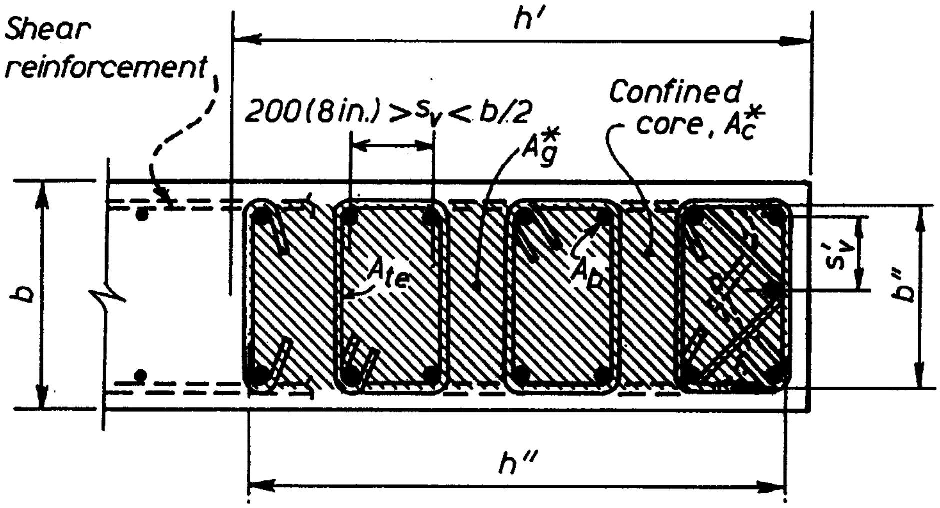


Fig. 28. Dimensions of a confined region of a wall.

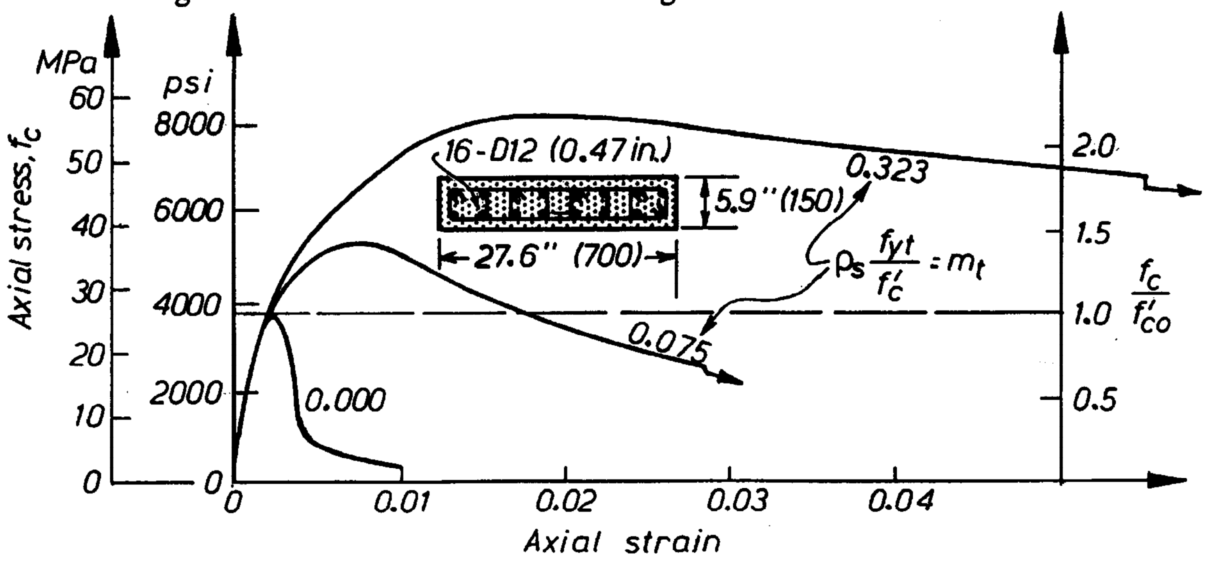


Fig. 29. Stress-strain response of a confined wall element.

This amount of transverse reinforcement, when required, should be provided over the height of potentially plastic region, estimated as l_w or $h_w/6$, whichever is more. Additional functions of the transverse reinforcement are commented on in Section 15.

Figure 29 shows the stress-strain response of a confined wall element (Mander *et al.*, 1988). It demonstrates the gain in both strain capacity and strength of the compressed concrete, resulting from increased quantities of transverse reinforcement quantified by the mechanical reinforcement ratio, $m_t = \rho_s f_{yt} / f_c'$. The maximum compression strain at the extreme fiber of wall sections will seldom exceed 0.01. When $c = 0.25\ell_w$ (Fig. 27) and typical values of properties and dimensions, such as in Fig. 28, are used, the mechanical reinforcement ratio m_t required (Fig. 29) will be of the order of 0.050.

12.4 The Stability of Walls

Aesthetic or functional requirements may necessitate the use of relatively thin rectangular walls. Clearly in the potential plastic region of such walls, instability resulting from *out-of-plane buckling* may arise, particularly in the first storey. Concepts of Eulerian buckling suggested that the thickness of the flexural compression zone in such a region should not be less than approximately 10% of the clear height of the first storey. Subsequent studies, both experimental and theoretical, indicated, however, that out-of-plane buckling may be triggered by large inelastic steel *tensile* strains rather than by high concrete compression stresses.

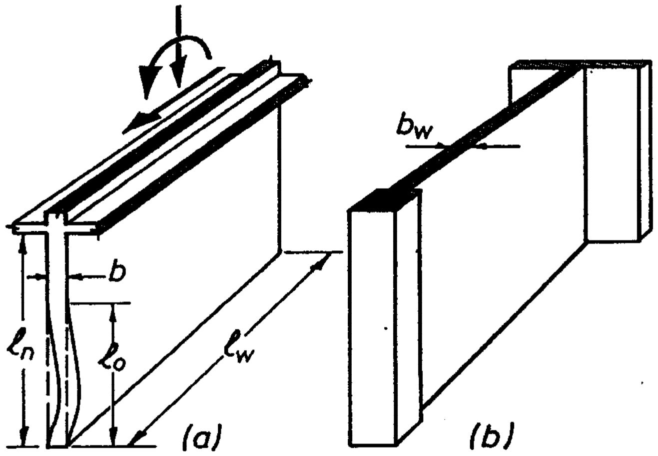


Fig. 30. Typical sectional configurations of walls.

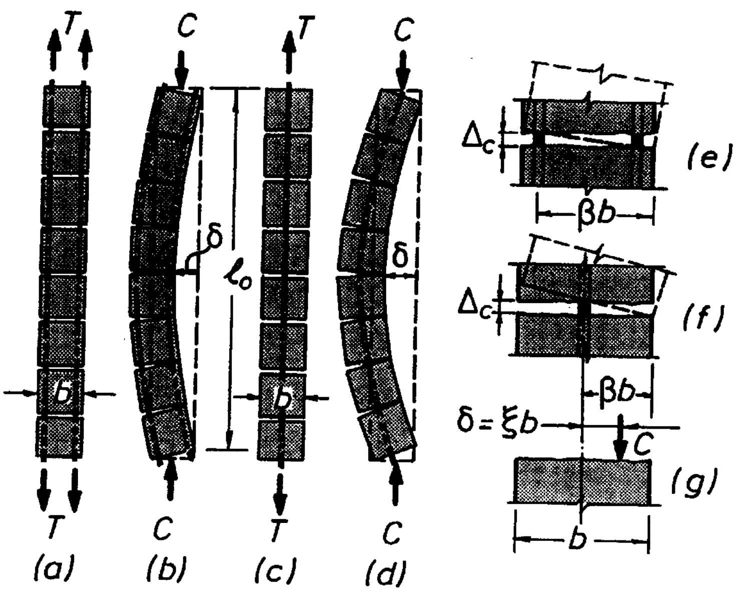


Fig. 31 Crack developments and subsequent deformations leading to out-of-plane buckling in walls.

As Fig. 30 suggests, this failure mode can be prevented by providing a sufficiently enlarged boundary element. This is well established in engineering practice. When restrictions on the selection of wall thickness exist, as they often do, some estimate needs to be made as to the admissible minimum wall thickness, b, shown in Fig. 30(a), corresponding with the expected ductility demand. Quantifying the problem is complex. However, an assessment of at least those parameters that are considered to be important is likely to give some indication of their criticality.

Strains in the vicinity of the extreme tensile edge of a rectangular wall at the development of significant inelastic curvature in the plastic hinge region are shown approximately to scale in Figs. 22(c) and 27. As a consequence, large permanent cracks with width Δ_c, will remain after the disappearance of the tensile force, T, in this region, as seen in Figs. 31(a) and (c). Upon an earthquake induced displacement reversal, when the same region is subjected to a flexural compression force, C, the wide cracks must first close *before* concrete compression stresses can be initiated. An even closure of cracks cannot be assured and hence, as Figs. 31(b) and (d) suggest, this is the stage when transverse curvature and as a consequence eccentricity, δ, may develop. A comparison of Figs. 31(e) and (f) indicates that a single layer of vertical wall reinforcement will accentuate transverse curvature.

By considering the geometrical relationships between buckling length, ℓ_o, critical wall thickness, b_c, and the position of the reinforcement as defined by the parameter β in Figs. 31(e) and (f), it may be shown from first principles that the critical wall thickness is

$$b_c = \ell_o \frac{\sqrt{\epsilon_{sm}}}{8\beta\xi} \tag{24}$$

where ϵ_{sm} is the mean residual steel tensile strain in the boundary segment, as shown in Fig. 27. This strain can be estimated from the expected maximum curvature ductility demand, $\mu_\phi = \phi_u/\phi_y$ (see Fig. 21). However, in routine design it is not necessary to evaluate the curvature ductility, μ_ϕ, because with conservative assumptions with respect to the theoretical plastic hinge length, strains can be readily related to displacement ductility, μ_Δ, the fundamental parameter used by the designer when selecting the desired level of seismic resistance of a ductile system.

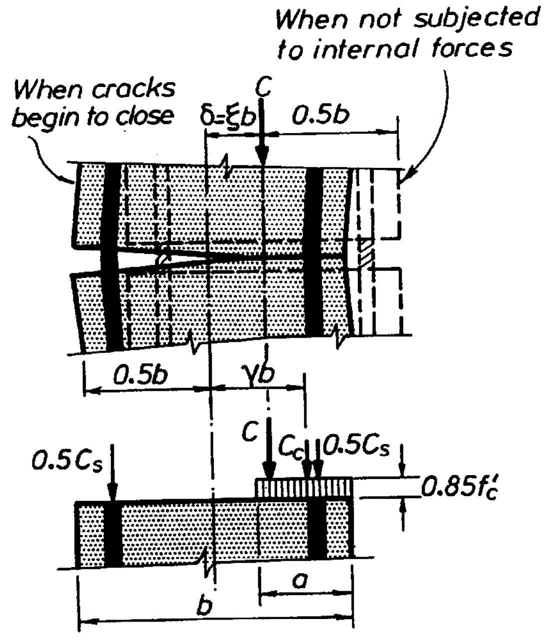


Fig. 32. The relation in a wall section of internal forces to eccentricity.

A more difficult task is to estimate the critical eccentricity, shown in Fig. 31(g) as $\delta = \xi b$, at which failure in lateral bending will occur. To this end the position of the total internal compression force, C, acting within the wall thickness, b, as shown in Fig. 32 for a wall with two layers of reinforcement, needs to be determined. With the notation shown in Fig. 32, simple equilibrium considerations show that

$$C = C_c + C_s = C_s \Big/ \left(1 - \frac{\xi}{\gamma}\right) \tag{25}$$

As the quantity of wall reinforcement, A_s, increases, both the total and the concrete compression force, C_c, necessary to close at least partly the crack, will increase and thus the eccentricity, ξb, at which out-of-plane buckling occurs, will decrease. In the extreme hypothetical case, when the reinforcement content in the wall approaches zero, $C = C_s$ will be very small and instability should occur when $\gamma = \xi = 0.5$, a situation depicted in Figs 31(e) and (f). Thus, the larger the amount of reinforcement for unit length of wall, expressed by the mechanical reinforcement ratio $m = A_s f_y/(bf_c')$, the greater the resistance against a uniform closure of cracks, resulting in reduced critical eccentricity and hence earlier onset of out-of-plane buckling.

For design purposes the "exact" solution of the problem can be approximated by linear relationships. These lead to the following easily applied criteria (Standards New Zealand, 1995) with respect to the critical wall thickness:

$$b_c = \frac{k_m\ (\mu_\Delta + 2)\ (A_r + 2)}{1700\ \sqrt{\xi}}\ l_w \tag{26}$$

where $k_m = 1.0$, unless it can be shown that for long walls

$$k_m = \frac{l_n}{\left(0.25 + 0.055A_r\right)\ l_w} < 1.0 \tag{27}$$

and where

$$\xi = 0.3 - 0.4m \geq 0.1 \tag{28}$$

Figure 30(a) defines the length, l_w, and the clear unsupported height, l_n, of the wall. Equation (26) shows that the phenomenon is sensitive to the aspect ratio $A_r = h_w/l_w$, where h_w is the total height of the wall.

One of the assumptions in deriving these relationships, difficult to verify, is that the effective buckling length, shown as l_o in Figs. 30 and 31(b), is equal to the length of the plastic hinge, l_p at the cantilever wall base, given by

$$l_p = l_o = (0.20 + 0.044\ A_r)\ l_w \tag{29}$$

The aspect ratio of a wall, A_r, is a measure of the shear-span to wall length ratio and thus that of the moment gradient at the base. This in turn affects the length of plastic hinge, l_p. When walls are interacting with frames in dual structural systems, wall moment patterns are associated with the lateral force pattern shown in Fig. 37(c). The beneficial effects in such cases of a greater moment gradient and hence of reduced buckling length, may be allowed for by replacing A_r in Eqs. (26) (27), and (29) by $1.5M/(Vl_w)$, where both the moment, M, and the shear, V, refer to the wall base.

It has been recommended (Standards New Zealand, 1995) that when a single layer of reinforcement is used, the displacement ductility capacity of the wall relied on should be less than 4. Moreover, the critical wall thickness, b_c, should be taken as 1.25 times that given by Eq. (26).

Where instability criteria control the contemplated wall thickness in the boundary region, an enlarged boundary element with gross area, A_{wb}, satisfying the limitations

$$b_c^2 \le A_{wb} \ge b_c l_w/10 \tag{30}$$

should be provided.

For walls of small aspect ratio, A_r, and low ductility demands, μ_Δ, stability criteria will not be critical. In such cases Eq. (26) may indicate excessively thin sections. It is considered that the wall thickness to clear height ratio, b/l_n, in the potential plastic regions should not be less than 1/20 and 1/16 in the ductility range of approximately $\mu_\Delta = 2$ and $\mu_\Delta = 6$, respectively. The expression in Fig. 33

$$b \ge 0.04\,(1 + 0.1\mu_\Delta)\,l_n \le b_1 \tag{31}$$

quantifies this requirement.

The recommendations for the dimensional limitations of the boundary regions of ductile walls are illustrated and summarized in Fig. 33.

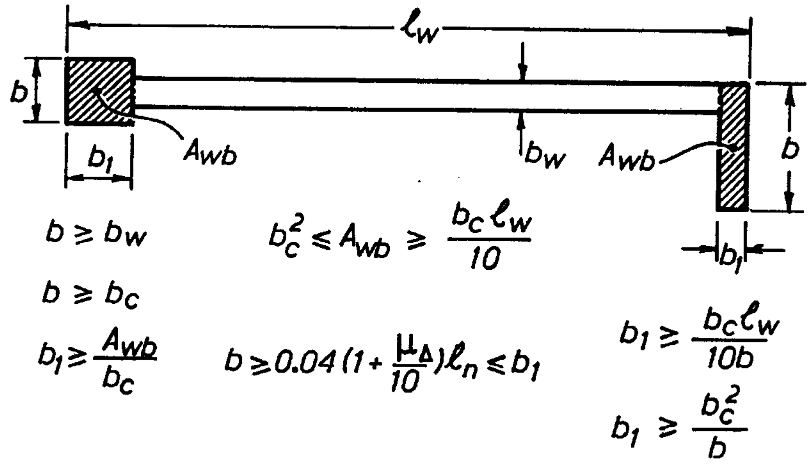


Fig. 33 Minimum dimensions of boundary elements of wall sections in the plastic hinge region.

12.5 Walls with Openings

Special consideration must be given to the design of walls which, by necessity, will have significant openings for windows, doors, the passage of services and other purposes. The approach to the design based on ductile response will depend on the degree of regularity in the patterns of openings and this pattern will control the flow of internal forces which the designer must identify.

12.5.1 Walls with regular openings. The great majority of walls with openings have them arranged in one or several vertical rows with uniform spacing. Using familiar frame models with deep members or those simulating coupled structural walls (Fig. 3(g)), the analysis of these systems is well established. The strength hierarchy between the beams, often rather deep, and the piers, in accordance with the principles of capacity design, may be established as in conventional ductile frames. However, the detailing for ductility of deep members, such as coupling beams (Section 15.3.3), requires attention because the dominance of shear effects in such members.

12.5.2 Walls with Irregular Openings. Sometimes openings are arranged in an irregular pattern. At first sight such systems often appear to defy a rational assessment of structural behaviour or the formulation of a reasonable model. In such structures, common in medium to low rise buildings, designers often used additional nominal reinforcement around the edges of openings to replace those which had to be left out because of the opening. To control early cracking resulting from stress concentrations at corners of openings, some diagonal bars crossing such potential diagonal cracks were used. As a general rule no attention was given to *sources of ductility*.

With the recent emergence of the more widely used *strut-and-tie models* to trace possible load paths in reinforced concrete systems, walls with irregular penetrations can be designed much more rationally than in the past. Extensive techniques with particular appeal to *structural designers* (Schlaich *et al.*, 1987), addressing structures subjected to monotonic loads, have become available. As yet issues relevant to load reversals and ductility do not seem to have been considered. With the aid of a simple example, shown in Fig. 34, a seismic design approach applicable to this type of structures, ensuring at least a limited ductility capacity, is presented.

A statically admissible truss model with rationally placed nodes, to dispose of the intended earthquake design forces via compression on tension members, as shown in Fig. 34, can be readily conceived. It is evident that, because of the absence of symmetry, the *truss models* will be different for each sense of the lateral design forces. Member forces satisfying equilibrium requirement and the modes of moment and shear transfer to the foundation system, are readily determined.

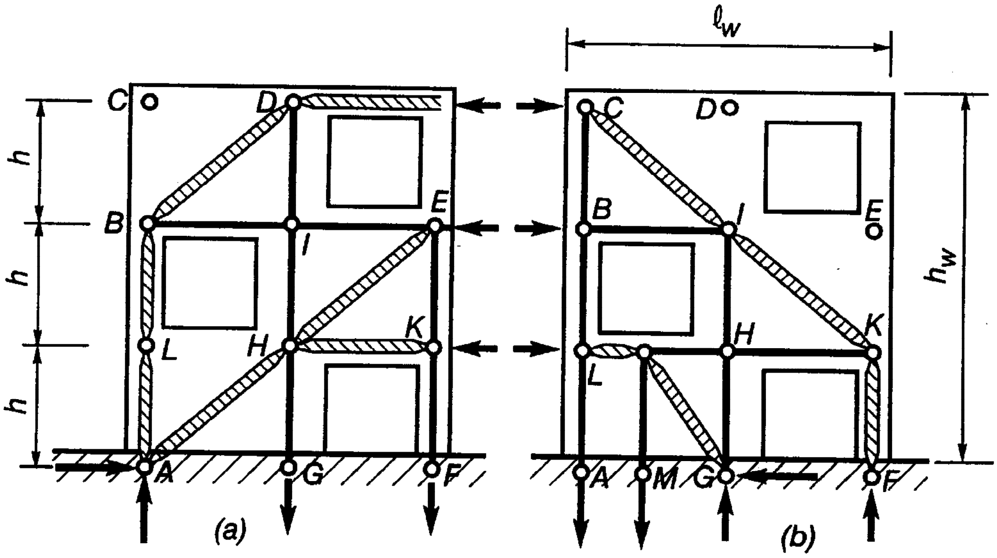
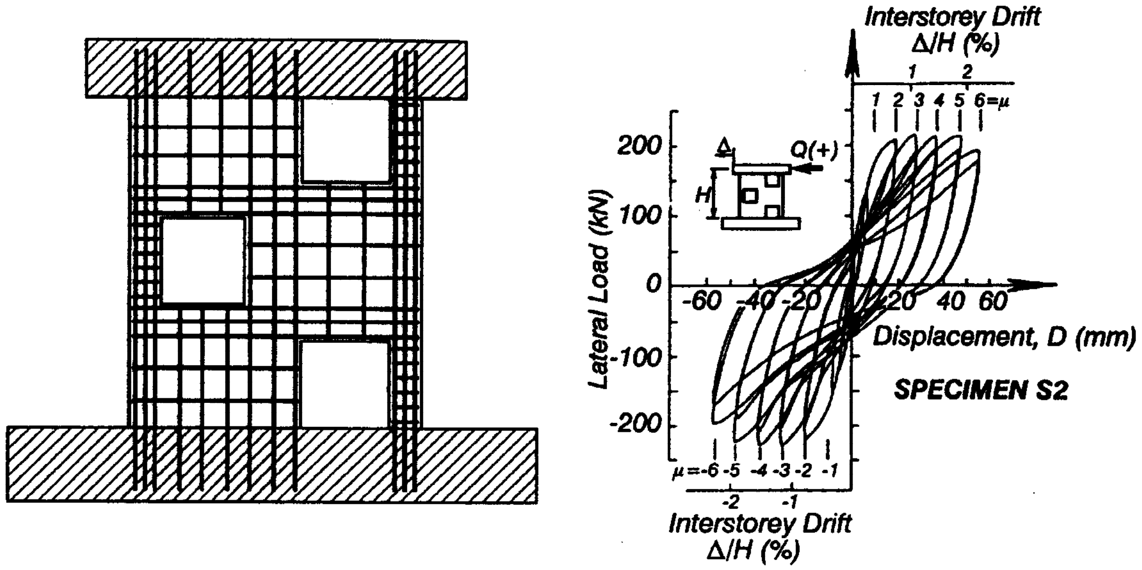


Fig. 34 A model for a wall with irregular openings.

The obvious sources of ductility are the vertical tension members consisting of a number of reinforcing bars arranged in bands. The detailing of the anchorages of bars making up a tension member must be such as to lead to low concrete compression stresses in the vicinity of the idealised nodes. With allowances for concurrent transverse tensile strains, typical concrete compression stresses at the ultimate limit state should be of the order of 0.4 f_c'.

The role of the horizontal tension members (Fig. 34) is the same as that of stirrups in a beam. They transfer, where necessary, the shear force from one vertical chord of the wall to the other. Therefore they should *not be utilised* for energy dissipation. Inelastic strains in such relatively long members could lead to excessive horizontal distortions within a wall. Moreover, with reversing ductility demands, they will be cumulative. For example yielding of member H-K (Fig. 34(b)) could impose large deflections at the top of the pier F-K relative to its fixed base. Thereby unintended shear and flexure is introduced to this important member, expected to function primarily as a strut.

Capacity design philosophy suggests that the vertical tension member, designed to yield, should furnish the desired ductility capacity of the system. By evaluating subsequently the overstrength of these members, the correspondingly increased lateral forces and those in all compression and horizontal tension members can be readily estimated. These members can then be proportioned to ensure that they remain elastic. The satisfactory response of such a unit is shown in Fig. 35 (Yanez *et al.*, 1991).



(a) Test Unit (b) Hysteretic Response

Fig. 35 Performance of a test wall with openings.

13 STRUCTURES DESIGNED FOR ELASTIC RESPONSE

In certain situations, particularly when systems of importance are involved, the structure is designed so as to respond within the elastic domain. Earthquake response spectra specified by building codes for nominally elastic structures require large seismic design forces to be used. However, as recent seismic events have shown, there is *no assurance that earthquake-induced forces so predicted may not be exceeded.* Reinforced concrete components designed in accordance with the general requirement of relevant codes, such as ACI 318, are considered to possess some inherent, albeit limited, capacity for ductility. Therefore, in terms of accommodating ductility demands in the design of *well-conditioned structures*, the application of the additional seismic requirements of such codes should not need to be considered. In any case a clear *identification of plastic mechanisms* that could be mobilised, should larger than anticipated ductility demands arise, *is necessary.*

When the selected structural system is such that in terms of regularity and the relative strengths of members as built, the system would qualify to be designed as a ductile one or one with restricted ductility, the exemption from the additional seismic requirements should apply. Typical examples are nominally elastic multistorey frames in which under the action of exceptionally large earthquake forces, the formation of a "soft storey" is *not* to be expected. Potential mechanisms for such *well-conditioned* structural system are those shown in Figs 3(a)(b) and (d).

When the configuration of the structural systems is such that a plastic mechanism, should it be required, is inadmissible in terms of the requirements for ductile structures or those of restricted ductility, attention must be given to local ductility demands. These may be significant. With the identification of members that may be subjected to inelastic deformations clearly in excess of those envisaged for nominally elastic structures, the relevant additional design requirements for seismic effects should be applied. Examples of such *ill-conditioned* structures are multistorey frames in which, because in the absence of the application of capacity design, the possibility of plastic hinge formation at both ends of columns in a storey is not excluded. The end regions of columns in such frames should be detailed for at least limited curvature ductility capacity. In frames with more than 3 storeys and where, because of their dominant strength, plastic hinges in beams could not develop, particular attention should be given to possible large ductility demands on columns. Ductility in such frames, if it arises, should be expected to develop only in one of the storeys. The same principles should apply to piers formed in between openings in walls, and also to walls with irregular openings.

14 DIAPHRAGMS

Floor systems, acting as diaphragms, are generally assumed to have infinite in-plane stiffness and sufficient strength. This enables a linear displacement relationship between the elements resisting horizontal forces, such as frames and walls, at different locations in plan, to be assumed. Floor slabs cast together with beams usually meet this assumption, unless the floor plan is grossly irregular or when large openings are present. In many parts of the world prefabricated concrete floor elements, designed for gravity load resistance, are extensively used. Diaphragm performance, which is vital for the efficient interaction of all lateral force resisting components of the structural system, is then assigned to a relatively thin cast-in-place topping slab with nominal mesh reinforcement. Such diaphragms in buildings, which are subjected to significant ductility demands during earthquake shaking, may not perform as well as expected. In the seismic design of diaphragms three aspects need to be addressed.

14.1 Floor Inertia Forces

The total floor force consistent with the maximum acceleration of the mass at the given level, needs to be determined. For elastic dynamic systems this is a routine operation. For inelastic, i.e., ductile systems, some estimate may be made to evaluate the maximum horizontal floor forces which correspond with the overstrength of the plastified frames and walls. The concepts of capacity design apply. A high degree of accuracy is, however, seldom warranted.

14.2 The Determination of Diaphragm Actions

Once the inertia force generated in a diaphragm at a given level has been estimated, actions, such as moments and shear forces, need to be determined. These will depend on the components of the inertia force at a level to be resisted by the lateral force-resisting vertical elements. Such actions are seldom critical when only ductile frames control the seismic response. However, in dual systems, where frames and walls, having markedly different storey displacement characteristics, contribute to seismic resistance, diaphragm actions resulting from floor masses identical to those in frame systems, may be significantly larger.

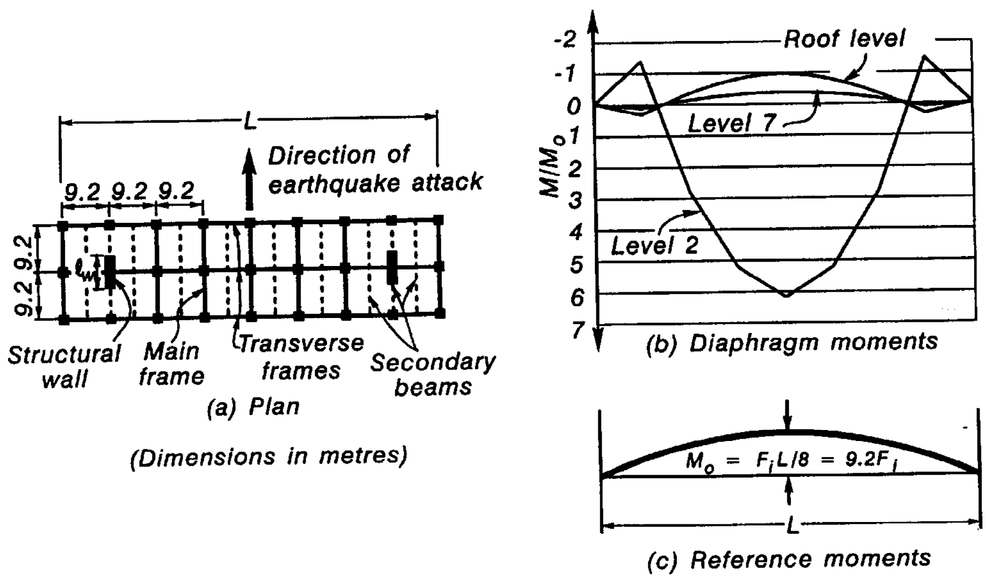


Fig. 36 Diaphragms in a 12 storey example wall-frame system.

Figure 36(a) shows the plan of a regular dual structural system, used here (Paulay and Priestley, 1992, Goodsir, 1985) to illustrate the actions, particularly bending moments, to which the diaphragms at various level could be subjected to. Figure 37(a) shows to scale the total lateral earthquake design forces considered at each level. An analysis of the elastic structure predicted the share of each of the walls and frames in resisting the total storey shear forces as shown in Fig. 37(b). From this the lateral forces applied separately to the walls and frames could be derived, as shown to scale in Figs. 37(c) and (d) respectively. It is seen that at about the midheight of the building the walls and frames combine in resisting the floor inertia forces. However, in the lower levels and particularly at the roof (level 13) the corresponding wall and frame forces, act in opposite directions and are significantly larger than the total inertia forces at these levels. These typical results also show that design recommendations, used for many years in the US and still retained in building codes of some countries, whereby a fixed proportion of the total lateral forces, typically 25%, be assigned to the frames, are unsound. This approach grossly underestimates the storey shear forces assigned to the frames in the upper parts of the structure.

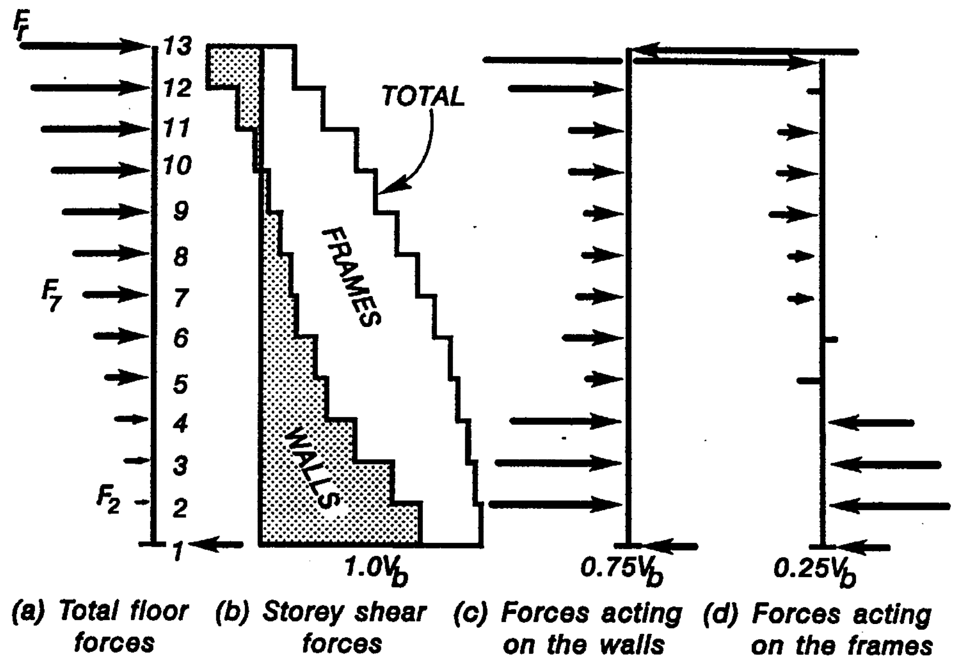


Fig. 37 Internal forces generated in an elastic dual system by lateral static forces.

The diaphragm moments, resulting from the design forces shown in Fig. 37(a), M, have been evaluated and these are shown in Fig. 36(b). To allow an appreciation of the relative magnitudes of these moments, they have been normalised in terms of $M_o = F_iL/8$, the maximum moment that would be developed if the horizontally distributed inertia forces, F_i, would be applied to a simply supported beam with a span of $L = 8 \times 9.2 = 73.6$ m. These are shown in Fig. 36(c). Because of the larger inertia force at roof level, F_r, (Fig. 37(a)) the absolute maximum moment at this level is 65% larger than that due to F_2 at level 2. The designer might be tempted to consider the inertia force acting on the diaphragm at level 2 to be of the order of F_2, shown in Fig. 37(a). The analysis indicates that the maximum diaphragm moment should correspond with a total inertia force of the order of $10F_2$. The example illustrates that the diaphragm actions in dual systems do require special attention. Note that for a given direction of the lateral design forces (Fig. 37(a)), the sense of the diaphragm moments at level 2 and at roof level is different.

When the diaphragm actions are established, the mode of the required resistance of the diaphragms and the necessary tension reinforcement may be determined. Strut-and-tie models are very suitable to determine the internal flow of forces, particularly when large openings in the floor are present. Fig. 38 shows the plan of an example diaphragm in which maximum in-plane shear forces are expected adjacent to the stiff perimeter frames. The simple strut-and-tie model shows that tensile forces can be conveniently resisted along beams, and that the floor panels can transmit the necessary diagonal compression forces. Figures 38(a) and (b) show admissible strut-and-tie models corresponding with different directions of the applied diaphragm inertia forces.

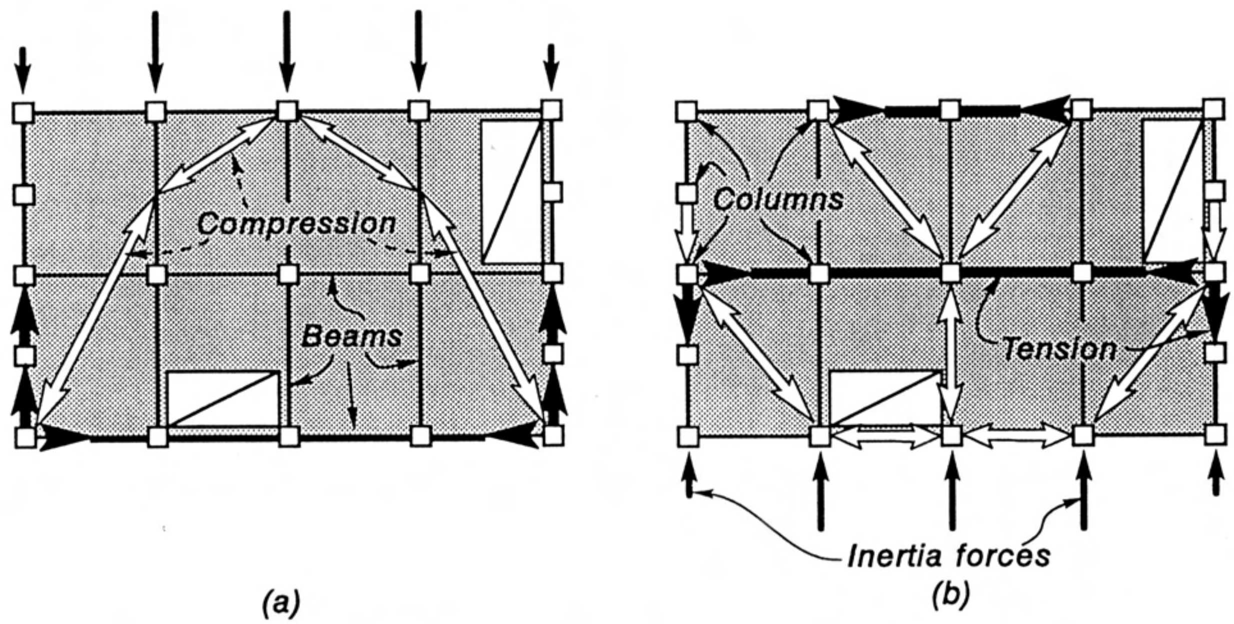


Fig. 38 The resistance, using diagonal compression fields, of an example floor diaphragm with openings.

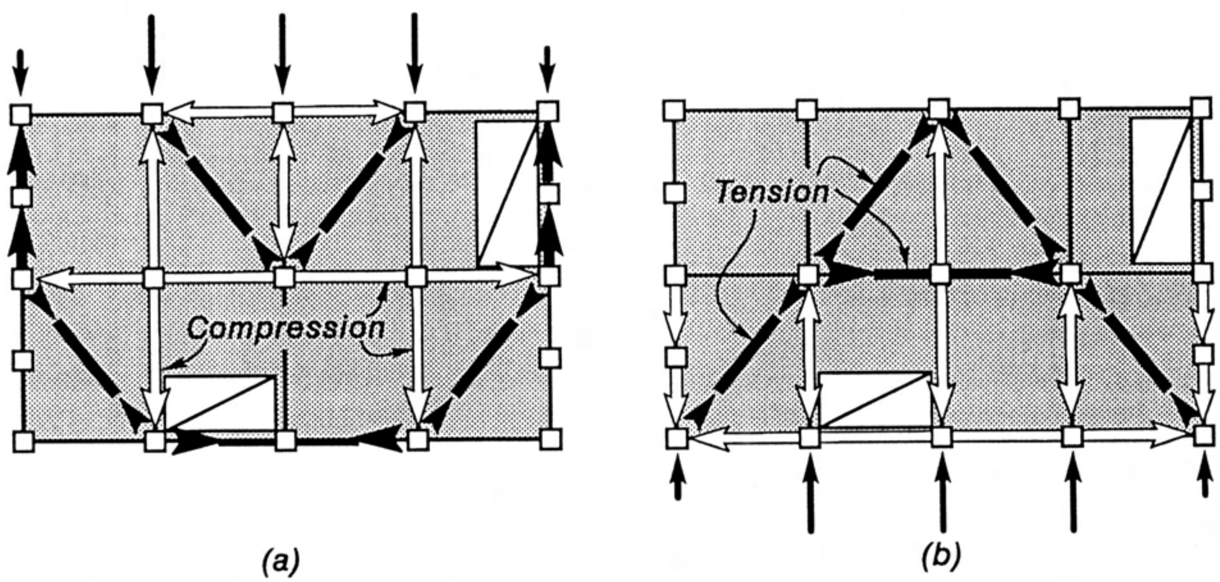


Fig. 39 The resistance, using diagonal tension fields, of an example floor diaphragm with openings.

When large ductility demands are imposed on the frames, generated by forces in the direction shown in Fig. 38, significant beam elongations in this direction, as shown in Figs. 12 and 13, may be expected. Diaphragm action in precast floor systems usually relies on a relatively thin topping slab with a light mesh of reinforcement. Large concentrated cracks can be expected where beam plastic hinges introduce significant tensile strains to the topping. The wide cracks may render the floor panels ineffective in diagonal compression. To enable under these circumstances a strut-and-tie mechanism within a diaphragm to function, the panels would need to provide a diagonal tension field while the North-South beams become compression members, as shown in Fig. 39. A light reinforcing mesh provided in topping slab is likely to be inadequate to transmit concentrated tension forces at the node points of the tension field, even if these forces are not significant in terms of nominal shear stresses. In such situations it is advisable to provide extra reinforcement in the topping slab, for example in the form shown in Fig. 40. This reinforcement should be well anchored in the diaphragm to allow a dispersal of the tension force within the panel and hence to enable the mesh reinforcement to be engaged efficiently in tension.

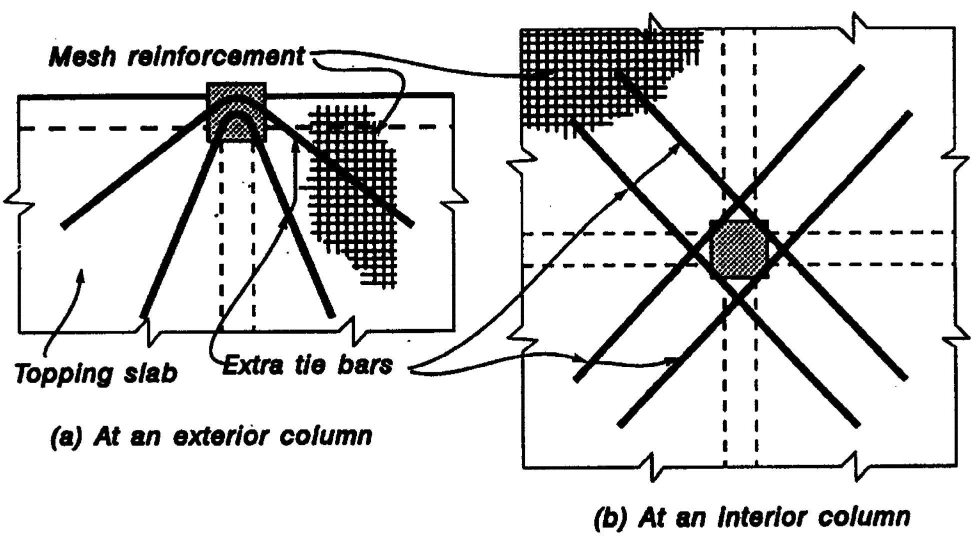


Fig. 40 Additional reinforcement recommended to develop a diagonal tension field in the topping of a precast floor system.

15 DETAILING FOR DUCTILITY

As a general rule, rationally-detailed structures can be made very ductile with relative ease involving little, if any, additional cost. Thereby a considerable reserve for inelastic deformations, that is *ductility capacity*, can be imparted to structural systems. *Detailing of reinforced concrete structures, very often considered a subordinate, depreciated drafting activity with apparent lack in intellectual appeal, deserves at least as much attention as the analytical work used to estimate design actions.* Faults in detailing are the first that will be revealed during earthquakes. They are predominant causes of structural distress. The detailing of potential plastic regions is partly an art. It relies on feel for and understanding of the natural disposition of internal forces and often invites innovations. Judiciously-detailed ductile systems will be *tolerant* with respect to imposed seismic displacements, an extremely valuable feature of structural response, that will compensate for the crudeness in predicting the magnitudes of such displacements.

Although this section addresses issues relevant to the estimated maximum ductility demand, it should be noted that many features of adequate detailing will have very beneficial effects on meeting serviceability criteria (Section 5.1). Moreover, well detailed structures, when subjected to intermediate ductility demands, will more readily meet the intents of the damage control limit state (Section 5.2), and are not likely to necessitate structural repair of significance.

The quality of detailing to be effective requires an assurance that the designer's intentions are in fact fulfilled during construction.

The correct placing and configuration of transverse reinforcement in the form of stirrups, hoops, spirals or ties is of paramount importance in potential plastic regions. The purpose of such transverse reinforcement is to provide: (i) shear resistance, (ii) lateral stability to principal reinforcing bars when these are subjected to compression, (iii) lateral restraint to confine concrete under large compression strains and (iv) clamping forces where bars are spliced by lapping. Often more than one of these actions need to be activated simultaneously. The following sections present associated features which may arise in various members. Where possible, comparisons are made between common practices considered to be *inappropriate* for seismic conditions with those recommended.

Many details that appear here have already been codified in building regulations. However, in the codes of several countries, no corresponding provisions can be found. Designers in such countries often adopt traditional detailing techniques which are associated with gravity load demands.

15.1 Beams

15.1.1 Curvature ductility demands. It was pointed out earlier that the concrete compression strain in the extreme fibre of the critical section is a critical parameter of curvature ductility capacity. Therefore in beams, which are the major sources of energy dissipation in frames, the concrete should be relieved from carrying excessive flexural compression forces. To this end a fraction of the quantity of tension reinforcement should be provided as compression reinforcement.

The possibility of major moment reversals must also be expected. Therefore some tension reinforcement must be present at both to top and bottom of beam sections. In earthquake dominated beams, such as shown in Fig. 3(a), about the same amount of flexural tension reinforcement may need to be placed at the top and bottom of the beam section. Therefore in such beams ample curvature ductility capacity is assured.

15.1.2 The anchorage of beam bars. Because plastic hinges in beams usually develop at the faces of columns, the anchorage of beam bars in the beam-column joint, taking into account reversible steel stresses possibly at overstrength ($\lambda_o f_y$), assumes vital importance. Codes specify anchorage lengths beyond the critical section over which bars can, be means of bond, develop full strength. Under earthquake actions, however, the mechanisms by which such bond forces can be equilibrated within a joint core must also be considered.

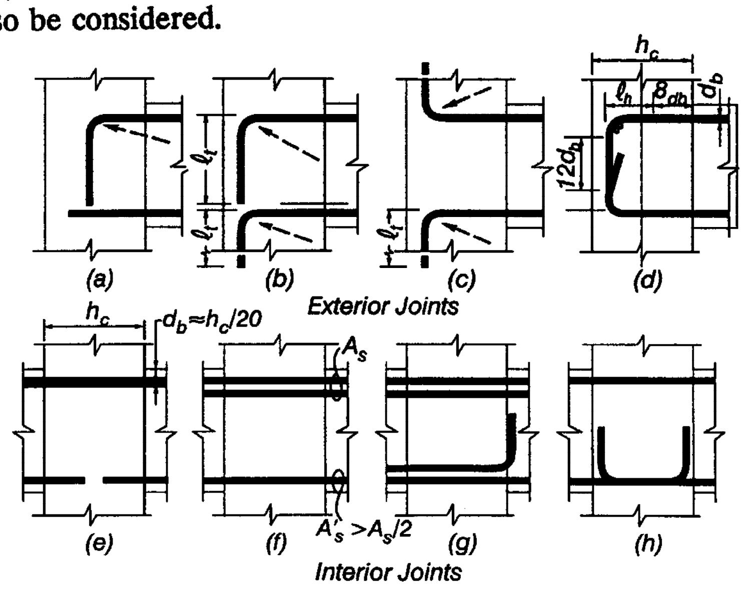


Fig. 41. Anchorage of beam bars in beam-column joints.

Figure 41(a) shows an unsatisfactory, yet not uncommon, anchorage detail at an exterior beam-column joint. The diagonal compression force generated at the bend of the top bar cannot be properly resolved into efficient internal components. The anchorage of the bottom reinforcement, typical in structures designed to resist gravity loads only, is inadequate when significant tension stresses are induced in these bars by seismic displacements. For convenience some designers are tempted to bend bottom beam bars down into a column, as shown in Fig. 41(b). It is evident that the major fraction of the anchorage force from the bottom reinforcement in tension will be directed into the column, instead of into the joint core, where a major diagonal strut, similar to that shown in Fig. 8(b), should be developed. Extended straight length, ℓ_t, provided beyond a 90° bend, is sometimes perceived to provide improved anchorage (Fig. 41(b)). However, tensile strains decrease rapidly over the vertical part of the hooks shown. Hence a straight extension of $12d_b$, recommended in several codes, should be considered as being more than adequate. Fig. 41(c) shows another example of unsatisfactory beam bar anchorage, used to ease bar congestion, particularly in joints of two-way frames.

As Fig. 41(d) shows, the aim should be to place standard hooks of beam bars as close as practicable to the far face of the column. The anchorage length, ℓ_h, stipulated for hooked tension bars, should ignore a length of the order of $8d_b$ or $0.5h_c$ from the inner face of the column as being effective in bond transfer. This is in recognition of the effects of reversing cyclic plastic hinge rotations which cause significant yield penetration along bars into the joint core. If the available length, ℓ_h, beyond the point at which effective bond transfer can be assumed is less than that specified by codes, transverse bearing bars should be placed within the bend, as shown Fig. 41(d), to prevent the concrete from being overstressed in compression within the bend. Particularly efficient anchorage at exterior joints can be provided when the use of stub-beams, as shown in Fig. 42, if possible.

Figure 41(e) shows unsatisfactory anchorage details at an interior joint. While the relatively large diameter top beam bars, violating the requirements of Eq. (5), will not be able to transmit bond forces without excessive bond-slip, the bottom bars would not be capable of developing significant tensile stresses at the face of the column.

To satisfy the severe bond requirements presented in Section 8.4.3, a larger number of smaller diameter bars, satisfying the limitations of Eq. (5) should be selected, while all bars should, as a general rule, pass continuously through the joint. Such a detail is shown in Fig. 41(f). However, the anchorage in tension of beam bars by means of a standard hook at or beyond the far face of an interior column, as seen in Fig. 41(g), is very satisfactory. This particular example suggests that an earthquake dominated short span beam (Section 9.1) with similar top and bottom reinforcement may be required at the left of the interior column, while the magnitude of the positive moment immediately to the right of the column is to be kept lower because of significant gravity loads on a long span beam.

When precast beams with cast in place floor systems are used, the splicing within the joint of the bottom bars, as shown in Fig. 41(h), before the joint core concrete is cast, provides very satisfactory anchorage.

15.1.3 <u>The curtailment of beam reinforcement.</u> To satisfy the intent of the design strategy outlined in Section 7.2, it is essential to ensure that yielding of the beam reinforcement can occur *only* at clearly defined plastic hinge regions. Therefore in other regions along the span of a beam, flexural reinforcement in excess of that indicated by the bending moment envelope must be provided to ensure in any event the elastic response of these regions. As a consequence such regions do not need to be detailed for ductility.

In the curtailment of the flexural reinforcement the effect of "tension shift" caused by shear in diagonally cracked members must be taken into account. The increase of internal tension forces at any section in the shear span to a value above that implied by the bending moment, caused by this effect, depends on the inclination of the diagonal compression field and the shear force resisted by the shear reinforcement that has been provided. As a simple and conservative approach, the flexural reinforcement may be provided so as to be able to resist at a section at full strength the larger moment that would occur at section located at a distance equal to that of the effective depth of the beam.

Figure 42 shows qualitatively the flexural capacity that should be provided in a span in relation to the moment envelope. The transverse reinforcement to be provided at the potential plastic hinges with negative moments should extend over a distance 2h from the relevant critical section. This is in recognition of the likelihood that at the development of flexural overstrength, yielding of the reinforcement may spread over a length larger than the overall depth of the beam.

15.1.4 Shear strength. Because of the inevitable deterioration of mechanisms associated with shear resistance, the entire shear force at the plastic hinges in a beam, such as seen in Fig. 42, should be assigned to shear reinforcement assuming a 45° diagonal tension failure plane. Moreover, to prevent premature crushing of the concrete due to reversing cyclic shear in the plastic regions, the shear stress at flexural overstrength should not exceed $0.16f_c'$ or $0.85\sqrt{f_c'}$ (MPa).

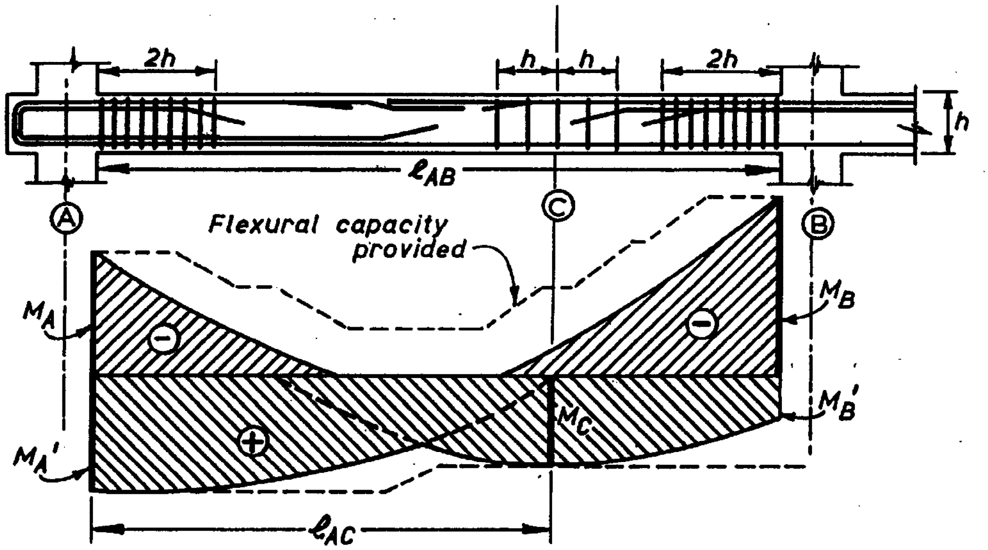


Fig. 42. Location of plastic hinges where special stirrup-ties are required.

With high reversing shear stresses in plastic hinges a *sliding shear failure* can occur after a few shear and moment reversals. Therefore, it has been recommended (Standards New Zealand, 1995) that when the nominal shear stress, v_n, exceeds $0.25\ (2 + r)\sqrt{f_c'}$ (MPa), diagonal shear reinforcement should be provided across the web in the plastic hinge in one or both directions to resist the fraction, V_{di}, of the total shear force, V_n, thus:

$$V_{di} = 0.7\left(\frac{v_n}{\sqrt{f_c'}} + 0.4\right)(-r)\,V_n \tag{32}$$

where r is the algebraic ratio at the plastic hinge section of the numerically smaller to the larger shear force, V_n, when reversal of the direction of shear force can occur, always taken negative. V_{di} need to be considered only when $-1 \le r \le -0.2$. Typical details of such reinforcement are shown in Fig. 43, where from Eq. (32)

$$A_{sd1} + A_{sd2} = V_{di}/(f_y \sin\alpha) \tag{33}$$

Note that the transverse components of both the diagonal tension and compression steel forces may be utilized to resist V_{di}.

15.1.5 The stability of compression reinforcement. The failure of the compression reinforcement by buckling in plastic hinges of beams, columns and walls is the largest single cause of collapse of buildings during major earthquakes. The factors contributing to bar buckling, some of which are illustrated in Fig. 44, are: the spalling of the concrete cover due to excessive compression strains or due splitting along bars originating from the transmission of bond forces, premature yielding of the

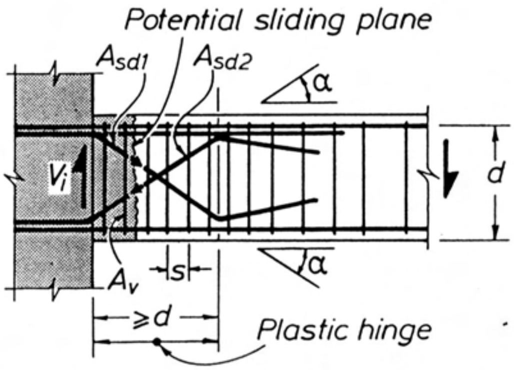


Fig. 43. Control of sliding shear in the potential plastic hinge regions of beams.

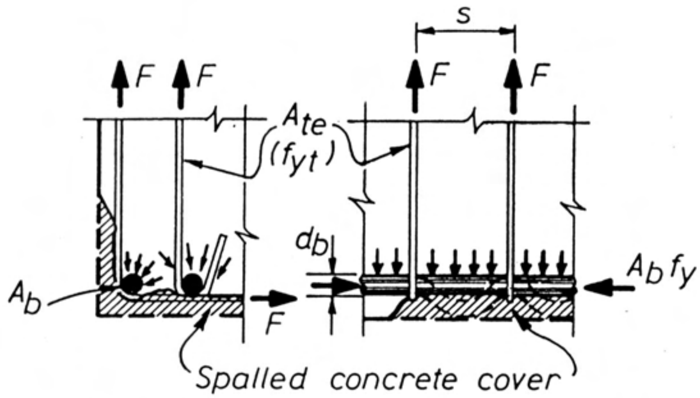


Fig. 44. Lateral restraint to prevent the premature buckling of compression bars in plastic hinge regions.

transverse shear or confining reinforcement, transverse sliding shear displacements along interconnecting flexural cracks, transverse pressure exerted by the progressively damaged concrete in the core of the section and Bauschinger effects leading to the dramatic reduction of the modulus of elasticity of steel. Because of the complex interaction of these effects during reversing inelastic deformations at plastic hinges, the theoretical prediction of the buckling phenomena is in most situations difficult. An empirical approach, introduced some 15 years ago in New Zealand and subsequently found in numerous experiments to result in satisfactory performance with displacement ductility demands up to $\mu_\Delta = 8$, suggests that the strength in tension of one transverse tie F, shown in Fig. 44, provided at $s = 6\,d_b$ centres, should be not less than 1/16 of the yield strength in compression of the bar with area A_b, or group of bars, ΣA_b, to be restrained. Using the notation in Fig. 44 this requirement reduces to

$$A_{te} \geq \frac{\Sigma A_b f_y}{96 f_{yt}} \frac{s}{d_b} \qquad (34)$$

with the tie spacing, s, not exceeding $6d_b$ in plastic hinge regions. It is seen that the area of transverse reinforcement required to stabilise a compression bar increases with the diameter, d_b, of principal reinforcement. This requirement is also applicable to bars in all regions of columns or walls where yielding of the principal reinforcement could occur.

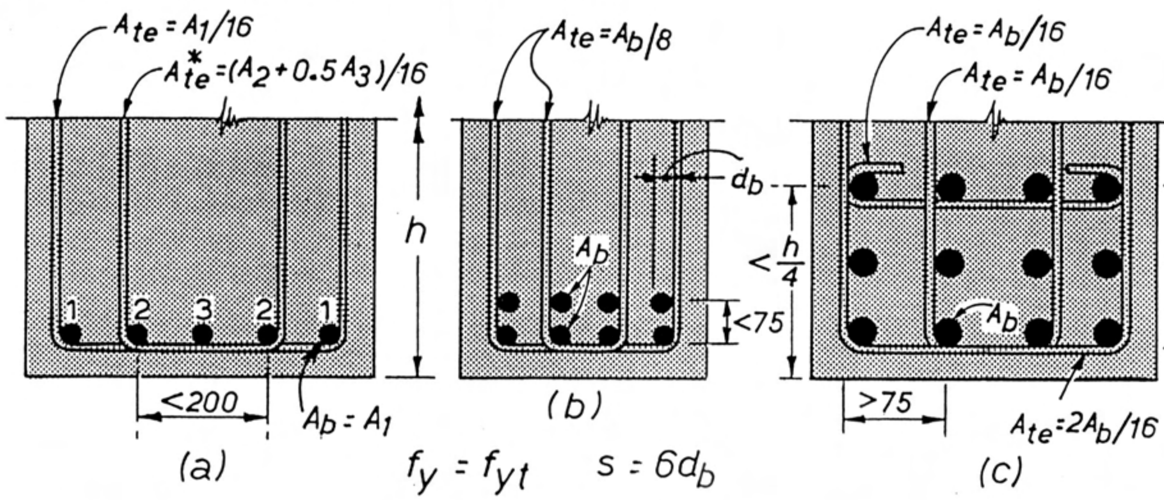


Fig. 45. The arrangement and size of stirrup-ties at $6d_b$ centres in potential plastic hinge regions.

Figure 45 illustrates the interpretation of these recommendations for typical bar arrangements used in beams. Figure 45(a) shows that a bar need not be tied when it is placed between tied bars that are not further than 200 mm apart. However, the area of ties needs to be correspondingly increased. Figures 45(b) and (c) show interpretations when beam bars are arranged in more than one layer.

15.2 Columns

15.2.1 The confinement of the compressed concrete in columns. It is inevitable that in a plastic hinge of a column and sometimes in that of a wall, as outlined in Section 12.3, the concrete compression strain developed under earthquake imposed curvature ductility demands, becomes excessive. In particular this is the case in the presence of significant axial compression load. In such cases it is necessary to confine the compressed concrete against lateral expansion. Thereby the compression strain and hence the curvature ductility capacity of the region can be increased. How this can be achieved in walls was discussed in Section 12.3.2 and shown in Fig. 28. In the potential plastic hinge regions of rectangular columns, typically at the base of a frame, as shown in Fig. 3(a), confining rectangular hoops with or without supplementary ties in each of the principal directions, with an effective area to, A_{sh}, should be provided, so that

$$A_{sh} = \frac{(1.3 - p_t m_f)\, s_h h''}{3.3} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{P_u}{\phi f_c' A_g} - 0.006 s_h h'' \qquad (35)$$

where
p_t = ratio of non-prestressed longitudinal column reinforcement
m_f = $f_y/(0.85 f_c')$
A_g = gross area of section, mm^2
A_c = area of concrete core measured to outside of peripheral hoop, mm^2
P_u = design axial load at the ultimate limit state, N
f_c' = specified compression strength of concrete, MPa
f_{yt} = lower characteristic yield strength of transverse reinforcement, MPa
ϕ = strength reduction factor, taken as unity when column actions are derived using capacity design procedures
s_h = centre-to-centre spacing of horizontal hoop sets, mm
h'' = dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm (see Fig. 46(c))

The following restrictions also apply:

(1) A_g/A_c not be taken less than 1.2
(2) $p_t m_f$ not be taken greater than 0.4
(3) f_{yt} not be taken larger than 800 MPa
(4) s_h not to exceed the smaller of 1/4 of the least lateral dimension of the cross section or 6 times the diameter of the longitudinal bar to be restrained.

This recommendation originates from extensive theoretical and experimental studies (Mander *et al.*, 1988; Zahn *et al.*, 1986; Watson *et al.*, 1994) carried out in New Zealand.

It should be noted that:

(i) It is intended to develop effective lateral confining pressure by means of the large diameter vertical bars which bear against the core concrete of the column. These bars in turn are held in position by closely spaced legs of hoops or ties. This is implied in Figs. 46(a) and (b), which show by shading the area of concrete which, because of spalling, must be expected to become ineffective in carrying compression stresses. Fig. 46(a) is an example of less effective confinement.
(ii) The amount of confining reinforcement must be increased as the axial compression load, P_u, increases, because, as a consequence, the portion of the section area subjected to flexural compression increases and hence the expected curvature ductility will result in increased strains at the extreme compression fibre. This requirement of Eq. (35) is *different* from that implied by those of some other codes, which consider the quantity of confining reinforcement as being independent of the intensity of the axial load.

(iii) The use of 135° hooks with the straight leg following the bend *anchored in confined concrete* is essential. Under significant ductility demands 90° hooks in the plastic hinge regions of columns, often used, will straighten and hence the effective anchorage of the transverse tie will be lost. The phenomenon has been repeatedly observed in many earthquakes.

(iv) The transverse reinforcement provided for confinement must not be less than that required by Eq. (34) given in Section 15.1.5.

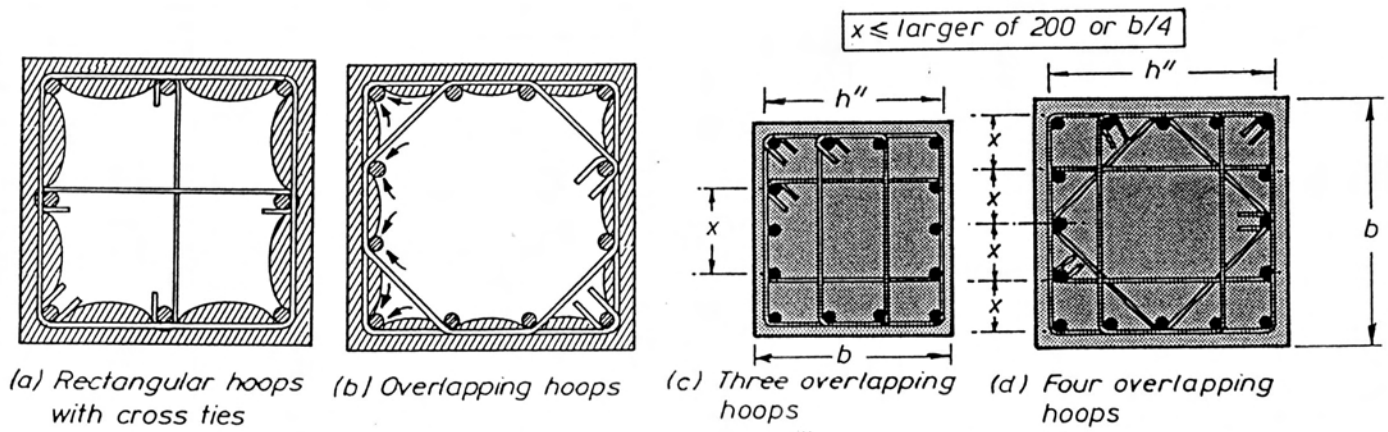


Fig. 46. Typical overlapping hoops used to confine the concrete core in columns.

15.2.2 Regions to be confined. Figure 47 shows a typical exterior column in the first storey with both ends detailed to sustain a plastic hinge. Ductility demand at the top end of the column would arise in circumstances as outlined in Section 10. The moment demand corresponding to the seismic design forces is M_E. The nominal flexural strength of the column as finally detailed, with an axial compression force, P_u, is M_n. After the imposition of the expected maximum curvature ductility demand at the base, typically of the order of $\mu_\phi \approx 20$, the flexural overstrength of the section, $\phi_o M_n$, will be mobilized (Fig. 47(b)). The significant increase of the moment of resistance of the base section of this column subjected also to significant earthquake induced axial compression load, results from both the increased compression strength of the confined concrete in the core of the section and strain hardening of the tension reinforcement.

Likely limits of the moment gradients developed during the inelastic dynamic response of the structure are also shown in Fig. 47(b). These emphasize that moments significantly larger than the nominal flexural strength of the column, M_n, may be developed also at sections a short distance away from the base. If adequate confining reinforcement is not provided in this region, a situation often encountered in practice, unexpected failure, as illustrated in Fig. 47(a) and observed in tests, may occur.

Therefore it is recommended that the amount of confining transverse reinforcement be reduced only gradually toward the midheight of such columns. In some cases it will be more practical to provide the full amount of required transverse reinforcement (Eq. 36) over the full height in the first storey, particularly when $P_u > 0.4 f'_c A_g$.

15.2.3 Lapped splices. It is generally recognized that reinforcing bars should not be spliced in regions where high steel stresses are expected. For this reason in columns where plastic hinges can be expected to develop at either or both ends, such as shown in Figs. 3(b), (d), (e), (h) and (i), splices must be located within the mid-height region. However, lapping of bars in a column should be acceptable at its ends, provided that the column is protected against plastic hinge formation (Fig. 3(a)) by capacity design procedures, such as outlined in Section 8.3. A convenient arrangement of a splice for bars in such a column is shown in Fig. 48.

During an earthquake, high reversible stresses below yield level will occur at the ends of protected columns in the upper storeys of ductile frames (Fig. 3(a)). To ensure that over the splice length, ℓ_s, the required forces with a large number of stress reversals can be transferred from on bar to the adjacent lapped bar, a significant amount of transverse reinforcement is required.

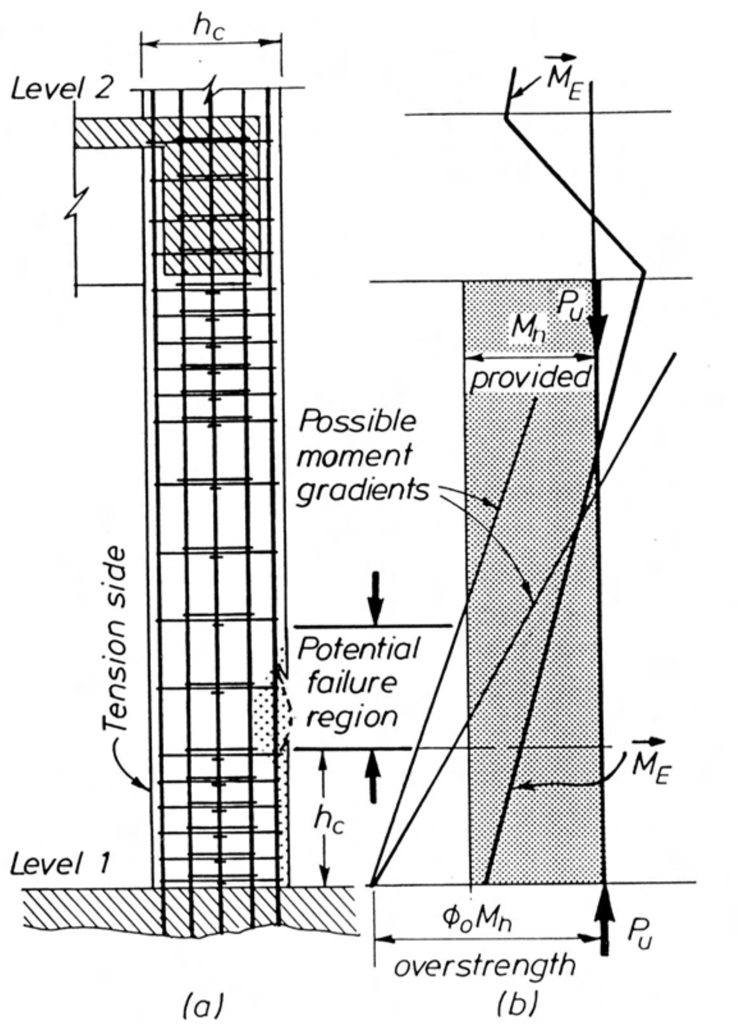


Fig. 47. The length of confinement in columns as affected by the overstrength of the base.

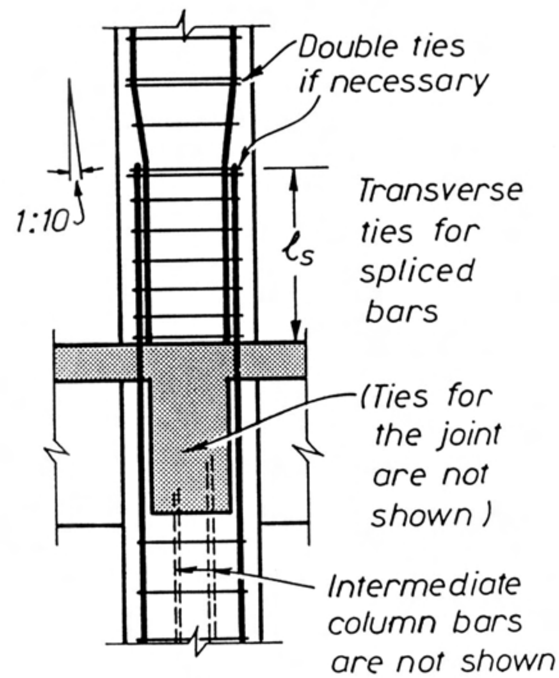


Fig. 48. Splice details at the end region of a column.

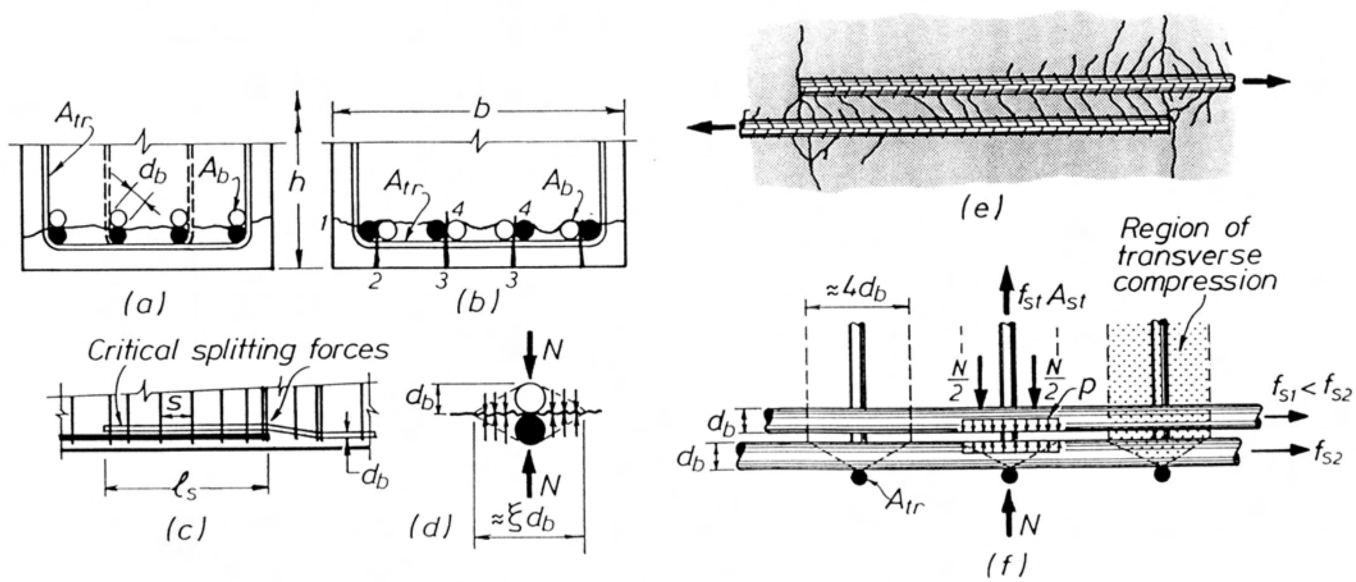


Fig. 49. Force transfer mechanism at a lapped splice.

Figure 49 shows two different arrangements for lapped bars and the assumed mechanisms of bond transfer. To enable shear to be transferred by the concrete between two bars (Fig. 49(e)), an externally applied transverse compression force, N, is also required (Fig. 49(d)). As Fig. 49(f) shows, this may be introduced by transverse reinforcement with reasonably close spacing, s. The relevant recommendations of Eqs. (34) and (35) may be considered to satisfy the spacing requirements at splices. After the formation of splitting cracks, shown in Fig. 49(a), a shear-friction mechanism is mobilized. Using simple equilibrium criteria, it may be shown that the area of the required transverse reinforcement across each pair of lapped column bars of the type shown in Fig. 48 is

$$A_{tr} \geq \frac{d_b f_y}{48 f_{yt}} s \tag{36}$$

where the notation is defined in Fig. 49.

15.2.4 Footings. As a general rule footings and components of the foundation structure should respond to earthquakes within the *elastic domain* (Paulay and Priestley, 1992). With the evaluation of the shear and axial forces, and moments at the base of columns, such as shown in Fig. 50, at the development of the overstrength of the superstructure, the input to the foundation system is known. Hence the required strength of the components of the foundation system, such as footings, corresponding with nominal yield strength can be readily established. Accordingly such components will be protected and need thus not be detailed for ductility.

Figure 50(a) reproduces widely used details of a footing supporting a single column transmitting typical seismic actions. Three *unacceptable features* should be noted. The convenient column starter bars are spliced with the main bars where a plastic hinge is expected to form. The bending outward of the column bars at the bottom of the footing does not ensure continuity of earthquake induced moment transfer from the column to the footing. The column-footing junction, behaving like a knee-joint of a portal frame, has insufficient joint shear reinforcement.

Features of detailing considered to be appropriate for this region are shown in Fig. 50(b). The splice is located approximately at the midheight of the column (Section 15.2.3). The splice between bent-over column bars and the footing reinforcement should enable effective moment transfer within the elastic domain. Properly designed joint shear reinforcement is assumed to have been provided.

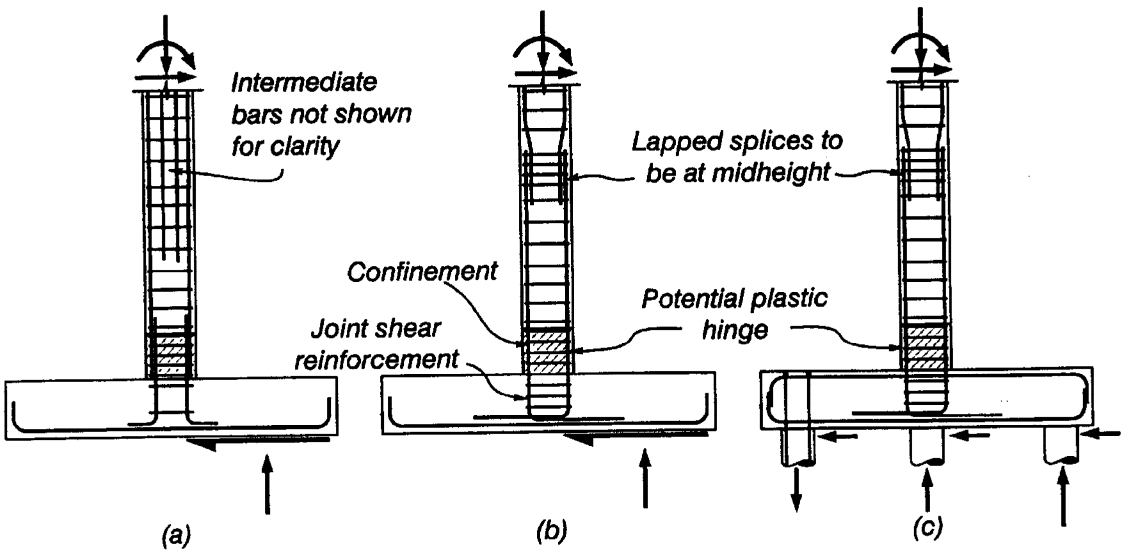


Fig. 50. Seismic design considerations for footings.

Similar details apply in the example shown in Fig. 50(c), where a pile cap is used. The design moments derived from the specified earthquake design forces may not indicate tension in any of the piles. However, under large ductility demands the flexural overstrength of the plastic hinge at the base of the column, may be much larger than the initial design moment. Hence tension in some piles may develop. This in turn requires reinforcement in the top of the pile cap. This example demonstrates the *ability of capacity design* to disclose vital features associated with real seismic response. A conventional design based on the ultimate limit state may never have indicated that reinforcement in the top of the pile cap, shown in Fig. 50(c), would also be required.

15.3 Structural Walls

Some features of the detailing of walls for ductility have been reviewed in Section 12. Only three aspects are briefly addressed here.

15.3.1 Design moment envelopes. The choice of rational plastic mechanisms (Section 7.2) and subsequent execution of the intents of capacity design enables most parts of the structure to be *protected* against inelastic deformations. Thereby, as stated earlier, these parts do not require special detailing for ductility. For a cantilever wall the obvious choice for the location of a potential plastic hinge is at its base. While the flexural overstrength at this level can be determined with sufficient accuracy, the limits of moment intensities over the remaining height of the cantilever will not be unique but will change during the ground shaking. The reason for this is that the major elastic portion of the wall above the base remains sensitive to response in the higher modes of vibrations.

Instants of typical cantilever moment patterns that can be expected, are shown in Fig. 20. To enable simple detailing of the reinforcement to be used over the very considerable extent of the wall, a *moment envelope* should be used which will ensure that yielding of significance will not occur outside the plastic hinge region. For this purpose the envelope shown in Fig. 51 has been recommended (Standards New Zealand, 1995). Accordingly, bars that are being curtailed should extend by the full development length beyond the level at which according to the horizontally shaded envelope they are not required to contribute to flexural strength.

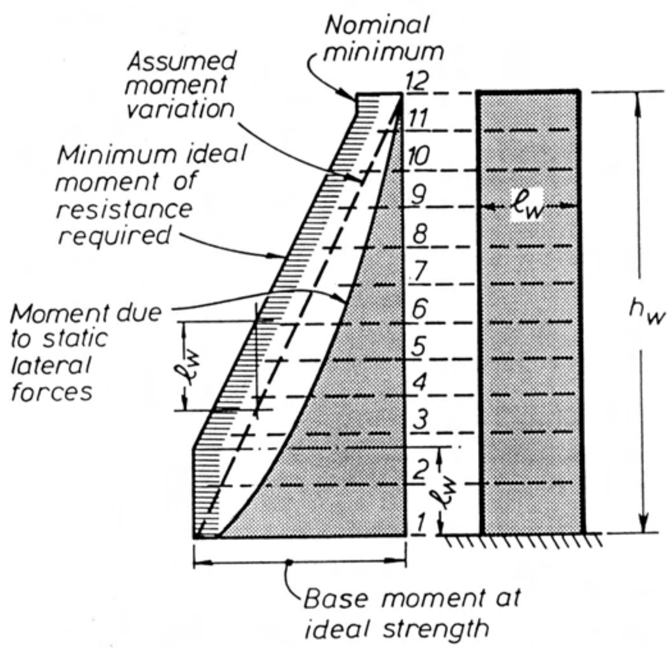


Fig. 51. Recommended design moment envelope for cantilever walls.

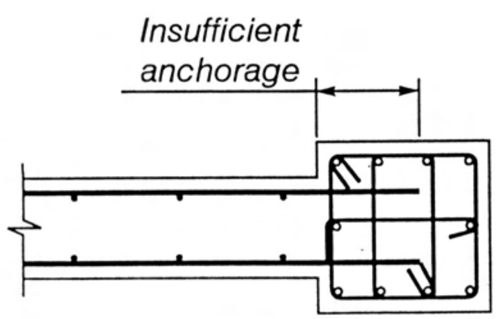


Fig. 52. Unsatisfactory anchorage of the shear reinforcement in walls.

15.3.2 The anchorage of shear reinforcement. Figure 52 shows the often used unsatisfactory termination of the horizontal wall reinforcement in enlarged boundary elements. It should be noted that the primary role of the horizontal bars is, being part of a truss mechanism, to resist shear forces. To enable the diagonal concrete compression and the vertical bond forces, introduced by the tension reinforcement in the boundary element, to be equilibrated, the horizontal bars in the wall should be anchored with a standard hook as close as practicable to the outer face of the boundary element. Such a detail is shown by the dashed lines in Fig. 28.

15.3.3 Coupling beams of structural walls. Two or more walls in multistorey buildings are often interconnected at each floor by relatively short and deep beams. The dimensions of these beams depend on the size of openings arranged in one or more vertical rows. These connecting elements, generally referred to as coupling beams, are usually subjected to high shear stresses. Hence, when adequately reinforced to ensure that diagonal tension failure cannot occur, they are prone to fail by *sliding* at a section of maximum bending. Because of their geometry, it is convenient to reinforce them by diagonal bars instead of conventional horizontal top and bottom flexural reinforcement. A specific example, giving also various construction details, is shown in Fig. 53. By resolving the design shear force, Q, the diagonal tension force, T_b, and hence the required amount of reinforcement, can be derived from elementary principles. Coupled walls reinforced in this manner and extensively used, have been found to promise excellent seismic performance (Park and Paulay, 1975).

It is recommended that diagonal reinforcement in the form seen in Fig. 53 be provided in coupling beams of ductile wall systems, whenever the nominal shear stress associated with the design shear force at the development of beam overstrength is such that

$$v_n > 0.08\, \phi_o \frac{L_n}{h} \sqrt{f_c'} \text{ (MPa)} \tag{37}$$

where L_n and h are the clear span and the overall depth of the coupling beam, respectively. As Fig. 53 shows, ties, often in the form of rectangular spirals, in accordance with Eq. (34), must be provided around the main diagonal bars to prevent their premature buckling when subjected to compression.

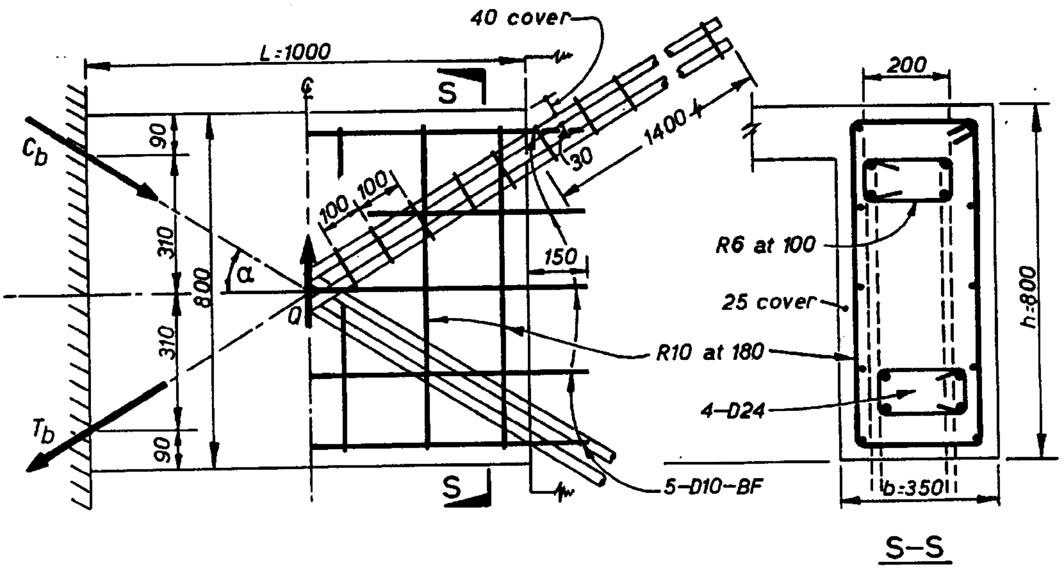


Fig. 53. Details of typical diagonally reinforced coupling beams.

16 CONCLUSIONS

Within the framework of our ambitions to mitigate the effects of natural disasters, particularly in developing countries, this address presented some features of the design of building structures exposed to the effects of earthquakes. In this it was suggested that engineers have an obligation, beyond the lofty aims of reaching into the future to ensure further progress and safety for developed societies, to make their knowledge accessible *now* to less developed regions of great seismic risk.

A design strategy and its application were outlined for reinforced concrete buildings in which earthquake resistance was provided by ductile structural systems. Emphasis was placed on design features which over the past 25 years were primarily developed in New Zealand. The design strategy described evolved from the following precepts:

(1) In the context of the state of the art in structural engineering, current predictions of the probable characteristics of large earthquake-generated ground motions are crude. Under these circumstances an aim to achieve a degree of *precision* in analytical techniques, comparable to those developed for structures to satisfy serviceability and "hypothetical" ultimate limit states, to predict both earthquake induced actions and deformations within the structure, is *not justified*.

(2) Provided that a reasonable level of resistance to lateral forces, such as prescribed for various seismic regions by relevant national building codes, is chosen, then the inevitable errors arising from crude estimations of the characteristics of ground motions will manifest itself only in *erroneous predictions* of earthquake imposed displacements, that is *ductility demands*. Thus *deformation capacity reserve* is an extremely important structural property in areas of high seismic risk.

(3) Types and localities of energy dissipation mechanism need to be chosen as part of a *capacity design procedure*, in which a unique hierarchy of strengths is established. All weak and necessarily ductile links must satisfy requirement of the stipulated level of lateral force resistance. In such an inelastic system the *maximum resistance that may be developed* during a major earthquake can be predicted with a relatively high degree of precision. However, ductility demands during an earthquake, being dependent on ground motions, may differ from those anticipated or assumed in building codes.

(4) As a general rule, *rationally detailed* reinforced concrete structures can be made very ductile with relative ease and little, if any, additional cost. Thereby a considerable reserve for potential inelastic deformations, that is *ductility capacity*, can be imparted to structural systems. Detailing of reinforced concrete structures deserves at least as much attention than the analytical work used to estimate design actions. Faults in detailing are the first that will be revealed during earthquakes. They are predominant causes of structural distress. Judiciously detailed ductile systems will be *tolerant* with respect to imposed seismic displacements, a valuable feature of structural response, that will compensate for the crudeness in predicting magnitudes of such displacements.

(5) Various steps in the description in previous sections of the design procedure were intended to emphasise the designer's *determination* to simply "tell the structure what to do". It is in this respect that the design strategy is *deterministic*. It inhibits the activation of mechanisms other than *those chosen*. The numerous detailed recommendations presented were intended to manifest unambiguously the *goodness of detailing*. Thereby reinforced concrete buildings can be made very *tolerant* to a wide range of ductility demands. Hence they can be expected with *confidence* to perform "as they were told to".

The primary intention of this presentation was to address structural design practitioners who are in the best position to utilize knowledge transfer with the greatest immediate benefit to those societies that are exposed to seismic hazards. In this context the quality of construction in the relevant country requires particular attention. Informed design practitioners are best qualified to cope with possible local shortcomings in construction activities, such as poor quality control, the scarcity of skilled personnel and inadequate field supervision. To make disaster mitigation more effective promptly, self reliance and determined commitment, including that for training, must be promoted.

This role could be eminently fulfilled by national earthquake engineering societies which make up the International Association for Earthquake Engineering. This indeed should be their foremost aim rather than representation. With the intimate knowledge of national needs, economic constraints, facilities for training and societal traditions, such national associations should have the ability to achieve significant advancement in earthquake engineering by identifying and hence addressing relevant needs that exist. National workshops relying on the informal and usually voluntary combined contribution of dedicated engineering practitioners and researchers, may be very effective in serving this purpose. The most advanced levels in applied structural design, meeting the challenges of future earthquakes, developed in a few countries, have largely been achieved this way.

17 ACKNOWLEDGEMENTS

The progress in seismic design reported here would not have been possible without the sustained support of and motivation by the New Zealand National Society for Earthquake Engineering, which for decades fostered steady dialogues between relevant branches of the engineering profession. This led to the transfer of the findings of the numerous study groups into the relevant New Zealand Standard, a comprehensive and detailed document.

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