SEISMIC DESIGN IN REINFORCED CONCRETE: THE STATE OF THE ART IN NEW ZEALAND

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SUMMARY

Highlights of the evolution over the past two decades of a seismic design strategy, used in New Zealand for reinforced concrete buildings, are reviewed. After a brief outline of some philosophical concepts of the capacity design methodology, the main features of its application with respect to ductile rigid jointed frames, structural walls and hybrid structural systems are sketched. Another aim of this strategy, complementary to ductility requirements, is to strive for high quality in detailing. Numerous examples are presented to illustrate how this can be achieved. A specific intent of this state of the art review is to report on features of design and detailing which are considered to have originated primarily in New Zealand.

1. INTRODUCTION

This review attempts to highlight certain features in recent developments of the structural design of reinforced concrete buildings in New Zealand. It concentrates on issues of seismic resistance as they emerged over the last two decades. Discussed in some detail are those aspects of design which were identified and studied primarily in New Zealand. Some of these studies resulted in recommendations which in due course were embodied in relevant building codes. Some of these code provisions, to be examined subsequently, appear to have no parallel recommendations in codes of other countries, which are being frequently consulted in New Zealand. Emphasis in presentation is, however, placed on structural behaviour under seismic actions and concepts of design strategies and not on codes.

The developments reported did not evolve in isolation. The reported research findings of other countries, particularly those in the United States and Japan, were also exploited.

After the general acceptance of the strength method of design for concrete structure in the decade following the second world war, it was increasingly recognized that those of its precepts which were relevant to a seismic scenario, required thorough reexamination. In particular the effects of reversed cyclic deformations imposed by large earthquakes, well beyond elastic limits, upon properties such as strength, stiffness, stability and energy dissipation had to be evaluated and translated into recommendations in terms of usable design office practice. While the great majority of the features of the seismic design methodology reviewed here were developed from extensive theoretical and experimental research work, there are others which are based on less quantifiable common sense engineering judgements. The latter emerged during an extensive and continuous dialogue between design engineers and researchers in New Zealand.

The abandonment of the use of permissible stresses and the subsequent de-emphasizing of the importance of accuracy in the prediction of quantities, relevant to elastic structural response to code prescribed forces or to simulated earthquake excitations, led to a relatively simple deterministic design philosophy. Because attention is primarily focussed on the likely effects of very large earthquakes, this philosophy readily allows the designer to prescribe the details of a desirable response in the fully plastic state without jeopardising the requirements of damage control. Amongst other features, the methodology enables a relatively flexible but intelligent selection of member capacities to be made, leading to a uniquely defined and enforceable strength hierarchy within the structural system. The philosophy is deterministic in the context that, irrespective of the severity of the seismic event, the designer can "tell the structure what to do".

First the simple concepts of this seismic design strategy, as currently used in New Zealand, are briefly reviewed. Subsequently some features of its application are illustrated. In this, reinforced concrete rigid jointed ductile frames, structural wall systems and hybrid structures are considered. A corner-stone of implementation is the requirement that rational analyses and the proper derivation of design quantities be accompanied by the kind of detailing of the construction which will satisfactorily meet in critical regions of the structure the exceptional ductility or strength demands of a large earthquake. For this reason the second and major part of this review is devoted to the quality in detailing. Specific examples, relevant to the basic types of building structures, have been chosen to illustrate how attempts were made in New Zealand to quantifiy high quality detailing. The review concludes with the sketching of a few innovative solutions, developed to meet

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specific demands arising from seismic effects.

2 CONCEPTS AND APPLICATION OF CAPACITY DESIGN PRINCIPLES

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

When the strength of one element is compared with that of another element, it was recognized that it is necessary to evaluate the likely strengths mobilized during large displacements imposed by severe earthquakes. To quantify various kinds of strength, new definitions had to be introduced [25]. Ideal [41] or nominal [2] strength, S_i, is a common term used in $S_e \leq S_{ed}$ = the strength design approach. This is obtained from first principles and using a failure geometry and specified material strengths. The dependable strength is then obtained from $S_d = \phi S_i$, where ϕ is a specified strength reduction factor [2,41]. During a large inelastic seismic pulse, material strengths considerably larger than values assumed or specified may be mobilized. For example steel strength at strain hardening may develop. Concrete strength may be enhanced by confinement. Moreover, for practical or other reasons, more reinforcement may have been provided at critical sections than what design equations indicated. All factors taken into account allow the over-strength to be estimated as

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

where λ_o is the overstrength factor relevant to a particular section. Its value ranges typically from 1.25 to 1.60, depending mainly on the grade of steel used and the ductility to be developed [1].

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

$$\phi_o = S_o/S_e = M^o/M_E \qquad (2)$$

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

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

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

$$S_i = \omega \phi_o S_e \qquad (3)$$

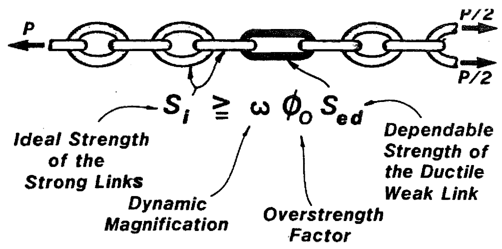


FIG. 1 - STRENGTH HEIRARCHY OF LINKS IN A CHAIN

where S_i = the required ideal strength of a strong link, S_{ed} = the dependable strength assigned to the adjacent weak link with the use of a code specified lateral earthquake loading, ϕ_o = overstrength factor (Eq. (2)) derived from the maximum likely strength of this weak link, and ω = dynamic magnification factor which estimates the deviation of load patterns during the inelastic dynamic response of the structure from corresponding patterns predicted by elastic analyses for specified lateral static loads. Only the highlights of the application of this simple concept are illustrated in the following sections with respect to two different structural systems. Capacity design procedures for concrete structures are formally in use in New Zealand since 1982 [41]. They have also been incorporated in parts of other codes [7,8].

As the ideal strength, assigned by this technique to the strong links in the chain of resistance (Eq. (3)), is an upper bound estimate of load input at a stage when significant damage has already occurred in the structure, no further precautions should need be applied. Hence the ideal strength S_i of the strong links need not be reduced by the customary strength reduction factor, ϕ [2,41].

2.1 Moment Resisting Ductile Frames

2.1.1 The aims of the design approach.

For multistorey frames the primary aim is to ensure that a storey column mechanism, consisting of simultaneous plastic hinges at both ends of all columns in any storey, will not occur. This principle is generally accepted in most codes [15]. It led to the "weak beam - strong column" system in ductile frames. However, the amount of reserve strength of strong columns in comparison with the weak beam, is seldom fully quantified. Some current procedures [16] aim at reducing the possibility of storey mechanisms while admitting the development of plastic hinges in columns either at the top or the bottom of a storey.

Case studies [31] indicated that, without imposing economic penalties, it is possible to provide columns with sufficient flexural strength so that plastic hinges should not be expected in any column in the upper storeys. In this context limited yielding of the steel is not synonymous with a plastic hinge and associated plastic rotations. This approach may require slightly larger amounts of vertical reinforcement in columns but accrued benefits are substantial. These are:

1. Ductilty demand in the end regions of columns does not arise or is negligible. Hence, there is no need to provide substantial transverse confining reinforcement. Congestion may thus be reduced.

2. As reversed cyclic inelastic response is not expected at the ends of columns, the contribution of concrete mechanisms to shear strength during an earthquake is not diminished. Therefore less shear reinforcement may be used.

3. Columns, which are more difficult to repair, enjoy a much greater degree of protection against structural damage.

4. Lapped splices, which should never be located in potential plastic hinge regions, can be used immediately above floors. If a plastic hinge in a column is to be expected, and this is the case of most current design procedures used in other countries, splices must be located at the midheight of columns. This represents some inconvenience in construction.

5. The prevention of yielding of column bars will improve the response of beam-column joints.

6. Slightly larger columns provide improved anchorage for beam bars, a task which is often difficult to fulfill in beam-column joints of ductile frames.

Through the work of the New Zealand National Society for Earthquake Engineering [21], the subsequent publication of the Code of Practice for the Design of Concrete Structures [41] and its commentary, and incorporation in the curricula of the schools of engineering at the Universities of Auckland and Canterbury, many features of the procedure are now used by a large section of designers in New Zealand. Two of the more interesting features of this design procedure, as yet not widely used outside New Zealand, are briefly reviewed in the following sections.

2.1.2 Moment redistribution

The redistribution of design actions, well known since the use of plastic design of steel structures, has seldom been used in the design of reinforced concrete frames. Routine procedures utilized the superposition of actions, obtained with separate analyses of the elastic structure for both gravity and lateral loading. With the recognition of the great importance of the inelastic response of structures, the useful role of the redistribution of design actions, such as bending moments, is now better appreciated.

Moment redistribution is a useful technique with which the designer can influence structural behaviour so as to achieve certain aims. Uneven moment demands, as illustrated by the top curves in Fig. 2, often do not permit the efficient strength utilization of the structural depth in prismatic concrete members. A reassignment of design moments from sections with large demand for flexural reinforcement to subcritical sections assists in the reduction of congestion of bars. Because columns need to be stronger than beams, it should be the designer's aim to produce beams which do not have capacities well in excess of those required by the design earthquake loading. Beams with unnecessary strength reserve will require matching columns with excess capacities. These in turn may require excessive foundations, if a weak link in the foundations is to be avoided. It must be appreciated that even a significant increase in the capacities of beams at a particular floor, will not prevent during a large earthquake the formation of plastic hinges at overstrength (Eq. (1)). The major aims of moment redistribution along continuous beams, in achieving an efficient design of ductile reinforced concrete frames, are:

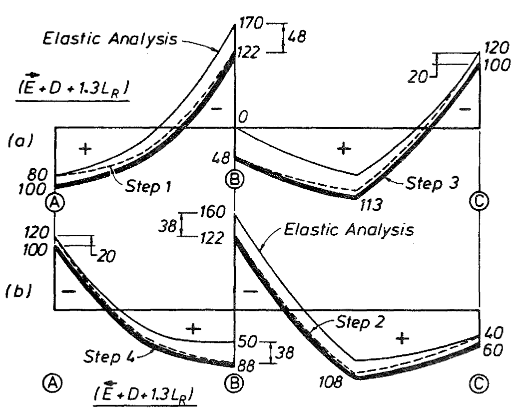


FIG. 2 - MOMENT REDISTRIBUTION ALONG A TWO SPAN BEAM OF A FRAME FOR TWO LOADING CASES

(i) Reduce the absolute maximum design moment, usually in the negative moment region of the beam, and compensate for this by increasing the moments at non-critical (usually positive) moment regions. In the example of Fig. 2(a) this is achieved in two steps. In this figure, within the expressions for load combinations, D denotes dead load, L_R denotes the reduced live load and E indicates the code specified lateral earthquake load applied to the building in the direction shown by the arrow.

(ii) Equalize the critical moment demands in a beam at either side of an interior column, arising from earthquake attacks from two different directions. This will obviate the necessity of having to terminate and anchor top beam bars at an interior beam-column joint, if excess strength development is to be avoided. A comparison of Figs. 2(a) and (b) show how this was achieved in the example structure.

(iii) Utilize the potential positive moment strength of beam sections at or near column faces, where, according to most building codes [2,41], this must be at least one half of the negative moment capacity provided at the same section. The approach to achieving this aim may be seen at Column B of Fig. 2.

(iv) Reduce the moment demand on columns subjected to small axial compression or to axial tension. Thereby the flexural tension reinforcement in such, typically exterior, columns can be significantly reduced, without affecting the earthquake resistance of the entire bent.

While the requirements of equilibrium must be scrupulously satisfied, limits for moment redistribution must also be established. A drastic departure from the elastic pattern of moments might result in unacceptable crack widths during moderate earthquakes. Another, more common, concern is the availability of curvature ductility at beam sections where the moments obtained for the elastic structure are to be significantly reduced. The redistribution of beam moments, or moments and shear forces in adjacent columns, relies on plastic hinge rotations in the beams only. The concern for the availability of curvature ductility may, however, be dismissed because ductility demands are very small when compared with those required to ensure plastic hinge rotations during a large earthquake. Therefore a reduction of design moments in a beam span by up to 30 % of the maximum value obtained for that span from elastic analyses for any combination of earthquake and factored gravity loads, has been adopted. In practical situations, optimum design moment patterns can be obtained usually with moment reductions considerably less than 30%.

2.1.3 Dynamic effects on columns

Once the beams, being the weak links with two plastic hinges in each span, have been designed, the moment input from beams to adjacent columns can be uniquely determined. However, the share of each column, above and below a floor, in resisting this moment input is uncertain. The original moment pattern for a column, which resulted from an elastic analysis and the use of lateral static loading, is usually a good representation of the response in the fundamental mode of vibration. However, higher modes of vibration and plastification of parts of the structure, may result in very significant changes in this moment pattern. An example is given in Fig. 3.

If columns are to be prevented from developing plastic hinges at either end, the disproportionate distribution of moments, such as shown in Fig. 3, must be accounted for. The dynamic moment magnification factor, ω, introduced with Eq. (3), intends to achieve this. Its value, ranging from 1.3 to 1.9 and derived

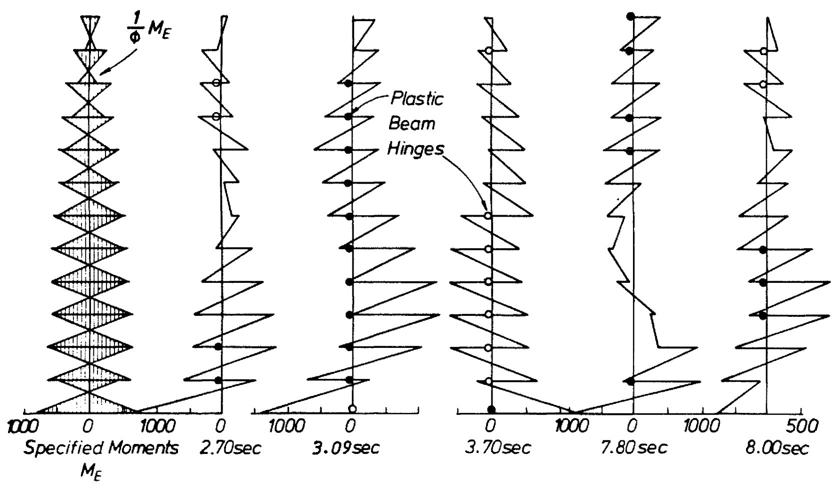


FIG. 3 - A COMPARISON OF COLUMN MOMENT PATTERNS DUE TO LATERAL STATIC AND DYNAMIC LOADINGS

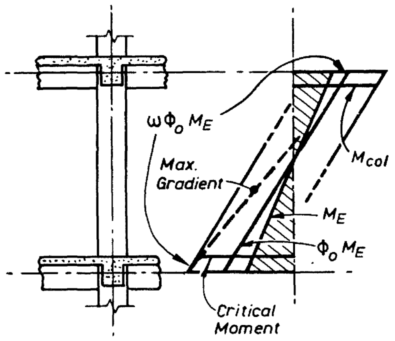


FIG. 4 - MOMENT MAGNIFICATIONS FOR AN UPPER STOREY COLUMN

from numerous case studies, is given in the commentary of the concrete code [41]. In recognition of the greater significance of the higher mode contributions, its magnitude increases with the first mode period of a frame.

The two-stage increase of design moments for a typical upper storey column is shown in Fig. 4. Column moments during the elasto-plastic dynamic response of a frame may increase significantly at either the top or the bottom of the storey, but not simultaneously at both ends. Figure 4 also shows this feature.

From considerations of the maximum probable axial load input into a column, due to plastic hinge developments in all or most of the beams in the floors above the storey concerned, and the maximum likely moment gradient, the critical total axial and shear forces for a particular direction of the earthquake attack can be readily estimated. Hence the critical sections of these "strong" columns can be proportioned and detailed [41].

2.2 Structural Walls

The role of structural walls in the earthquake resistance of buildings was studied for over 20 years. The thrust of research in New Zealand was first directed towards establishing the reasons behind certain code provisions in the United States. These codes implied that wall systems possess limited ductility and that for this reason they should be designed for considerable larger lateral load resistance. The challenge presented itself in finding out whether reinforced concrete walls could be made more ductile and if so, how. In making progress invaluable benefit was obtained from the extensive experimental program conducted simultaneously by the Portland Cement Association Research Laboratories in the United States [22].

Design procedures currently used in New Zealand [41] rest on the premise that, provided that the appropriate detailing of the critical wall regions is assured, ductilities approaching those in ductile frames can also be attained in buildings in which earthquake resistance is assigned entirely to structural walls. With the appropriate application of the capacaity design philosophy, embodied in Eq. (3), non-ductile failure mechanisms, particularly those due to shear, which may jeopardize overall ductile response, may be suppressed. For this reason the required lateral load resistance of ductile wall systems is only 25% larger than that of ductile frames [42]. This policy intends to recognize also the fact that wall systems generally possess less redundancey than frame systems.

2.2.1 Cantilever walls

A feature which distinguishes the wall design procedure for flexure adopted in New Zealand [41] from those of other countries, is the consideration of curvature ductility in the potential plastic hinge regions. Figure 5 shows that identical curvatures (lines (1) and (2')) may result in distinctly different concrete compression strains. For most walls in buildings, axial compression is relatively small, and hence large curvature ductilities may be

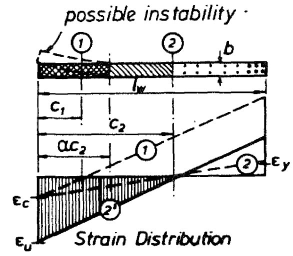


FIG. 5 - STRAIN PATTERNS IN RECTANGULAR WALL SECTIONS

attained before concrete compression strains, ϵ_c, would become critical. A simple way to check the criticality of curvature ductility is to compare the computed neutral axis depth, c, with a critical value, c_c [41]. Concrete compression strains are expected to be excessive where the neutral axis depth is larger than this critical value. In the latter case, a part of the theoretical flexural compression zone of the wall section, shown with double shading in Fig. 5, must be provided with transverse confining reinforcement. Some details of this are summarized in Section 3.2.1.

Because the potential plastic hinge region requires careful and more costly detailing, it makes good sense to restrict it to the base of cantilever walls. This means that the flexural strength of the wall in the upper storeys must be in excess of the moment demands indicated by the code specified static loading which has been assigned to the wall. This excess must be sufficient to allow for variations in bending moment distributions during the dynamic response to ensure that the wall remains essentially elastic in those upper storeys. The design moment envelope suggested [32] for walls is similar to that shown in Fig. 14(c). This envelope requires somewhat more flexural reinforcement than the traditional design procedure, but it leads to very considerable savings in transverse reinforcement in the upper storeys of walls.

To provide significant protection against a shear failure, cantilever walls are designed, in accordance with Eq. (3), for a shear force of

$$V_{wall} = \omega_v \phi_{o,w} V_{code} \qquad (4)$$

where V_{code} = shear force derived from the code specified lateral load assigned to the wall by the initial elastic analysis, $\phi_{o,w}$ = flexural overstrength factor based on properties of the wall section at the base as constructed, and ω_v = dynamic shear magnification factor. The latter allows for the fact that due to the effects of higher modes of vibrations, shear forces much larger than those derived from the applied static code load, can be developed. Its value increases with the fundamental period, T, of the structure [41]. The design of walls for shear resistance is a particularly good example of the application of the capacity design philosophy.

2.2.2 Coupled structural walls

The study of structural walls coupled by beams of various stiffness and strength began in New Zealand shortly before the March 27, 1964 Alaska earthquake. This exceptional M 8.4 event caused some spectacular failures in the city of Anchorage in multistorey buildings with coupled structural walls. The study led to the development of a seismic design strategy which incorporated also the philosophy of capacity design. The modelling of such structures, examples of

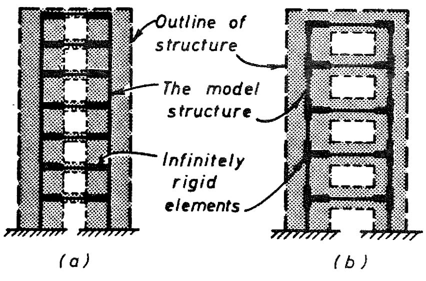


FIG. 6 - THE MODELLING OF COUPLED STRUCTURAL WALLS

which are shown in Fig. 6, are well established. Typical distributions of bending moments along the height of walls, M_1 and M_2, and vertical shear forces, q, across the coupling beams, obtained from elastic analyses, are shown in Fig. 7(a) (b) and (c). It was envisaged, however, that in recognizing the ductile properties of such a system, the designer should utilize the ability of the structure to redistribute internal actions without any reduction in lateral load resistance [32]. Redistribution of design moments from a tension wall (Fig. 7(a)) to a compression wall (Fig. 7(b)) is an example of enforcing more desirable structural behaviour when an earthquake imposes considerable ductility demands. Moreover such moment redistribution usually results in a reduction in the necessary tension

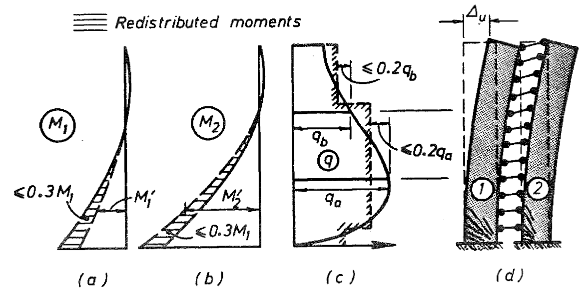


FIG. 7 - THE RESPONSE OF COUPLED WALLS TO HORIZONTAL STATIC LOAD

reinforcement. Similarly the redistribution of shear forces over the height of the structure among coupling beams may be utilized, as shown in Fig. 7(c), to arrive at simple and more practical solutions. To ensure that premature yielding during moderate earthquakes will not occur, the reduction of design quantities, such as wall moments and beam shear fores, must be limited. Suggestions for such limitations are also shown in Fig. 7.

Redistribution of design action can be relied on only if each component which is expected to become inelastic, possesses adequate ductility. The purpose of the code [41] requirements for the detailing of such regions is to ensure that the available ductility is ample. The typical response of one well detailed wall of a structure, shown in Fig. 7(d), which will be subjected to variable axial loads during an earthquake, is of the type seen in Fig. 8. The ductility demand on the coupling beams may be very large. For this reason special detailing of such beams is required [41], and this is briefly reviewed in Section 3.2.2.

The appeal of coupled walls in seismic resistance stems from their relatively large stiffness and their ability to dissipate, when required, large amounts of energy in the coupling system, which has a negligible role in gravity load resistance.

2.2.3 Squat walls

Squat structural walls with a height, h_w, to length, ℓ_w, ratio of less than two find wide application in low-rise buildings. However, they are also used in high rise structures, where they may make a major contribution to earthquake resistance in the lower storeys.

The capacity of a squat wall is often limited by its foundation. Because of their large potential strength, it is relatively easy to ensure that they will respond within the elastic domain. Nevertheless brittle failures of such walls have often been encountered during earthquakes. It is a common notion that squat walls are destined to fail in shear, usually as a result of diagonal tension, and hence at best they may exhibit only limited ductiliy.

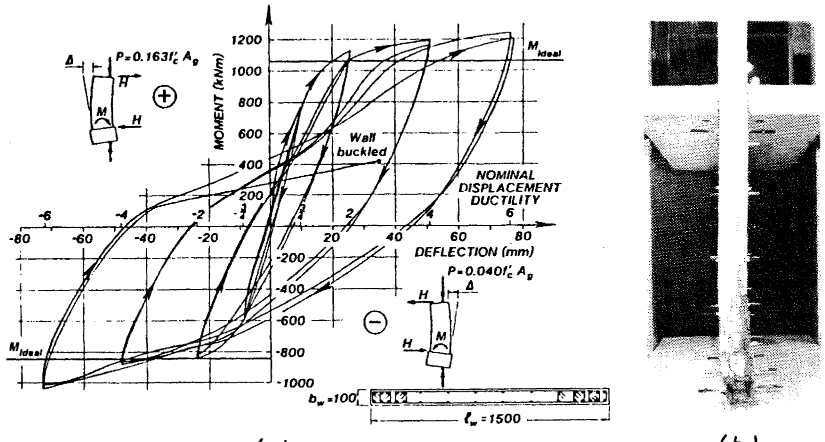


FIG. 8 - THE HYSTERETIC RESPONSE OF A DUCTILE WALL (A) AND ITS FAILURE DUE TO OUT OF PLANE BUCKLING (B)

Pilot studies attempted to exploit the principles of capacity design philosophy [34]. Consequently shear reinforcement was provided in such a way that squat walls were forced to develop their full flexural strength. This proved to be feasible.

In more recent studies [20] the behaviour of walls, which were considered to be typical of those used in New Zealand before the introduction of capacity design procedures, were investigated. These may be classified as walls of limited ductility. Their theoretical shear strength, considered to have satisfied code [2,41] requirements for earthquake resistance, was typically of the order of 75% of the flexural strength provided. The fact that such test walls developed their ideal flexural strength, confirmed the conservatism of existing shear provisions catering for monotonic loading. After some load reversals, however, the strength and energy dissipating capacity of these walls deteriorated. Eventually they failed in diagonal tension. Figure 9 shows an example of such a wall with $h_w/\ell_w = 1.0$. These test walls fulfilled current expectations with regards walls of limited ductility. A drawback in the behaviour of such walls is, however, that the width of the main diagonal cracks is large. Diagonal cracks, typically 1.5 mm wide, develop suddenly with a thud when about one half of the design shear strength of the wall is attained.

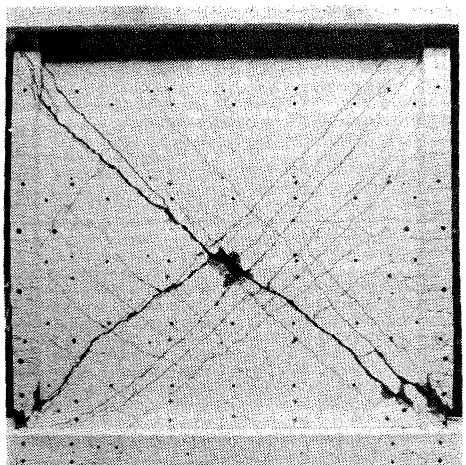


FIG. 9 - FAILURE OF A SQUAT WALL WITH LIMITED DUCTILITY

The response to reversed cyclic loading of squat walls, the strength of which is controlled by flexure, is illustrated in Fig. 10. In squat walls, carrying small gravity loads, a continuous wide horizontal crack may develop at the base after one displacement excursion involving flexural yielding (Fig. 10(b)). At this stage shear transfer by aggregate interlock along this failure plane is dimininshed, and a much more flexible mechanism, that of dowel action, is mobilized. The response of such walls is very ductile. However, because of increasing sliding along the base, energy

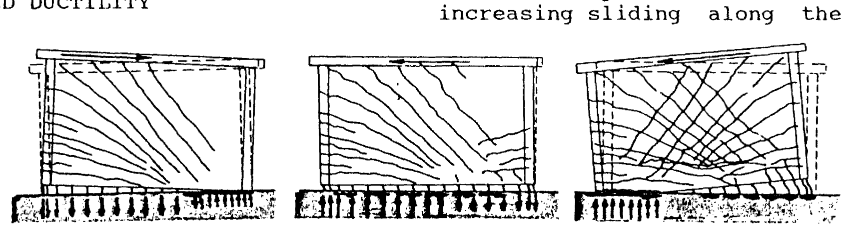


FIG. 10 - DEVELOPMENT OF SLIDING SHEAR MECHANISM IN SQUAT WALLS

dissipation may be dramatically reduced. Suitable detailing, to improve hysteretic response of ductile squat walls is briefly described in Section 3.2.3. Damage in such walls is confined to the wall base. Diagonal cracks, such as shown in Fig. 10, remain very small at all stages of the response.

2.3 Hybrid Structural System

In many buildings earthquake resistance is provided by interacting ductile frames and structural walls. While capacity design procedures were gradually developed and are being used in New Zealand for both frames and structural wall system, the application of this philosphy to the very common dual or hybrid systems is largely left to the ingenuity of individual designers. This section reports briefly on some features of the behaviour of such mixed systems and on relevant design concepts which lend themselves to further development.

2.3.1 Types of hybrid systems

Familiar models, representing interacting frames and walls, are shown in Fig. 11. The first diagram shows a frame, condensing the properties of several similar frames,

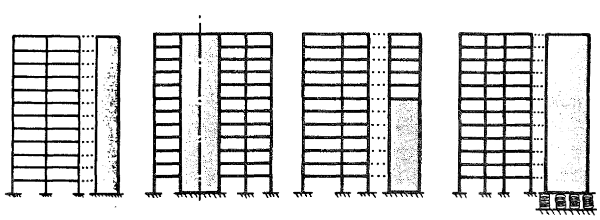


FIG. 11 - MODELS OF HYBRID STRUCTURAL SYSTEMS

coupled to a cantilever wall assumed to condense the properties of several similar walls. The coupling is via links simulating floor diaphragms. Frames may also be connected rigidly to walls, as shown in Fig. 11(b). The last two diagrams in Fig. 11 show interacting partial height walls and walls resting on flexible foundations.

There are several possibilities for the selection of viable energy dissipating mechanisms. Within a capacity design strategy, this selection is a very important step. One might select plastic hinges in frames, as shown in Fig. 12(a), the same way as in framed buildings without any walls. The intent is then to prevent plastic hinge formation in columns of upper storeys, largely for the sake of more convenient detailing, as outlined in the introduction to Section 2. A similar strategy may be followed when beams of frames are rigidly connected to cantilever walls, as shown in Figs. 11(b) and 12(b). Because of the presence of walls, "soft storeys" should never develop, and hence the possible formation of plastic hinges in columns need not be of particular concern. In cases when gravity rather than earthquake load considerations govern the

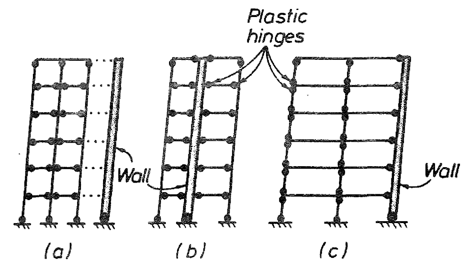


FIG. 12 - ENERGY DISSIPATING MECHANISM IN HYBRID STRUCTURAL SYSTEMS

required strength of beams, it may even be preferable to choose the system shown in Fig. 12(c). In this case plastic hinges, which may spread over the full height of the structure, will primarily develop in the columns. This mechanism necessitates, however, the careful detailing of both ends of these columns, discussed in Section 3.1.2, to accommodate the imposed ductility demands.

2.3.2 The behaviour of interacting frames of walls

The relative participation of frames and walls in the resistance of the total lateral static load applied to buildings is readily established with the use of standard elastic analysis programmes. Examples of the sharing of overturning moments and storey shear forces between a number of identical frames and walls of different dimensions, are shown in Fig. 13. It shows the well known feature of behaviour, that cantilever walls make a significant contribution to both moment and shear resistance only in the lower storeys. In the upper half of the building height, wall contributions are negligible and often negative. A designer may well be tempted to omit walls above a certain height, if functionality permits, as shown in Fig. 11(c). It must also be appreciated that the single most important parameter affecting the stiffness of cantilever walls, is the degree of restraint which is provided at the base (Fig. 11(d)). While base rotations do not affect significantly the behavior of interacting slender walls, the effects on stiff walls may be profound.

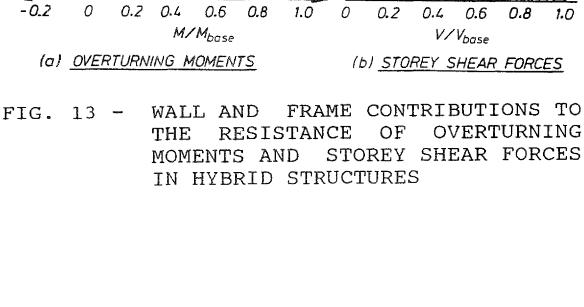


FIG. 13 - WALL AND FRAME CONTRIBUTIONS TO THE RESISTANCE OF OVERTURNING MOMENTS AND STOREY SHEAR FORCES IN HYBRID STRUCTURES

A major issue in the application of capacity design procedures to hybrid systems, is the questionable relevance of the results of elastic analyses, such as shown in Fig. 13, to the resistance demands which arise during the inelastic dynamic response of such systems to major seismic events. The question to be satisfied is thus; what adjustments are required in the application of capacity design procedures, developed separately for frames and to walls, if they are also to assure the desired seismic response of hybrid systems to severe earthquakes.

2.3.3 The capacity design of hybrid systems

During the dynamic response of the structure, walls can be expected to protect columns against effects due to higher modes of vibrations. Therefore, if it is desired to avoid plastic hinge formation in columns, design moments for columns of hybrid systems need not to be magnified to the same extent as those of ductile frames, Suggested values for the dynamic moment magnification, in accordance with Eq. (3), are shown in Fig. 14(a). If walls of partial height are used, the columns in the upper storeys, where no wall is present, should be designed the same way as those of ductile frames using appropriate moment magnification ω_p, as shown in Fig. 14(b).

The design quantities for walls of hybrid structures, derived from elastic analyses for code specified lateral static loads, are more sensitive than those of the wall systems reviewed in Section 2.2. During the inelastic dynamic response of the structure, departures of both moments and shear forces from those shown for example in Fig. 13, can be very significant. From numerous cases studied [13], design envelopes for wall moments and shear forces, in terms of the maximum values at the wall base, have been derived. These, shown in Fig. 14(c) and (d), should ensure that plastic hinges are restricted to wall base and that adequate reserve shear strength is available throughout the height of the walls.

3 QUALITY IN DETAILING

Concrete is inherently brittle. Deformed reinforcing bars, however, can be readily manufactured to possess ample ductility. The art in the design to survive catastrophic earthquakes, consists primarily in the skillful combination of these two materials to produce sufficiently ductile composite response in all critical regions, such as at plastic hinges of the structure. Detailing of the reinforcement includes this important aspect of seismic design.

It is emphasized that a structure need not be ductile everywhere. Only in chosen regions (plastic hinges) are large inelastic deformations to be expected. One of the aims of the previously outlined capacity design procedure is to precisely locate the potential plastic hinges, where special detailing for ductility is essential. Irrespective of the intensity of ground motions, clearly defined major parts of the body of the structure should remain elastic during an earthquake. These elastic regions require no special detailing.

It is increasingly recognized that good detailing of regions, assigned to dissipate energy, is at least as important as analyses required to determine the necessary strength of structural components. A considerable part of the research effort in New Zealand, directed over the past 20 years to the seismic performance of reinforced concrete components or structural subassemblages, attempted to quantify in unambigious terms the quality of goodness in detailing. In the following sections a few examples of these endeavours are given.

3.1 Ductile Frames

3.1.1 Plastic hinges in beams

(a) Inelastic buckling of beam bars

The dominant source of energy dissipation in inelastic frames is a number of plastic hinges in beams.

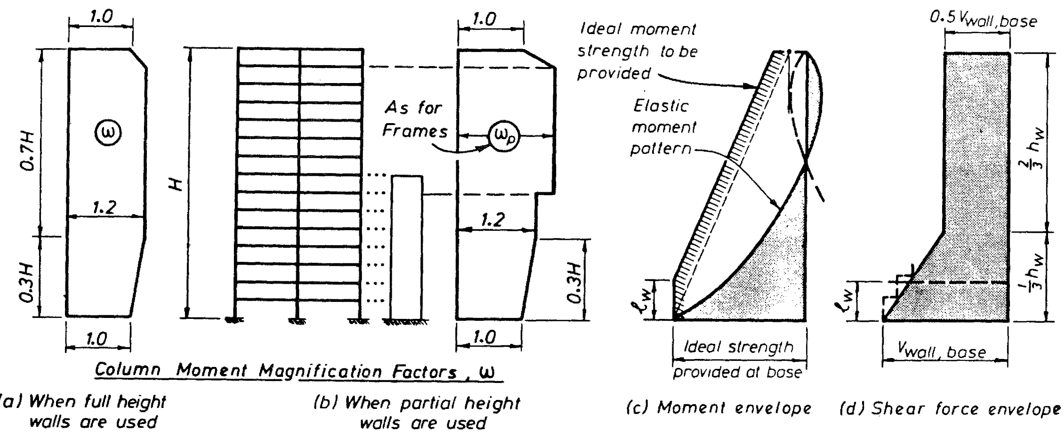


FIG. 14 - DYNAMIC MOMENT MAGNIFICATION FOR COLUMNS, AND MOMENT AND SHEAR ENVELOPES FOR WALLS IN HYBRID STRUCTURES

The major part of the moments, to be transmitted in the critical regions during inelastic cyclic displacements, are carried by internal steel forces. In doubly reinforced beams, concrete in compression has a relatively minor role. Nevertheless there must be sufficient transverse reinforcement present to preserve the integrety of the thoroughly cracked concrete. A frequent cause of flexural failures, defined here as a significant loss of moment resistance, is inelastic buckling of compression bars after the cover concrete is lost.

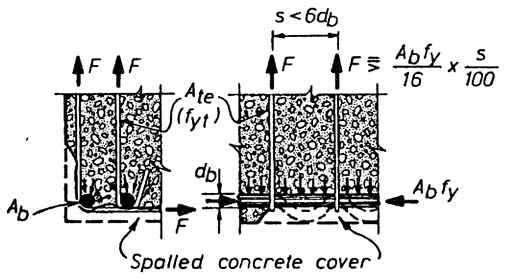


FIG. 15 - LATERAL RESTRAINT TO PREVENT PREMATURE BUCKLING OF COMPRESSION BARS SITUATED IN PLASTIC HINGE REGIONS

Experiments combined with engineering judgement lead to the conclusion that to prevent premature inelastic buckling of compression bars subjected to Bauschinger effect, associated with strains necessary to develop the intended large curvature ductility, beam bars should receive lateral support at distances not less than 6 bar diameters, as shown in Fig. 15. Even this spacing, which is less than that generally recommended in codes of other countries, will not be sufficient if extremely large ductility is imposed. Figure 16 shows an example. It is well established practice in steel design to provide for a restraining force, shown as F in Fig. 15, which is at least 1/40th of the load on the strut. A compressed bar in a beam is, however, subjected along its length also to lateral forces exerted by the expanding cracked compressed concrete in the core of the beam section. To allow for the effects of this transverse pressure on the bars, shown by small arrows in Fig. 15, it was concluded [41] that the restraining force should not be less than 1/16th of the bar strength at every 100 mm along the bar. This resulted in the requirement that the area of a tie required to restrain one compression bar should be

$$A_{te} = \frac{A_b}{16}\,\frac{f_y}{f_{yt}} \times \frac{s}{100} \qquad (5)$$

where the symbols are identified in Fig. 15. Through numerous tests with beams and columns, it was found that

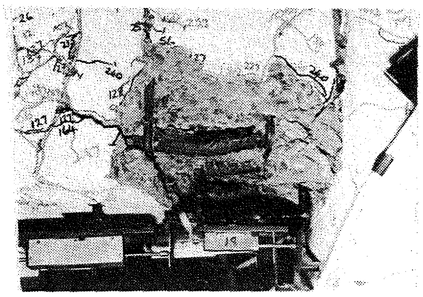


FIG. 16 - FAILURE AT A PLASTIC HINGE SATISFYING THE REQUIREMENTS SHOWN IN FIG. 15 AT A DISPLACEMENT DUCTILITY OF μ_Δ = 10

the intended large ductility could be readily attained in plastic hinges so detailed. The total amount of transverse reinforcement may be governed by requirements of shear strength. Bar buckling may also be initiated at the development of relatively small ductilities if, because of excessive shear deformation, dowel action of the beam bars is mobilized. This issue is reviewed in the next paragraph.

(b) The shear response of plastic hinges

The shear strength of plastic hinges in beams was viewed in New Zealand with some concern. It was generally recognized that due to reversed cyclic inelastic deformations, leading to wide cracks, the "contribution of the concrete" to shear strength must necessarily diminish. Consequently there is general concensus [2,7,8,41] that shear reinforcement should be provided to resist the entire shear which, in accordance with the traditional truss analogy, will need to be transmitted across the potential plastic hinges of beams. In spite of this apparently conservative procedure, shear deformations in the plastic hinge region may become excessive, particularly when curvature ductility demands and shear stresses associated

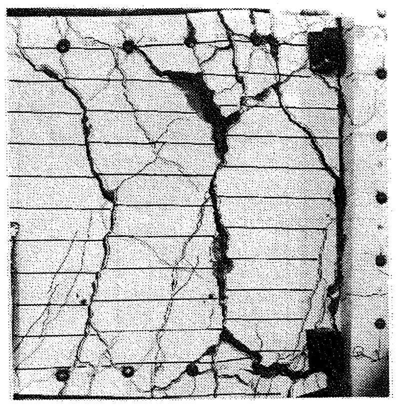


FIG. 17 - LARGE SHEAR DISPLACEMENTS ALONG INTERCONNECTING FLEXURAL CRACKS ACROSS A PLASTIC HINGE OF A BEAM

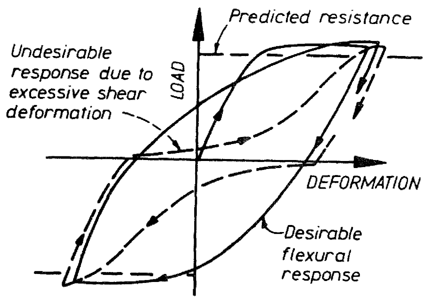


FIG. 18 - A COMPARISON OF HYSTERETIC RESPONSES

with each direction of reversed shear loading, are large. Figure 17 shows the visible sliding shear displacements along interconnected flexural cracks in the plastic hinge region of a beam. It is also evident that an increase of stirrup reinforcement is not likely to reduce markedly such sliding displacements. The consequence of this is significant reduction of beam stiffness, particularly at small shear loads, as shown in Fig. 18, leading to some loss in energy dissipation.

It has also been established [12] that the diagonal compression field in the web, necessary to transmit shearing forces and to engage stirrups in tension, develops at an angle considerably steeper than the traditionally assumed 45°, as shown

To control both excessive sliding shear displacements and premature yielding of stirrups within a beam plastic hinge, it is recommended [41] that some diagonal reinforcement be used when nominal alternating shear stresses are in excess of $0.3\sqrt{f'_c}$ MPa. Because diagonal bars, as shown in Fig. 20, will immediately respond to a vertical sliding displacement both in tension and compression, a significant improvement in the hysteretic response may be obtained even in cases when the diagonal bars can resist as little as 30% of the total shear force. The need for diagonal shear reinforcement will arise only in relatively short beams, such as exterior spandrels.

3.1.2 Confinment in the plastic hinges in columns

The catastrophic consequences of earthquake induced failures in columns or piers of bridges are well known. Therefore significant research efforts have been devoted in many countries to the behaviour of reinforced concrete columns in seismic environments. Of particular importance is the extensive research conducted over the past 20 years at the University of Canterbury [17,18,23,24,25,26,27,29,38,39,40]. Several features of the results have already been incorporated into design practice in New Zealand [41]. Unfortunately in this brief review no justice can be given to the extent and quality of this research program, and in particular to its

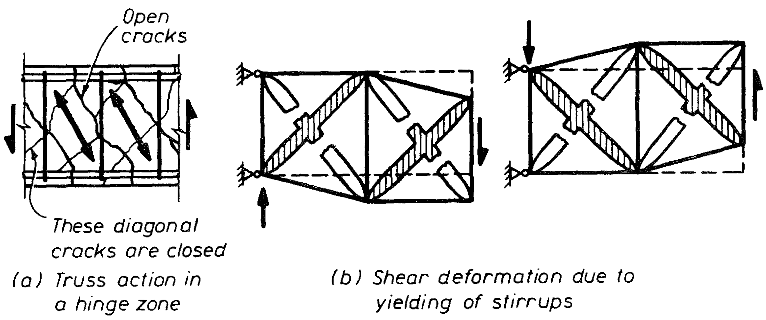


FIG.19 - MECHANISMS OF SHEAR TRANSFER IN PLASTIC HINGES [11]

in Fig. 19(a). This means that, in spite of providing web reinforcement to transmit the entire shear across a 45° crack, stirrups will yield and consequently diagonal cracks will become large [11]. The closure of diagonal cracks upon shear reversal will then lead to significant pinching during the hysteretic response, as shown in Fig. 18. The mechanism associated with such response is shown in Fig. 19(b). It was identified at the University of Auckland during extensive experimental studies of beam responses to reversed cyclic shear loading [10,11,12].

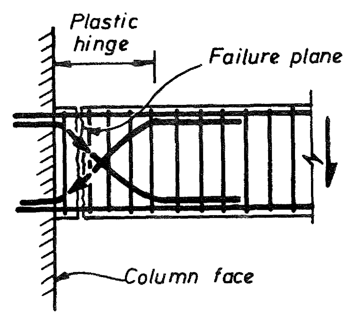


FIG. 20 - THE CONTROL OF SLIDING SHEAR IN A PLASTIC HINGE BY DIAGONAL REINFORCEMENT

impact on the understanding of behaviour and on seismic design in general.

With no or very small axial compression loads, columns behave as beams. Consequently the requirements for transverse reinforcement are those for beams, reviewed in Section 3.1.1. However, when axial compression is significant, adequate curvature ductility in the plastic hinge of a column can generally be developed only at the expense of large concrete compression strains. Therefore in such very common cases the compressed concrete must be laterally confined to prevent the unrestricted volumetric increase of the column core. Thereby compressed concrete, an inherently brittle material, can be made remarkably ductile.

Early specifications (ACI, SEAOC) for spiral binding, aimed at boosting the axial load carrying capacity of the confined concrete core to that of the gross column section before the spalling of the cover concrete. Such columns will rarely occur in buildings subjected to earthquakes. Because of very significant flexural actions, the aim must be to impart rotational ductility while preserving adequate axial load carrying capacity. This was the reason for the focussing of research interest at the University of Canterbury primarily on factors affecting plastic hinge rotations in columns.

Previous ACI specifications for the confinement of rectangular columns with rectangular hoops and ties assumed that these are much less effective than spirals because, as Fig. 21(a) shows, to sustain lateral pressure, hoop legs would need to span between effective lateral supports. The large flexural deformability of small diameter ties renders the mechanism illustrated in Fig. 21(a) inefficient. A greately improved mechanism can be developed, however, if the confined concrete can rely principally on small horizontal arches, for which the larger diameter column bars act as abutments. These bars in turn are held in position by ties, which being in tension, represent a stiff system. As Fig. 21(b) shows, column bars, reasonably closely spaced along the edges of the column section and acting as

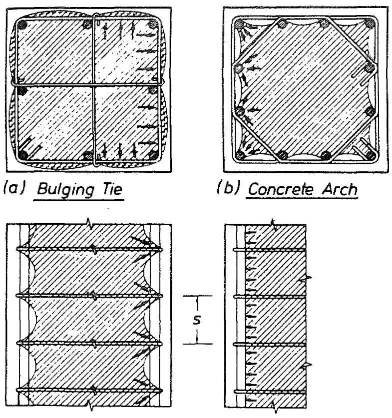


FIG. 21 - CONTRIBUTION OF OVERLAPPING TIES TO THE CONFINEMENT OF COMPRESSED CONCRETE IN COLUMNS

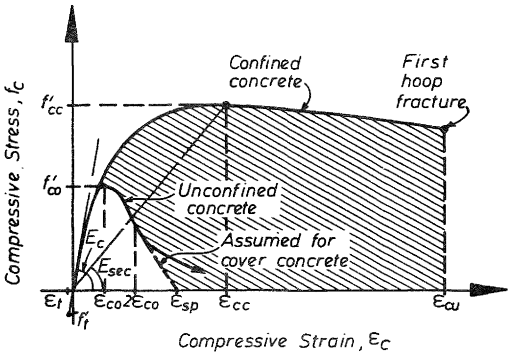


FIG. 22 - STRESS-STRAIN CURVES FOR CONFINED AND UNCONFINED CONCRETE [18]

short span continuous beams, can sustain significant confining pressure developed in the concrete. This mechanism, with restrictions on the horizontal spacing between column bar and the vertical spacing of tie legs, is the basis of design recommendations developed and used in New Zealand [41]. It is an illuminating example of the use of rational yet simple principles in the development of "good" detailing.

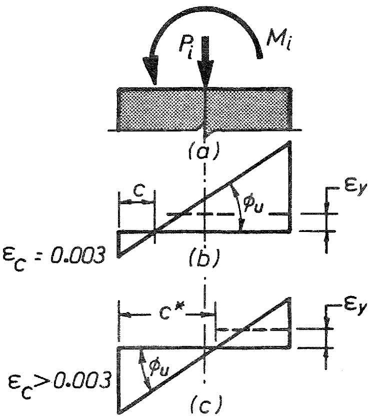


FIG. 23 - STRAIN PROFILES IN COLUMNS RESULTING FROM IDENTICAL CURVATURES AT THE DEVELOPMENT OF IDEAL STRENGTHS

It is well established [25] that adequate amount of confinement, for example the type shown in Fig. 21(b), has two significant effects on the stress-strain relationship of concrete. It transforms the relatively brittle (unconfined) material into a very ductile one. This is seen in Fig. 22. This property enables large plastic rotations to be developed in column plastic hinges, even when large axial compressive loads are also present. The other effect is the increase of the compression strength of the confined concrete, shown as f'_{cc} in Fig. 22. In general this strength enhancement more than offsets the reduction of column resistance which would result from the loss of the unconfined cover concrete.

Figure 23 shows strain profiles across a column section subjected to axial

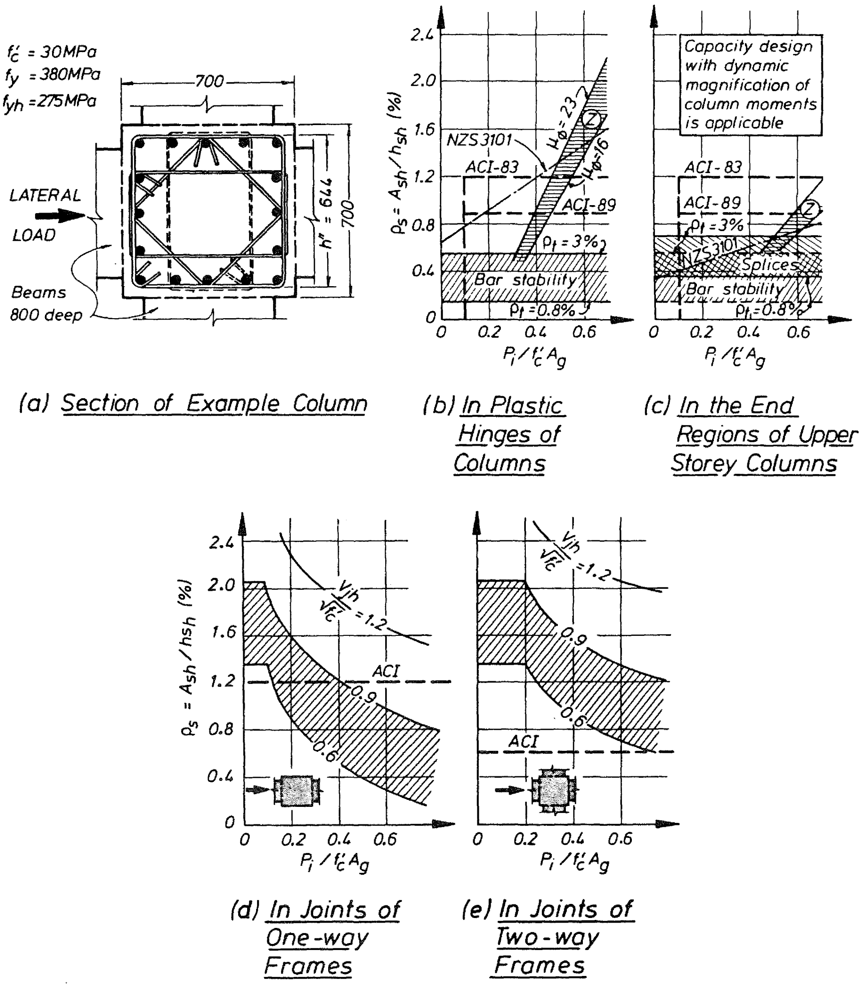


FIG. 24 - A COMPARISON OF REQUIREMENTS FOR TRANSVERSE REINFORCEMENT IN THE END AND JOINT REGIONS OF COLUMNS

compression P_i and moment M_i. When the axial compression is relatively small, a curvature of ϕ_u may be developed without generating excessive concrete compression strains (Fig. 23(b)). Large axial compression load requires a larger flexural compression zone within the section. Hence for the same curvature, ϕ_u, in this case much larger concrete strains ϵ_c need to develop (Fig. 23(c)). This well known simple principle is recalled here only to show that to sustain a given curvature ductility, the amount of confining reinforcement must increase as the axial compression load P_i, and consequently the depth of the flexural compression zone, c*, increases. Theoretical and experimental work at the University of Canterbury verified this simple principle [39] and led to the presently used design procedures [41]. Figure 24 shows a specific example column and compares the trends in various requirements for transverse reinforcement in the potential plastic hinge region. In many countries affected by earthquakes, particularly the Americas, ACI 318-83 recommendations, are used [2]. As Fig. 24(b) shows, the amount of confining reinforcement according to this requirement is constant, i.e. effects of axial load intensity are not considered. The current procedure used in New Zealand [28,41] requires some transverse reinforcement even in the case of no axial load. This (NZS 3101) is also shown in Fig. 24(b). However, often other requirements, to be reviewed subsequently, will necessitate more transverse reinforcement. More recent research [44] indicated that reversed cyclic loading with large axial compression load on the column leads to greater deterioration of the concrete in the core. This necessitates a larger amount of transverse confining reinforcement. It was also found that for small axial load intensity, existing requirements [2,41] are unduly conservative. Theoretical and experimental studies [44] suggest the trend shown in Fig. 24(b) and (c) by the cross hatched band marked Z. The quantity of this reinforcement is, as expected, also a function of the curvature ductility demand, μ_ϕ, which may arise during a large earthquake.

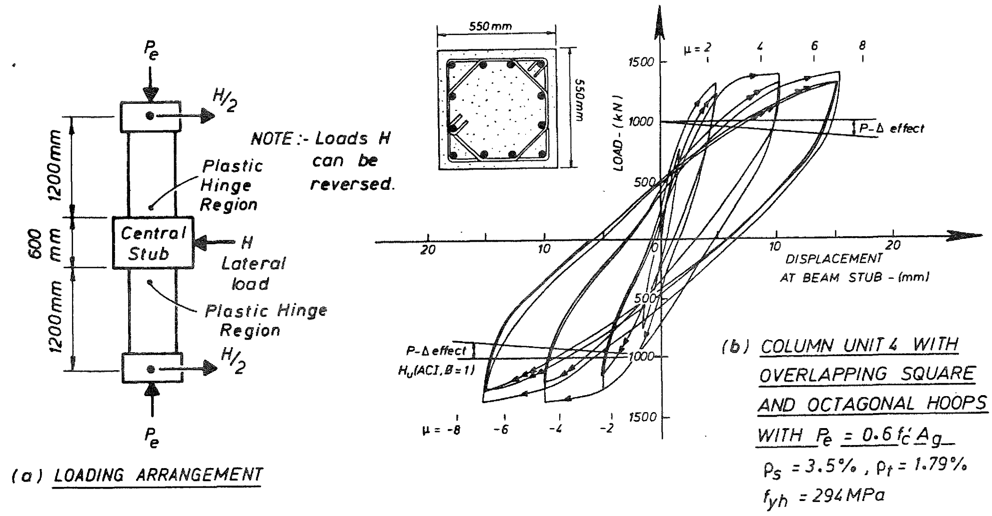


FIG. 25 - HORIZONTAL LOAD-DISPLACEMENT RESPONSE OF A COLUMN SUBJECTED TO LARGE AXIAL COMPRESSION [29]

Relaxations in future ACI requirements are currently being considered and this is shown in Fig. 24(b) and (c) by the lines marked ACI 318-89. It is seen that disturbing discrepancies in design approaches for confinement exist for columns subjected to significant axial compression.

For the purpose of seismic design, it is of great significance that large ductilities have been shown to be attainable in columns also in the presence of large axial compression [39]. Figure 25 shows the horizontal load-displacement response of a column during progressively increased ductility demands [29]. It is seen that the hysteretic behaviour of the column for a given ductility is very stable i.e. the degradation in both stiffness and resistance is negligible. Moreover, due to the substantial confinement and in accordance with the stress-strain response of confined concrete, shown in Fig. 22, the flexural resistance of the column is considerably in excess of the conventionally computed value, which is based on the contribution of unconfined concrete, including that of the cover concrete.

Vertical bars in the end regions of columns must be guarded against premature buckling the same way as in beams. Therefore Eq. (5) is applicable [41]. The quantity of transverse reinforcement required to stabilize the 5 bars in one face of the example column in Fig. 24(a), is shown in Figs. 24(b) and (c) for the common range of total reinforcement content, i.e. $0.008 \le \rho_t \le 0.030$. It is seen that, apart from shear resistance, the requirements of bar stability are likely to govern when axial compression on a column is small.

In the end regions of columns in the upper storeys of multistorey frames, designed in accordance with capacity design principles, reviewed in Section 2.1.3, curvature ductility demands, μ_ϕ, should be insignificant even during the largest expected seismic event. As Fig. 24(c) shows, one half of the amount of confining reinforcement required in potential plastic hinges, was considered in New Zealand [41] to be adequate. However, no relaxation in the protection against bar buckling is warranted.

3.1.3 Lapped splices in columns

In columns of upper storeys which have been designed, in accordance with the capacity design principles reviewed in Section 2.1, lapped splices may be used at the traditional location, immediately above a floor. This is because reversed cyclic inelastic strains are not expected to occur in these bars. However, several cycles of reversed stresses close to the level of yield may occur. For this reason adequate transverse reinforcement, crossing the potential failure plane between each pair of spliced bars, as shown in Fig. 26(a), must be provided. The purpose of transverse bars with area A_{tr} is to provide a clamping force so as to enable a shear friction mechanism in the plane of a splitting (Fig. 26(b)) to be mobilized. As Fig. 26(c) shows, there are two possibilities for splitting when spirals or circular hoops are used.

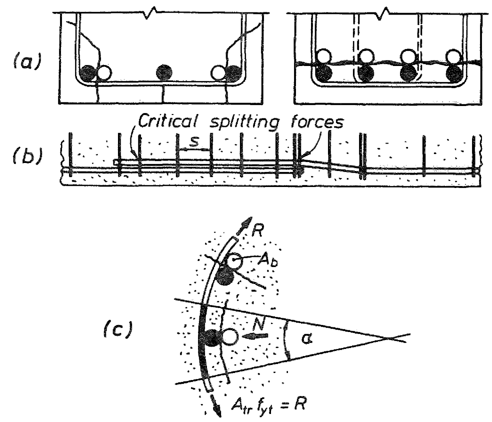


FIG. 26 - LOAD TRANSFER BY SHEAR FRICTION AT LAPPED SPLICES

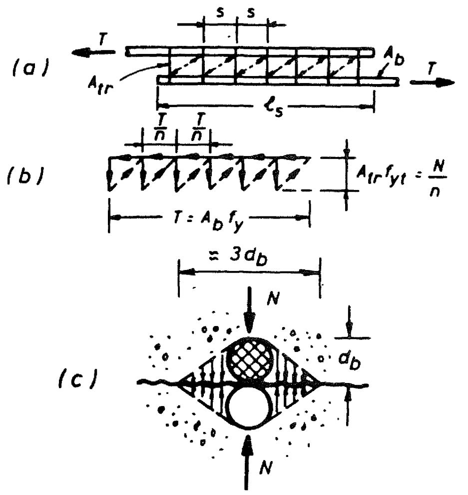


FIG. 27 - THE ROLE OF TRANSVERSE TIES IN SUSTAINING A DIAGONAL COMPRESSION FIELD ACROSS TWO SPLICED BARS

The design of this "clamping" transverse reinforcement may be based on the simple mechanism shown in Fig. 27. In this it is assumed that diagonal compression struts, forming at 45°, transfer the bond forces from the ribs of one deformed bar to those of the other. As Fig. 27 shows, the clamping force to be developed across the two spliced bars by all the ties spaced along the lap, is equal to the force T to be transmitted between the two bars. By making conservative assumptions with regards the moment gradient along a column and the necessary length of a splice, the simple expression for the area of a tie leg, A_{tr}, transverse to the two lapped bars with diameter d_b was derived [41].

$$A_{tr} = \frac{8sd_b}{f_{yt}} \qquad (6)$$

where s is the tie spacing and f_{yt} is the yield strength of the tie.

Tests have verified [33] that with this amount of transverse reinforcement a high level of fully reversed cyclic loading could be maintained with little stiffness deterioration. Figure 28 shows the results of a typical test in which 28 mm diameter bars, spliced in a heavily reinforced column, were subjected to 10 cycles of reversed loading corresponding with 70%, 85% and 95% of the ideal flexural strength of the column. Very satisfactory displacement response, with some deterioration only at the maximum load intensity, was obtained, in spite of the fact that only 77% of the transverse reinforcement required by Eq. (6) was provided. However, after imposing a displacement ductility demand of 4 in each direction of loading, steady and rapid deterioration of the column strength was observed. These tests confirmed that lapped splices should not be used in regions of potential plastic hinges.

To enable a comparison to be made with other requirements for the quantity of transverse reinforcement, the demands resulting from Eq. (6) [41] are also shown in Fig. 24(c). It is seen that this criterion is likely to govern the nececessary amount of transverse reinforcement at the lower end of upper storey columns which, according to the previously outlined capacity design strategy, are intended to remain elastic.

With a significant increase of the transverse reinforcement around lapped splices, it is possible, even under simulated severe cyclic earthquake loading, to develop the full flexural strength at the critical column section. In this case yielding is restricted to the highly stressed end of the splice. Thereby spreading of yielding along a column bar is restricted. For ductility demands to be expected during a very large earthquake this leads to extremely high strains, over a short length of bars, which can lead to bar fracture. For this reason lapped splices, even when reinforced to ensure that a bond failure will not occur, should not be located in regions where ductility needs to be developed.

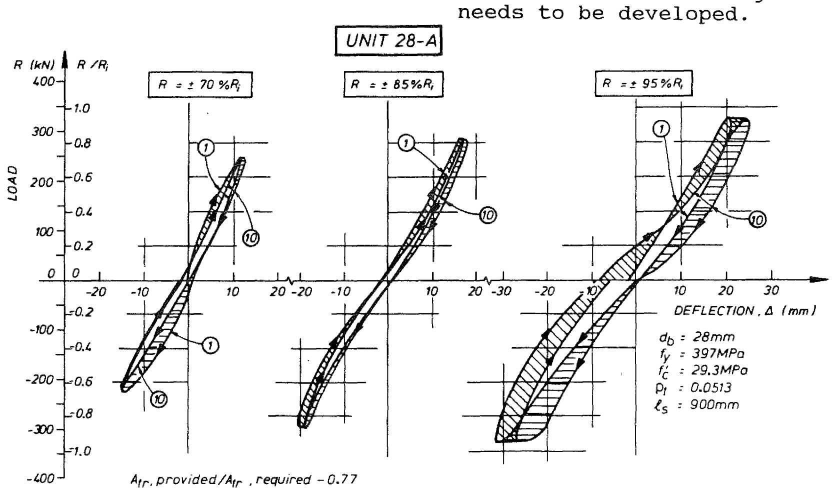


FIG. 28 - TYPICAL LOAD DEFLECTION RESPONSE OF AN ELASTIC COLUMN WITH LAPPED SPLICES

3.1.4 Beam-column joints

(a) Design criteria

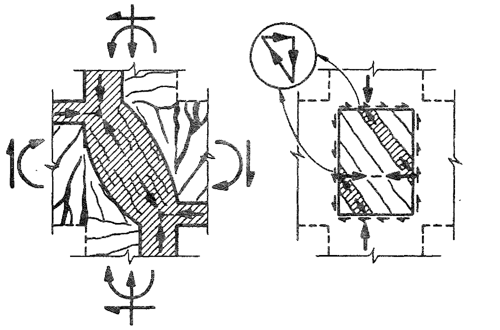
The criteria proposed in New Zealand for the design of joints in ductile reinforced concrete frames [35] are briefly:

(i) The strength of a joint should not be less than the maximum strength of the weakest member it connects.
(ii) Because of the difficulties in repair and the degrading nature of energy dissipation in joint mechanisms, joints should respond preferably within the elastic domain.
(iii) The capacity of a column should not be jeopardized by the behaviour of the adjacent joint.
(iv) Joint deformations should not significantly increase storey drift.
(v) Joint reinforcement provided should not cause undue construction difficulties.

The strategy for the design of joints adopted in New Zealand attempted to ensure that with the application of relatively simple yet rational rules, the above criteria are satisfied. The design procedure is based to a large extent on theoretical considerations and experimental work which originated in New Zealand. Attention to possible problems with joints subjected to seismic loading was drawn by the Portland Cement Association [14] in the United States some 20 years ago. The intensive study of joints began in New Zealand in 1971, and related research work at the universities of Auckland and Canterbury and within the New Zealand Ministry of Works and Development, continued ever since.

(b) Failure modes

There are two failure modes which need to be controlled. Of this the more important is that associated with shear strength. The shear forces, readily derived form first principles [25], which are typically 4 to 5 times as large as those in adjacent columns, may lead to a diagonal tension failure when no or insufficient amount of joint shear reinforcement has been provided. This failure may occur well before the intended ductility in a frame has been attained by means of plastic hinges in beams. The other failure mode is associated with bond. A simple check will show that bond stresses along reinforcing bars passing through an interior joint may be 3 to 4 times larger than maxima envisaged by most codes [2,41]. An anchorage failure by a pull-out of beam bars at exterior joints is catastrophic. At interior joints slipping of bars through the joint core may occur, and this results in significant loss of stiffness and



(a) Concrete Struts *(b) Diagonal Compression Field*

FIG. 29 - MECHANISMS OF SHEAR RESISTANCE AT AN INTERIOR BEAM-COLUMN JOINT

ability of a frame to dissipate energy.

(c) Joint shear strength

The design for shear strength, for example at an interior beam-column joint, is based in New Zealand on the interaction of two mechanisms, shown in Fig. 29 [19]. The flexural concrete compression forces in the four adjacent members, as shown in Fig. 29(a), combine to develop a diagonal strut across the joint. When plastic hinges are restricted to adjacent beams and the nominal joint shear stresses are not excessive, which is commonly the case, the diagonal compression stresses in the joint core are moderate and hence readily sustained.

The role of the second mechanism, shown in Fig. 29(b), is to equilibrate the bulk of the bond forces transmitted from the beam and column bars to the concrete of the joint core. It is seen that, after the development of diagonal cracks, the peripheral shear flow necessitates the formation of a diagonal compression field. Numerous diagonal struts, shown somewhat idealized in Fig. 29(b), can readily transmit compression stresses provided that horizontal and vertical forces respectively , acting at the edges of the joint core, can also be developed. These forces, which enable the resolution of bond forces into suitable components, also shown in Fig. 29(b), require horizontal shear reinforcement. The corresponding vertical forces at the edge of the joint core may originate from compression forces in the column, or in the absence of these from vertical joint shear reinforcement. The primary role of this mechanism (Fig. 29(b)) is to enable the beam and column reinforcement to function as intended. Large bond forces are expected to be developed in the joint core to enable each bar to be subjected simultaneously to tension and compression at opposite edges of the joint. This may involve forces at yield strength with strain hardening. In cases of a bond failure the mechanism shown in Fig. 29(b) is negated. The structure will then attempt to redistribute the joint shear force so lost to the mechanism of Fig. 29(a). When this occurs the joint becomes slack.

One of the aims of the research carried out in New Zealand was to quantify the contribution of each of these two mechanisms to the total joint shear strength. It was found for example that with plastic hinges developing in beams at the two vertical faces of a joint, the horizontal concrete compression forces in the beams progressively diminish with reversed cyclic loading [3]. Hence the contribution of the mechanism in Fig. 29(a) also diminishes. If the capacity of the beams and their contribution to frame stiffness is to be sustained, the contribution of the mechanism of Fig. 29(b) will need to increase correspondingly. This will then necessitate increased horizontal joint shear reinforcement.

ACI specifications [2] consider that the amount of transverse confining reinforcement to be used in the end regions of columns, when carried through the joint, is also adequate to ensure satisfactory joint performance. The emphasize is on confinement rather than on shear strength. For this reason only one half of the above transverse reinforcement is specified [2] when beams of sufficient width frame into all four sides of a column. It is assumed that these beams confine the joint core.

This is an area in which large differences exist between American and New Zealand design approaches [37]. For a specific example Figs. 24(d) and (e) compare the requirements for the amount of horizontal joint shear reinforcement, ρ_h%, as a function of the axial compression load intensity $P_i/(f'_c A_g)$ on the column, for joints on one-way and two-way frames. While the ACI requirements lead to constant amounts of joint reinforcement, the NZS 3101 approach considers both the intensity of joint shear stress, v_{jh}, and a beneficial effect of axial compression P_i. The shaded area indicates the range of joint shear stresses commonly encountered in practice. the discrepancies are particularly large in the case of two-way frames shown in Fig. 24(e) [30,35].

(d) Bar anchorage in joints

The bond strength of bars, for example that of a beam bar shown in Fig. 30, is strongly influenced by the conditions at the joint edges. Figure 30(a) shows steel (f_s) and bond (u_a) stresses which are expected when adjacent beams are still elastic. After some inelastic cycles and beam hinge development, the mean bond stresses in the core, u_b, will increase, as shown in Fig. 30(b) because the increased steel stresses of up to $\lambda_o f_y$ and the loss of anchorage in the cover concrete outside the column bars. Yielding of the steel gradually destroys bond to the surrounding concrete. Hence progressive yielding, penetrating into the joint core will occur. As Fig. 30(c) shows, this may eventually lead to excessive bond stresess, u_c, and consequent failure. The bar will then slip through the joint core.

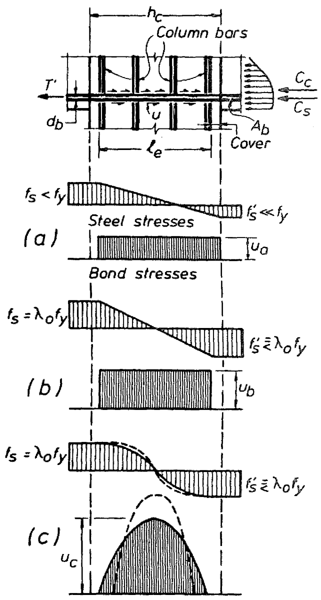


FIG. 30 - LONGITUDINAL AND BOND STRESSES ALONG A BEAM BAR PASSING THROUGH A JOINT

From numerous tests [3.4] it was that such bond failures can be delayed till after the development of a resonable number of load reversals corresponding with the expected ductility demand on the structure, if the diameter of the beam bar passing through the joint, d_b, its yield strength, f_y is related to the overall depth, h_c, of the column with small axial compression, thus

$$d_b \leq 11\, h_c/f_y \qquad (7)$$

Therefore when f_y = 275 MPa, $d_b \leq h_c/25$. When large axial compression load acts on the column, beam bar diameters can be somewhat increased.

In drafting corresponding requirements for the ACI code [2], the above requirement was considered unacceptably severe. It is to be noted that the above limit for the commonly used Grade 60 reinforcement in the US (f_y = 415 MPa) is indeed severe, i.e. $d_b \approx h_c/38$. Current ACI

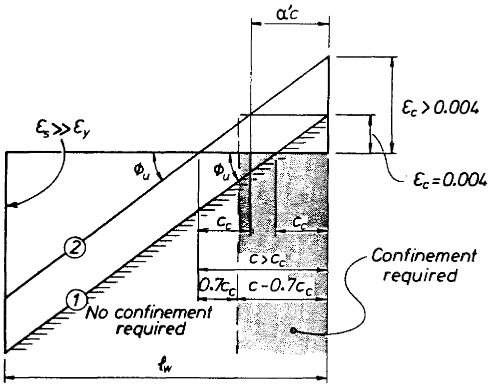


FIG. 31 - THE CONFINED REGION OF WALL SECTIONS RELATED TO DIFFERENT STRAIN PROFILES

policy is to admit bar slip and to accept the consequences of it [2].

A current cooperative project, with participion of researchers from United States, New Zealand, Japan and China, is expected to assist in the developement of more widely accepted design approaches for beam-column joints.

3.2 Structural Wall Details

3.2.1 Wall sections

Consideration of curvature ductility demands in the plastic hinge region of cantilever walls, discussed in Section 2.2.1 with the aid of Fig. 5, lead to the conclusion that a length of the flexural compression zone equal to $(c - 0.7\,c_c)$ should be confined to enable compression strains larger than $\epsilon_c = 0.004$ to be developed in the confined core. This is shown in Fig. 31 where c = the computed neutral axis depth, the critical value of which is

$$c_c = \frac{55\,\phi_{o,w}\,\ell_w}{\mu_\Delta} \qquad (8)$$

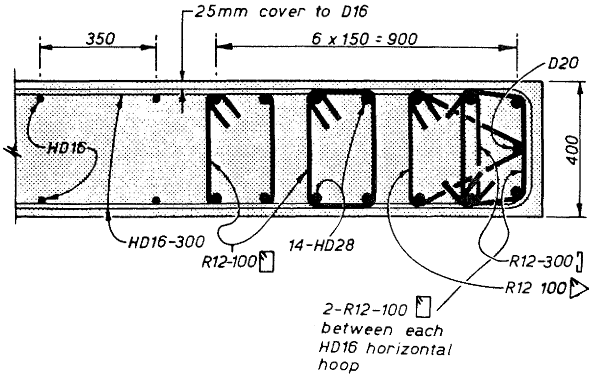


FIG. 32 - THE CONFINEMENT OF COMPRESSED REGIONS OF A WALL SECTION

and where $\phi_{o,w}$ = flexural overstrength factor for the base wall section, ℓ_w = length of wall, and μ_Δ = the displacement ductility intended to be developed. For example when μ_Δ = 5 and typically $\phi_{o,w}$ = 1.4, Eq. (8) will give $c_c \approx 0.15\,\ell_w$.

An example of the detailing of such a region is shown in Fig. 32. The approaches to the determination of transverse reinforcement in the end regions of walls, to confine compressed concrete or to stabilize vertical bars, such as shown in Fig. 32, aginst buckling, are similar to those described for columns in Section 3.1.2.

Some attention was also paid in New Zealand to lateral instability of wall sections, such as shown in Fig. 32, in the potential plastic hinge regions. Limitations on wall unsupported height to thickness ratio were introduced [41] in recognition of the dramatic softening which occurs in the plastic hinge zone when large amplitude inelastic reversed cyclic displacements are imposed by a severe earthquake. The complex behaviour of walls during out of plane buckling is as yet not fully understood. Limited tests [36] have shown, however, that the existing limitation [41] on wall thickness is likely to assure sufficient rotational ductility in walls before the onset of a failure by out of plane buckling. Figure 8(a) shows the excellent hysteretic response of a test wall which was adequately detailed in the end regions. With the recommended [41] maximum limit on the height to width ratio

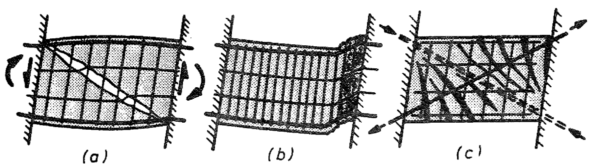


FIG. 33 - THE MECHANISMS OF SHEAR RESISTANCE IN COUPLING BEAMS

of 10 in the plastic hinge zone, the wall failed due to out of plane buckling when a 3rd cycle to a displacement ductility of 6 was attempted. The buckled edge of this wall is seen in Fig. 8(b). The phenomenon points to the obvious need to provide, whenever possible, compact boundary elements in wall sections to stabilize inelastic regions against out of plane displacements.

3.2.2 Coupling beams

Beams connecting coupled walls, as shown in Fig. 6, are often relatively short and deep. Therefore the shear forces, generated when the flexural strength of these beams is developed, can be critical. Conventionally reinforced beams designed in accordance with traditional code [2] provisions, invariably fail by diagonal tension, as shown in Fig. 33(a). This failure mode can be readily suppressed if additional shear reinforcement is provided, so that the shear force associated with the

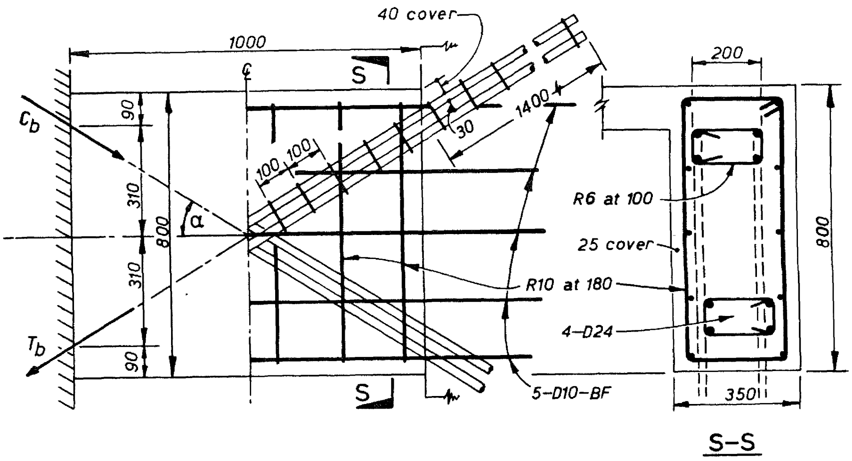


FIG. 34 - DETAILS OF A TYPICAL DUCTILE COUPLING BEAM

development of the flexural overstrength at both ends of the beam can be transferred across the potential diagonal failure crack without causing yielding in the stirrups. The performance of such beams is thus much improved, but the ductility developed is much less than is necessary if a ductile mechanism of the type shown in Fig. 7(c) is to be mobilized. Such beams will fail by sliding shear [25] as shown in Fig. 33(b). It is for this reason that in New Zealand diagonally reinforced coupling beams, shown in Fig. 33(c), have been used.

The disposition of internal forces and details of the reinforcement for such a beam are shown for an example coupling beam in Fig. 34. The extremely ductile response of such beams stems from the gradual transfer of shear resistance during inelastic cyclic response to the diagonal reinforcement. Eventually the entire shear may be resisted by this diagonal reinforcement at yield strength causing tension in one and compression in the other direction. At this stage the concrete need not participate in load transfer. This detail has been adopted also in several other countries.

3.2.3 Diagonally reinforced squat walls

It was shown in Section 2.2.3 that at the development of the flexural strength of a squat wall at its base, the associated shear force, V_a may be so large as to cause significant sliding after a few inelastic load reversals. To control it effectively, as in the case of short coupling beams, diagonal reinforcement may be used. The relevant principles are illustrated in Fig. 35. It is tempting to use a few diagonal bars as shown in Fig. 35(b). It is evident that this wall would be capable of sustaining a shear force of V_b because of the considerable flexural resistance provided at the base. Thus when these diagonal bars are placed in the wall of Fig. 35(a), the total capacity of the wall would be increased to $V_i = V_a + V_b$, while the two diagonal bars would offer resistance of $A_{sd} f_y \cos\alpha > V_b$ against sliding shear. However, this force may be only a relatively small fraction of the total shear demand, V_i. The aim of diagonal reinforcement to control sliding should be, however, to provide significant shear resistance without simultaneously increasing the flexural strength of the wall. This may be achieved with the arrangement shown in Fig. 35(c), where $V_c = A_{sd} f_y \cos \alpha_c$. Thus when the reinforcement of the wall in Fig. 35(c) is added to that of Fig. 35(a), the strength of the wall is not increased, but significant control of sliding displacement, with corresponding increase in energy dissipation capacity, will be achieved.

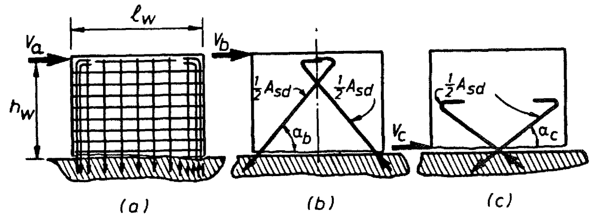


FIG. 35 - MODES OF SLIDING SHEAR RESISTANCE IN SQUAT STRUCTURAL WALLS

4 INNOVATIVE SOLUTIONS

Through progressive research our understanding of the behaviour of reinforced concrete structures at various stages of seismic load demands expanded. Critical areas of structures, previously not considered in buildings subjected to gravity and wind loadings only, have been identified. Attempts were then made to solve these new critical issues in traditional ways, usually by providing more reinforcement in some form. In certain cases these solutions lead to construction difficulties, particularly to congestion of reinforcement in critical regions where the quality of concrete, to be assured by proper compaction, was also of paramount importance. In certain cases satisfactory seismic performance with conventional detailing of the reinforcement was difficult to ascertain.

"If you can't solve a problem, try to avoid it". This was the guide in developing new

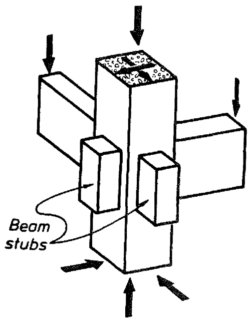


FIG. 36 - BEAM STUBS AT EXTERIOR JOINTS TO ACCOMMODATE BEAM BAR ANCHORAGES [25]

detailing strategies for difficult situations. The principles used were, however, as old as the theory of reinforced concrete. Some example solutions are presented here.

4.1 Beam-Column Joints

It was pointed out in Section 3.1.4(d) that undesirable consequences of plastic hinge formation adjacent to joints are the deterioration of bond and subsequent yield penetration along bars into the joint core. This then may result in excessive slip or to complete anchorage failure.

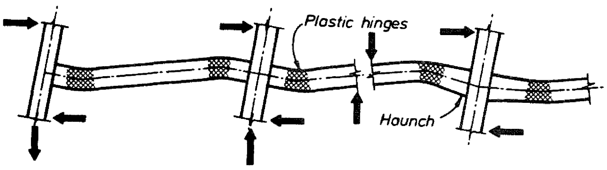


FIG. 37 - BEAMS WITH RELOCATED PLASTIC HINGES

At exterior joints, particularly at corner columns, serious congestion may arise because of the large number of hooked anchorages of both top and bottom bars. A small beams stub, as shown in Fig. 36, overcomes this problem [25,41]. It allows a much longer straight length of bar embedment to be used. Moreover, the vital hook anchorages are then located in a mass of concrete which is not affected by diagonal cracking.

Another way to eliminate bond deterioration within a joint core, is to ensure that, irrespective of the magnitude of inelastic seismic displacements, yielding of the beam reinforcement cannot occur at column faces. This necessitates the deliberate relocation of potential plastic hinges away from column faces, as shown in Fig. 37. It may be readily achieved by either appropriate curtailment of the flexural reinforcement [19,35] or by the use of vertical haunches [43], as shown at the right hand column in Fig. 37. In two-way frames horizontal beam haunches may be used, as shown in Fig. 38. This arrangement offers additional advantages, to be discussed subsequently.

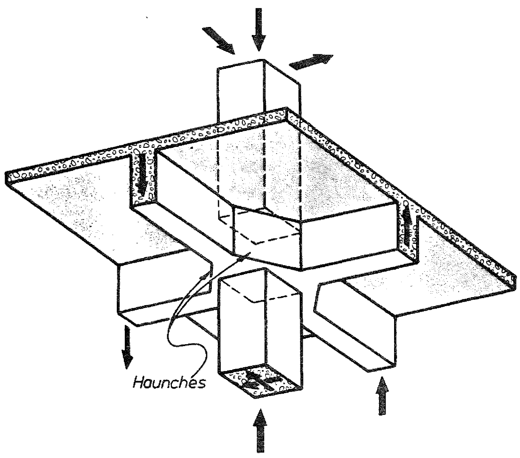


FIG. 38 - AN INTERIOR JOINT FORMED BY HORIZONTAL BEAM HAUNCHES [35]

A full exploitation of the very effective shear transfer mechanism by one diagonal concrete strut, shown in Fig. 29(a), can be made with the use of special anchorage plates [9]. Typical details of an example joint are shown in Fig. 39. In this mechanism bond transfer from the beam bars to the concrete of the joint is abandoned. Instead the beam bar forces, both tension and compression, are transmitted to a suitably dimensioned welded plate, which in turn transmits the combined forces to the concrete core by bearing. One anchorage plate transmits thus all the beam forces for one direction of the earthquake attack. To ensure that the distance between the anchorage plates does not increase significantly during cyclic loading, it is important to ensure that within the joint no yielding will occur along the beam bars.

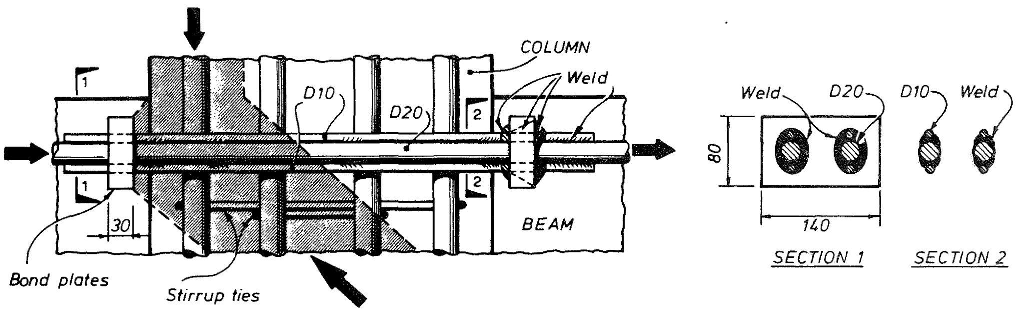


FIG. 39 - BEAM BAR ANCHORAGES AT INTERIOR JOINTS BY MEANS OF WELDED ANCHORAGE PLATES [9]

This may be achieved by increasing the area of bars, for example by the addition with welding of smaller bars, as shown in Fig. 39. Without this precaution, yielding of the beam bars would occur also between anchorage plates leading to slack joints with greatly reduced capacity to dissipate energy.

When beams and columns have suitable dimensions, the majority of beam bars at

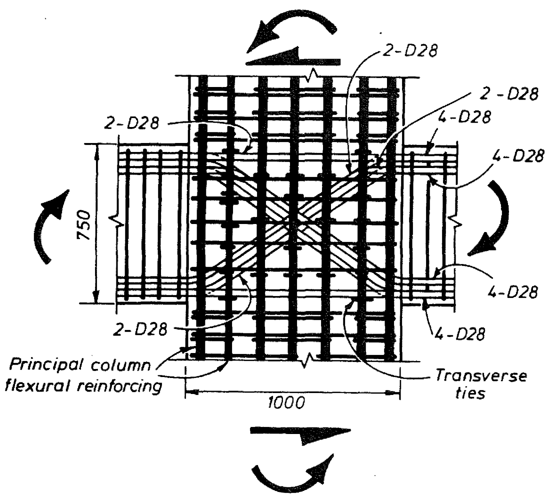


FIG. 40 - BEAM REINFORCEMENT BENT DIAGONALLY ACROSS A BEAM-COLUMN JOINT CORE

interior joints of one-way frames may be bent across the joint, as shown in Fig. 40. It is seen that diagonally bent bars are subjected to tension or to compression throughout the joint region. Thereby negligible or no bond forces need to be transferred to the surrounding concrete. The beam bars may thus transfer by diagonal tension and compression the major fraction of the necessary horizontal and vertical joint shear forces. Careful placement of the bent bars within the column must ensure the proper transfer of radial bearing stresses to the surrounding concrete [35].

The above examples illustrated efforts to enable joint shear forces to be resisted by mechanisms other than the diagonal compression yield of the truss mechanism shown in Fig. 29(b). It was seen that this mechanism requires joint shear reinforcement. Consequently the advantages in using the above "unconventional" solutions stem from a drastic reduction of horizontal joint shear reinforcement. Moreover, because of the elimination of yield penetration into the joint core and significant improvements in bond performance larger diameter and hence lesser numbers of beam bars may be used.

The use of rectangular and intermediate ties of usual diameter, for example with shapes seen in Fig. 24, for horizontal joint shear reinforcemnt may lead to serious congestion in the joint core. This is particularly the case at interior beam-column joints of two-way frames when beam plastic hinges are to be expected at all four faces of the column. In this case the entire joint shear force [41] may need to be resisted by the mechanism of Fig. 29(b). Congestion is alleviated if fewer large diameter bars with large yield strength (f_y = 380 MPa) are used [35]. Such bars, however, cannot readily be bent into shapes shown in Fig. 24. If sufficient space is provided, large hoops can be provided around the group of column bars. Such a solution is shown for a specific example structure in Fig. 41. The horizontal haunches, shown in Fig. 38, allow fewer 20 mm hoops to be used. It is seen, that within the beam depth, ties engaging individual column bars have been omitted. Even more space may be provided in the joint region when these few special hoops are formed by butt welding.

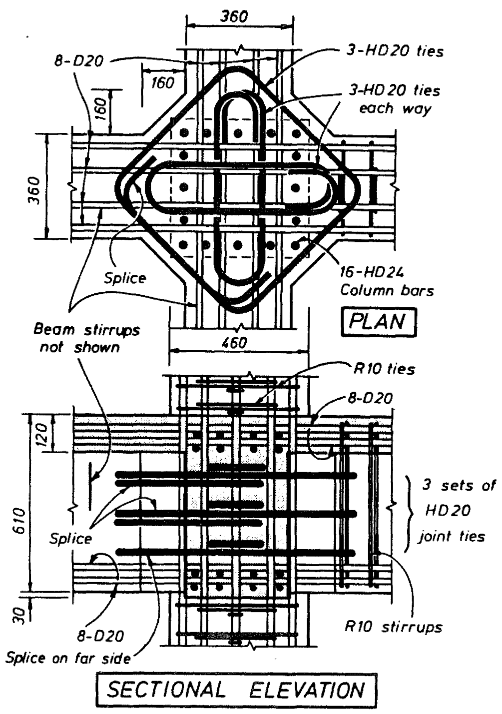


FIG. 41 - JOINT SHEAR REINFORCEMENT FORMED WITHIN HORIZONTAL BEAM HAUNCHES [35]

To eliminate yield penetration along beam bars into the joint core the area of flexural reinforcement may be locally increased by additional bars welded to the main bars [6]. This is similar to the details shown in Fig. 39 except that no anchorage plates are used. The additional bars in the joint region should extend by a small distance into the adjacent beam plastic hinges, as shown for a specific example in Fig. 42.

4.2 Spandrel Beams in Tube Frames

In certain multistorey buildings it is advantageous to assign the entire earthquake resistance to peripheral frames only. Closely spaced columns with relatively short spandrel beams may then be employed. Beams in these tube frames will

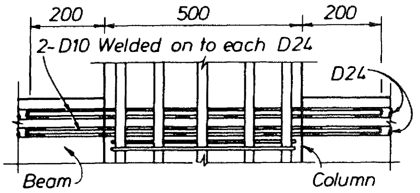


FIG. 42 - ADDITIONAL BEAM REINFORCEMENT AT AN INTERIOR COLUMN TO ENSURE THAT YIELDING OF BEAM BARS WILL NOT OCCUR WITHIN BEAM-COLUMN JOINTS [6]

reach the stage of strain hardening. Moment demands corresponding with code requirements, M_E, and those at the development of overstrength, M_o, are compared in Fig. 44 with the envelope of flexural resistance at ideal strength, M_i. The deformations in the centre region of a beam, to be expected during a large earthquake are similar to those shown in Fig. 33(c).

This system has been developed further to allow speedier erection of multistorey frames by using precast elements. As Fig.45* shows, the critical joint region of such a beam-column assembly is factory

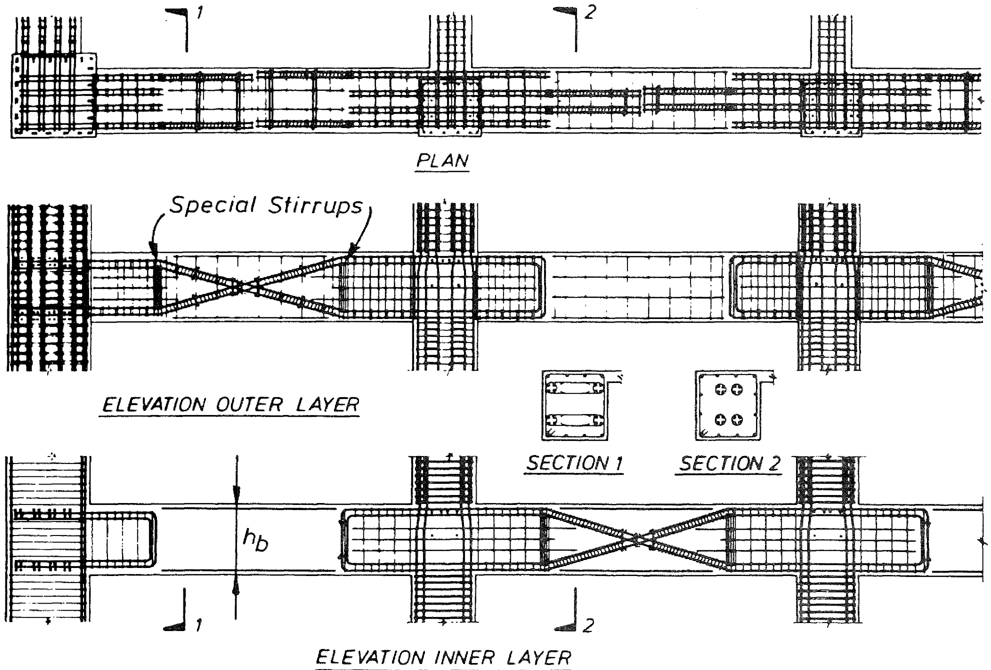


FIG. 43 - DIAGONALLY REINFORCED SPANDREL BEAMS FOR TUBE FRAMES [5]

be subjected to approximately equal shear forces for both directions of earthquake attacks. These are constant over the span. Gravity load effects are generally insignificant. To reduce the amount of joint shear reinforcement required, the beam plastic hinges may be relocated as shown in Fig. 37. However, because the spandrel beams may be short, the two potential plastic hinges required in each span, may be too close to each other. Thereby the rotational ductility demand on these hinges, corresponding with an acceptable overall displacement ductility for the structural system, may become excessive. In such situations the principles used in the design of diagonally reinforced coupling may be employed. Details of the first frames, so designed and constructed in New Zealand [5], are shown in Fig. 43. Moment resistance along each beam span is to be provided in such a way that the moments developed at column faces should not result in yielding of the horizontal beam bars, when the diagonal bars in the centre portion of the span

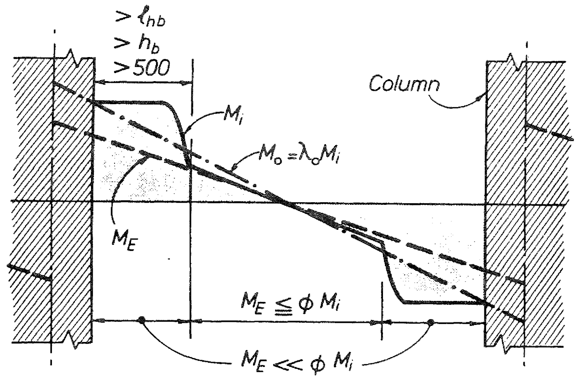


FIG. 44 - MOMENT ENVELOPES FOR DIAGONALLY REINFORCED SPANDREL BEAMS

cast. A simple site connection for the spandrel beams is provided at midspan, where, by virtue of the diagonal reinforcement, no moments are transferred. Splices for the vertical column bars are required immediately above each floor. These, however, are located in an elastic

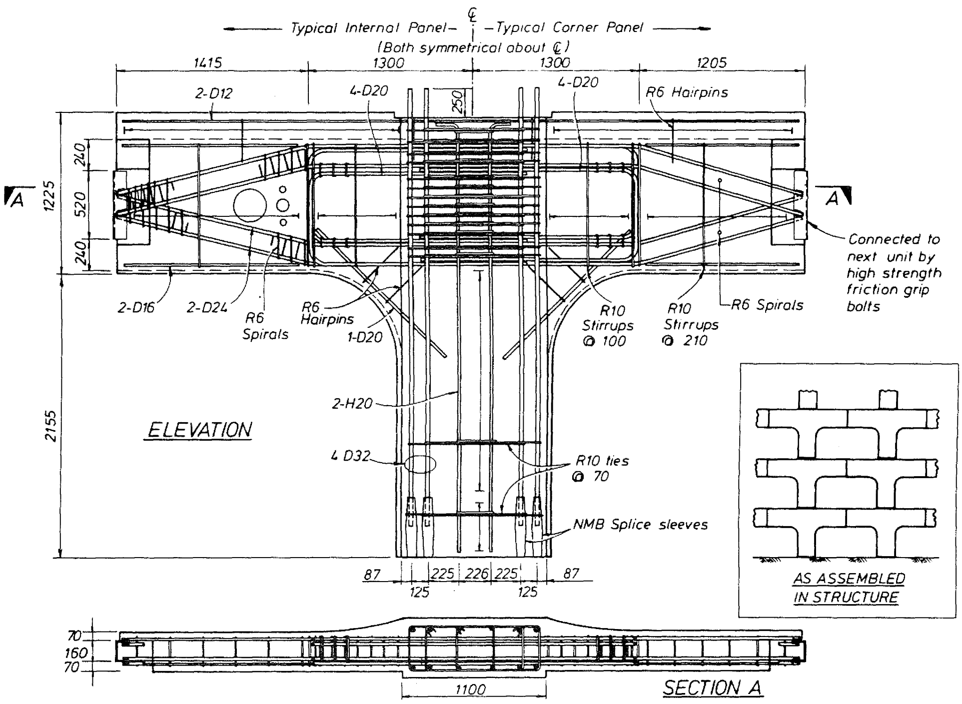


FIG. 45 - PRECAST BEAM-COLUMN UNIT*

region when the columns are designed according to the principles outlined in Section 2.1.3.

5 CONCLUSIONS

This review attempted to sketch highlights in the evolution of the seismic design strategy currently used for reinforced concrete buildings in New Zealand. The review is biased. It deliberately set out to emphasize those features of design which were thought to have had a significant input from work carried out in New Zealand. Descriptions of design procedures intended to illuminate that designer's determination to "tell the structure what to do". In spite of its simplicity, this design approach should ensure excellent inelastic structural response, provided that, as a complementary task, all critical regions are judiciously detailed. Examples were presented to manifest attempts to unambigously quantify the goodness of detailing. Thereby reinforced concrete buildings can be made extremely tolerant to a wide range of seismic demands. That is, they can be expected to perform "as they were told to".

6 ACKNOWLEDGEMENTS

It was not possible to acknowledge all contributions to the state of art of concrete design in New Zealand. The roles of a host of engineers, while pursuing their graduate studies, technical and administrative staff of our research organisations and universities have been invaluable. The progress made over the last two decades would not have been possible without the generous financial support provided by the New Zealand Ministry of Works and Development, the National Roads Board, the University Grants Committee, the Building Research Association and the Universities of Auckland and Canterbury. The speedy dissemination and acceptance of these developments by the engineering profession was to a great extent due to related activities sponsored by the New Zealand National Society for Eartqhuake Engineering and the New Zealand Concrete Society.

*Courtesy Holmes Wood Poole and Johnstone Ltd. Consulting Engineers, Christchurch.

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The references given here report only on a small fraction of the developmental work carried out in New Zealand. Wherever possible, papers and reports published in Bulletins of the New Zealand National Society for Earthquake Engineering have been selected. For convenience the abbreviation "Bulletin NZNSEE" has been used for these references.

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EDITOR'S NOTE

In 1987 the New Zealand National Society for Earthquake Engineering established a Travelling Lectureship. The aim of the Earthquake Engineering Lecture, to be given in public every two or three years, is to inform the New Zealand public about the aims and achievements in earthquake engineering in a broad spectrum, and the Society's role in these activities.

This paper presents, in somewhat greater detail, the contents of the inaugural lecture, given by the author as a keynote address in August 1987 during the Pacific Conference on Earthquake Engineering held in Wairakei, and delivered subsequently in Dunedin, Christchurch, Auckland and Wellington.