3.

Ductile Design Approach for Reinforced Concrete Frames

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In the design of multistorey moment-resisting reinforced concrete frames to resist severe earthquakes the emphasis should be on good structural concepts and detailing of reinforcement. Poor structural concepts can lead to major damage or collapse due to column sidesway mechanisms or excessive twisting as a result of soft storeys or lack of structural symmetry or uniformity. Poor detailing of reinforcement can lead to brittle connections, inadequate anchorage of reinforcement, or insufficient transverse reinforcement to prevent shear failure, premature buckling of compressed bars or crushing of compressed concrete. In the seismic provisions of the New Zealand concrete design code special considerations are given to the ratio of column flexural strength to beam flexural strength necessary to reduce the likelihood of plastic hinges forming simultaneously in the top and bottom of columns, the ratio of shear strength to flexural strength necessary to avoid shear failures in beams and columns at large inelastic deformations, the detailing of beams and columns for adequate flexural strength and ductility, and the detailing of beams, columns and beam-column joints for adequate shear resistance and bar anchorage. Differences exist between current United States and New Zealand code provisions for detailing beams and columns for ductility and for the design of beam-column joints.

1. INTRODUCTION

It is well known that when a structure responds elastically to ground motions during a severe earthquake, the maximum response acceleration may be several times the maximum ground acceleration and depends on the stiffness of the structure and the magnitude of the damping. For example, Fig. 1 shows the maximum acceleration response of a simple structure in the form of a single degree of freedom oscillator responding elastically to typical North American earthquake ground motions. Generally it is uneconomical to design a structure to

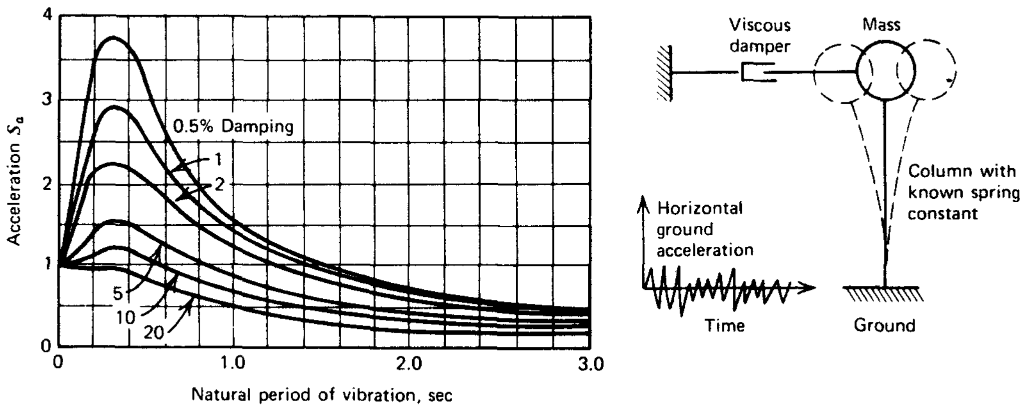


Figure 1 : Design Spectrum Giving Acceleration as a Function of Damping and Period of Vibration for a Single Degree of Freedom Oscillator Responding Elastically to Typical North American Earthquake Ground Motions.

respond in the elastic range to the greatest likely earthquake induced inertia forces. As a result, the design seismic horizontal forces recommended by codes are generally much less than the elastic response inertia forces induced by a major earthquake.

Experience has shown that structures designed to the level of seismic horizontal forces recommended by codes can survive major earthquake shaking. This apparent anomaly has been attributed mainly to the ability of well designed structures to dissipate seismic energy by inelastic deformations, helped by such other factors as reduced response due to decrease in stiffness and soil-structure interaction. Fig. 2 illustrates the lateral load-deflection response of an oscillator which is not strong enough to resist the full elastic response inertia load elastically and develops a plastic hinge with elasto-plastic characteristics. Because of the inelastic behaviour of the plastic

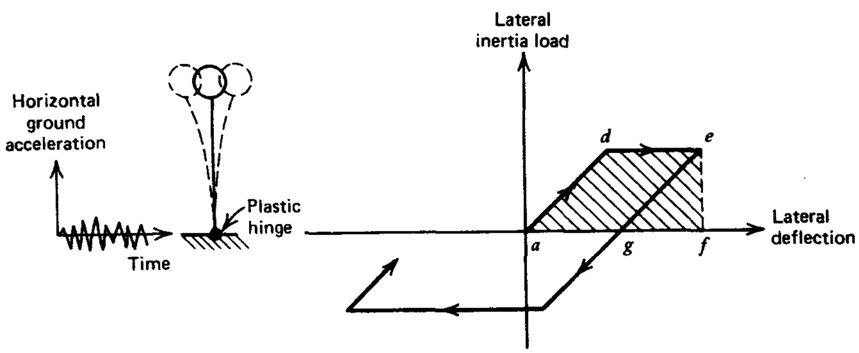


Figure 2 : Elasto-plastic Response of Ocillator to Earthquake Ground Motions.

hinge the maximum deflection of the elasto-plastic structure may not necessarily be much greater than that of the structure if behaving elastically. It is evident that use of the level of static seismic design loads recommended by codes implies that the critical regions of those members should have sufficient ductility to enable the structure to survive without collapse when subjected to several cycles of loading well into the inelastic range. This means avoiding all forms of brittle failure and achieving adequate ductility by flexural yielding of members.

The importance of good detailing of steel reinforcement in earthquake resistant reinforced concrete structures cannot be overemphasized. Significant protection against damage will be provided by carefully detailed reinforcement. As well as serving the normal function of providing resistance to tensile and compressive forces in concrete elements and their connections arising from bending, shear and normal forces, steel reinforcement is necessary to prevent compressed bars from buckling and to provide confinement to concrete in highly stressed areas of compression. Detailing should be based on a thorough understanding of the behaviour of reinforced concrete, thus ensuring that the requirements of all the internal forces in the structure are considered from the point of view of the serviceability, strength and ductility of the structure.

This paper will summarize developments in the ductile design approach for reinforced concrete frames subjected to seismic loading.

2. GENERAL PRINCIPLES OF DUCTILE DESIGN APPROACH

2.1 Design Strength and Ductility

It is evident that since it is generally uneconomical to design a structure to withstand the greatest likely earthquake without damage, the cost of providing strength must be weighed against the likely damage. The criteria for the level of loading of most seismic codes are as follows: buildings should be able to resist minor earthquakes without damage, to resist moderate earthquakes without structural damage but possibly with some nonstructural damage, and to resist major earthquakes without collapse but possibly with some structural and nonstructural damage. Hence the possibility of damage caused by a major earthquake is accepted, but not loss of life. The definitions of minor, moderate and major earthquakes vary from country to country. In many countries, for example New Zealand, only one level of earthquake load is considered in design, the level being that corresponding to a major earthquake.

In setting the levels of design static seismic loading, codes also have to consider the associated post-elastic deformations of the structure designed to that strength. Nonlinear dynamic analyses of code designed multistorey structures responding to typical major earthquake ground motions have given an indication of the order of post-elastic deformations, and hence the "ductility factor" required. However the number of variables involved in such analyses is so great that no more than qualitative statements concerning ductility demand can be made. For example, the type of ground motion has a considerable influence. Nevertheless some general conclusions can be drawn.

A measure of the ductility required of a structure is the displacement ductility factor μ defined as

$$\mu = \Delta_u/\Delta_y \tag{1}$$

where Δ_u = maximum horizontal deflection of the structure during severe earthquake shaking, generally measured either at the top of the structure or at the point of action of the resultant horizontal seismic load.

Δ_y = horizontal deflection at that point of the structure at first yield.

A number of dynamic analyses have indicated that the maximum horizontal deflection reached by a structure, which is not strong enough to resist the full elastic response inertia load and yields with elasto-plastic load-deflection characteristics, may be approximately the same as that of a structure which is strong enough to respond in the elastic range. This "equal maximum deflection" response is illustrated in Fig. 3. The comparison is for structures with the same initial stiffness, the same percentage of critical viscous damping, and responding to the same major earthquake.

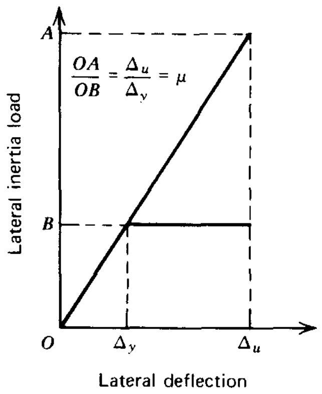


Figure 3 : Equal Maximum Deflection Response of Elastic and Elasto-plastic Structures to Earthquake Ground Motions.

Typically seismic codes assume an available displacement ductility factor of about μ = 3 to 5 for ductile frame structures and the design loads can be regarded as being approximately $1/\mu$ of the elastic response inertia loads. However the actual design spectrum used for seismic loading does not follow the exact shape of the spectrum shown in Fig. 1 but may be of trilinear or some other shape. Also the equal maximum displacement concept may not be particularly accurate for small periods of vibration. For small-period structures the maximum deflection reached during elasto-plastic response may be considerably greater than that reached during elastic response. Also the particular earthquake ground motion record will affect the comparison of elasto-plastic and elastic response. Hence the real displacement ductility demand on a structure may be quite different from code assumed values.

2.2 Structural Configuration

As far as possible the structural configuration of a building should have symmetry and uniformity.

The arrangement of the seismic load resisting elements in a building should, as nearly as is practicable, be located symmetrically about the centre of mass of the building. This requirement is in order to minimize the torsional response of the structure during an earthquake. Unsymmetrical structural arrangements, for example, walls enclosing a service core at one end of the building only which is structurally connected to the remainder of the building, can result in significant twisting about the vertical axis of the building and hence lead to greater ductility demand on some parts of the structure than for symmetrical arrangements. Such twisting may become critical for the overall stability of the building. Furthermore, due to numerous uncertainties, the actual behaviour of an unsymmetrical building structure is difficult to predict, even with elaborate computer models. Note that the participation of nonstructural elements in the response of the structure may result in unexpected and undesirable torsional effects. The designer should endeavour to anticipate the influence of nonstructural elements on the response of the structure.

It is undesirable for discontinuities in stiffness and/or strength of the structural system to exist up the height of the building. For example, the absence of some vertical structural elements in one storey of a building can lead to a dangerous concentration of ductility demand in the remaining elements of the storey. Similarly, sudden variations in building plan dimensions up the height of the building can result in equally dangerous large local deformations.

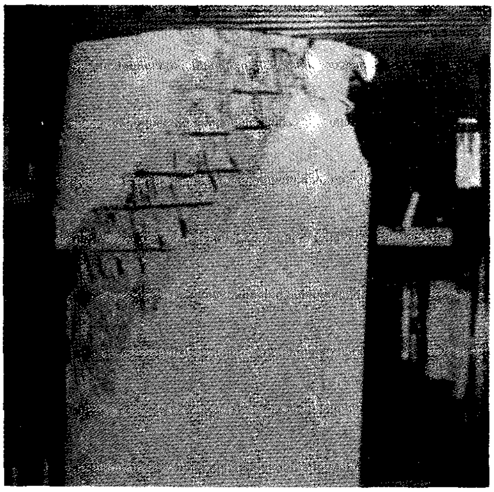
There are many examples of severe earthquake damage occurring in reinforced concrete buildings as a result of poor structural configurations. Fig. 4a shows the Macuto Sheraton Hotel which suffered very severe structural damage during the 1967 Venezuela earthquake, as well as damage to tile infill walls [1]. The building had a poor structural feature - an abrupt change of stiffness from frames to structural walls above the mezzanine floor. This more flexible and less strong portion under the structural walls led to a concentration of inelastic strains and energy dissipation in this portion. Figs. 4b and c show typical compression and shear failures which occurred in the 1.1 m (43 in) diameter reinforced concrete columns under the shear walls. As another example, Fig. 5a shows part of the main building of the Olive View Hospital which suffered very severe damage during the 1971 San Fernando earthquake [2]. Structural walls were present in the upper storeys but were not present in the first storey. A sidesway mechanism developed in the columns of the first storey resulting in approximately a 0.6m (2 ft) permanent lateral deformation in the tops of those columns after the earthquake. Fig. 5a illustrates the deformations in the first storey. Fig. 5b indicates that the corner columns of that storey, which were only lightly tied, were insufficiently ductile to maintain their load carrying capacity at such large deformations. Fig. 5c shows that the spiral columns of that storey were able to preserve sufficient strength to prevent collapse, the circular spiral being able to effectively confine the concrete and prevent buckling of longitudinal bars. Again the more flexible and less strong portions under the structural walls led to a concentration of deformations in that storey of the building.

2.3 Interstorey Deflections

The horizontal deflection of a structure during an earthquake should not be so large as to cause extensive damage to nonstructural elements.

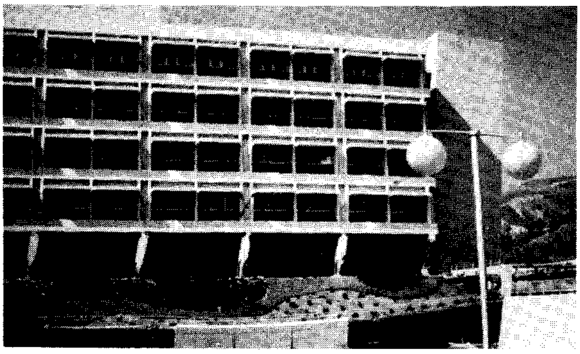
(a) The hotel building after the earthquake

(b) View of failed columns at mezzanine level



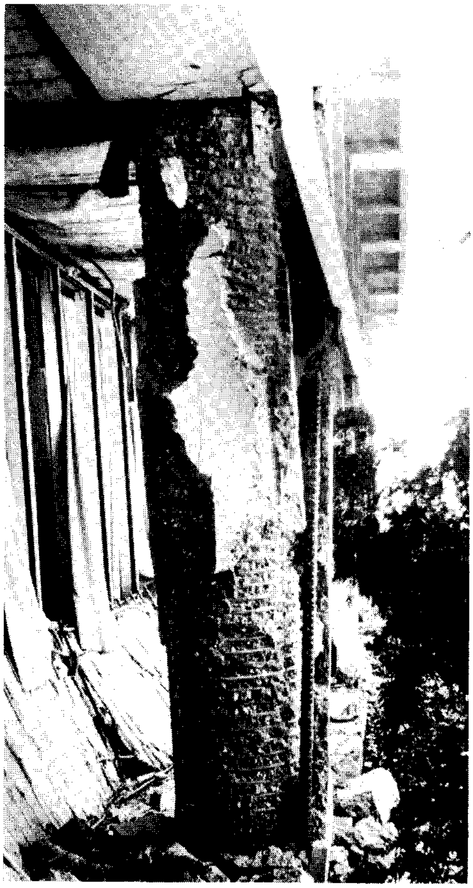
(c) Close up of damage to a column

Figure 4 : Damage to Macuto Sheraton Hotel Caused by the 1967 Venezuela Earthquake [1].



(a) A wing of the hospital building illustrating the sidesway deformations in the columns of the first storey

(b) Shattered column with inadequate tie reinforcement



(c) Core of spiral column maintaining load carrying capacity

Figure 5 : Damage to Olive View Hospital Building Caused by 1971 San Fernando Earthquake [2].

Also, large horizontal deflections result in high PΔ moments in the structure, where P is the column compressive load and Δ is the horizontal deflection of the column, which could lead to a significant reduction in the ability of the structure to resist further seismic loading. Codes normally place limits on the interstorey deflections to control damage and PΔ effects.

2.4 Achieving Adequate Ductility in Structural Systems

The exact characteristics of the earthquake ground motions that may occur at a given site cannot be predicted with certainty and the analytical modelling of some aspects of the behaviour of complete building structures is still open to question. Hence it is impossible to evaluate all aspects of the complete behaviour of a reinforced concrete building when subjected to very large seismic disturbances. Nevertheless it is possible to impart to the structure features that will ensure the most desirable behaviour. In terms of damage, strength and ductility (including energy dissipation) this means ensuring a desirable sequence in reaching the strengths of the various modes of resistance of the structure. It implies a desired heirarchy in the failure modes of the structure.

The rational approach for achieving this aim in earthquake resistant design is to chose the most suitable energy dissipating mechanism for the structure and to ensure by appropriate design procedures that yielding will occur only in the chosen manner during a severe earthquake. For reinforced concrete frames the best means of achieving energy dissipation is by flexural yielding at selected plastic hinges, since with proper design the plastic hinge positions can be made adequately ductile.

The ductility required at a plastic hinge in a yielding structure may be expressed by the curvature ductility factor ϕ_u/ϕ_y, where ϕ_u is the maximum curvature (rotation per unit length) at the critical section and ϕ_y is the curvature at the section at first yield. It should be emphasized that the required curvature ductility factor ϕ_u/ϕ_y at plastic hinge sections will generally be much greater than the required displacement ductility factor Δ_u/Δ_y value for the structure, since once yielding commences further displacement occurs mainly by rotation at the plastic hinges. This aspect of behaviour in the yield range is discussed further below.

For frames, mechanisms which involve flexural yielding at plastic hinges are shown in Fig. 6. If yielding commences in the columns of a frame before the beams, a column sidesway mechanism can form. In the worst case the plastic hinges may form in the columns in only one storey as in Fig. 6b, since the columns of the other storeys are stronger. Such a mechanism can make very large curvature ductility demands on the plastic hinges of the critical storey [3], particularly for tall buildings. The curvature ductility required at the plastic hinges of a column sidesway mechanism may be so large that it cannot be met and in that case collapse of the structure will occur. On the other hand if yielding commences in the beams before in the columns, a beam sidesway mechanism, as illustrated in Fig. 6c, will develop [3], which makes more moderate

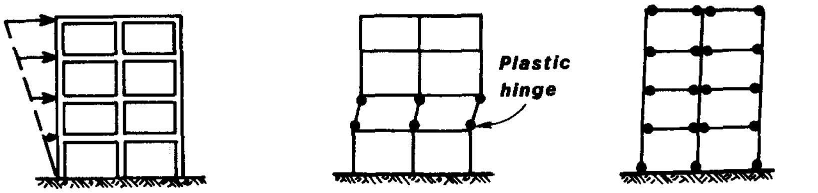


Figure 6 : Moment Resisting Frames With Horizontal Seismic Loading and Possible Mechanisms.

demands on the curvature ductility required at the plastic hinges in the beams and at the column bases. The curvature ductility demands at the plastic hinges of a beam sidesway mechanism can be met by careful detailing. Therefore for tall frames a beam sidesway mechanism is the preferred mode of inelastic deformation and a strong column-weak beam concept is advocated to ensure beam hinging. For frames with less than about three storeys, and for the top storey of tall frames, the curvature ductility required at the plastic hinges if a column sidesway mechanism develops is not particularly high. Hence for one and two storey frames, and in the top storey of taller frames, a strong beam-weak column concept can be permitted.

The required curvature ductility factor ϕ_u/ϕ_y which should be available at the plastic hinge locations in frames will depend on the many variables involved, such as the geometry of the members and the relative strengths of sections. This required ϕ_u/ϕ_y value for a particular frame can be calculated from the static collapse mechanism [3]. However it would appear that an available ϕ_u/ϕ_y of at least 3 should be provided at potential plastic hinge regions of regular frames, where μ is the required displacement ductility factor.

It should be appreciated that the static collapse mechanisms of Fig. 6 are idealised in that they involve possible behaviour under code type equivalent static seismic loading. The actual dynamic situation is different, due mainly to the effects of higher modes of vibration. Considerations such as in Fig. 6 can only be regarded as providing the designer with a reasonable feel for the situation. In important cases of some unusual structures it may be necessary to use time-history nonlinear dynamic analysis of the structure responding to severe earthquakes to obtain a better indication of the required curvature ductility at the critical plastic hinge sections.

3. NEW ZEALAND GENERAL SEISMIC DESIGN PROVISIONS

3.1 Introduction

The New Zealand general seismic design provisions are contained in a code for general structural design and design loadings for buildings [4]. This code was first published in 1976 and a second edition was published in 1984. The code sets out the following general seismic design principles.

3.2 Adequate Ductility

Structural systems intended to dissipate energy by ductile flexural yielding should have adequate ductility. Adequate ductility may be considered to have been provided if all primary elements resisting seismic forces are detailed for ductility in accordance with the seismic provisions of the concrete design code [5].

An approximate criteria for adequate ductility, applicable to reasonably regular symmetrical frames without sudden changes in storey stiffness, given in the commentary of the code [4], is as follows: the building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the design static seismic loading, calculated on the assumption of appropriate plastic hinges, is at least four times that at first yield, without the horizontal load carrying capacity of the building being reduced by more than 20%. The horizontal deflection at the top of the building at first yield can be taken as that when yield first occurs in any main structural element or that at the design static seismic load calculated on the assumption of elastic behaviour, whichever is the greater.

3.3 Capacity Design

Building frames designed for flexural ductile yielding should be the subject of "capacity design". In the capacity design of earthquake-resistant structures, energy-dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur.

3.4 Concurrent Earthquake Loading Effects

Columns, joints and foundations, which are part of two way frames, should be designed for concurrent earthquake load effects resulting from the simultaneous yielding of all beams in both directions.

3.5 Energy Dissipating Mechanisms

Ductile frames should be capable of dissipating seismic energy in a flexural mode at a significant number of plastic hinges in the beams, except that energy dissipation by column plastic hinge mechanisms is permitted in single or two-storey structures and in the top storey of multistorey structures. Apart from those specific cases, columns should be designed to have adequate strength to avoid column hinge mechanisms, taking into account possible distributions of column moments which may be different from that derived from elastic analysis with code static seismic loading applied and column loads appropriate to the simultaneous formation of plastic hinges in beams in several storeys.

3.6 Interstorey Deflections

For ductile frames in the worst seismic zone the horizontal deflection between two successive floors, computed for the design static seismic loading and assuming that the frame remains elastic, should not exceed 0.0032 of the storey height. In effect this means that, assuming the structure yields at the design static seismic loading, the permitted horizontal deflection at a displacement ductility factor of $\mu = 4$ is 1.3% of the storey height. The permitted interstorey deflection is less than this value if the nonstructural elements are not effectively separated from the structural elements.

4. NEW ZEALAND SEISMIC DESIGN PROVISIONS FOR DUCTILE REINFORCED CONCRETE FRAMES

4.1 Introduction

A considerable amount of research and development into the design of reinforced and prestressed concrete moment resisting frames and structural walls for seismic resistance has been conducted in New Zealand in recent years. The New Zealand seismic design provisions for ductile reinforced concrete frames are contained in a code for the design of concrete structures [5] published in 1982. The design provisions for gravity loading are based mainly on the 1977 building code of the American Concrete Institute. The seismic design provisions are based mainly on the research conducted in New Zealand and elsewhere. Papers resulting from deliberations of discussion groups [6,7,8] organised by the New Zealand National Society for Earthquake Engineering were used as a basis for the seismic provisions of the new code. The emphasis of the code is on good structural concepts and good detailing of reinforcement. It is also recognised that a proper assessment of the strength and ductility of a structure cannot be made using the working stress design method. It is required that the strength design method be used for seismic design.

The critical regions in frames to be designed for seismic resistance are illustrated in Fig. 7. The design actions at those regions and the design of the regions for adequate strength and ductility require the

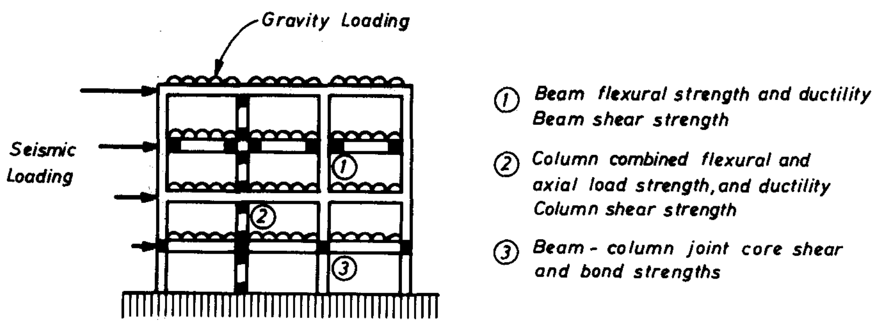


Figure 7 : Critical Regions in Frame Designed for Seismic Resistance.

particular attention of the designer.

The provisions of the concrete design code [5] for the seismic design of ductile reinforced concrete frames, and the background to those provisions, are outlined below.

4.2 Material Behaviour

4.2.1 Steel

Fig. 8 shows stress-strain curves measured for typical steel reinforcing bars under monotonic loading. In practice the actual yield strength of the steel will normally exceed the specified yield strength f_y. Also, in the plastic hinge regions the longitudinal reinforcement may reach strains in the order of 10 or more times the strain at first yield [3] and an increase in steel stress due to strain hardening may occur. The resulting increase in the flexural strength of beams in plastic hinge regions due to these two factors is of concern since it is accompanied by: (a) an increase in the shear forces in the members which could result in brittle failure, (b) an increase in the column moments which could cause a column sidesway mechanism, and (c) an increase in the shear forces in beam-column joints. A capacity design procedure is used to ensure that failure does not occur away from plastic hinge locations during a major earthquake.

In the capacity design procedure it is assumed that the plastic hinge regions develop their maximum likely flexural strength, referred to as "flexural overstrength". It is recommended that the flexural overstrength should be calculated assuming that the actual strength of the longitudinal reinforcing steel is $1.25f_y$ when f_y = 275 MPa (40 ksi) and $1.40f_y$ when f_y = 380 MPa (55 ksi), where f_y = specified yield strength of the reinforcing steel. These are the two grades of reinforcing steel in common use in New Zealand. The overstrength moment at plastic hinges takes into account both the actual steel yield strength being greater than specified and the effect of strain hardening at high strains. The higher overstrength factor for Grade 380 steel is due to the earlier strain hardening of that steel (see Fig. 8). The beams and joint shears and column actions used in design are the maximum likely values calculated on the basis of the overstrength beam actions.

Fig. 9a and b show stress-strain curves measured for reinforcing steel under cyclic loading. The "rounding" of the stress-strain curve during the loading reversals in the inelastic range due to the Bauschinger effect is of interest. This reduction in the tangent modulus of the steel at relatively low compressive stresses during reversed loading makes the buckling of compression steel more likely than would be expected in monotonic load tests. It is recommended that to prevent premature buckling of reinforcing bars during cycles of reversed loading the centre to centre spacing of transverse reinforcement restraining the longitudinal bars should not exceed six longitudinal bar diameters in the potential plastic hinge region of members.

Fig. 9a shows measured stress-strain behaviour for cyclic stressing with strain mainly in the tensile range. This cyclic strain history would

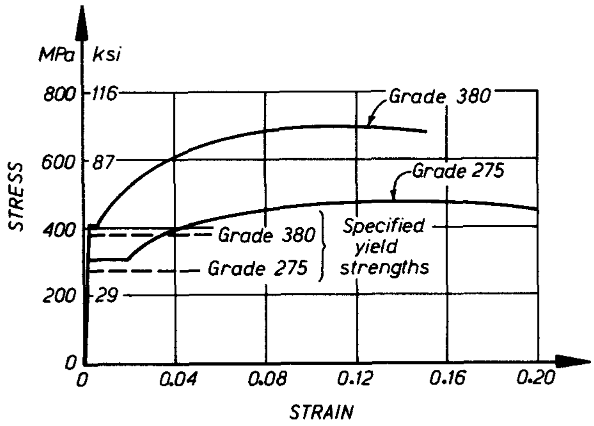
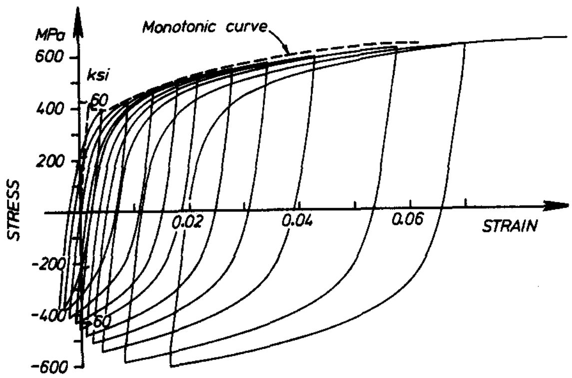
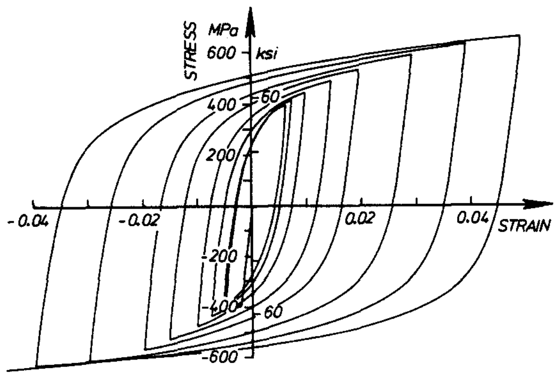


Figure 8 : Stress-Strain Relations for Typical Reinforcing Steel With Monotonic Loading.



(a) Grade 380 ($f_y \geq 55$ ksi) reinforcing steel with unsymmetrical strain cycles.



(b) Grade 380 ($f_y \geq 55$ ksi) reinforcing steel with symmetrical strain cycles.

Figure 9 : Stress-Strain Relations for Typical Reinforcing Steel With Cyclic Loading.

be typical of a longitudinal reinforcing bar in a beam during cyclic flexure where, since the neutral axis is close to the extreme compression fibre of the beam section, the steel strains in compression would be much smaller than the steel strains in tension. The stress-strain curve for monotonic loading is also shown in Fig. 9a. For the type of cyclic strain history shown in the figure it is evident that the monotonic stress-strain relation with origin at the original position gives a satisfactory envelope for the cyclic stress-strain behaviour of reinforcing steel in beams. Hence overstrength factors for beams can be based on monotonic stress-strain relations for the reinforcing steel.

Fig. 9b shows the results for cyclic stressing with gradually increasing symmetrical straining in the tension and compression range. This strain history would be typical of a longitudinal reinforcing bar in a building column during cyclic flexure with high axial load in which the neutral axis is close to mid-depth of the column section. The particular specimen in Fig. 9b was yielded in compression first. It is evident that symmetrical straining of the type shown in Fig. 9b results in a build up of steel stress to stress levels which are much greater than that given by the monotonic stress-strain curves with the origin in the original position. The build up of steel stress for this type of loading with increasing strain amplitudes is particularly high for steel which strain hardens early. Hence overstrength factors for columns used in design may need to be greater than those used for beams.

4.2.2 Concrete

In order to achieve ductile plastic hinge behaviour it is essential to avoid sudden failure of the concrete when it reaches its compressive strength. Concrete can be made to act in a ductile manner by providing adequate transverse confining reinforcement in the form of arrangements of spirals, hoops, stirrup ties or cross ties. The concrete becomes confined when at strains approaching the unconfined strength the transverse strains become very high and the concrete bears out against the transverse reinforcement which then applies a passive confining pressure. The strength and ductility of concrete is considerably improved by confinement. The confinement arises because of arching of the concrete between the transverse bars and the longitudinal bars. The cover concrete, including that concrete outside the arching forces, is not confined and will be lost as in the case of unconfined concrete.

It is evident from Fig. 10 that the confinement of concrete is improved if the transverse reinforcement is placed at relatively close spacing, and if there are a number of longitudinal bars well distributed around the column section and ties across the section, because then the arches between the bars are shallower and hence more of the concrete section is confined. It is recommended that in the potential plastic hinge regions of columns the centre to centre spacing of sets of transverse confining reinforcement should not exceed 0.2 of the smaller cross section dimension or 200 mm (7.9 in), whichever is smaller, and the spacing between tied longitudinal bars in columns with rectangular hoops should not exceed 200 mm (7.9 in).

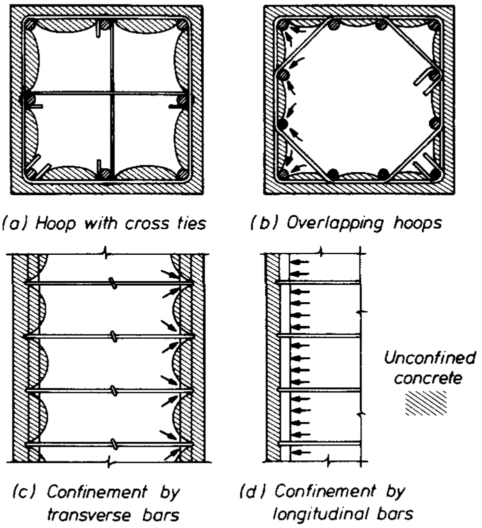


Figure 10 : Confinement of Concrete by Reinforcement.

4.3 Design of Beams

4.3.1 Beam Dimensions

To prevent lateral instability of beams, particularly after a reduction in stiffness resulting from cyclic flexure in the inelastic range, the dimensions of T and L beam members to which moments are applied at both ends should be such that

$$\ell_n/b_w \leq 37 \qquad (2)$$

$$\ell_n h/b_w^2 \leq 150 \qquad (3)$$

where ℓ_n = clear span, h = overall depth and b_w = width of web. For beams with rectangular cross sections the limitations on the right hand side of Eqs. 2 and 3 should be reduced to two-thirds of the values shown. These equations are approximations based on the theory for lateral instability of beams [3].

Also, to keep the longitudinal beam reinforcement reasonably close to the column core for effective moment transfer, the width of the beam web should not be more than the width of the column plus a distance each side of the column equal to one quarter of the overall depth of the column.

4.3.2 Design Moments

The design moments in beams should be calculated by elastic frame analysis for the code seismic and gravity loadings and the factored (ultimate) load combinations. Moment redistribution from the derived moment envelope may be used to gain a more advantageous design moment envelope and thus allow a more efficient design of the beam. The amount of moment redistribution permitted for any span of a beam forming part of a continuous frame should not exceed 30% of the maximum elastic moment derived for that span for any combination of design earthquake and gravity loading. The modified moments are used to calculate moments elsewhere in the span. This is, static equilibrium between the internal forces and the external loads must be maintained.

4.3.3 Plastic Hinge Regions

The plastic hinge regions are considered to extend over a length equal to twice the beam overall depth (2h) in the vicinity of the maximum moment sections, as illustrated in Fig. 11.

4.3.4 Longitudinal Reinforcement

The sections are designed for flexure by strength theory using a strength reduction factor ϕ = 0.9.

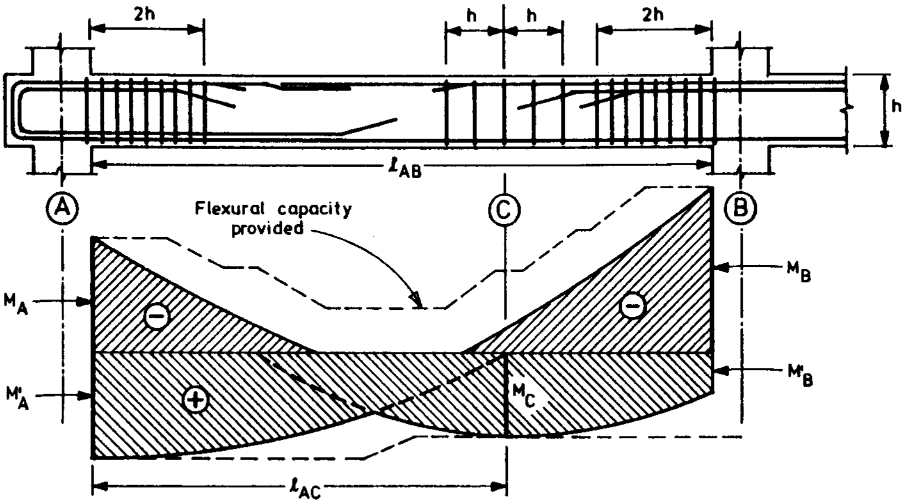


Figure 11 : Localities of Plastic Hinge Regions in Typical Beam and Bending Moment Envelope Due to Design Seismic Plus Gravity Loading [5].

In order to ensure ductile flexural behaviour during reversed loading at any section with a plastic hinge region:

(a) Tne tension steel ratio should not be greater than

$$\rho = \frac{1 + 0.17\left(\frac{f'_c}{7} - 3\right)}{100}\left(1 + \frac{\rho'}{\rho}\right) \tag{4}$$

$$\text{or } \rho = 7/f_y \tag{5}$$

whichever is smaller, where $\rho = A_s/b_w d$, $\rho' = A'_s/b_w d$, A_s = area of tension reinforcement, A'_s = area of compression reinforcement, b_w = width of beam web, d = effective depth of beam, f'_c = concrete compressive cylinder strength (MPa), and f_y = steel yield strength (MPa), where 1 MPa = 145 psi.

(b) The compression steel should be such that

$$A'_s \geq 0.5A_s \tag{6}$$

The upper limit on ρ, as given by Eq. 5, is 2.55% for Grade 275 ($f_y \geq 40$ ksi) steel and 1.84% for Grade 380 ($f_y \geq 55$ ksi) steel. Generally the requirements of Eq. 4 will govern the maximum permitted ρ. For example if f'_c = 25 MPa (3,630 psi) and $\rho'/\rho = 0.5$, Eq. 4 gives ρ = 1.65% for both Grade 275 and Grade 380 steel. Eqs. 4 and 5 are plotted for Grades 275 and 380 steel in Fig. 12.

Note that these maximum permitted ρ values are more severe than currently allowed in the United States. For example, the seismic provisions of the ACI building code [9] permit ρ = 2.5% to be used.

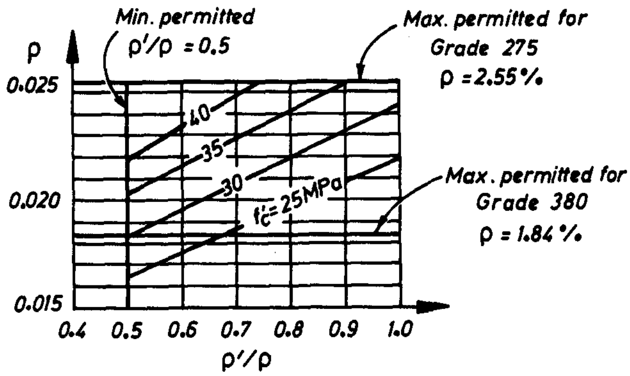


Figure 12 : Maximum Tension Steel Ratios for Beams (1 MPa = 145 psi) [5].

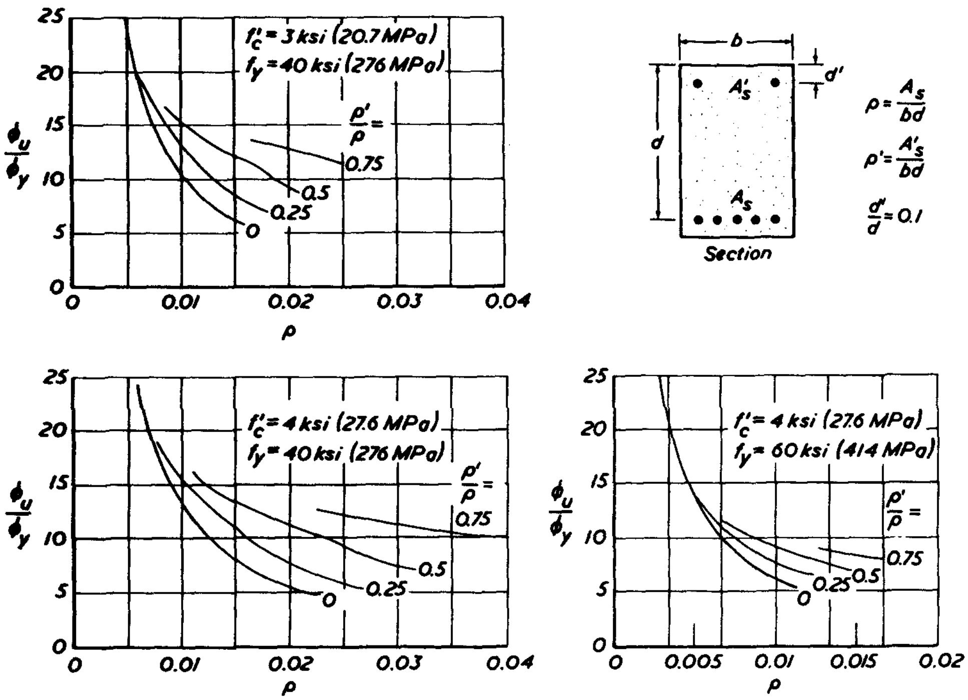


Figure 13 : Variation of Curvature Ductility Factor ϕ_u/ϕ_y for Doubly Reinforced Beam Section Assuming an Extreme Fibre Concrete Compressive Strain of 0.004 [3]

Fig. 13 shows the available curvature ductility ϕ_u/ϕ_y for a range of doubly reinforced rectangular concrete beam sections calculated assuming an extreme fibre concrete compressive stress of 0.004 [3]. The figure illustrates the increase in available ϕ_u/ϕ_y when ρ decreases and ρ'/ρ increases. The importance of the section containing a reasonable proportion of compression steel is evident from the figure and is recognised by the code requirement that $A'_s \geqq 0.5A_s$.

From Fig. 13 it can be determined that the use of Eqs. 4 to 6 will result in a curvature ductility factor ϕ_u/ϕ_y of at least 12 being available for a rectangular section reinforced with Grade 275 ($f_y \geqq 40$ ksi) steel when an extreme fibre concrete compressive strain of 0.004 is reached. It is to be noted that at that curvature there would be significant flexural cracking of concrete but no significant crushing of the compressed concrete. For sections with the same ρ, the available ϕ_u/ϕ_y when Grade 380 ($f_y \geqq 55$ ksi) steel is used will be less than when Grade 275 steel is used.

Outside the plastic hinge regions at least two reasonable size longitudinal reinforcing bars should exist in both the top and bottom of the beam throughout its length. The top reinforcement should be not less than one quarter of the top reinforcement at either end and the bottom reinforcement should not be less than $\rho = 1.4/f_y$, where f_y = specified yield strength of the longitudinal reinforcement (MPa, where 1 MPa = 145 psi). This is to allow for unexpected deformations and moment distributions which may occur during major earthquakes.

In T and L beams built integrally with slabs the slab longitudinal reinforcement placed within the shaded areas shown in Fig. 14 should be considered to participate in resisting the negative moments at the supports of continuous beams. The widths of slab considered effective in Fig. 14 depend on whether the column is interior or exterior and whether transverse beams exist. The widths shown are conservative estimates of the effectiveness of force transfer from the slab bars to the joint core. During large inelastic displacement of the frame it is likely that there will be a contribution from slab bars at larger distances from the column than shown in Fig. 14.

Splices in longitudinal reinforcement in beams should not be located within beam-column joint cores or within twice the effective depth of the beam from the critical section of a plastic hinge in a beam. This is because of the possible degradation of anchorage during cycles of severe seismic loading.

4.3.5 Transverse Reinforcement to Prevent Premature Buckling of Longitudinal Reinforcement and to Confine Compressed Concrete

In potential plastic hinge regions where moment reversal may cause the longitudinal bars to yield both in compression and tension at each face of the beam (for example, the end regions of Fig. 11), the maximum permitted centre to centre spacing of stirrup ties is 150 mm (5.9 in) or d/4 or six longitudinal bar diameters, whichever is least, where d = effective depth of beam.

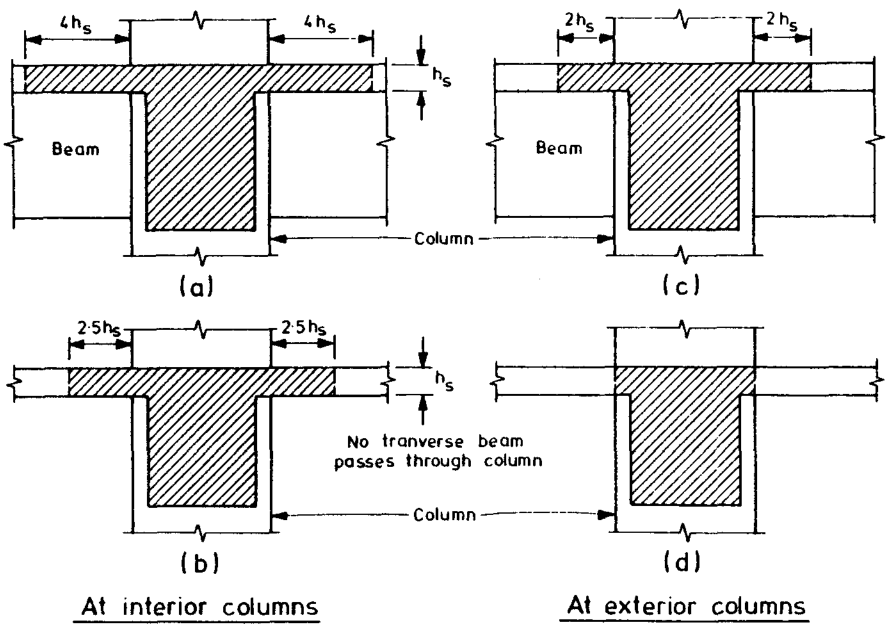


Figure 14 : Longitudinal Steel in Shaded Area to be Included in the Flexural Strength of the Beam at Column Face [5].

In potential plastic hinge regions of beams where yielding of longitudinal bars occurs only for one direction of moment (for example, the region C of Fig. 11) the maximum permitted centre to centre spacing of stirrup ties is 200 mm (7.9 in) or d/3 or twelve longitudinal bar diameters, whichever is least.

The above difference in the required spacing of transverse reinforcement is because premature buckling of longitudinal reinforcement can be prevented by wider spacing when yielding occurs only in compression. When yielding occurs in both tension and compression there is a reduction in the tangent modulus of the steel at low stress levels which makes the bars more prone to buckle (see Fig. 9). It is considered that the above recommended spacing of transverse reinforcement is also adequate to provide some confinement to the concrete in the compression zone. In beam the prevention of premature buckling of compression steel is a more important requirement than the confinement of concrete.

It is also necessary to ensure that an adequate quantity of stirrup ties are present in potential plastic hinge regions to provide the necessary lateral forces to the longitudinal steel to prevent premature buckling. It is recommended that the area of one leg of a stirrup tie should be not less than

$$A_{te} = \frac{1}{16} \frac{\Sigma A_b f_y}{f_{yh}} \frac{s}{100} \quad (\mathrm{mm}^2) \tag{7}$$

where ΣA_b = sum of areas of longitudinal bars reliant on the tie (mm^2), f_y = yield strength of longitudinal reinforcement (MPa), f_{yh} = yield strength of transverse reinforcement (MPa) and s = centre to centre spacing of ties (mm). (1 mm = 0.039 in, 1 MPa = 145 psi). This requirement is illustrated in Fig. 15 for the case where s = 100 mm (3.9 in).

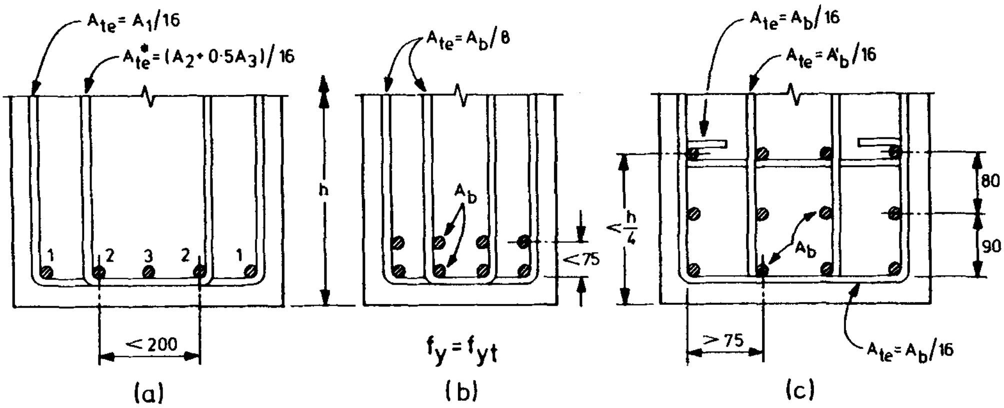


Figure 15 : Arrangement and Size of Stirrup Ties in Potential Plastic Hinge Regions of Beams [5] (Dimensions in mm, 1 mm = 0.039 in).

Note that the New Zealand requirements for the spacing of transverse reinforcement are more severe than currently recommended in the United States. The ACI code [9] in its seismic provisions requires that in potential plastic hinge regions the spacing of stirrup ties should not exceed d/4, eight longitudinal bar diameters, 24 transverse bar diameters and 12 in (305 mm), where d = effective depth of beam, whichever is least.

Outside the plastic hinge regions the detailing requirements are similar to those for gravity loading, but stirrup spacing should not exceed d/2, where d = effective depth of beam.

4.3.6 Transverse Reinforcement for Shear

The design shear force in beams should be determined by a capacity design procedure for when the flexural overstrength is reached at the most probable plastic hinge locations within the span and the factored gravity load is present. For example, for the beam in Fig. 11 the design shear force at B will be

$$V_{uB} = \frac{M'_{oA} + M_{oB}}{\ell_{AB}} + \frac{w\ell_{AB}}{2} \qquad (8)$$

where M'_{oA} and M_{oB} are the flexural overstrength capacities of the sections for positive moment at A and negative moment at B, and w is the factored uniform dead and live load considered to be present per unit length. The flexural overstrength capacities are calculated using the overstrength factors for the reinforcing steel discussed in Section 4.2.1.

Since the design shear forces are found from the assumed flexural overstrengths, it is reasonable for the design of sections for shear to be carried out using a strength reduction factor ϕ for shear of 1.0. However it has been found from tests [3] that reversed flexure in plastic hinge regions causes a degradation of the shear carried by the concrete shear resisting mechanisms of aggregate interlock, dowel action and across the compression zone. Therefore it is required that shear reinforcement, normally in the form of vertical stirrups, should be provided to carry the total shear force in plastic hinge regions. Thus the area of vertical stirrup in plastic hinge regions is given by

$$A_v = \frac{V_u s}{d f_{yh}} \quad (\mathrm{mm}^2) \tag{9}$$

where V_u = design shear force (N), s = centre to centre spacing of stirrups (mm), d = effective depth of beam (mm), and f_y = yield strength of stirrups (MPa)(where 1 N = 0.225 lb , 1 mm = 0.039 in, 1 MPa = 145 psi).

Also, tests [3] have demonstrated that full depth flexural cracks can exist in the plastic hinge regions, as well as inclined diagonal tension cracks, during much of the reversed loading range. This is because when longitudinal steel yields in tension for loading in one direction, open cracks will be present in the concrete "compression" zone when the load is applied in the opposite direction. These cracks will remain open until that steel yields in compression and allows the cracks to close and the concrete to carry some compression (see Fig. 16). Thus for parts of the loading cycles the bending moment will be carried by a steel couple alone. If the shear stress at the section is high a sliding shear deformation can occur along a full depth vertical crack (see Fig. 16) and the load-deflection hysteresis loop for the structure will become "pinched", resulting in reduced energy dissipation. To avoid a sliding shear failure and loss of energy dissipation, inclined shear reinforcement (for example, bent up bars), is required in plastic hinge regions when the shear stresses to be carried are high. Therefore it is recommended that the design shear forces are resisted only by vertical stirrups if

$$\frac{V_u}{b_w d} \leq 0.3\ (2 + r)\sqrt{f'_c} \quad (\mathrm{MPa}) \tag{10}$$

where V_u = design shear force (N), b_w = web width of beam (mm), d = effective depth of beam (mm), f'_c = concrete compressive cylinder strength (MPa) and r = ratio at the plastic hinge section of the maximum shear force developed with positive moment hinging to the maximum shear force developed with negative moment hinging, always taken as negative (where 1 N = 0.225 lb, 1 mm = 0.039 in, 1 MPa = 145 psi). For higher values of $V_u/b_w d$ than permitted by Eq. 10, diagonal shear reinforcement should be

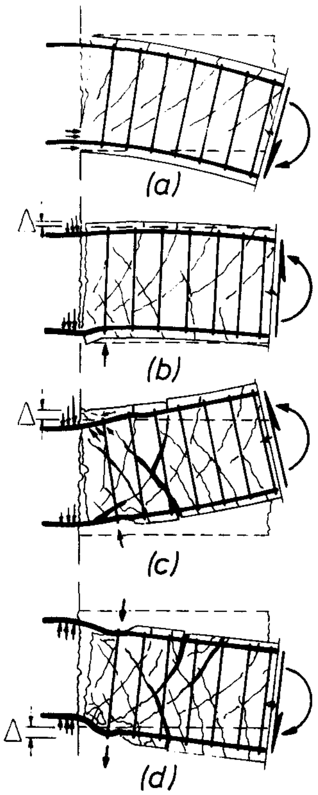


Figure 16 : Significant Stages of Development of Plastic Hinge During Cyclic Flexure With High Shear [10].

provided across the web in the plastic hinge region in one or both directions to resist a specified proportion of the shear force.

Outside the plastic hinge regions the shear may be considered to be carried by both transverse reinforcement and the concrete shear resisting mechanisms, and the shear reinforcement may be designed by the shear strength equations used for gravity load design.

4.4 Design of Columns

4.4.1 Design Moments and Axial Loads

A strong column-weak beam concept is adopted for frames with more than two storeys, in order to prevent as far as possible column sidesway mechanisms occurring during a major earthquake. To achieve this aim a capacity design approach is used to determine the design actions for the columns. In this approach the column bending moments found from elastic frame analysis for the code factored (ultimate) load combinations are amplified to take into account:

(1) The flexural overstrength at the beam plastic hinges, which results in higher moments being applied to the columns.

(2) The higher mode effects of dynamic loading, which can cause much higher column moments than calculated from the code static seismic loading which is based on mainly a first mode response. Nonlinear dynamic analyses have shown that in frames, due to higher modes of vibration, the points of contraflexure may occur well away from the mid-height of columns at various stages during an earthquake [3]. For example Fig. 17 shows a possible bending moment distribution in a column at an instant during an earthquake. It is evident that the total beam input moment $M_{b1} + M_{b2}$ may have to be resisted almost entirely by one column section, rather than be shared almost equally between the column sections above and below the joint as would be implied by the bending moment diagrams obtained using the code static seismic loading.

(3) The greater column moments in a two-way frame caused by the possible simultaneous yielding of beams in two directions due to seismic loading acting in a general (skew) direction and having components of load along both principal axes of the structure. For example, for the symmetrical building shown in Fig. 18, if a displacement ductility factor of 4 is reached in direction 2, it only requires $\Delta_1 = \Delta_2/4$ to cause yielding in direction 1 as well, and this occurs when θ is only 14°. Thus yielding

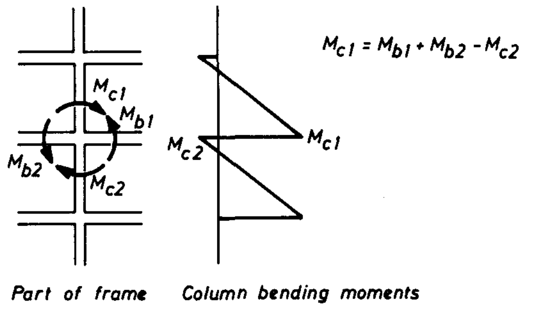


Figure 17 : Possible Column Moments During Dynamic Response.

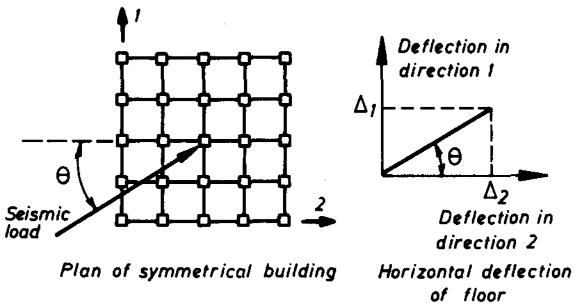


Figure 18 : General Direction of Earthquake Loading on Building.

in the beams in both directions may occur simultaneously for much of the seismic loading, and for a structure with beams of equal strength in each direction, the resultant beam moment input applied biaxially to the columns is $\sqrt{2}$ times the uniaxial beam moment input. Also biaxial bending will generally reduce the flexural strength of the column. Typically the flexural strength of a square column for bending about a diagonal may be 15% less than the flexural strength for uniaxial bending [3]. Therefore concurrent earthquake loading may cause the columns to yield before the beam unless columns are strengthened to take this effect into account.

It is evident that column flexural strengths much greater than the bending moments derived from the code static seismic loading are needed if plastic hinges in columns are to be avoided. The difficulty of preventing plastic hinges from forming in columns is such that some column yielding must be considered to be inevitable. The degree of protection of columns against plastic hinging is a debatable issue and needs to be approached on a probabilistic basis.

A method for the evaluation of column actions in ductile multistorey frames due to Paulay [6] has been placed in the Commentary of the code [5]. This procedure is aimed at giving reasonable protection against column yielding. In the procedure, the design uniaxial bending moment for the column, acting separately in each of the two principal directions of the structure, is given by

$$M_{col} = \phi_o \omega M_{code} \tag{11}$$

where M_{code} = column moment at the beam centre line derived from the code static seismic loading and to be reduced as indicated by the moment gradient to give the column moment at the beam face,

ϕ_o = ratio of overstrength flexural capacity of the beams as detailed to the dependable moment capacity required by the code $\geq 1.25/0.9 = 1.39$ for Grade 275 ($f_y \geq 40$ ksi) steel reinforcement,

ω = factor allowing for higher mode and concurrent load effects given for one-way frames as

$$\omega = 0.6T_1 + 0.85 \tag{12}$$

but not less than 1.3 or more than 1.8, and for two-way frames as

$$\omega = 0.5T_1 + 1.0 \tag{13}$$

but not less than 1.5 or more than 1.9, where T_1 = fundamental period of vibration of the structure.

Note that for two-way frames the columns are designed for uniaxial bending only, since ω includes some moment enhancement to make allowance for the effect of biaxial bending. The values of ω are based on dynamic analyses and judgement [6].

The design axial loads in columns P_{col} to be used with M_{col} for section design should be derived from the shear forces applied at the column faces by the gravity loads from the beams and the moment induced shears from the beam plastic hinge moments in both directions acting at flexural overstrength, except that a reduction in the moment induced shears is allowed to take into account the probability that not all beam plastic hinges have reached their overstrength up the height of the frame.

The maximum design axial load in columns is limited to $0.7f'_cA_g$ or $0.7P_o$, which ever is greater, where f'_c = concrete compressive cylinder strength, A_g = gross area of column section, and P_o = axial (concentric) load strength of column, since the ductility of very heavily loaded columns may be small even with extensive confining steel.

The column sections are designed for P_{col} and M_{col} by strength theory assuming a strength reduction factor $\phi = 1.0$, since the design actions are maximum probable values.

It is evident that the multiplier $\phi_o\omega$ in Eq. 11 can vary from a minimum of 1.39 x 1.3 = 1.81 to much higher values. By comparison the approach of the seismic appendix of the ACI code [9] is to require the sum of the moments at the centre of the joint corresponding to the design flexural strength of the columns at the joint to be at least equal to 6/5 times the sum of the moments at the centre of the joint corresponding to the design flexural strength of the beams at the joint. If a strength reduction factor ϕ of 0.75 is assumed for the columns and 0.9 for the beams, this requirement involves a multiplier of 1.2 x 0.9/0.75 = 1.44. Note that in the New Zealand approach $\phi = 1.0$ is assumed for the columns and the effect of $\phi = 0.9$ for the beams is already included in $\phi_o\omega$, and hence the ACI multiplier of 1.44 can be compared directly with the New Zealand multiplier of 1.81 or greater.

4.4.2 Potential Plastic Hinge Regions

The length of the potential plastic hinge regions in the ends of columns to be confined is dependent on the magnitude of the ratio of $P_e/\phi f'_c A_g$, where P_e = column load in compression due to the design gravity and seismic loading, ϕ = strength reduction factor = 0.9 for confined members or 1 for confined members protected by capacity design, f'_c = concrete compressive cylinder strength, and A_g = gross area of column cross section. When $P_e \leqq 0.3\phi f'_c A_g$, the potential plastic hinge region is taken as the longer column section dimension in the case of a rectangular section or the diameter in the case of a circular section, or where the moment exceeds 0.8 of the maximum moment at that end of the member, which ever is larger. When $P_e > 0.3\phi f'_c A_g$, the potential plastic hinge region is increased by 50%. This increase is because tests in New Zealand have shown that at high axial load levels the plastic hinge region tends to spread along the column because the flexural strength at the critical section is enhanced by the larger confining steel content. Thus flexural failure could occur in the less heavily confined adjacent regions of columns unless the heavy confining steel is spread over a longer length of the column.

4.4.3 Longitudinal Reinforcement

The area of longitudinal reinforcement should not be less than $0.008A_g$, nor greater than $0.06A_g$ for Grade 275 ($f_y \geqq 40$ ksi) steel or $0.045A_g$ for Grade 380 ($f_y \geqq 55$ ksi) steel, where A_g = gross area of column cross section. In potential plastic hinge zones the longitudinal bars should not be spaced further apart than 200 mm (7.9 in) between centres.

In a column the centre of a splice is required to be within the middle quarter of the column height, unless it can be shown that plastic hinging cannot develop at the column end.

Tests in New Zealand [11] have shown that lap splices should not be placed in plastic hinge regions of columns. To develop the lap in a plastic hinge region subjected to cyclic flexure requires a large amount of transverse reinforcement. If sufficient transverse reinforcement is available to transfer the longitudinal bar force satisfactorily, it was found that there is an undesirable concentration of curvature at the critical section which could lead to low cycle fatigue of the longitudinal steel there. The restricted length of plastic hinge occurs because yielding concentrates in only a small length of bar near the end of the lap at the end of the member. Therefore lap splices should be positioned away from the plastic hinge regions.

4.4.4 Transverse Reinforcement to Prevent Premature Buckling of Longitudinal Reinforcement and to Confine Compressed Concrete

(a) Spacing of Transverse Reinforcement in Potential Plastic Hinge Regions of Columns

For both circular and rectangular shaped transverse reinforcement, the centre to centre spacing in the longitudinal direction of the column in potential plastic hinge regions is required not to exceed the smaller of one-fifth of the least lateral dimension of the cross section, six longitudinal bar diameters, or 200 mm (7.9 in).

For rectangular hoops the centre to centre spacing between hoop legs or supplementary cross ties across the section should not exceed 200 mm (7.9 in). Also, longitudinal bars should not be spaced further apart between centres than 200 mm (7.9 in).

For rectangular hoops, each longitudinal bar or bundle of bars should be laterally supported by a corner of a hoop having an included angle of not more than 135° or by a cross tie, except that longitudinal bars are exempted from this requirement if either:

(1) they lie between two bars supported by the same hoop where the distance between the laterally supported bars does not exceed 200 mm (7.9 in) between centres, or
(2) they lie within the concrete core of the section centred more than 75 mm (3.0 in) from the inner face of the hoop.

In addition, for rectangular hoops the yield force in the hoop bar or cross tie shall be at least one-sixteenth of the yield force of the longitudinal bar or bars it is to laterally restrain, including the contribution by any bar or bars excempted above from the direct support.

With regard to the limits on longitudinal spacing of transverse reinforcement, the one-fifth of the least column lateral dimension requirement is to ensure that the bars are close enough to effectively confine the compressed concrete and the six longitudinal bar diameter requirement is to ensure that premature buckling of compressed longitudinal reinforcing bars does not occur. The ACI code [9] requires the spacing of transverse reinforcement not to exceed one-quarter of the minimum member dimension or 4 in (102 mm).

Fig. 19, taken from the Commentary of the New Zealand code [5], illustrates typical arrangements of transverse reinforcement in potential plastic hinge regions of rectangular columns.

(b) <u>Anchorage of Transverse Reinforcement in Potential Plastic Hinge Regions of Columns</u>

It is required that transverse steel should be anchored by at least a 135° hook around a longitudinal bar plus an extension of at least eight transverse bar diameters at the free end of the bar into the core concrete of the member. Alternatively, the ends of the bars should be welded.

Note that the loss of concrete cover in the plastic hinge region, as a result of the cover concrete spalling during plastic hinge rotation, means that the transverse steel must be carefully detailed. It is inadequate to lap splice a spiral or a circular hoop in the cover concrete. If the cover concrete spalls the spiral or circular hoop will be able to unwind (as was observed in some bridge piers in the 1971 San Fernando earthquake [2]). Therefore if the transverse bars are lapped in the cover concrete full strength welds are required at the laps. Similarly, rectangular hoops and cross ties should be anchored either by bending the ends back into the core of the column or by welding.

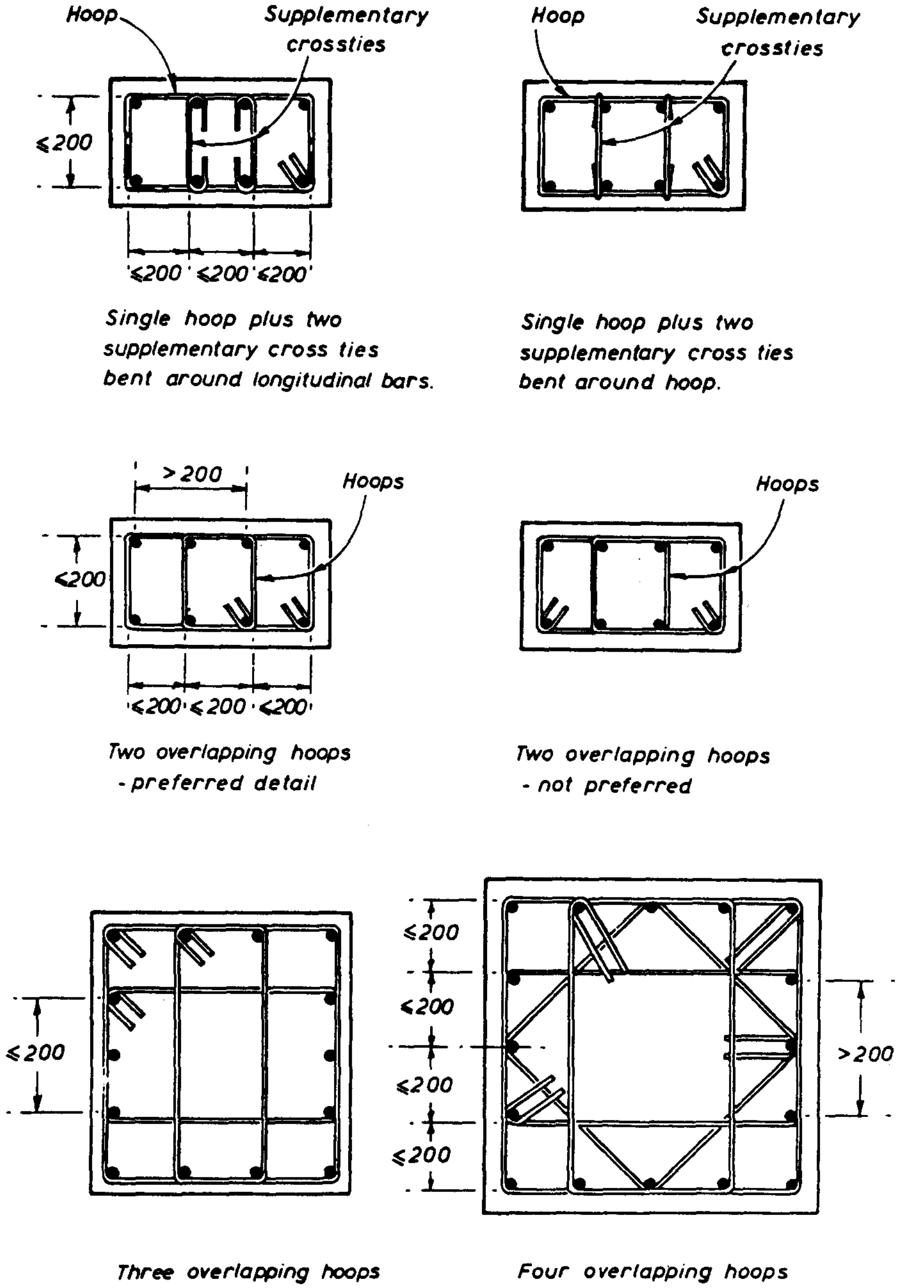


Figure 19 : Examples of Transverse Reinforcement in Columns According to Commentary of New Zealand Code [5]. (Dimensions in mm, 1 mm = 0.039 in).

Examples of anchorage details for cross ties taken from the Commentary of the ACI code [9] and the SEAOC recommendations [12] are shown in Figs. 20 and 21. A cross tie with a 90° bend at one end and a 135° bend at the other as shown in Fig. 20 has the construction advantage that it can be inserted into the reinforcing cage from the side when placing the steel. However it is considered in the New Zealand code that the 90° bend is undesirable since the extension of bar beyond the 90° bend is not embedded in the concrete core. Thus when the cover concrete spalls the 90° bend anchorage may become ineffective. One column recently tested in New Zealand [13] has shown that the 90° bend anchorage does eventually become ineffective at large column deformations, but that this detail may be satisfactory for limited ductility demands, for example for use in columns protected from plastic hinging by a capacity design procedure. Two columns recently tested in New Zealand [13] have indicated that the tension splice detail for cross ties shown in Fig. 21 can be satisfactory providing sufficient lap length is provided for adequate anchorage.

(c) Quantity of Transverse Reinforcement Required for Concrete Confinement in Potential Plastic Hinge Regions of Columns

The quantity of transverse reinforcement of circular shape for confinement is expressed in terms of ρ_s, which is the ratio of the volume of spiral or circular hoop reinforcement to the volume of the concrete core. Therefore

$$\rho_s = \frac{A_{sp}\pi d_s}{s\pi (d_s/2)^2} = \frac{4A_{sp}}{d_s s} \qquad (14)$$

where A_{sp} = area of spiral bar, d_s = diameter of spiral, and s = centre to centre spacing of spiral bar. For transverse reinforcement of rectangular shape for confinement with or without cross ties, the

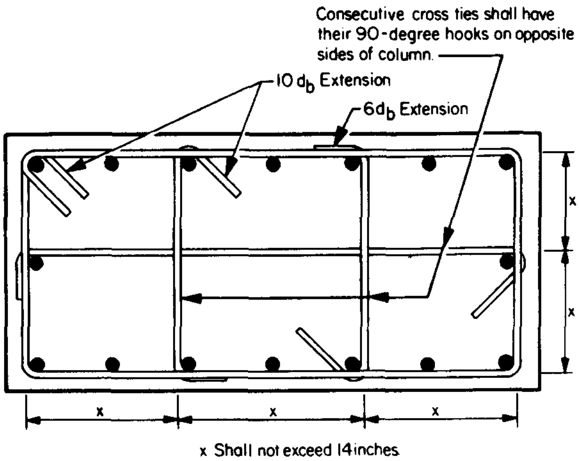


Figure 20 : Examples of Transverse Reinforcement in Columns According to the Commentary of the ACI Code [9].

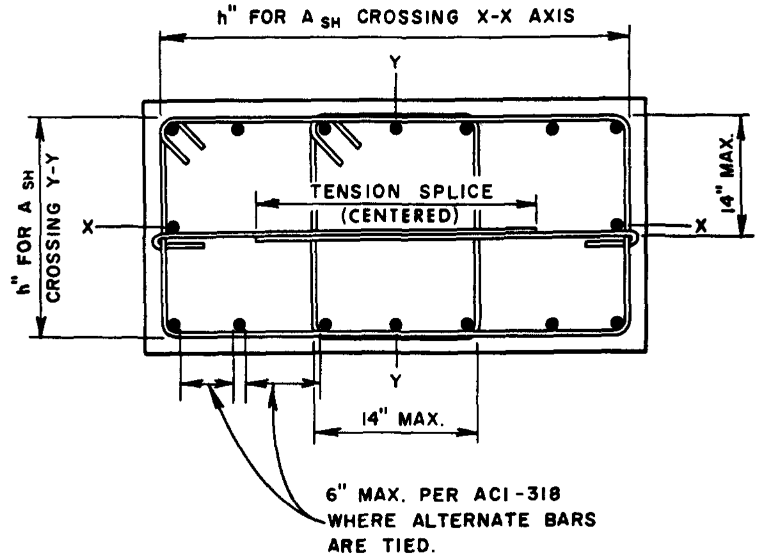


Figure 21 : Example of Transverse Reinforcement in Columns According to Commentary of SEAOC Recommendations [12].

quantity of transverse reinforcement is generally expressed in terms of A_{sh}, which is the total area of transverse reinforcement, including cross ties, in the direction under consideration within longitudinal spacing s_h.

The quantity of reinforcement specified is intended to ensure adequate ductility at the potential plastic hinge regions in the columns, in the event of plastic hinging occurring, for the level of seismic design force used. In potential plastic hinge regions when spirals or circular hoops are used the volumetric ratio ρ_s should not be less than

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right) \tag{15}$$

or

$$\rho_s = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right) \tag{16}$$

whichever is greater, where A_g = gross area of column cross section (mm^2), A_c = area of concrete core of section measured to outside of peripheral transverse steel (mm^2), f_{yh} = yield strength of transverse reinforcement (MPa), f'_c = concrete compressive cylinder strength (MPa), P_e = axial load on column (N), and ϕ = strength reduction factor = 0.9 if plastic hinging can occur or 1.0 if the column is protected from plastic hinging by a capacity design procedure. In potential plastic hinge regions when rectangular hoops with or without cross ties are used, the total area of transverse bars A_{sh} in each of the transverse directions within spacing

s_h should not be less than

$$A_{sh} = 0.3 s_h h'' \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_{yh}} \left(0.5 + 1.25 \frac{P_e}{\phi f'_c A_g}\right) \quad (17)$$

or

$$A_{sh} = 0.12 s_h h'' \frac{f'_c}{f_{yh}} \left(0.5 + 1.25 \frac{P_e}{\phi f'_c A_g}\right) \quad (18)$$

whichever is greater, where h" = dimension of concrete core of the section measured perpendicular to the direction of the hoop bars to the outside of the perimeter hoop (mm), s_h = centre to centre spacing of hoop sets (mm), and the other notation is as for Eqs. 15 and 16 (where 1 N = 0.225 lb, 1 mm = 0.039 in and 1 MPa = 145 psi).

In frames where a capacity design procedure is used to provide protection from plastic hinging the required quantity of transverse reinforcement in the potential plastic hinge regions may be one-half of that required by Eqs. 15 to 18, but the previous spacing and anchorage requirements should be maintained. This reduction in the quantity of transverse reinforcement is not permitted at the base of columns of frames nor in columns forming part of frames designed using a weak column-strong beam concept.

(d) United States Provisions for Quantity of Transverse Reinforcement Required in Potential Plastic Hinge Regions of Columns

In the ACI code [9] and in the SEAOC recommendations [12] it is specified that in the potential plastic hinge regions when spiral reinforcement is used ρ_s should not be less than

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_{yh}} \quad (19)$$

or

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \quad (20)$$

whichever is greater. In the potential plastic hinge regions where rectangular hoops with or without cross ties are used A_{sh} should not be less than

$$A_{sh} = 0.3 s_h h'' \frac{f'_c}{f_{yh}} \left(\frac{A_g}{A_c} - 1\right) \quad (21)$$

or

$$A_{sh} = 0.12 s_h h'' \frac{f'_c}{f_{yh}} \quad (22)$$

whichever is greater. The notation used in Eqs. 19 to 22 is the same as for Eqs. 15 to 18, except that in the ACI code h" is the dimension of the concrete core measured to the centres of the perimeter hoop.

(e) Background of New Zealand and United States Provisions for Quantity of Transverse Reinforcement Required for Concrete Confinement in Potential Plastic Hinge Regions of Columns

It is evident that the New Zealand Eqs. 15 to 18 are similar to the United States Eqs. 19 to 22 except for the term $[0.5 + 1.25P_e/(\phi f'_c A_g)]$. The reason for this difference is that the US equations are based on a philosophy of preverving the axial load carrying capacity of the column after spalling of the cover concrete has occurred rather than aiming to achieve a particular curvature ductility factor for the section [3]. The philosophy of maintaining the axial load strength of the section after spalling of the cover concrete does not properly relate to the detailing requirements of adequate plastic rotation capacity of eccentrically loaded members. A more logical approach for the determination of the amount of transverse reinforcement necessary to achieve adequate curvature ductility would be based on ensuring a satisfactory moment-curvature relationship.

Theoretical moment-curvature analyses have been conducted in New Zealand by Park et al (3,14,15] to determine the influence of quantity of confining steel on the available curvature ductility. The idealised stress-strain curve for confined concrete used in these analyses were based on a limited number of tests on small concentrically loaded column specimens and the curves used are now known to be conservative. The complete stress-strain curve for confined concrete was used in the analyses. That is, the full extent of the "falling branch" of the stress-strain curve after the maximum stress was reached was utilised and hence no arbitrary value for the "ultimate" concrete compressive strain was assumed. Instead, the available curvature ductility factor of the section was judged by assessing the curvature after maximum moment was reached when the section was still carrying a reasonable proportion of the maximum moment. Fig. 22 shows diagramatically the form of the moment-curvature curves obtained for a particular column section with different transverse steel contents and a

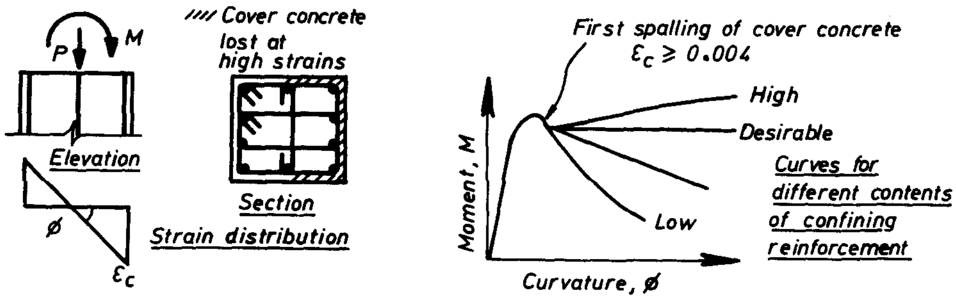


Figure 22 : Moment-Curvature Relations for a Reinforced Concrete Column Section With Constant Axial Load P and Different Contents of Confining Reinforcement.

constant axial load level. The theoretical moment-curvature relations for a range of transverse steel contents and axial load levels were computed. The analyses showed that the quantity of transverse steel specified by the ACI and SEAOC equations are conservative for low axial load levels but are unconservative for high axial load levels. Hence Eqs. 15 to 18 are based on the ACI and SEAOC equations but with a modification factor $[0.5 + 1.25P_e/(\phi f'_c A_g)]$ introduced to account for the effect of axial load level.

The amount of transverse reinforcement required by Eqs. 15 to 18 increases with the axial load level because a higher axial load means a larger neutral axis depth which in turn means that the flexural strength of the column is more dependent on the contribution of the concrete compressive stress block. Thus the higher the axial load the more important it becomes to maintain the strength and ductility of the concrete, thus leading to a greater quantity of transverse steel.

The behaviour of reinforced concrete columns subjected to simulated seismic loading has been studied extensively at the University of Canterbury in recent years. The test results from a wide range of columns have indicated that the New Zealand equations for the quantity of confining reinforcement results in columns with very satisfactory available ductility. A summary of the test results may be seen elsewhere [16].

(f) Examples of Comparisons of the Quantity of Confining Steel Required in Potential Plastic Hinge Regions of Columns by New Zealand and United States Codes

The difference between the provisions for transverse reinforcement of the New Zealand code [5] and the ACI and SEAOC codes [9,12] illustrated in Fig. 23 for a typical 1.5 m (59.1 in) diameter circular column confined by a spiral, and in Fig. 24 for a typical 700 mm (27.6 in) square column confined by an arrangement of square and octagonal hoops.

It is evident from Figs. 23 and 24 that the ACI and SEAOC provisions stipulate a constant quantity of transverse steel, regardless of the level of axial compressive load on the column. It should be noted however that for column compressive loads less than $0.1f'_c A_g$ the member could be defined as a beam according to the ACI code and then contain the reduced quantity of transverse steel specified for a beam. The New Zealand code quantity of transverse steel shows a linear increase with axial load from 50% of the ACI/SEAOC quantities at zero axial load, to 100% of the ACI/SEAOC amount at an axial load of $0.36f'_c A_g$, to 1.47 times the ACI/SEAOC amount at an axial load of $0.7f'_c A_g$, assuming $\phi = 0.9$ in the New Zealand code equations.

For the circular column of Fig. 23, the quantity of spiral steel required by the New Zealand code at an axial load level of $0.36f'_c A_g$, and by the ACI and SEAOC codes at all load levels, could be met using a spiral from 20 mm (0.79 in) diameter bar at 74 mm (2.9 in) centre to centre spacing. If a higher strength spiral steel was used the spiral bar diameter could be reduced or the spacing increased or both. For example, if f_{yh} = 60 ksi (414 MPa) spiral steel is used, a 3/4 in (19.1 mm)

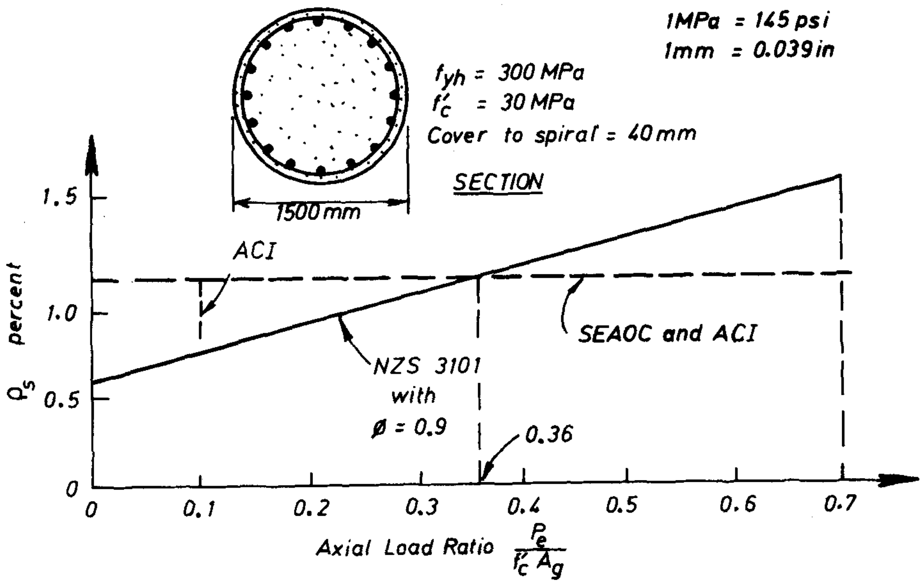


Figure 23 : Comparison of United States and New Zealand Code Spiral Steel Requirements for a Circular Column.

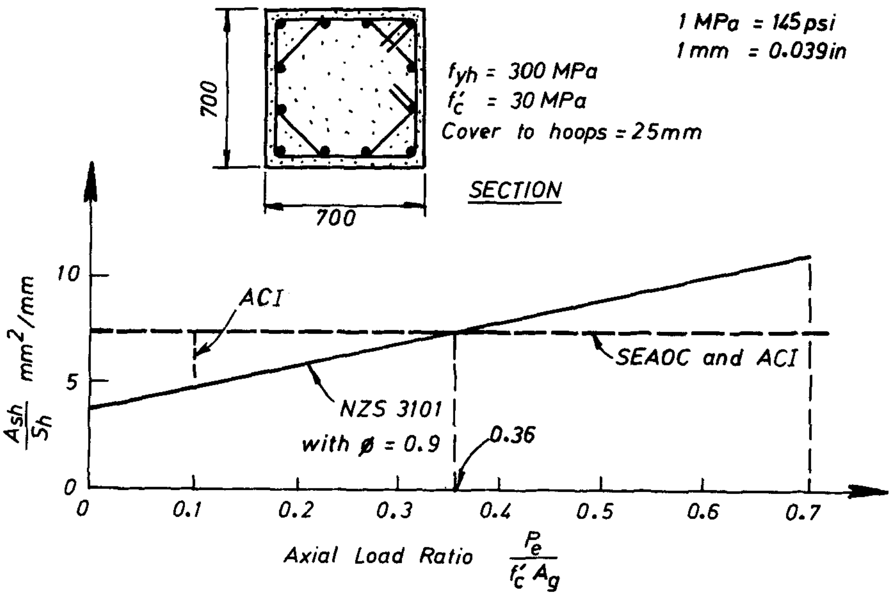


Figure 24 : Comparison of United States and New Zealand Code Hoop Steel Requirements for a Square Column.

diameter spiral bar at 3.7 in (93 mm) centre to centre spacing would be sufficient. It is obvious that large diameter columns require large diameter spiral steel bars at close centres for confinement.

For the square column of Fig. 24 the quantity of hoop steel required by the New Zealand code at an axial load level of $0.36f'_cA_g$, and by the ACI and SEAOC codes at all load levels, could be met using hoops from 16 mm (0.63 in) diameter bar with the hoop sets placed at 88 mm (3.5 in) centres. Again the quantity of transverse steel used could be reduced by using higher strength steel. For example, if f_{yh} = 60 ksi (414 MPa) hoop steel is used, a 5/8 in (15.9 mm) diameter hoop bar with hoop sets at 4.7 in (120 mm) centre to centre spacing would be sufficient.

Figs. 25 and 26 show theoretical moment-curvature relations derived for the columns of Figs. 23 and 24 for three different axial load levels, namely 0, $0.36f'_cA_g$ and $0.72f'_cA_g$. The stress-strain relation for the longitudinal steel used in the analysis included the effect of strain hardening. The stress-strain relation for confined concrete used was that due to Mander et al [17]. It is of interest that at the high axial load level of $0.72f'_cA_g$ the curve obtained for the ACI/SEAOC amount of confining steel shows a significant reduction in moment at curvatures beyond maximum moment (that is, relatively poor ductility), while the curve obtained for the New Zealand amount of confining steel shows more ductility. At zero axial load the curves obtained for the ACI/SEAOC and NZ amounts of confining steel are almost identical and show good ductility, indicating that the smaller NZ amount of confining steel is adequate.

(g) Examples of Comparisons of the Quantity of Confining Steel Required in Potential Plastic Hinge Regions of Circular and Square Column Sections

It is of interest to compare the volumes of transverse steel required for the three column sections shown in Fig. 27. Each section has twelve longitudinal bars. The circular column has spiral reinforcement. The square columns have two alternative arrangements of hoops. Case 1 has three overlapping hoops per set, made up of one square perimeter hoop surrounding all twelve bars and two interior rectangular hoops each surrounding four bars. Case 2 has two overlapping hoops per set, made up of one square perimeter hoop surrounding all twelve bars and one octagonal interior hoop surrounding eight bars.

The relative sizes of the circular and square cross sections are such that the nominal (ideal) flexural strengths of the three columns are approximately the same if the columns all have Grade 380 (f_y = 55 ksi) longitudinal reinforcement with $\rho_t = A_{st}/A_g = 0.02$, concrete with f'_c = 30 MPa (4,350 psi) and an axial load level of $0.3f'_cA_g$, where A_{st} = total area of longitudinal reinforcement, A_g = gross area of column section, and f'_c = concrete compressive cylinder strength. This equality of flexural strength may be demonstrated as follows. In both columns the distance between the bar centroids in extreme faces of the column is close to 0.8 of the overall column depth (within 2% of this proportion), and $\rho_t f_y/0.85f'_c = 0.02 \times 380/(0.85 \times 30) = 0.30$. If the strength reduction factor ϕ is 1.0, the nominal (ideal) flexural strength is 1690 kNm (1246 kip ft) for the circular column and 1730 kNm (1275 kip ft) for the

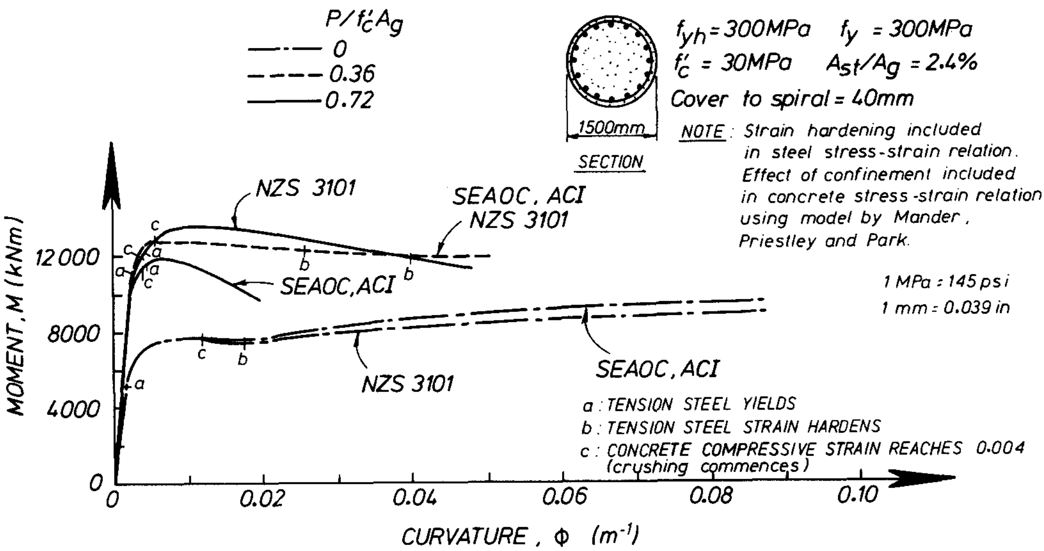


Figure 25 : Theoretical Moment-Curvature Relations for a Circular Column.

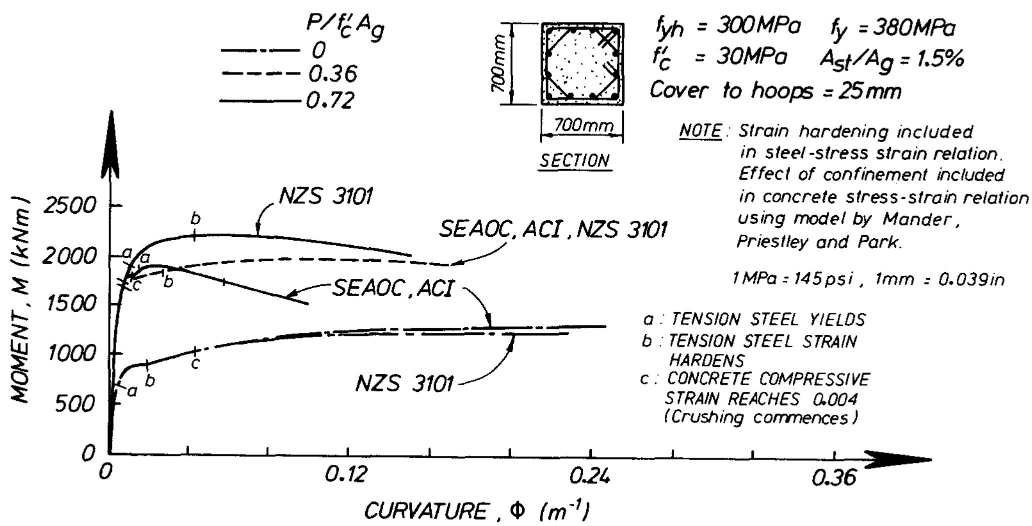


Figure 26 : Theoretical Moment-Curvature Relations for a Square Column.

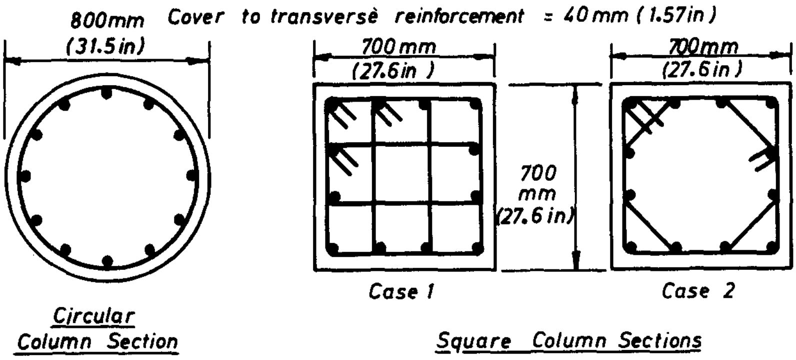


Figure 27 : Cross Sections of Example Column.

square column, as may be obtained from design charts, tables or by calculation from first principles.

For the same axial load level $P_e/\phi f'_c A_g$ on all three columns it can be shown using Eqs. 15 to 18 to calculate the quantities of transverse reinforcement that:

1. The volume of overlapping hoops in the square column is 2.0 to 2.2 times the volume of the spiral in the circular column.

2. The square column with sets of overlapping square and octagonal hoops (Case 2) requires 11% less transverse reinforcement volume than the square column with sets of overlapping square and rectangular hoops (Case 1).

3. For the same transverse bar diameter, the clear spacing between transverse bars is greatest in the circular column with the spiral. For example, if $P_e/\phi f'_c A_g = 0.4$ and if 16 mm (0.63 in) diameter bar from Grade 380 steel (f_{yh} = 55 ksi) is used for transverse reinforcement, the centre to centre spacing of the spirals or hoop sets is as follows from Eqs. 15 to 18.

   Circular Column:

   s = 118 mm (4.6 in)

   Clear spacing between spirals = 118 - 16 = 102 mm (4.0 in)

   Square Column, Case 1 (three overlapping hoops):

   s_h = 137 mm (5.4 in)

   Clear spacing between hoop sets = 137 - 48 = 89 mm (3.5 in)

   Square Column, Case 2 (two overlapping hoops):

$s_h = 117$ mm (4.6 in)

Clear spacing between hoop sets = 117 - 32 = 85 mm (3.3 in).

Note that this example points to the considerable reduction in the volume of transverse reinforcement possible according to the code if circular columns with spirals are used. The circular and the square columns had approximately the same concrete cross sectional area and for the same longitudinal steel content gave the same flexural strength at $P_e = 0.3f'_cA_g$.

4.4.5 Transverse Reinforcement for Shear

The design shear force is estimated from a probable critical moment gradient along the column. According to the method for the evaluation of column actions in ductile multistorey frames due to Paulay [6], placed in the Commentary of the code [5], the design shear force can be taken for columns of one-way frames as

$$V_{col} = 1.3\phi_o V_{code} \tag{23}$$

and for columns of two-way frames as

$$V_{col} = 1.6\phi_o V_{code} \tag{24}$$

where ϕ_o = beam overstrength factor defined as for Eq. 11, and V_{code} = column shear force derived for code static seismic loading. The larger value for two-way frames is to include the effect of concurrent seismic loading. That is, the design shear forces need only to be considered in each principal direction independently.

Since the design shear forces are found using flexural overstrengths, a strength reduction factor $\phi = 1$ can be used in the design of shear reinforcement. In the potential plastic hinge regions at the ends of columns the shear stress carried by the concrete v_c is assumed to be zero unless the minimum design axial compression force produces an average stress in excess of $0.1f'_c$ over the gross concrete area. When the average compression stress exceeds $0.1f'_c$ the value of v_c is taken as follows:

$$v_c = 4v_b\sqrt{\frac{P_e}{f'_cA_g} - 0.1} \quad \text{(MPa)} \tag{25}$$

where v_b = shear stress carried by the concrete when $P_e = 0$ in gravity load design (MPa), P_e = minimum design compressive force (N), f'_c = concrete compressive cylinder strength (MPa), and A_g = gross area of column section (mm^2) (where 1 N = 0.225 lb, 1 mm = 0.039 in and 1 MPa = 145 psi). This equation gives a gradual transition from $v_c = 0$ to the full gravity load design value of v_c as P_e/f'_cA_g increases from 0.1.

The assumption of $v_c = 0$ for small axial load levels is to take into account the possible deterioration of the shear carried by the concrete during high intensity cyclic loading. Reversal of moment in plastic hinge regions causes a reduction in the shear transferred by the concrete across the compression zone and in the shear force carried by aggregate interlock and dowel action [3].

The transverse reinforcement present should be capable of carrying by truss action that shear not carried by the concrete. The requirements of shear may govern the quantity of transverse reinforcement necessary in columns.

Away from the potential plastic hinge regions the shear may be considered to be carried by both the transverse reinforcement and the concrete, and the shear reinforcement may be designed by the shear strength equations used for gravity load design.

Column shear is an important design consideration since column shear failure can be extremely brittle. Note that the shear strength of a square column with load acting along a diagonal of the section and the shear strength of the same column with loading acting along a major axis of the section are almost identical. This is because for diagonal shear, although the component of transverse bar force in the direction of the shear force is smaller, the diagonal tension crack at 45° to the longitudinal axis of the member has a greater projected length and therefore intercepts more transverse bars.

4.5 Design of Beam-Column Joints

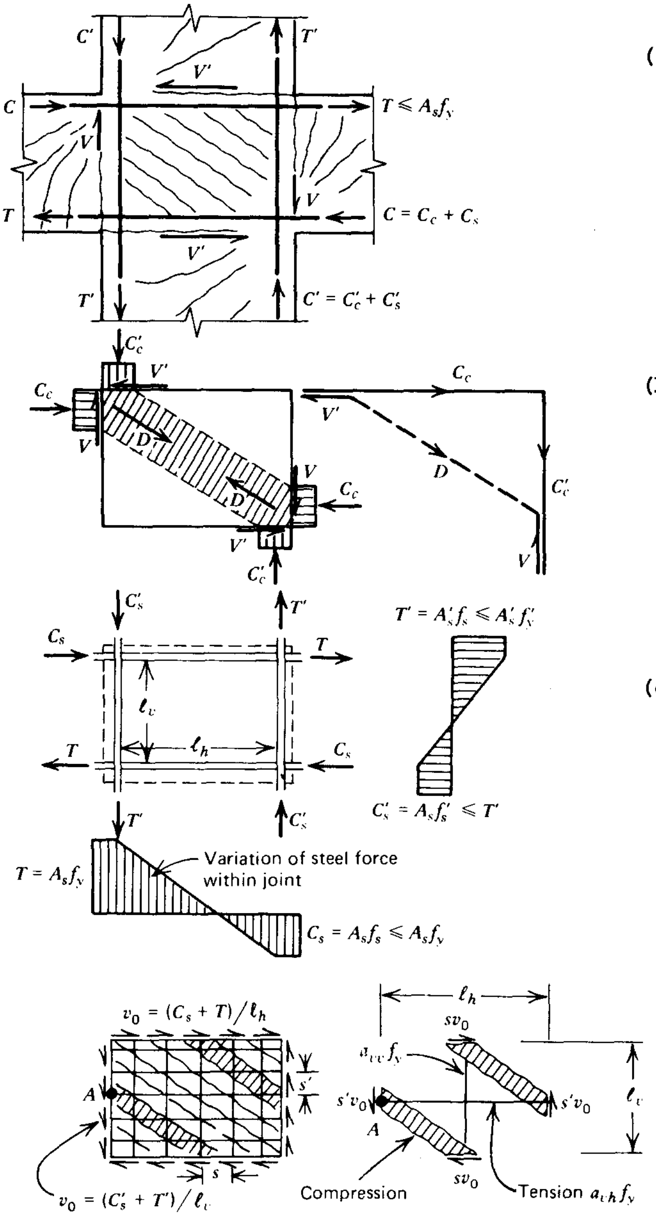
4.5.1 General

The strength of a beam-column joint core should be greater than the strength of the members it joins, since the joint core strength may degrade rapidly with cyclic loading, the joint core is difficult to repair, and failure of the joint core could lead to collapse of the column.

In past years designers have tended not to give much attention to the detailing of beam-column joints. However very few failures of beam-column joint cores have been observed during severe earthquakes. Collapses of frames during severe earthquakes have normally occurred as a result of inadequately detailed columns, particularly poor arrangements of transverse reinforcement. However beam-column joint cores can be subjected to extremely high shear and bond stresses when subjected to seismic loading. If the beams and columns are detailed for adequate ductility the joint cores could become the critical regions of the structure unless also carefully designed. The behaviour of beam-column joint cores has been studied extensively in New Zealand in recent years. A summary of the results of that work may be seen elsewhere [18].

4.5.2 Shear Resistance

Fig. 28 illustrates an interior beam-column joint core which forms part of a frame subjected to horizontal seismic loading. Consideration of the concrete and steel forces in the adjacent beams and columns acting at the boundaries of the joint core indicates that to satisfy the equilibrium requirements of the joint core there must be two mechanisms of joint core shear resistance [3], namely:



(a) Internal actions and crack patterns

(b) Shear transfer of concrete compression forces by diagonal compression strut mechanism

(c) Forces in the reinforcement only

(d) Shear transfer of reinforcement bond forces by truss mechanism

Figure 28 : Idealised Behaviour of Reinforced Concrete Beam-Column Joint of Frame Subjected to Horizontal Loading [3].

(1) a diagonal compression strut carrying the concrete compressive forces across the joint.
(2) a truss mechanism of joint core reinforcement carrying the longitudinal bar forces across the joint.

The diagonal compression strut mechanism transfers mainly the forces from the concrete compression zones of the adjacent beams and columns across the joint core. The truss mechanism is necessary to transfer the bond forces from the longitudinal beam and column reinforcement, which act along the boundaries of the joint core, across the joint core. The first mechanism is commonly referred to as the "shear carried by the concrete", and the second mechanism as the "shear carried by the shear reinforcement".

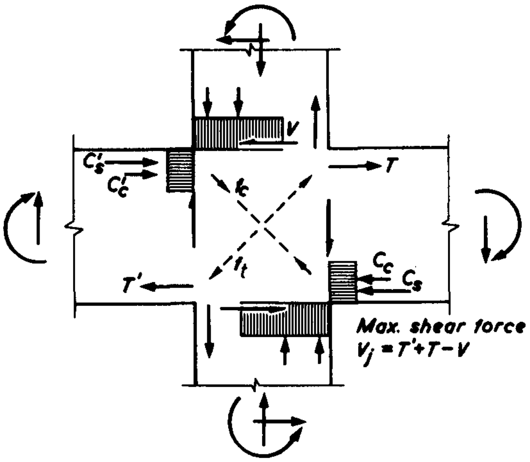
During reversed seismic loading full depth vertical cracks can occur in the beams at the column faces, as illustrated in Fig. 16. When full depth cracking occurs in the beams at the column faces the diagonal compression strut mechanism becomes much less effective, unless the axial compression on the column is large. Diagonal tension cracking in alternating directions in the joint core can also cause a degradation of the strength of the diagonal compression strut mechanism. Hence cyclic loading causes a transfer of the joint core shear resistance from the diagonal compression strut mechanism to the truss mechanism. The potential failure plane is a corner to corner crack in the joint core.

It is evident that the truss mechanism requires the presence of both horizontal and vertical shear reinforcement and a diagonal concrete compression field to satisfy the equilibrium requirements of the mechanism (see Fig. 28d). The horizontal and vertical reinforcement necessary for shear resistance can be provided by horizontal column hoops between the top and bottom longitudinal beam bars and longitudinal column bars between the corner longitudinal column bars.

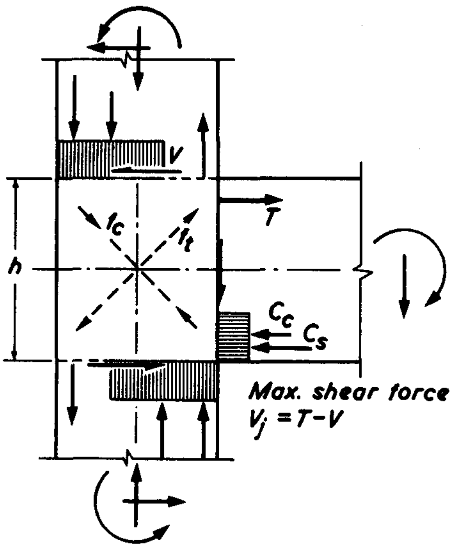
4.5.3 Design Shear Forces

In order to ensure that energy dissipation occurs in the plastic hinge regions of the adjacent members and not in the joint core regions, the joint core should be designed to resist the forces arising when the overstrength of the framing members is developed (Section 4.2.1). That is, the stresses in the flexural steel at the plastic hinges are assumed to be 1.25 times the yield strength in the case of Grade 275 ($f_y \geqq 40$ ksi) steel, or 1.4 times the yield strength in the case of Grade 380 ($f_y \geqq 55$ ksi) steel. The design horizontal shear force V_{jh} and the design vertical shear force V_{jv} are found by taking into account the effect of the normal and shear forces acting on the joint core. When beams frame into the joint in two directions, these forces need only be considered in each principal direction independently.

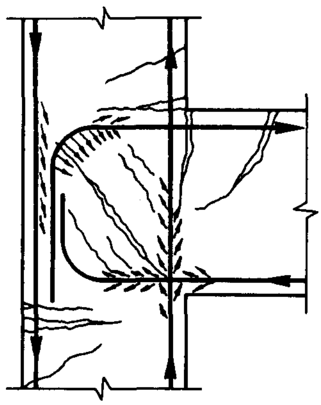
The forces acting at the boundaries of joint cores of exterior and interior reinforced concrete beam-column joints are shown in Fig. 29. The horizontal shear force acting on the joint core is given by the resultant horizontal force acting above or below a horizontal plane at the mid-depth of the joint core. For the exterior joint the horizontal design shear force is



(a) Actions and stress resultants of interior joint



(b) Actions and stress resultants of exterior joint



(c) Cracking and bond forces of exterior joint

Figure 29 : Reinforced Concrete Beam-Column Joints of Frame Subjected to Horizontal Loading.

$$V_{jh} = T - V \tag{26}$$

and for the interior joint the horizontal design shear force is

$$V_{jh} = C'_s + C'_c + T - V \qquad \text{where} \quad C'_s + C'_c = T'$$

$$= T' + T - V \tag{27}$$

where the notation is as shown in Fig. 29. The vertical design shear force may also be found from the forces acting on the boundaries of the joint core. As an approximation the vertical design shear force may be taken as

$$V_{jv} = V_{jh}\frac{h_b}{h_c} \tag{28}$$

where h_b = overall depth of beam and h_c = overall depth of column in the direction being considered.

In order to prevent the concrete diagonal compression strut from crushing, the nominal horizontal shear stress v_{jh} in either principal direction is limited to $1.5\sqrt{f'_c}$ MPa (or $18\sqrt{f'_c}$ psi) where

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \tag{29}$$

and where f'_c = concrete compressive strength, h_c = overall depth of column in the direction being considered and b_j is the effective joint width defined as

When $b_c > b_w$

either $b_j = b_c$ or $b_j = b_w + 0.5h_c$, whichever is smaller.

When $b_c < b_w$

either $b_j = b_w$ or $b_j = b_c + 0.5h_c$, whichever is smaller.

where b_c = width of column and b_w = width of beam web.

4.5.4 <u>Horizontal Joint Shear Reinforcement</u>

The total area of horizontal shear reinforcement placed between the outermost layers of top and bottom beam reinforcement should not be less than

$$A_{jh} = \frac{V_{jh} - V_{ch}}{f_{yh}} \tag{30}$$

where V_{jh} = horizontal design shear force, f_{yh} = yield strength of horizontal shear reinforcement, and V_{ch} = shear carried by diagonal compression strut mechanism should be taken as zero unless one of the following applies:

(1) When the minimum average compressive stress on the gross concrete area of the column above the joint exceeds $0.1f'_c/C_j$

$$V_{ch} = \frac{2}{3}\sqrt{\frac{C_j P_e}{A_g} - \frac{f'_c}{10}}\,(b_j h_c) \quad (N) \qquad (31)$$

where $C_j = V_{jh}/(V_{jx} + V_{jy})$ where V_{jx}, V_{jy} = horizontal design shear forces in joint core in the two principal directions (= 1 for one-way frame or 0.5 for symmetrical two-way frame), P_e = minimum axial compressive column load (N), A_g = gross area of column cross section (mm^2) and f'_c = concrete compressive cylinder strength (MPa) (where 1 N = 0.225 lb, 1 mm = 0.039 in and 1 MPa = 145 psi). The value for V_{ch} given by Eq. 31 recognises the degradation in the shear carried by the concrete diagonal compression strut during reversals of seismic loading, but acknowledges that an increase in the axial compressive load on the column results in an increase in V_{ch} due to the formation of a wider compression strut.

(2) The degradation of shear carried by the concrete diagonal compression strut during reversals of seismic loading can be greatly reduced if yielding of the longitudinal steel is forced to occur away from the faces of the joint core, since then full depth cracking will not occur at the faces of the joint core and the diagonal compression strut mechanism will be preserved. Thus an attractive design concept involves deliberately designing plastic hinges to form in the beams away from the columns. The code allows a reduction in the content of joint core shear reinforcement for this design situation. Plastic hinging can be forced away from the column faces by suitable reinforcing details or by haunching the beams, as shown in Fig. 30. For exterior joints the degradation in the shear carried by the diagonal compression strut mechanism during seismic load reversal is not so great as for interior joints. This is because it is possible for the diagonal compression strut to act between the

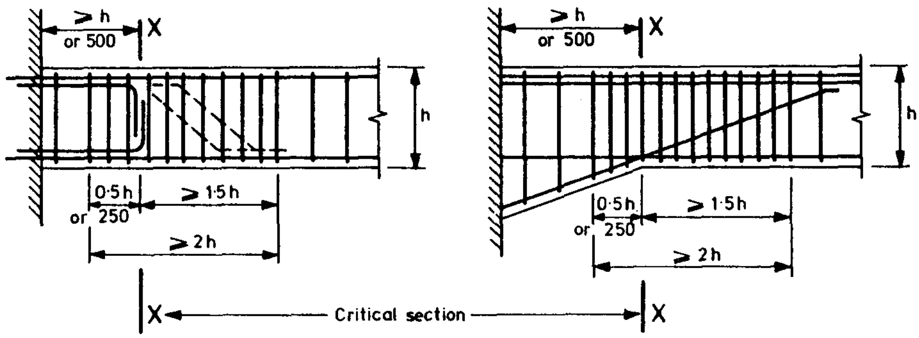


Figure 30 : Plastic Hinges Located in Beams Away from Column Faces [5].

the bend in the 90° hook of the flexural reinforcement and the hoops outside the joint core in the column placed near the beam bars (see Fig. 29). Hence even when full depth flexural cracks due to yielding beam steel exist in the beam at the column face V_{ch} has a significant value for exterior joints.

Hence when the design is such that plastic hinging occurs in the beam at a distance away from the column face not less than the beam depth nor 500 mm (19.7 in), or for exterior joints where the flexural steel is anchored outside the column core in a beam stub, the value of V_{ch} may be increased to

$$V_{ch} = 0.5 \frac{A'_s}{A_s} V_{jh} \left(1 + \frac{C_j P_e}{0.4 A_g f'_c}\right) \quad \text{(N)} \tag{32}$$

where A'_s = area of compression reinforcement in beam (mm^2) and A_s = area of tension reinforcement in beam (mm^2) and other notation is as for Eq. 31. A'_s/A_s should not be taken larger than 1.0. When the axial column load results in tensile stresses over the gross concrete area exceeding $0.2f'_c$, $V_{ch} = 0$. For axial tension between these limits V_{ch} may be obtained by linear interpolation between zero and the value given by Eq. 32 when P_e is taken as zero.

(3) For exterior joints without beam stubs at the far face of column, Eq. 32 may be used when multiplied by the factor

$$\frac{3h_c (A_{jv} \text{ provided})}{4h_b (A_{jv} \text{ required})} \tag{33}$$

which should not be taken as greater than 1.0. Use of this factor requires that the beam bars be anchored using a 90° standard hook in the joint core.

(4) When the ratio h_c/h_b is greater than or equal to 2.0, V_{ch} need not be taken as less than

$$V_{ch} = 0.2 b_j h_c \sqrt{f'_c} \quad \text{(N)} \tag{34}$$

4.5.5 Vertical Joint Shear Reinforcement

The total area of vertical shear reinforcement, normally in the form of intermediate column bars at the side faces of the column, should not be less than

$$A_{jv} = \frac{V_{jv} - V_{cv}}{f_{yh}} \quad (\text{mm}^2) \tag{35}$$

where V_{jv} = vertical design shear force (N), f_{yh} = yield strength of horizontal reinforcement (MPa), V_{vc} = shear carried by diagonal compression strut mechanism (N) (where 1 N = 0.225 lb, 1 mm = 0.039 in and 1 MPa = 145 psi). When plastic hinging is not expected to occur in the column above or below the joint core

$$V_{cv} = \frac{A'_{sc}}{A_{sc}} V_{jv} \left(0.6 + \frac{C_j P_e}{A_g f'_c}\right) \quad \text{(N)} \qquad (36)$$

except where axial load results in tensile stresses over the column section, where A'_{sc} = area of compression reinforcement in one face of column (mm^2), A_{sc} = area of tension reinforcement in one face of column (mm^2), and other notation is as for Eq. 31. When P_e is tensile, the value of V_{cv} is interpolated linearly between the value given by Eq. 36 when P_e is taken as zero and zero when the axial tensile stress over the gross concrete area is $0.2f'_c$. However, if plastic hinges are expected to form in the column above or below the joint core, but not when elastic behaviour is assured in the column or column stub on the opposite side of the joint, V_{cv} should be taken as zero for any axial load on the column.

The spacing of vertical shear reinforcement in each plane of any beam framing into the joint should not exceed 200 mm (7.9 in) and in no case should there be less than one intermediate bar in each side of the column in that plane.

4.5.6 Confinement of Joint Core

The horizontal transverse confinement reinforcement in the joint core should not be less than that required in the potential plastic hinge regions in the ends of the columns. In no case should the spacing of transverse reinforcement in the joint core exceed 10 times the diameter of the longitudinal column bar or 200 mm (7.9 in), whichever is less.

4.5.7 Anchorage of Longitudinal Reinforcement

It is clear from Figs. 28 and 29 that bond conditions for the longitudinal beam and column bars at the boundaries of the joint cores are unfavourable, because large steel forces need to be transferred to the concrete over relatively short lengths of bar, flexural and diagonal tension cracks are present which will alternate in direction during cyclic loading,and bond deterioration will occur during cyclic loading.

For interior joints, when plastic hinges form in the beams at the column faces, a beam bar will be yielding in tension on one side of the joint core and in compression on the other side of the core, and hence twice the yield force of the bar will need to be transferred by bond to the joint core, which may require extremely high bond stresses. Hence the bar diameters need to be limited to prevent excessive slip of bars through the joint core.

For exterior joints, degradation of bond strength will also cause yielding of longitudinal beam bars to penetrate into the joint core, thus reducing the effective anchorage length and possibly result in loss of anchorage. Therefore, it is recommended that at exterior beam-column joints in which plastic hinging occurs in the beam at the column face, the anchorage of beam steel should be considered to commence within the joint core. Also for exterior joints, the outer longitudinal column bars in Fig. 29 will be subjected to high bond stresses which can result in vertical splitting of concrete along those bars.

Bond in the joint core is not so critical when plastic hinges form some distance away from the joint core, since then yield penetration into the joint core does not occur.

(1) Bar anchorage at exterior joints

The basic development length of a deformed bar in tension terminating with a standard 90° hook is

$$\ell_{dh} = \frac{66d_b}{\sqrt{f'_c}} \frac{f_y}{275} \text{ (mm)} \tag{37}$$

where d_b = diameter of longitudinal bar (mm), f_y = yield strength of longitudinal reinforcing steel (MPa), and f'_c = compressive cylinder strength of concrete (MPa) (where 1 mm = 0.039 in and 1 MPa = 145 psi). When the bar diameter is 32 mm (1.3 in) or smaller, with side cover not less than 60 mm (2.4 in) and cover on tail extension not less than 40 mm (1.6 in), the value may be reduced by multiplying by 0.7. Where the concrete is suitably confined the value may be reduced by multiplying by 0.8.

The basic development length for a deformed bar in compression is

$$\ell_{db} = 0.24 d_b f_y / \sqrt{f'_c} \text{ (mm)} \tag{38}$$

but not less than $0.044 d_b f_y$. Where the concrete is suitably confined the value can be reduced to $0.75 \ell_{db}$.

The anchorage is considered to commence within the column at distance $0.5h_c$ or $10d_b$ from the column face whichever is less (see Fig. 31), except that when the plastic hinge is located sufficiently far away from the column face anchorage may be considered to commence at the column face (see Fig. 32), where h_c = column overall depth.

When the column depth is not great enough to accommodate the required development length for beam bars, a beam stub at the far face of the column, such as shown in Figs. 33 and 34, may be used to increase the available concrete length for anchorage. The presence of such a stub has been shown to result in considerable improvement in joint performance and are being used by some designers in New Zealand.

(2) Bar anchorage in interior joints

To keep bond stresses to an acceptable level, the diameters of longitudinal bars d_b passing through a joint core are limited as follows:

(i) Beam bars:
When plastic hinging can occur adjacent to the column face:
$d_b \leq h_c/25$ when f_y = 275 MPa, or
$d_b \leq h_c/35$ when f_y = 380 MPa.

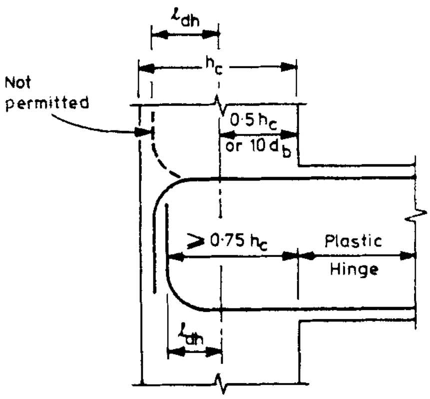


Figure 31 : Anchorage of Beam Bars When Critical Section of Plastic Hinge Forms at Column Face [5].

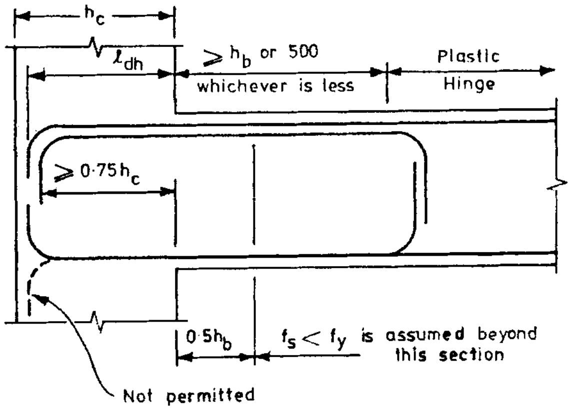


Figure 32 : Anchorage of Beam Bars When Critical Section of Plastic Hinge is Located at Sufficient Distance From Column Face [5].

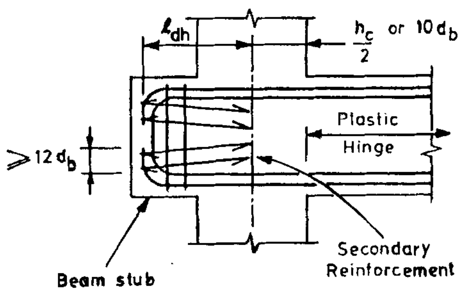


Figure 33 : Anchorage of Bars in Beam Stub [5].

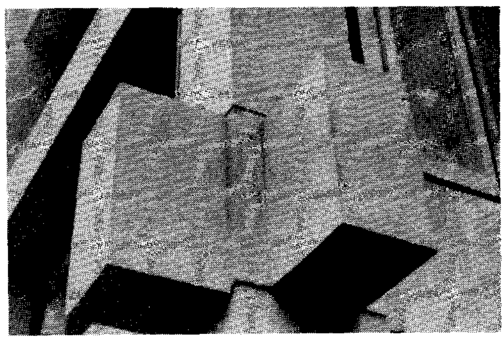


Figure 34 : Building with Beam Stubs.

When plastic hinging is located at a distance from the column face of at least the beam depth or 500 mm (19.7 in), whichever is less:

$d_b \leqq h_c/20$ when f_y = 275 MPa, or

$d_b \leqq h_c/25$ when f_y = 380 MPa.

where h_c = column overall depth and 1 MPa = 145 psi.

(ii) Column bars:

When columns are intended to develop plastic hinges:

$d_b \leqq h_b/20$ when f_y = 275 MPa, or

$d_b \leqq h_b/25$ when f_y = 380 MPa.

When columns are not intended to develop plastic hinges:

$d_b \leq h_b/15$ when f_y = 275 MPa, or
$d_b \leq h_b/20$ when f_y = 380 MPa.
where h_b = beam overall depth and 1 MPa = 145 psi.

4.5.8 American Concrete Institute Code Provisions for Beam-Column Joints

The 1983 ACI building code [9] in its seismic provisions has adopted a fundamentally different approach to the problem of beam-column joint shear. The design rules are based on the assumption that a joint can carry a design horizontal shear force if the concrete in the joint core is confined by the same quantity of transverse reinforcement as used to confine the ends of the column.

The design horizontal shear force in the joint core V_u is calculated in the same manner as for the New Zealand code. The stresses in the steel are assumed to be 25% greater than the yield strength regardless of the grade of steel. The calculated value of V_u should satisfy

$$V_u \leq \gamma \sqrt{f'_c} A_j \quad \text{(N)} \tag{39}$$

where γ = 1.67 for a confined joint and 1.25 for other joints, f'_c = concrete compressive cylinder strength (MPa) and A_j = effective joint area (mm^2) (where 1 MPa = 145 psi and 1 mm = 0.039 in). A joint is considered to be confined if members frame in on all vertical faces and if at least 75% of each face of the joint is covered by the framing member. The effective joint area is the area of the column but if the column has a larger width than the beam the effective joint width should not be taken to exceed the width of the beam plus the overall depth of the column.

Transverse reinforcement as specified by Eqs. 19 and 20 or 21 and 22 should be provided within the joint core, except that if the joint is a confined joint as defined above it is permitted to reduce the transverse reinforcement in the joint to one-half of that specified by those equations.

Hence there is no specific calculation for the horizontal joint core shear reinforcement required for the horizontal shear force V_u. Also there is no calculation procedure or specific requirement for vertical column reinforcement crossing the joint core. The spacing between longitudinal bars is governed indirectly by the maximum permitted horizontal spacing of the tie legs across the section which is 335 mm (14 in). This means that no intermediate column bars between corner bars need be used when the column size is less than about 450 mm (18 in).

No special requirements exist for the anchorage of beam bars at interior joints and there are no limitations on usable bar diameters. At exterior joints the development length for hooked bars anchored in the joint core is taken from the face of the column.

It is evident that the ACI code approach [9] is not based on a rational model for joint core shear behaviour. The design of joint core hoop reinforcement on the basis of the quantity of transverse steel required to confine the ends of columns is illogical and cannot produce any degree of accuracy because it does not take into account the possible varying conditions for shear in the joint cores. This is especially the case when the wide range of joint types and column axial loads used in design in practice is considered. Also, the ACI approach does not give any attention to the requirements of vertical shear reinforcement, or to the anchorage of beam bars passing through interior joints.

4.5.8 Congestion of Reinforcement

A serious problem in the construction of ductile reinforced concrete frames is the congestion of steel reinforcement in the critical regions adjacent to and in the beam-column joints, which makes the fabrication of the reinforcement cages and the placing the concrete difficult. Fig. 35 shows some reinforcement arrangements in the potential plastic hinge regions of beams and columns and beam-column joint cores of ductile frames. The congestion in those regions arises mainly from the close spacing and the multi-leg arrangements of the transverse reinforcement.

The fabrication of reinforcement cages is best carried out as far as possible under factory conditions where good workmanship is more readily achieved than on the building site. Construction problems can be eased by giving careful attention to the manner in which the reinforcement will be placed on the site in the frames. For work to proceed rapidly and efficiently, standardization of detailing of the reinforcement should be sought.

Congestion can be made a less serious problem by using larger cross sectional dimensions for the concrete members so that relatively low reinforcement ratios are adequate to achieve the required flexural strength. For example, a tension steel ratio in beams of approximately 1% is considered a practical maximum in New Zealand since the design shear forces in beam-column joint cores are then moderate and the required quantity of horizontal joint core hoops for shear is reduced and can be placed with less difficulty. Similarly, the use of larger column sections results in lower average compressive stresses on the gross area of the column and then the required quantity of horizontal confining steel in the column ends is not large.

The placement of concrete in congested reinforced concrete cages is difficult. However, proper compaction of concrete is obviously essential if the structure is to perform adequately under service load conditions and when subjected to large deformations during major earthquakes.

5. CONCLUSIONS

The emphasis in the seismic design of reinforced concrete frames should be on good structural concepts and detailing. It is recognised that uncertainty exists regarding the selection of the mathematical model representing the behaviour of the structure and the form of the imposed ground shaking. Major damage observed in earthquakes has been shown to be due mainly to poor structural concepts (for example: column sidesway mechanisms and/or considerable twisting, due to soft storey, or lack of symmetry and uniformity), and poor ductile detailing (for example:

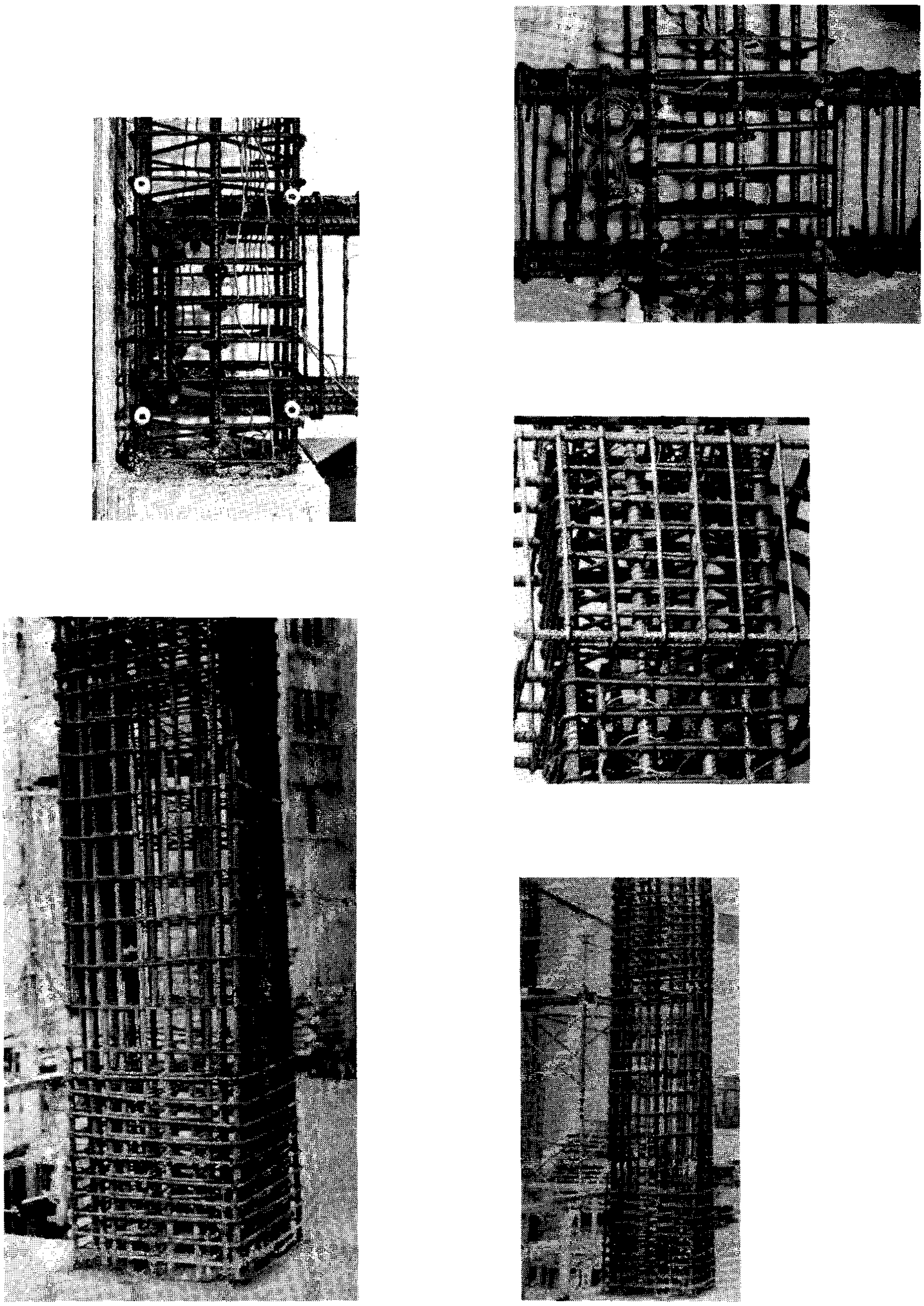


Figure 35 : Some Reinforcement Arrangements for Ductile Reinforced Concrete Frames.

brittle connections, inadequate anchorage of reinforcement or insufficient transverse reinforcement to prevent shear failure, premature buckling of compressed bars and crushing of concrete).

The aim in seismic design should be to impart to the structure features which will result in the most desirable behaviour which implies establishing a desirable hierarchy in the possible failure modes for the structure. In the New Zealand concrete design code this philosophy is incorporated in a rational capacity design procedure which considers the required levels of flexural and shear strength of the members and joints. A proper assessment of the strength and ductility of a structure cannot be made using the working stress design method and hence strength design is used.

In the ductile design approach for reinforced concrete frames specified in the New Zealand concrete design code special consideration is given to the ratio of column to beam flexural strength necessary to reduce the likelihood of plastic hinges forming simultaneously in the top and bottom of columns, the detailing of beams and columns for adequate ductility, and the mechanisms of shear resistance and bar anchorage in members and in beam-column joints.

ACKNOWLEDGEMENTS

The author acknowledges many informative discussions with professional engineers in New Zealand, particularly with his university colleagues Professor T. Paulay and Dr. M.J.N. Priestley.

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NOTATION

A_b = area of a longitudinal bar

A_c = area of concrete core of column measured to outside of the peripheral transverse reinforcement

A_g = gross area of column cross section

A_j = effective joint area

A_{jh} = total area of horizontal shear reinforcement placed between top and bottom beam reinforcement in a beam-column joint

A_{jv} = total area of vertical shear reinforcement, at the side faces of the column, placed between the corner bars of a beam-column joint

A_s = area of longitudinal tension reinforcement in a beam

A'_s = area of longitudinal compression reinforcement in a beam

A_{sc} = area of longitudinal tension reinforcement in one face of column

A'_{sc} = area of longitudinal compression reinforcement in one face of column

A_{sh} = total area of transverse reinforcement, including cross ties, in direction under consideration within longitudinal spacing s_h

A_{sp} = area of spiral bar

A_{te} = area of one leg of a stirrup-tie

A_v = area of vertical shear reinforcement in beam within longitudinal spacing s

b = width of compression face of member

b_c = width of column

b_j = effective width of joint

b_w = width of beam web

C_j = $V_{jh}/(V_{jx} + V_{jy})$

C'_c = compression force in concrete of beam during positive moment

C'_s = compression force in longitudinal beam reinforcement during positive moment

d = distance from extreme compression fibre of concrete to centroid of tension steel

d_b = diameter of longitudinal reinforcing bar

d_s = diameter of spiral

f'_c = concrete compressive strength

f_y = yield strength of longitudinal reinforcement

f_{yh} = yield strength of transverse reinforcement

h'' = dimension of concrete core of section, perpendicular to the direction of hoop bars, measured to the outside of the perimeter hoop in the New Zealand code or to the centres of the perimeter hoop in the ACI code

h_b = overall depth of beam

h_c = overall depth of column

ℓ_{db} = basic development length of a deformed bar in compression

ℓ_{dh} = basic development length of a deformed bar in tension terminating with a standard 90° hook

M_{code} = column moment derived from the code static seismic loading

M_{col} = design uniaxial bending moment for column

P_e = column load in compression due to the design gravity and seismic loading

P_{col} = design axial load for column

P_o = axial (concentric) load strength of column

r = see Eq. 10

s = centre to centre spacing of stirrups or ties or spirals or circular hoops

s_h = centre to centre spacing of hoop sets

T = tensile force in longitudinal beam reinforcement during negative moment

T' = tensile force in longitudinal beam reinforcement during positive moment

T_1 = fundamental period of vibration of the structure

v_b = shear stress carried by the concrete when $P_e = 0$ in gravity load design

v_c = shear stress carried by concrete of a column

v_{jh} = nominal horizontal shear stress in a beam-column joint

V = column shear

V_{ch} = horizontal shear carried by diagonal compression strut mechanism in a beam-column joint

V_{code} = column shear force derived from the code static seismic loading

V_{col} = design shear force for column

V_{cv} = vertical shear carried by diagonal compression strut mechanism in a beam-column joint

V_{jh} = horizontal design shear force in a beam-column joint

V_{jv} = vertical design shear force in a beam-column joint

V_{jx} = horizontal design shear force in x direction in a beam-column joint

V_{jy} = horizontal design shear force in y direction in a beam-column joint

V_u = design shear force

γ = see Eq. 39

Δ_u = maximum displacement

Δ_y = displacement at first yield

μ = Δ_u/Δ_y

ρ = $A_s/b_w d$

ρ' = $A_s'/b_w d$

ρ_s = ratio of total volume of spiral or circular hoop reinforcement to total volume of concrete core measured to the outside of the spirals or hoops

ϕ = strength reduction factor

ϕ_o = ratio of overstrength flexural capacity of beam to the dependable moment capacity of beam required by the code

ϕ_u = maximum curvature

ϕ_y = curvature at first yield

ω = factor allowing for higher mode and concurrent loading effects on the bending moments in columns

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