This paper is the result of deliberations of the Society's discussion group on
SEISMIC DESIGN OF DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES

SECTION E

BEAM FLEXURE AND HINGE ZONE DETAILING IN REINFORCED CONCRETE DUCTILE FRAMES REQUIRING BEAM SWAY MECHANISMS

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E1.0 INTRODUCTION

This report considers the provisions to be made when designing and detailing the longitudinal steel to be used within beams of structures types 1 and 2 as defined by Table 5 of NZS 4203, 1976[1]. Also discussed are the detailing standards required within the beam hinge zones.

While these provisions relate primarily to the seismic resisting frames it is envisaged that they should be applied to any frame that is subject to stress reversal in the longitudinal reinforcement when displaced to the provision of 4203:3.8. They could also govern the design of secondary frames where primary seismic resistance is provided by some other means.

Since capacity design procedures are required for structural types 1 and 2 this report considers only that method of design. It should be noted that one and two storeyed buildings and the top storey of a multi-storey building may be exempted from these requirements by Clause 4203:3.3.3.5.2.

E2.0 REQUIREMENTS OF EXISTING CODES

E2.1 Code of Practice for GENERAL STRUCTURAL DESIGN AND DESIGN LOADINGS FOR BUILDINGS.

In so far as NZS 4203:1976 governs the subject of this report the following clauses are of significance.

Clause 4203:1.2.4 requires the use of a capacity reduction factor (ϕ) as provided by the relevant material code. Where values of ϕ are not given then ϕ shall be taken as 1.0.

Clause 4203:1.3 specifies the design loads and their combinations for the strength method of design. Those of primary concern to the report are:

$$U = 1.4D + 1.7\ L_R$$

$$U = D + 1.3\ L_R + E$$

$$U = .9D + E$$

However it must be understood that for all flexural elements for which ductile detailing is required, as a consequence of seismic movements, the longitudinal steel and hence the subsequent design may be governed by any of the loading combinations of 4203:1.3.2.3.

Clauses 4203:3.2 and 4203:3.3.1 - 3.3.3 determine the general principles of design and the particular requirements for ductile frames. The directive that is of importance to this report is that the non ductile failure of beams is to be avoided and that the true capacities of the beams should be assessed on the basis of the actual reinforcement of the beam and slab.

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E3.0 BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-71)[2]

ACI 318-71 can be considered a relevant materials code within the meaning of NZS 4203 and as such contains the minimum requirements for the design of flexural reinforcement in beams. Although this code contains an appendix A - 'Special Provisions for Seismic Design' - these shall not be considered mandatory as such provisions will be modified by the findings of the Concrete Design Committee of SANZ and the NZNSEE Committee and documented in the reports produced by this group.

Clause 9.2.1.1 specifies the ϕ factor for bending at 0.9 while Clause 10.2 defines assumptions to be made when determining the ultimate strength of a section in flexure.

Clause 8.6 - moment redistribution of up to 20% is allowed by Clause 8.6, the amount depends on the percentages of positive and negative reinforcement at the section.

Clause A.5 - while Clause A.5 is not considered binding by this report it does contain requirements that are particularly relevant to the design of flexural members and many of the final recommendations are closely related to the provisions of this Clause.

E4.0 RECOMMENDED CRITERIA FOR THE DESIGN OF FLEXURAL REINFORCEMENT AND HINGE ZONE DETAILING

These criteria have been written with some modifications from the recommendations and information supplied by the MWD[3] and Professors Park[4,5] and Paulay[4,5,7,8].

E4.1 Dimensions

The depth (h) and width (b) of rectangular members subject to bending at both ends should be such that:

$$\frac{\ell_n}{b_w} \not> 10 \qquad \text{for cantilevers - E4-1 (a)}$$

$$\frac{\ell_n}{b_w} \not> 16 \qquad \text{for rectangular beams fixed at both ends - E4-1 (b)}$$

$$\frac{\ell_n}{b_w} \not> 24$$ for T or L beams fixed at both ends - E4-1 (c)

and

$$\frac{\ell_n h}{b_w^2} \not> 38$$ for cantilevers - E4-2 (a)

$$\frac{\ell_n h}{b_w^2} \not> 65$$ all other beams - E4-2 (b)

where ℓ_n is the clear span of the member and h the depth of the beam.

The width of a beam should not be less than 200mm nor more than the width of the supporting column plus a distance on each side of the supporting column equal to ¼ the overall depth of the column in the relevant direction or more than twice the width of the column.

The eccentricity (e) of any beam relative to the width of the column (b_c) into which it frames, as measured by the distance between the geometrical centre lines of the two members, should be limited as below.

$$e \not> \frac{b_c}{4} \qquad \text{E4-3 (a)}$$

E4.2 <u>Materials</u>

All concrete used in the primary structural elements should be high or special grade as defined by NZS 1900 Chapter 9.3A. Further it is recommended that a minimum specified strength of 25 MPa should be used, except that for buildings or parts of buildings satisfying NZS 4203:3.3.3.5.2, 20 MPa concrete may be used.

All longitudinal reinforcement should be of New Zealand manufacture to NZS 3402P. Grade 275 deformed steel should be used for all longitudinal beam reinforcement and plain round bars of the same grade for all transverse reinforcement except that plain round bars of grade 380 may be used for stirrups up to half the diameter of the longitudinal bars, provided that the plain round bars to grade 380 are permanently identified.

E4.3 <u>Moment Redistribution</u>

Except where empirical values of bending moment are used, moments within a frame may be redistributed according to the rule given below:

(a) In beams the amount of redistributable moment should not exceed at any point 30% of the absolute maximum moment derived for the beam for <u>any</u> of the loading combinations specified in NZS 4203:1.3.

(b) The positive span moments for all design combinations should be modified accordingly to satisfy the requirements of statics. (Note: ACI 318-71 Clause 8.6 does not apply.)

E4.4 <u>Longitudinal Reinforcement</u>

E4.4.1 <u>Reinforcement design and limiting ratios</u>

Flexural reinforcement should be designed in accordance with the assumptions of Clause 10.2 of ACI 318-71. A capacity reduction factor ϕ = 0.9 shall be used.

At any section subject to bending without axial compression, the tensile reinforcement ratio for the top or bottom reinforcement shall not be less than

$$\rho_{min} = \frac{1.4}{f_y} \qquad \text{(E4.4.1-1)}$$

nor shall it be more than

$$\rho_{max} = \frac{1 + 0.17\ (\frac{f'_c}{7} - 3)}{100} \left(1 + \frac{\rho'}{\rho_{max}}\right) \qquad \text{(E4.4.1.-2)}$$

or

$$\rho_{max} = \frac{7}{f_y} \qquad \text{whichever is less (E4.4.1-3).}$$

At least two 16mm bars should be provided at both top and bottom.

At any section expected to form a ductile hinge the percentage of steel in the compression face should not be less than half that in the tension face. This requirement shall apply even if the hinge does not reverse.

A minimum of 25% of the larger amount of the negative reinforcement should be continued throughout the length of the beam.

E4.4.2 <u>Placement of Reinforcement</u>

To be considered effective, longitudinal beam reinforcement at columns should be positioned to meet the following requirements.

(a) At interior columns, where a transverse beam of similar dimensions frames into the column, all bars within the width of the beam web and that part of the slab which extends a distance 4 times the thickness of the slab from each side of the column should be considered.

(b) At interior columns, where no transverse beam exists, all bars within the beam web and that part of the slab which extends a distance of 1½ times the thickness of the slab from each side of the column should be considered.

(c) At exterior columns where a transverse beam of similar dimensions is framing into the column, and where the beam reinforcement is to be anchored, all bars within twice the slab thickness on each side of the column should be considered.

(d) Where no transverse beam exists at an exterior column all effective beam bars shall be within the width of the column.

(e) In all cases at least 75% of the effective top reinforcement must pass through or be anchored in the column core. Where longitudinal reinforcement is governed by the load combination of U = 1.4D + 1.7 L_R then only 75% of the reinforcement required by the load combinations U = D + 1.3 L_R + E and U = .9D + E is required to pass through or be anchored in the column core.

E4.4.3 <u>Restrictions on Longitudinal Bar Sizes</u>

For conventionally reinforced beams with horizontal flexural reinforcement no longitudinal bar should have a greater diameter than 1/80th of the clear span.

Where beams frame into opposite sides of a column the maximum diameter of the longitudinal (grade 275) beam reinforcement should be limited to 1/25th the depth of the column, except that when beam end detailing is such that no yielding can occur in bars as a result of inelastic seismic displacements, bar diameters up to 1/20th the column depth may be used. Bars within the slab should be further limited to 1/5th the slab thickness unless they are adequately confined over the potential plastic hinge location. Bars greater than 1/5th the slab thickness should not be considered to contribute to beam flexural strength.

E4.4.4 Anchorage, Curtailment and Splicing of Longitudinal Reinforcement

The anchorage of longitudinal steel should comply with ACI 318-71 Chapter 12 except that the following clause shall not apply.

> Clause 12.5 (d) - shall not be considered to apply within column cores or beam stub anchorages.

When anchoring longitudinal beam steel into column cores or beam stub, the anchorage shall be deemed to commence ½ the depth of the relevant depth of the column or $10d_b$, whichever is less, from the face at which the steel enters the column. Except that when detailing is such that no yielding can occur in the bars as a result of inelastic seismic displacements, the anchorage can be considered to commence at the column face. Notwithstanding the adequacy of the anchorage, no bar should be anchored within a column core without terminating in at least a standard hook as near the far side of the core as is practically possible; and no closer than 3/4 the relevant depth of the column to the face of entry. For anchorages wholly within the column the standard radius bend should commence at least h/2 from the face of entry.

For situations in which beams frame into opposite sides of a column the limitation on longitudinal bar diameter of section E4.4.3 is deemed to provide adequate development. Where the moment envelopes are such that unequal reinforcement is required on opposite sides of a column such additional reinforcement may be anchored in the beam on the far side of the column by means of a standard hook plus an extension to the opposite face of the beam.

The distribution and curtailment of longitudinal reinforcement should be such that the yield moment of the section is exceeded only at hinge locations where special transverse reinforcement has been provided.

Splices in reinforcement should comply to the requirements of ACI 318-71 except that no part of a lap splice should occur within 1½ effective beam depths of a column face for top bars or within one beam depth for bottom bars.

E4.4.5 Overstrength Capacity of Longitudinal Reinforcement

The overstrength capacity of beams for use in subsequent capacity design procedures should be based on the following factors. All grade 275 steel within the effective beam and slab zone, as defined in 4.4.2, shall be considered to have a strength at limiting rotation of 340 MPa; the capacity should be calculated in accordance with the assumptions of ACI 318-71 Clause 10.2 with the ϕ factor taken as 1.0.

E5.0 HINGE ZONE SPECIAL TRANSVERSE REINFORCEMENT

Hinge zone special transverse reinforcement consisting of stirrup ties of not less than 6mm diameter should be provided in the following regions:

(a) Over a length equal to twice the flexural member depth, measured from the critical section at the face of the supporting column, wall, haunch or beam towards midspan, at both ends of the member, or over a length where a rational analysis of the moment diagram (taking into account the effect of diagonal cracking and based on the overstrength capacities of the beam sections with gravity loading of 0.9D and the curtailment of the reinforcement as detailed) indicates that expected steel stress is more than 0.5fy in either steel layer, whichever is less, but not less than 1½ effective depths of the member.

Ties should be arranged so that each upper or lower face bar (or bundle of bars) should be restrained against lateral buckling by a 90° bend of a closed confining tie, except that when 2 or more bars at not more than 20 tie diameters (or 200mm, whichever is less) apart are so restrained by the same tie, any bars between them are exempted from this requirement, except that not more than two bars are permitted to be exempt.

The area of ties should be computed as

$$A_{te} = \frac{\Sigma A_b \ f_{yb}}{16 \ f_{yt}} \times \frac{S}{100}$$

where A_{te} = area of the tie leg(s) in the direction of potential buckling.

ΣA_b = sum of the areas of the longitudinal steel reliant on the tie leg(s) including the area of any bars exempted under the above provisions. Bars centred more than 75mm beyond any cover concrete need not be counted in ΣA_b

f_{yb} = minimum yield stress of the longitudinal steel

f_{yt} = minimum yield stress of the stirrups.

For beams containing more than two layers of longitudinal steel every alternate layer should be confined by a horizontal tie to prevent lateral movement. Horizontal ties should also be provided in those beams in which the depth of the compression block exceeds 150mm.

The first tie set should be placed as

close as possible to the column ties but no further than 50mm.

The spacing of these ties should not exceed 100mm, 6 times the diameter of the longitudinal bar to be restrained, or ¼ the effective depth of the member, whichever is least.

COMMENTARY

CE4.1 Dimensions

The criteria for the relationships between the length, depth and breadth are based on the dimensional limitations of the British Code of Practice, CP110, as recommended by Paulay[6,8]. In the derivation of these criteria the effect of stiffness degradation from reversed cyclic loading at yield was recognised. Hence only one third of the maximum slenderness ratios in CP 110 have been allowed. The unsupported length for a continuous beam has conservatively been assumed at 2/3 the clear span and the effective length factors for the cantilever distance from the point of support to the point of inflexion are as defined in Australian Standard 1250-1972 SAA Steel Structure Code[9].

The derivation of the recommended ratios is as follows:

CP110 limits

$$\frac{\ell}{b_w} < 25 \; , \; \frac{\ell_d}{b_w^{\,2}} < 100$$

for unrestrained cantilever beams; using stiffness of one third as discussed above.

$$\frac{\ell}{b_w} < 8 \; , \; \frac{\ell_d}{b_w^{\,2}} < 33$$

For continuous beams the length from point of support to point of inflexion is 2/3 length and the effective length factor is 0.75 l as per AS 1230-1972: 5.9.4 (b).

$$\text{Thus } 3/4 \times 2/3 \times \frac{\ell_n}{b_w} < 8 \text{ or } \frac{\ell_n}{b_w} < 16 \qquad \text{E4-1(b)}$$

$$\text{and } 3/4 \times 2/3 \times \frac{\ell_n h}{b_w^{\,2}} < 33 \text{ or } \frac{\ell_n h}{b_w^{\,2}} < 66 \text{ say} < 65 \qquad \text{E4-2(b)}$$

for members monolithic with the slabs they support, the effective length factor used has been 0.5 ℓ, as per AS 1230-1972: 5.9.4 (c) in the calculation of equation (4-1); equation (4-2) has been left unchanged.

Similarly for cantilever beams the length of the beam is used with an effective length factor of 0.85, as per AS 1250:1972: 5.9.4 (a).

These criteria are represented graphically in Figure CE.1 the unusable area being shaded. It should be noted that, while these criteria allow low $\frac{b_w}{h}$ ratios, the width of the beam is fixed primarily by the clear span. However, these rules do allow a more uniform approach in determining beam, column and wall sections.

The existing provisions of the SEAOC[10] code for the maximum usable width of beams in relation to the column width are considered too liberal. It is essential that the bulk of the reinforcement provided for lateral load effects be anchored within or pass through the column core to enable satisfactory joint performance.

Thus to keep the longitudinal reinforcement reasonably close to the column core the width of the beam in excess of the column has been specified in terms of the column depth rather than beam depth. Notwithstanding these provisions a proportion of the longitudinal steel is required to pass through the column core in Section E4.4.2.

A further effect of these dimensional and placement requirements is that the amount by which it is possible to place the beam eccentrically on the column is now limited. Figure CE.2 indicates these requirements graphically.

CE4.2 Materials

Reliable concrete strengths are essential for the concrete to be used in ductile flexural frames, hence the requirement for high or special grade concrete. The MWD requirement for 30 MPa concrete appears to be excessively stringent particularly when lapping and other details are related to the strengths used in the design.

Longitudinal reinforcement is required to be of New Zealand manufacture as no reliable information is available as to the average strengths and overstrengths to be expected from bars from other sources. Bars from these sources may be used when reliable long-term statistical data is available. Other properties being equal, overseas manufactured bars are adequate for transverse reinforcement as excessive strength in these situations is not detrimental in ductile flexural beams.

Sections detailed with grade 380 longitudinal steel are inherently less ductile and significant strain hardening can commence at section ductility factors as low as 3 or 4. If grade 380 bars are used the overstrength yield in section E4.4.5 should be taken as 530 MPa. Furthermore the limitations on reinforcement diameters in section E4.4.3 should be amended to 1/120th of the clear span, 1/35th the depth of the column or 1/25th with elastic joint zones and 1/8th the slab thickness.

There are difficulties too with grade 380 reinforcement when used for stirrup ties, the principal problems being:

(i) simple identification

(ii) bending sufficiently tightly to engage the longitudinal steel.

The use of deformed ties of any grade can lead to cracking at the root of the deformations on the insides of the bends, particularly if the tie is overbent and straightened. The better crack control, normally associated with deformed steel, is of little consequence as it is expected that cover concrete will be lost in the hinge zone regions.

It is not intended that the use of deformed ties be prohibited in deep, non-hinging foundation beams.

CE4.3 Moment Redistribution

The design and detailing of longitudinal reinforcement will result in sections having greater ductilities than those envisaged in the ACI Code. Hence it has been considered appropriate that up to 30% redistribution, as allowed in many continental codes, should be allowed.

Designers are warned that in structures in which the gravity load effects dominate, cracking is likely to occur when the full redistribution allowed in this report is used.

The advantages of allowing moment redistribution are as follows:

(a) When the negative or positive beam moment demands at either sides of a column are very different these moments can often be equalised and thus the need to anchor some beam steel in an interior column or continue excess beam steel into the next span may be avoided.

(b) In beams where gravity produces only moderate effects it is possible to approach the ideal situation of equal positive and negative steel.

(c) In beams where gravity effects dominate the redistribution allows for greater use of the 50% negative reinforcement required by Section E4.4.1.

CE4.4 Longitudinal Reinforcement

The lower limit for reinforcement ratio is the normal minimum flexural steel except that the reduction below this is not allowed even if the reinforcement provided is 1/3 greater than that required. The upper limits are based on recommendations by Park as modified by Paulay to allow for the effects of increased compression steel. These depend on concrete strength and steel yield strength. The tables below indicate the limits derived from equations E4.4.1 - 2 and 3.

TABLE CE 4.4.1-1

$$\rho_{max} = \frac{1 + 0.17\,(\frac{f'_c}{7} - 3)}{100} \cdot \left[1 + \frac{\rho'}{\rho_{max}}\right]$$

f'_c	$\rho' = 0.5\ \rho_{max}$	$\rho' = .75\ \rho_{max}$	$\rho' = \rho_{max}$
25	0.0164	0.0192	0.0219
30	0.0183	0.0213	0.0244
35	0.0201	0.0235	0.0268
40	0.0219	0.0255	0.0292

TABLE CE4.4.1-2

$$\rho_{max} = \frac{7}{f_y}$$

f_y	ρ_{max}
275	0.0254
380	0.0184

It should be noted, however, that high percentages of longitudinal steel can lead to considerable difficulties at the beam column junction if yielding at this location occurs. It seems that ρ_{max} = 0.015 is the upper practical limit at plastic hinges and preferably ρ_{max} should be in the range 0.01 to 0.0125 for designs in which beam hinges form at the junctions and which influence the design of the joint shear reinforcement.

The requirement that ρ' is at least 50% of ρ_{max} is to ensure that the sections are capable of attaining the ductilities required. Top and bottom steel percentages should be as near equal as possible. The MWD proposal that $\rho' = .7\ \rho_{max}$ has not been recommended as it was felt that the information in Table CE4.3.1 indicates sufficient penalty for structures in which gravity plays a significant part even when 30% moment redistribution is allowed.

CE4.4.2 Placement of Reinforcement

These requirements are basically as recommended by Paulay except for E4.4.2 (e) and are closely related to the dimensional requirements of Section 4.1. While the mechanism by which slab steel can be effectively anchored and the forces transferred to the column by compressive struts and/or torsion in the beams at right angles have been recognised, the more liberal requirements of the MWD have not been adopted for the following reasons:

(i) The mechanism by which slab bars may be able to resist yield stress reversal across the column depth are not clear. Note that Section E4.4.3 requires longitudinal reinforcement to be limited to 1/25th of the column depth.

(ii) It is undesirable to rely on the torsional strength of the transverse beams for transmitting moment to the column. In the extreme case they may have been cracked to the full depth when all torsional and normal shears would be required to be resisted by dowel action or special diagonal steel.

(iii) Attention is drawn to the fact that criteria for width of slab considered to be effective are somewhat arbitary. For this reason concentrations of slab steel just outside the regions deemed effective should be avoided. However a clear definition has been given so that the requirement for $\rho' = 0.5\rho$ can be adequately interpreted.

(iv) It should be noted that for the most effective action in resisting seismic forces, as many beam bars as possible should pass through the column core.

CE4.4.3 Restrictions on Longitudinal Bar Sizes

The requirement that the diameter be less than 1/80th of the clear span is based on the necessity that the span be at least two development lengths long. It should be noted that it would probably be difficult to lap reinforcement of this diameter without the use of welded or other mechanical splices, when the conditions of E4.4.4 are complied with.

The limitation of 1/25th of the column depth is based on experimental results obtained by the MWD. Bars larger than this are likely to produce excessively large slippage within the core of the beam column joint. It should be noted that bond stresses significantly larger than any code will accept are required to ensure that a bar changes from yield in tension to yield in compression across the column core.

In beams where the detailing at the column face is such that hinges cannot form at these locations, significantly improved performance of the joint can be expected and larger bar sizes can be used.

CE4.4.4 Anchorage, Curtailment & Splices of Longitudinal Reinforcement

The requirements of ACI 318-71 have been adopted except where they conflict with the requirement for continuous positive steel and allow increased bond stresses for confined concrete.

Figure CE.3 indicates the anchorage requirements at a terminal column. The MWD requirement to exclude the first 2/3 of the column depth has been considered extreme and the requirement of ½ the depth of the column or 10 d_b whichever is less is suggested as a reasonable allowance for the encroachment of the yield into the joint. Hitherto anchorage has been calculated from the face of columns.

The current requirement that no splice should occur within 2 times the effective depth (h) is considered to be both ambiguous and excessive. It requires spans to be at least 4h long and possibly 4h plus a lap length, depending on interpretation. It is impossible to realistically comply with these requirements in frames of all types. Perimeter frame buildings could be particularly adversely affected by that requirement. The requirement recommended is considered a reasonable solution in that beam spans are effectively required to be 3h plus a lap length.

Designers are reminded that for the determination of cut off points that beams should be considered for the gravity load case of D + 1.3 L_R and 0.9D and then subjected to seismic moments at each end until two hinges form within the span (one positive and one negative). However cut off points may be more critical when only one hinge is formed. Particular attention is drawn to the case when the negative hinge has just formed and the gravity loading is 0.9D.

CE4.4.5 Overstrength Capacity of Longitudinal Reinforcement

This section is intended to clearly define the procedure for computing the over capacity strength of a beam. The recommendation that the strength at limiting rotation be taken as 340 MPa is intended to apply to grade 275 reinforcement only. This figure should be replaced by 530 MPa for grade 380 steel as recommended in CE4.2.

These strengths at limiting rotation are intended to reflect the combined effects of elevated yield strength and the onset of strain hardening.

CE5.0 Hinge Zone Special Transverse Reinforcement

The length of the hinge zone has been modified from the commonly used 2 times the effective depth of the member. The purpose of this section is to define the zone over which lateral restraint to prevent buckling of the longitudinal reinforcement is required. It is considered that the special requirements, that are normally suggested to cope with the Bauschinger effect or stiffness degradation of the longitudinal steel, should not apply if it can be shown that the steel stress will never exceed 50% of yield.

Beam sections reliant on compression steel that may have previously yielded in tension also require some consideration as to the likely need for adequate lateral support.

The size of bar used to prevent lateral buckling is related to the force which is to be restrained. The customary value of 1/40 of the axial force has been conservatively increased to 1/16 when spaced at 100mm centres. The effect of this bar size requirement is that a 6.5mm bar can only be used to confine bars 24mm diameter and smaller and a 12mm bar can restrain 2 #32 bars provided the yield strength of the longitudinal and transverse steels are the same. Figure CE.4 illustrates the interpretation of this clause.

The loss of cover concrete that can be expected in a major earthquake is likely to be alarming and dangerous if excessively thick pieces of concrete are capable of spalling off. For this reason basketing reinforcement is recommended for bottom and side covers exceed 90mm to the main steel.

REFERENCES

1. NZS 4203:1976. Code of Practice for General Structural Design and Design Loadings for Buildings.
2. ACI 318-71. Building Code Requirements for Reinforced Concrete.
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4. R. Park and T. Paulay. Reinforced Concrete Structures. Wiley, 1975.
5. R. Park. Notes and Questions for Pre-workshop Discussion Group Dated 19.5.76.
6. T. Paulay. Memorandum dated 23.12.75. Suggested Dimensional limits on Reinforced Concrete Members in Earthquake Resistant Structures.
7. T. Paulay. Memorandum to Preworkshop Discussion Group dated 25.6.76.
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9. Australian Standard 1250-1972. SAA Steel Structures Code.
10. SEAOC:1973. Recommended Lateral Force Requirements and Commentary.

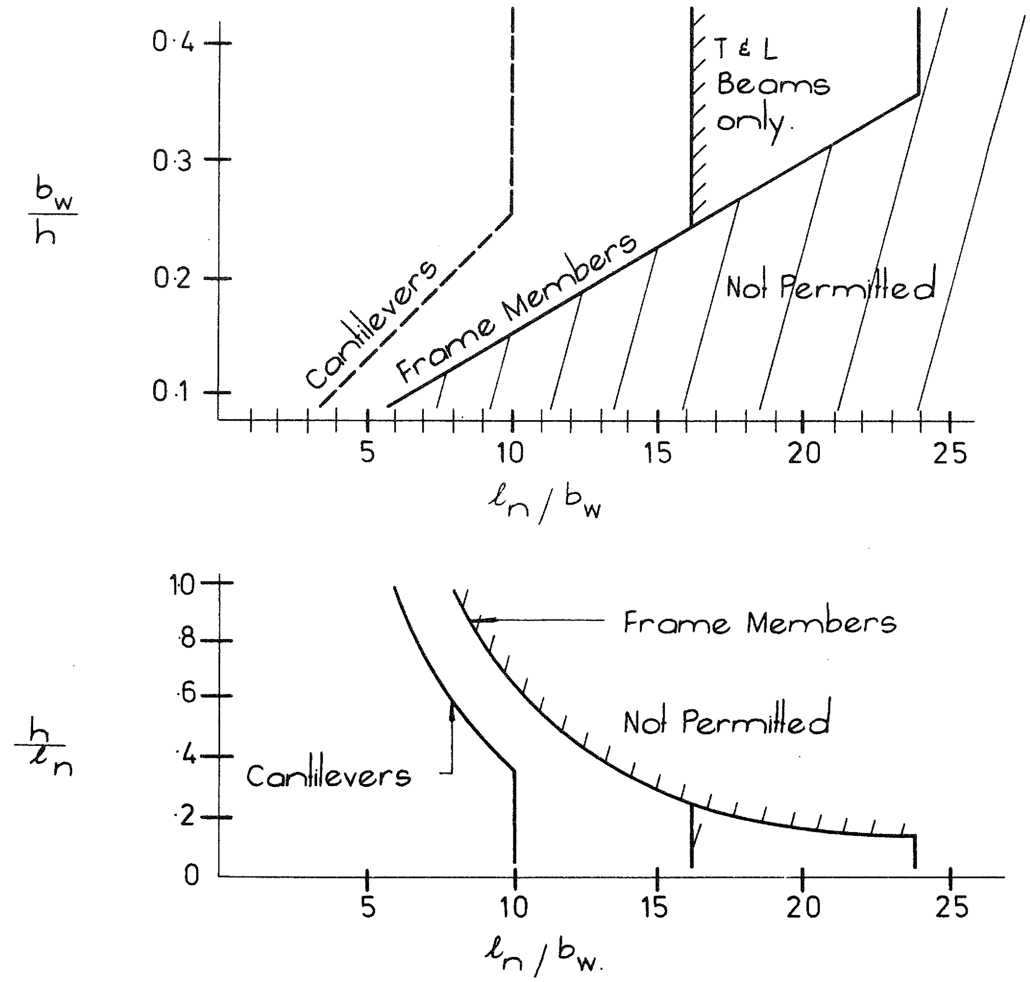
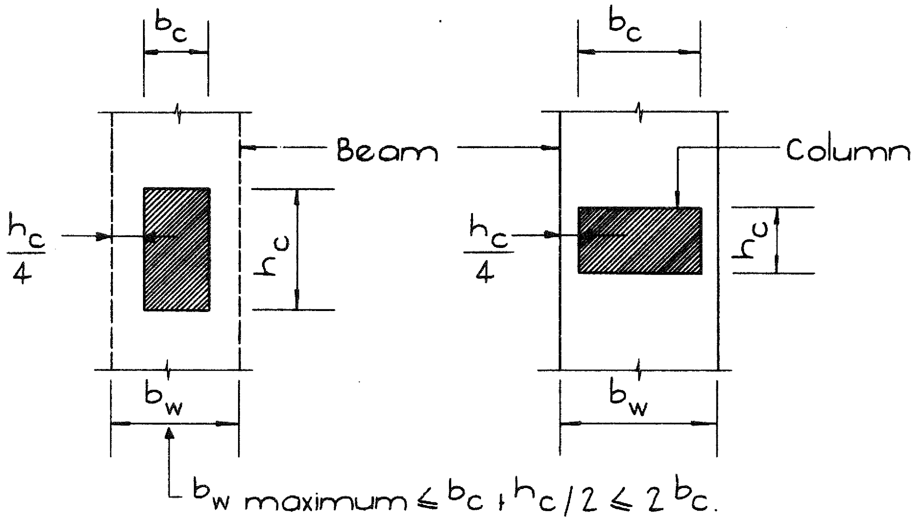


FIGURE CE.1



MAXIMUM WIDTH OF BEAMS

FIGURE CE. 2

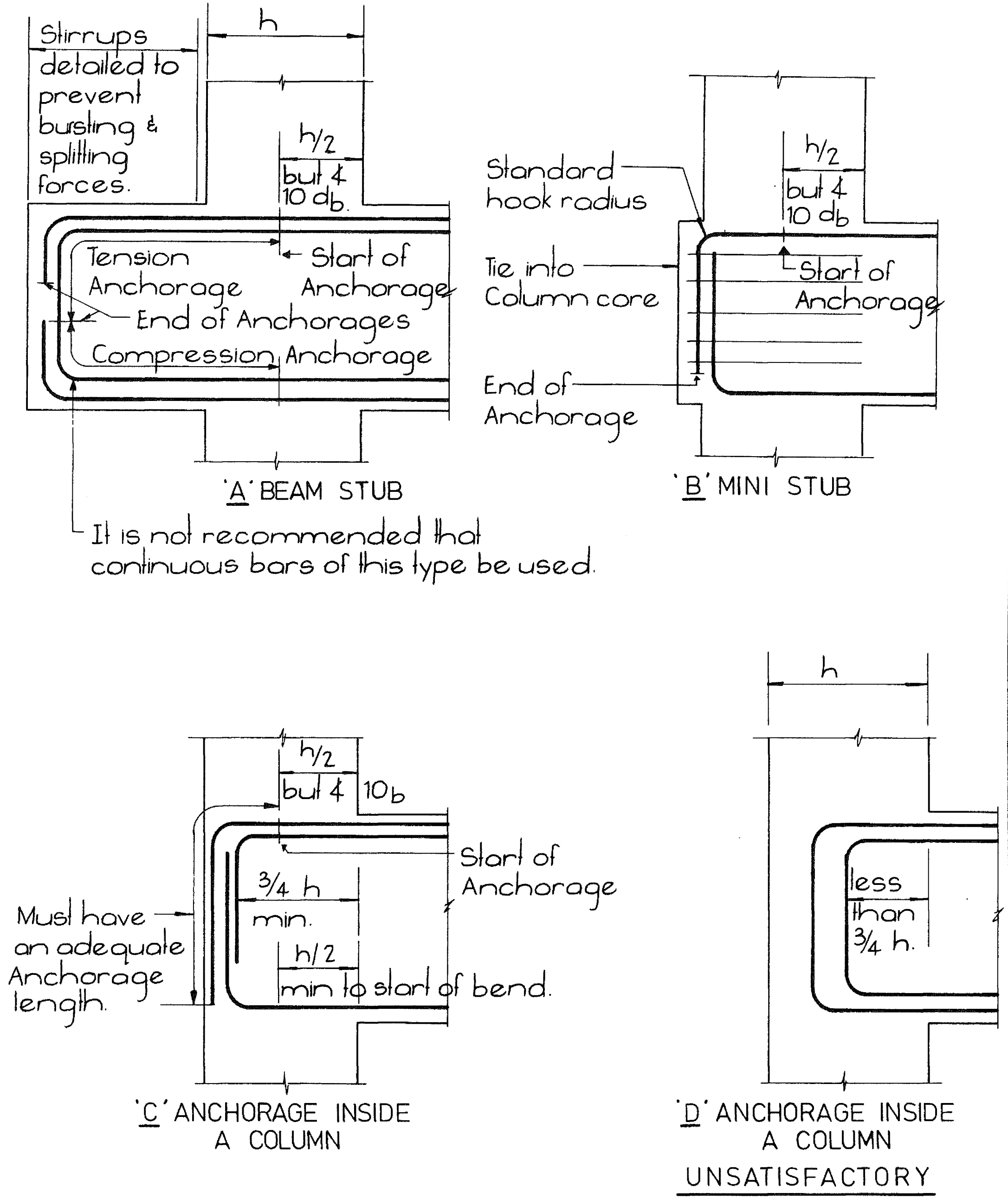


FIGURE CE.3

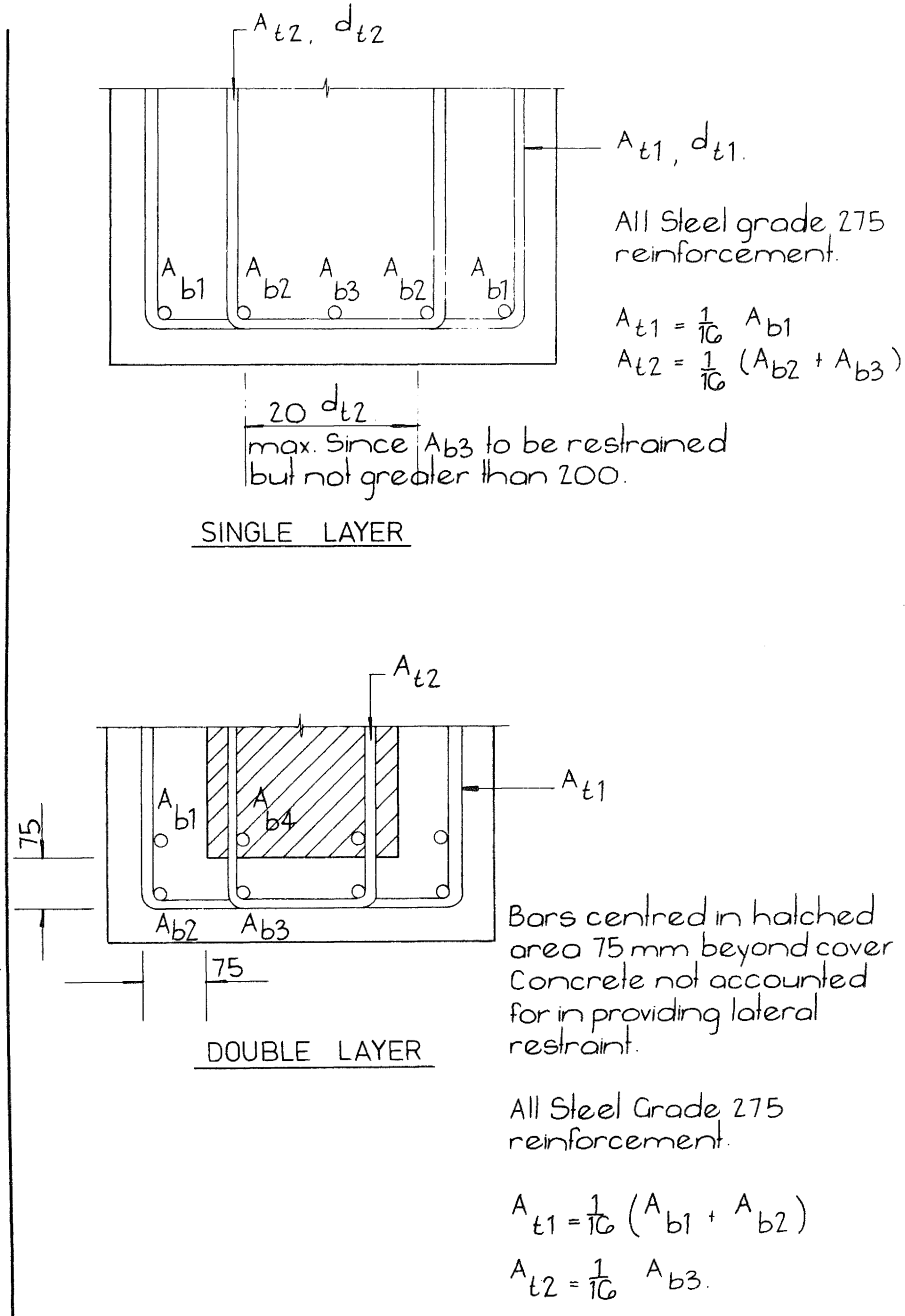


FIGURE CE.4