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NEW ZEALAND STANDARD

Commentary on

THE DESIGN OF CONCRETE STRUCTURES

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NEW ZEALAND STANDARD

COMMENTARY ON
Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

Superseding NZS 3101P and NZSR 32

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COMMITTEE REPRESENTATION

This Commentary was prepared under the supervision of the Concrete Industry Sectional Committee (31/–) for the Standards Council, established under the Standards Act 1965. The committee consisted of representatives of the following:

Association of Consulting Engineers New Zealand
Building Research Association of New Zealand
Department of Scientific and Industrial Research
Harbours Association of New Zealand
Institution of Professional Engineers New Zealand
Ministry of Works and Development
Municipal Association of New Zealand
New Zealand Concrete Masonry Association
New Zealand Concrete Research Association
New Zealand Concrete Society
New Zealand Counties Association
New Zealand Master Builders Federation
New Zealand Portland Cement Association
New Zealand Ready Mixed Concrete Association
New Zealand Universities

The Concrete Design Commitee (31/12) was responsible for the preparation of the Commentary and consisted of the following persons:

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RELATED DOCUMENTS

In this document reference is made to the following:

NEW ZEALAND STANDARDS

NZS 1900 : - - - - Chapter 8 : 1976	*Model building bylaw – General structural design and design loadings*
NZS 3152 : 1974 ® 1980	*Manufacture and use of structural and insulating lightweight concrete*
NZS 3402P : 1973	*Hot rolled steel bars for concrete reinforcement*
NZS 3404 : 1977	*Code for design of steel structures*
NZS 3422 : 1975	*Welded fabric of drawn steel wire for concrete reinforcement*
NZS 4203 : 1976	*Code of practice for general structural design and design loadings for buildings*
NZS 4702 : 1982	*Metal arc welding of Grade 275 reinforcing bar*
NZSR 32 : 1968	*Prestressed concrete* (revoked on declaration of this Standard)

BRITISH STANDARD

CP 110 : 1972	*The structural use of concrete*

AMERICAN STANDARDS

ACI 318–77	*Building code requirements for reinforced concrete* (Note – Reference is also made to earlier editions of ACI 318)
ASTM A497–79	*Welded deformed steel wire fabric for concrete reinforcement*

OTHER PUBLICATION

New Zealand Ministry of Works and Development CDP 701/D September 1978 Highway Bridge Design Brief.

NOTE – In addition to the above, comprehensive lists of references are included at the end of each section of this Commentary.

COMMENTARY

C1 GENERAL

C1.1 Scope

Some special structures involve unusual problems which may not be covered by this New Zealand Standard Code of Practice.

General structural design and design loadings for compliance with NZS 1900, Chapter 8, are given in NZS 4203 for buildings. Design loads for highway bridges are given in Ministry of Works and Development Highway Bridge Design Brief.

C1.2 Interpretation

This Commentary is intended to be read in conjunction with NZS 3101 Part 1. It not only explains the provisions of the code of practice, but in certain cases it suggests approaches which satisfy the intent of the Code. At the end of each commentary section a list of references is provided to assist designers in areas where standard design procedures have not yet been formulated.

Clause numbering of the commentary is identical to that of the Code except that clauses are prefixed with the letter 'C'. A cross-reference such as "5.5.1.3" refers to that clause in the code of practice, while "C5.5.1.3" refers to the corresponding commentary clause.

C2 DEFINITIONS

C2.1 General

For consistent application of the Code, it is necessary that terms be defined where they have particular meanings in the Code. The definitions given are for use in application of this Code only and do not always correspond to ordinary usage.

Reinforced concrete is defined to include prestressed concrete. Although the behaviour of a prestressed member with unbonded tendons may vary from that of members with continuously bonded tendons, bonded and unbonded prestressed concrete are combined with conventionally reinforced concrete under the generic term "reinforced concrete". Provisions common to both prestressed and conventionally reinforced concrete are integrated to avoid overlapping and conflicting provisions.

By definition, plain concrete is concrete that contains less than the minimum reinforcement required by this Code.

A number of definitions for loads are given as the Code contains requirements that must be met at various load levels. The term "dead load" and "live load" refer to the unfactored loads (service loads) specified or defined by the local building or other code. Service loads (loads without load factors) are to be used where specified in the Code to proportion or investigate members for adequate serviceability as in 4.4, Serviceability. Loads used to proportion a member for adequate strength are defined as "factored loads". Factored loads are service loads multiplied by the appropriate load factors specified in NZS 4203 or other appropriate loadings code for required strength.

COMMENTARY

C3 GENERAL DESIGN REQUIREMENTS

C3.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 3 of the Code.

h_w total height of wall from its base to its top

ℓ distance between vertical reference axes of coupled shear walls

ℓ_w horizontal length of wall in the plane of bending

M structural material factor as in NZS 4203

M_o total overturning moment at base of shear wall structure due to lateral design earthquake loading

N total number of storeys

T axial load induced at the base of coupled shear walls by design earthquake loading

V_{code} shear force derived from lateral design earthquake loading

V_{wall} design shear force for shear walls

Z factor allowing for effect of h_w/ℓ_w ratio in eq. 3-B

ϕ strength reduction factor

ϕ_o flexural overstrength factor

ω_v dynamic shear magnification factor

C3.2 Scope

Various aspects of design, applicable to all reinforced concrete structures and members considered in subsequent sections of this Code, are specified in this Section. The general principles and design requirements are given in three groups. In 3.3 requirements applicable to all structures and for any type of loading are assembled. This is followed by additional principles and requirements in 3.4, applicable when seismic conditions need not be considered. Finally in 3.5 the additonal principles and general requirements for structures subjected to seismic loading are presented in considerable detail. In these clauses concepts of design strategy are listed. Subsequent sections of the Code follow this tripartite presentation of design requirements.

C3.3 General principles and requirements for analysis and design

C3.3.1 *Methods of design*

The general design provisions of the Code are referred to as "strength design". The strength to be provided is to be at least equal to the strength demand resulting from the appropriate application of factored loads, as specified by NZS 4203 or other loadings code. In this the strength reduction factors of Section 4 must also be used.

The "alternative design method" is only permitted to be used for components not subjected to seismic design considerations. Therefore this method is covered in 3.4.

When members of ductile structures are subjected to seismic distortions, they are generally to be designed for the maximum credible internal actions that may be generated. For this, "capacity design procedures", covered in 3.5, need to be applied, in which strength reduction factors are not used.

C3.3.2 *Arrangement of live load for buildings*

The Code permits the far ends of columns to be considered as fixed for the purpose of analysis under gravity loads. This assumption does not apply to lateral load analysis. However, in analysis for lateral loads, simplified methods[3.1] may be used to obtain moments, shears, and reactions for structures that are regular in their outlay and for which the assumptions of such simplified analysis methods are satisfied. For irregular or very tall structures, more rigorous methods should be used.

For gravity load analysis only it can be assumed that all restrained columns are fixed at the far ends, and the designer must investigate conditions of pattern loading to obtain the most severe cases. A two-cycle moment distribution method described in Reference [3.2] provides a convenient means of determining moments and shears under these provisions.

Most approximate methods of analysis constitute a "first order" method since the effects of deflections and axial deformations are not included. Therefore, when relevant, beam and column moments must be amplified for column slenderness in accordance with 6.4.10.

C3.3.3 *Assumptions and methods of analysis*

C3.3.3.1 In all design methods of this Code, moments, shears, reactions and the like are obtained from elastic analysis. However, the internal actions so obtained may be subsequently modified, when inelastic behaviour is expected or approximations are acceptable.

C3.3.3.2–C3.3.3.3 The suggested moment coefficients give reasonable conservative values for the stated conditions whether the flexural members are simply supported or are part of a frame or continuous construction. Because the load patterns that produce critical values for moments in columns of frames differ from those for maximum negative moments in beams, column moments must be evaluated separately. (See 3.3.6.1).

C3.3.3.4 The interpretation of the provisions for moment redistribution in continuous non-prestressed flexural members is illustrated with fig. C3.1 where it is shown how bending moments develop in an elastic-plastic member. As the load is increased, the beam behaves elastically until the plastic moment of one or more critical sections is reached (in fig. C3.1 (a) the support moments). Further loading causes these hinges to rotate while the moments ideally do not change. The extra moment required to balance the load is carried by other parts of the member, that is, at mid-span. This continues until the mid-span section reaches its plastic moment, when the structure becomes a mechanism and collapses. Figure C3.1 (b) shows the final bending moment diagram when all the critical sections are carrying their plastic moments. The percentage by which a moment is reduced from the elastic value is a measure of the rotation of the hinge. Design can therefore conveniently be done by carrying out an elastic analysis and then applying a limited amount of moment redistribution. The design of the critical sections must then be such that they can carry the rotations implied by the redistribution.

Condition (b) is required to deal with serviceability conditions, where an elastic response will be appropriate. From fig. C3.1, it is seen that service loading in this case will produce hogging moments in region 'x'. Ultimate load conditions require no reinforcement here and plainly wide cracks could develop in this region if it is not suitably reinforced.

The limitation of 30% in reducing moments is to restrict the rotation which will develop at critical sections. It should be noted that no limit is placed on the amount by which moments can be increased.

The effect of condition (d) is to limit the neutral axis depth at the development of ideal strength to 0.3 of the effective depth if the full 30% reduction in moment has been made. As the neutral axis depth is increased, the amount of redistribution is reduced. Where the neutral axis depth exceeds 0.6 of the effective depth, no reductions in moment are permitted.

These principles also apply when the load on the beams originates from gravity and seismic loading.

Where confining reinforcement is used to increase the rotational capacity of a section and special studies are made the limitation of eq. 3-1 may be exceeded but the moment reduction should not exceed 30%.

C3.3.3.5 Beam moments obtained at the centre lines of columns may be reduced to the moments at the face of supports for design of beam members. Reference [3.2] provides an acceptable method of reducing the moments at centre line to that at face of supports. The assumption to neglect the width of the beam in analysis should be considered carefully with unusually wide beams. In some circumstances, torsional effects could make this assumption unsound. The span lengths to be used in the design of two-way slab systems are specified in Section 11.

C3.3.3.6 These assumptions with respect to stiffness (see 3.3.5) may be made when an elastic analysis for the specified loading is carried out.

For monolithic T and L beams, only one half of the over-hanging parts of flanges, used for the evaluation of flexural strength in accordance with 3.3.6.2, should be included in the evaluation of the moment of inertia of the section. With this allowance flanged members with uniform depth, such as beams cast together with floor slabs, may be assumed to be prismatic. This assumption is intended to compensate for the fact that the effective widths of flanges will vary along the span and that over considerable length tension may prevail in the flange area.

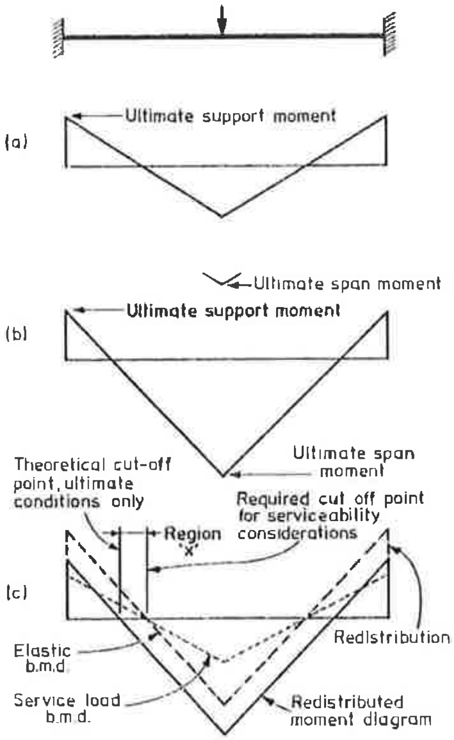


Fig. C3.1 BENDING MOMENT DEVELOPMENT IN AN ENCASTRE BEAM

C3.3.3.7 This Code does not specify an additional thickness for wearing surfaces subjected to unusual conditions of wear. Whether or not the separate finish is structural, the need for added thickness for unusual wear is left to the discretion of the designer.

As in previous editions of ACI Codes, a floor finish may only be considered for strength purposes if it is cast monolithically with the slab, but permission is now given to include a separate finish in the structural thickness if composite action is insured in accordance with Section 8.

All floor finishes may be considered for non-structural purposes such as cover and fireproofing. Provisions should be made, however, to insure that the finish will not spall off, thus causing decreased cover.

C3.3.4 *Material properties*

C3.3.4.1 Studies by Pauw [3.3] indicate that the modulus of elasticity of concrete with a density between 1 000 and 2 500 kg/m^3 can be represented with acceptable accuracy by the general formula stated. However, it is recognized that E_c can vary considerably.

C3.3.4.2 The value E_s = 200 GPa for non-prestressed steel represents a realistic average value obtained from many tests.

C3.3.5 *Stiffness*

C3.3.5.1 This clause allows the use of any reasonable assumption when computing the stiffnesses for use in a frame analysis provided the assumptions made are consistent throughout the analysis. Ideally, the sectional stiffnesses

EI and *GJ* should reflect the degree of cracking and inelastic action which has occurred along each member immediately prior to the onset of yielding. However, the complexities involved in selecting the ideal values of *EI* for all members of a frame would make frame analysis uneconomical in design offices. Hence, simpler assumptions are required to define the flexural and torsional stiffnesses for practical analyses.

For braced frames, only relative values of stiffnesses are required. The two most common assumptions in this case are to use gross *EI* values for all members or, to use half the gross *EI* of the beams and the gross *EI* for the columns. The latter assumption is more representative of the relative degree of cracking present at high loads. *EI* values for braced frames are discussed in References [3.4] and [3.5].

For frames which are free to sway, a more accurate estimate of *EI* may be desirable, and should be used if second order analyses are carried out. Guidance for the choice of *EI* for this case is given in C3.3.6.2.

Two conditions determine whether it is necessary to include the torsional stiffness in the analysis of a given structure: the relative magnitude of the torsional and flexural stiffnesses and, whether the torsion is required for equilibrium of the structure (equilibrium torsion) or simply arises due to members twisting to maintain compatibility of deformations (compatibility torsion). In the case of compatibility torsion, the torsional stiffness can frequently be neglected, while it generally must be included in the analysis if equilibrium torsion exists. Prior to torsion cracking, the torsional stiffness of a rectangular section will be in the order of 10 to 20% of its flexural stiffness. Torsional cracking will reduce the torsional stiffness of such a beam by 80 to 90% so that the torsional stiffness drops to a small fraction of the corresponding flexural stiffness. Because of this high reduction, the torsional stiffness of beams transverse to the frame being analysed are usually not considered in the analysis. It is, however, necessary to consider the torsional stiffnesses of edge beams in Section 11.

C3.3.5.2 Stiffness and fixed end moment coefficients for haunched members may be obtained from References [3.6] and [3.7].

C3.3.6 *Structural members*

C3.3.6.1 The Code requires that columns be designed for the loadings which produce the most adverse combinations of axial loads and moments. For gravity loads, the most general is the combination of factored loads on all floors above, which produces maximum axial force, and factored live load on a single adjacent span of the floor under consideration which produces the maximum axial load. In addition, it is required to consider the case which produces the maximum ratio of moment to axial load. This is generally the chequerboard loading pattern in multi-storey structures which results in maximum column moments but at a somewhat lower than maximum axial force. Because of the nonlinear nature of the column interaction diagram, both cases need to be examined to find which governs design of the column.

In structures where the loading patterns and type of structural system result in biaxial bending in compression members, the effect of moments about each of the principal axes must be considered. Design procedures for biaxial bending are outlined in Reference [3.10].

C3.3.6.2 The provisions for T beam construction are similar to those of previous ACI Codes for limiting dimensions related to stiffness and flexural calculations. Special provisions related to T-beams and other flanged members are stated in 7.3.7.7 with regard to torsion. Special provisions for seismic conditions are given in 3.5.5.

C3.4 Principles and requirements additional to 3.3 for members not designed for seismic loading

C3.4.1 *Method of design*

For engineers who prefer the alternative method for their designs, the provisions of Appendix B are reworded and expanded from those of ACI Code 318 to provide a more usable set of design provisions, including a table of permissible stresses. No changes in the design rules are made from those of the 1977 ACI Code.

The alternative method of design, as outlined in Appendix B, is similar to the working stress design method of previous ACI Codes. For members subjected to flexure without axial load the method is identical; differences in design procedure occur in design of columns, design for shear, anchorage length, and splices. No strength reduction factors are used, that is, ϕ is taken as 1.0.

The general serviceability requirements of the Code, such as the requirements for deflection control and crack control, must be met also when the alternative design method of Appendix B is used in proportioning for strength.

Since an Appendix is sometimes not judged to be an official part of a legal document (unless specifically adopted), specific reference is made in the main body of the Code to make Appendix B a legal part of the Code.

C3.4.2 *Joist construction*

C3.4.2.1 The size and spacing limitations for standard concrete joist construction meeting the limitations of 3.4.2.2 to 3.4.2.5 inclusive are based on successful performance in the past.

C3.4.2.5 A limit on the maximum spacing of ribs is required because of the special provisions permitting higher shear stresses and less concrete protection for the reinforcement for these relatively small, secondary members. The 750 mm maximum spacing of ribs follows the standards as given in "Types and Sizes of Forms for One-Way Concrete Joist Construction", NBS Voluntary Product Standard, No. PS 16-69, and "Forms for Two-Way Concrete Joist Floor and Roof Cosntruction", Simplified Practice Recommendation, No. R265-63, U.S. Department of Commerce.

C3.4.2.8 A 10% greater concrete shear stress is permitted for ribs than for other members. The increase is justified on the basis of

(1) Satisfactory performance of joist construction with higher shear stresses, as were designed under previous ACI Codes, which allowed shear stresses comparable to the 10% greater shear stress

(2) Distribution of local overloads possible because of the close spacing and relatively long spans associated with joist systems meeting the requirements of 3.4.2.2 and 3.4.2.3.

C3.5 Principles and requirements additional to 3.3 for the analysis and design of structures subjected to seismic loading

Whereas in the preceding clauses the general requirements of ACI 318-77 for the design of reinforced concrete structures have been adopted with relatively minor modifications, the additional seismic requirements presented in this clause and the corresponding clauses of subsequent sections are entirely new and markedly different from Appendix A of the ACI Code. These provisions intend to implement the requirements of NZS 4203. They are based on currently accepted design practice, on recent research findings that were available prior to 1980 and to an extent on expertise developed in New Zealand. The recommendations of other standards [3.8], information specifically related to earthquake resistant design in reinforced concrete structures [3.9, 3.10], studies of damage to buildings resulting from catastrophic earthquakes, namely Skopje (1963) [3.11], Anchorage (1964) [3.12], Caracas (1967)[3.13, 3.14], San Fernando (1971) [3.15], and the deliberations of the New Zealand National Society for Earthquake Engineering with respect to the seismic design of reinforced concrete frames [3.16, 3.17] and shear wall structures [3.18], have been considered in particular.

The earthquake loading, principles of seismic design, recommended analysis procedures and several other aspects of earthquake structural engineering are documented in detail in NZS 4203. Therefore the commentary of NZS 4203 should also be consulted when applying this Code. In particular, attention should be given to recommendations, specifically related to reinforced concrete buildings, such as:

(a) The aiming for structural symmetry and geometric uniformity in the planning of buildings

(b) The general requirements for ductility for the structure as a whole and for its components

(c) The development of desirable energy dissipating mechanisms in ductile moment resisting frames and shear wall structures

(d) The preservation of gravity load carrying capacity of members which are not required to be part of the horizontal force resisting system

(e) Capacity design in buildings where energy is expected to be dissipated by inelastic deformations involving flexural yielding

(f) The consideration of actions due to earthquake induced displacements concurrently occurring in the two principal directions of the framing.

No specific recommendations are made for the case where flexural members are slabs (plates) since the seismic behaviour of such members is still being studied. In general slabs are not likely to be useful as primary earthquake resistant elements in framed buildings because of the flexibility of the slab-column systems.

Numerous recommendations are made in the Code with respect to the detailing of primary earthquake resistant structural components because it is considered that this aspect of the design processes is one of the most important.

To clarify the meaning of various loads referred to in New Zealand Standards, in their commentaries and in the references quoted in these documents, the following definitions should be considered to apply to the seismic provisions of this Code of practice.

(a) Code loads are those specified by NZS 4203 for buildings or the MWD Highway Bridge Design Brief[3.22] for highway bridges

(b) Factored loads are those derived from the combinations of loads which in accordance with NZS 4203 are relevant to the strength method of design, which must be used. (See 3.5.1.2.)

(c) Capacity loads are those that, as a result of extreme seismic displacements, would be required to develop the flexural over-strength of members, sub-assemblages or the entire structure as appropriate, in accordance with the principles of capacity design.

(d) Design loads are those which determine the proportioning and detailing of structural components. Depending upon the requirements for the specific component the design load may be the capacity load or the factored load.

C3.5.1 *Methods of design*

C3.5.1.1 For seismic resistance this code considers three groups of reinforced concrete structural systems and requires specific design methods to be used for each.

Fully ductile structures for buildings, such as space frames or structural walls or the combination of these, and bridges, are those in which the lowest of seismic force resistance is provided.

Ductility in this context means ability to deform beyond the yield point into the plastic range of behaviour without excessive loss of strength. The magnitude of the ensuing damage will depend on the extent of yielding that is imposed during an earthquake.

In flexural ductile yielding the deflections into the inelastic range are provided by rotations at selected plastic hinges which normally form at points of maximum bending moments. The relevance of ductility is discussed in the Commentary of NZS 4203. NZS 4203 also provides the definitions associated with ductility, such as ductility factors. A definition of ductility factor for members or structures with no clearly defined yield point is given in Reference [3.22].

To ensure that energy will be dissipated at the selected localities only, capacity design procedures must be used. At specified potential plastic hinge localities, the ductility demand or availability, need not be computed if the detailed provisions of the subsequent sections are met, except for bridge structures in accordance with 3.5.9.

In certain structures, particularly those of irregular small buildings, it is often impractical to provide a structural system that could provide maximum ductility. For such situations structures of limited ductility are considered also to provide satisfactory protection against damage and, in case of an extreme seismic disturbance, collapse. Because such structures must still possess a considerable ability to be ductile, a capacity design procedure is the appropriate design method. However, as earthquake induced design actions are often not critical, a more conservative but less onerous design procedure is considered to lead to an equally satisfactory performance. For this reason the use of a modified strength design procedure, according to Section 14, is also permitted.

When a structure is designed to resist the much larger seismic forces, induced during its elastic response, corresponding with the appropriate structural type factors *S*, and materials factors *M*, required by NZS 4203, the strength method of design should be used. As no inelastic deformation is expected to occur in any part of the structure, the additional seismic detailing requirements, set out in subsequent sections, need not be satisfied.

C3.5.1.2 All provisions for minimum seismic requirements are based on the strength method of design and hence for these structures the Alternative Design Method, given in Appendix B, must not be used.

C3.5.1.3 In order to minimize the likelihood that brittle or other undesirable failures would occur while plastic hinges form during large earthquake induced inelastic displacements of the structure, the flexural overstrength of the potential plastic hinges needs to be evaluated. In accordance with the requirements of capacity design this is particularly important when the earthquake induced shear forces in beams (Section 7) or beam-column joints (Section 9) or moments and axial forces in columns of ductile frames (Appendix C3A to this commentary section) are evaluated. The actions derived in the capacity procedure will generally be greater than the corresponding actions calculated from the application of the design seismic loads of NZS 4203 or other appropriate code. The overstrength of reinforcing bars, however, need not be specifically considered when anchorage or splice requirements are established because these requirements are based on the full strength of any bar.

When using Grade 275 Flexural reinforcement the following relationships may be used to determine flexural strengths of beams:

Dependable strength	$\simeq$	0.90	Ideal strength
Probable strength	$\simeq$	1.15	Ideal strength
Overstrength	$\simeq$	1.25	Ideal strength
Overstrength	$\simeq$	1.39	Dependable strength
Probable strength	$\simeq$	0.90	Overstrength

where the ideal flexural strength is to be in accordance with Section 6, with the value of ϕ taken as unity. The use of probable strength properties is appropriate when, for special structures, the inelastic dynamic response to given earthquake excitations is being studied.

When using Grade 380 flexural reinforcement, greater enhancement of strength due to strain hardening is to be expected, and accordingly the above strength relationships should be modified as follows:

Overstrength $\simeq$ 1.40 Ideal strength

For prestressing steel the following assumptions may be used:

Probable strength $\simeq$ 1.05 Ideal strength
Overstrength $\simeq$ 1.10 Ideal strength

unless the properties are determined from the stress-strain relationship obtained for the prestressing steel to be used.

C3.5.1.6 Inertia forces due to distributed masses at floor levels need to be transferred to the lateral force resisting ductile frames or shear walls. It is necessary therefore that the connecting elements (diaphragms), which are normally non-yielding components, possess adequate strength and continuity for stress transfer. Hence the arrangement of openings, if any, must be rational and the connections must be adequate.

C3.5.1.7 The state of the art in seismic design is changing rapidly. More detailed studies of seismological data and structural response, made feasible by computers, are likely to lead to more realistic and necessarily more sophisticated methods of assessing seismic requirements. This Clause intends to accommodate such future useful developments. However, alternative systems or design methods should be supported by thorough analytical or experimental studies or both, to assure a proper combination of earthquake input and strength and ductility requirements. The acceptance or rejection of such alternative methods lies with the Engineer.

C3.5.2 *Seismic loading*

C3.5.2.1 For various levels of available ductility, NZS 4203 specifies appropriate values for the structural type factor *S*. Some of the equations in subsequent sections, which express dependence on ductility, incorporate this factor. The value used must be the same as that used in the derivation of the base seismic shear for the structure. Amendments to NZS 4203 may be published after the release of this Code. Therefore designers must ascertain that the appropriate structural type factors used are those given in NZS 4203. For the convenience of the designer the structural type factor *S*, relevant to each type of reinforced concrete structure covered in this Code, is listed within the appropriate clause of the commentary for this Section only. The structural material factor *M* to be used in conjunction with the *S* factors listed in this commentary are given in C3.5.4.1.

C3.5.2.3 Traditionally earthquake forces have been considered to act independently in the two principal directions of the framing system. However, NZS 4203 requires the effects of large inelastic displacements in any direction to be considered. Therefore:

(a) When significant inelastic displacements occur simultaneously in both principal directions of the framing system, it is considered necessary to make provisions for the simultaneous hinging of beams or yielding of all diagonal braces framing into a column or wall during a severe earthquake attack. The purpose of this procedure is to safeguard components, that are required to sustain the primary energy dissipating mechanisms, against premature failure. Columns subject to biaxial bending through two or more plastic hinges of beams at adjacent faces of the column, and to skew shear effects, are typical examples.

(b) In capacity design procedures the maximum load input from beams into columns or walls are considered. If additional factors, such as dynamic magnification of actions due to higher mode response or an artificial increase of design moments or forces to compensate approximately for concurrency effects, are considered, the design of components may be carried out on the bases of a unidirectional earthquake attack, applied separately in each of the two principal directions of the framing. This may involve less onerous computations. The Appendix to this commentary section sets out such a procedure for the columns of two-way ductile moment resisting frames. Where appropriate, specified mention is made in the subsequent sections when this design approach may be considered to have satisfied the intent of 3.5.2.3.

C3.5.3 *Assumptions and methods of analysis*

C3.5.3.1 Whenever the strength method of design is used to satisfy the strength requirements for static or dynamic seismic analysis, permitted for structures of limited ductility (3.5.1.1 (b)) or elastically responding structures (3.5.1.1 (c)), the strength reduction factor ϕ, specified in Section 4, must also be used.

C3.5.3.2 For ductile structures or structures of limited ductility, the design must be based on the assumption that under combined gravity and earthquake loading, the lateral deflections will be large enough to develop a complete plastic hinge mechanism in the structure. The lateral earthquake loading must be considered in both directions and hence this will usually involve reversed plasticity in plastic hinges.

C3.5.3.3 In capacity design the maximum likely actions, that could ever develop in a plastic hinge or in adjacent member or members, are being considered when the design of the member in question is being considered. For this extreme loading case, when significant inelastic deformations and damage in various parts of the structure are to be expected, additional reserve strength is not considered to be necessary. Therefore the ideal strength of a member should be equal to or more than the load demand derived from a capacity design procedure. (See C3.5.1.3.) Consequently in capacity design, strength reduction factors need not be used, or alternatively, where relevant it should be assumed that $\phi = 1$.

C3.5.3.4 Some of the advantages of moment redistribution for gravity loading were recognized and discussed in C3.3.3.4. The same principles and requirements also apply where seismic loading is being considered. The additional considerations for inelastic earthquake response are outlined in the following paragraphs.

Because beams at potential plastic hinges are required to be designed for maximum ductility, it is desirable that the benefits of moment redistribution be used. The purpose of moment redistribution in seismic design is to reduce the absolute maximum moment, usually in the negative moment region, and compensate for this by increasing the moments in the non-critical (usually positive moment) regions of a span. Thus a better distribution of strength demand along a span is attained. Moreover the critical moments at either side of a column, usually for different directions of lateral loading, can be equalized. This obviates the desirability of having to terminate and anchor top beam bars at interior column-beam joints. Also the positive moment potential of beams at column faces can be more fully utilized. The maximum moment that can be redistributed is defined, similarly to the requirement of 3.3.3.4 (c), as 30% of the maximum moment ordinate, derived from an elastic analysis, for either of the following load combinations, $1.4D + 1.7L_R$ or $E + D + 1.3L_R$ or $E + 0.9D$, as specified by NZS 4203. The redistribution involves plastic rotation at potential plastic hinges of beams only.

It is intended that moment redistribution should not change the magnitude of the shear force, derived from an elastic analysis for the design earthquake load E only, and to be resisted across any column of a bent, by more than 30%. This requirement may be considered to be satisfied when the sum of the end moments in a column, below and above a floor, obtained after the redistribution of adjacent terminal beam moments for any load combination listed in C3.5.3.4 (a), is not reduced to less than 70% of the sum of the corresponding column end moments derived for earthquake loadings E only. The redistribution of moments or shear forces between columns also involves plastic rotations at potential plastic hinges in the continuous beam. No plastic rotation within any column is implied. It is important to ensure that no shear force due to earthquake loading is "lost" in the process, otherwise lateral strength requirements are violated.

From considerations of equilibrium for any span it is evident that if the support moments of a given bending moment pattern are changed then corresponding adjustments must be made in the midspan region of the beam.

Moments are sometimes derived from nominal terminal moment values, such as given in 3.3.3.3. As these values may well involve some moment redistribution this Section prohibits the application of additional moment redistribution.

The full mechanism of an earthquake resisting structure, consisting mainly of structural cantilever or coupled walls, will comprise plastic hinges at the base of all these walls. It may be advantageous to allocate more or less lateral load to a structural wall than indicated by the elastic analysis. Through the formation of simultaneous plastic hinges in all walls this is possible, and therefore a moment redistribution of up to 30% is permitted. The designer must ensure that in the process the total lateral strength of the assembly of structural walls is not reduced.

C3.5.4 *Material properties*

C3.5.4.1 In accordance with NZS 4203 the structural material factors M to be used together with the appropriate structural type factor S are as follows:

(a) Reinforced concrete 0.8

(b) Prestressed concrete (when used in elements which resist seismic forces and moments by flexural yielding) 1.0

C3.5.4.3 The specified yield strength of reinforcement is limited to 415 MPa because higher strength steels used in New Zealand do not as yet have a satisfactory yield plateau.

The reinforcement used in areas where yielding due to seismic actions can occur, must not have a higher specified yield strength than that called for in the plans and specifications for each particular placement in the structure.

C3.5.4.4 When Grade 275 reinforcement is specified for a particular member, then it must not be replaced by Grade 380 or any other steel without special approval. The unintended use of higher grade steels may result in unintended large internal forces during inelastic displacement of the structure, for which that component or adjacent components will not have been designed.

The longitudinal reinforcement in beams or bridge piers which are normally the primary sources of energy dissipation, should consist of Grade 275 deformed bars to ensure that the intended large section ductilities can be developed without excessive gain in strength due to strain hardening. Grade 275 steel from overseas sources should only be used if reliable statistical information with respect to its average yield strength and overstrength is available.

If Grade 380 deformed bars are used in beams of ductile frames or in shear walls, the overstrength of the bars should be estimated as in C3.5.1.3. In addition more severe dimensional limitations apply in joint regions (5.5.2.5) because of the critical conditions for bond.

In columns, shear walls, diaphragms, or members which are not part of the primary lateral force resisting system or in which yielding under seismic load conditions is unlikely, such as footings and foundation walls, Grade 380 can be used efficiently. In such elements either the expected ductility demand is moderate or the gain in strength during large straining is not detrimental to the response of the structure.

C3.5.4.5 Under inelastic reversed load conditions, to be encountered during earthquakes, the bond performance of plain bars is considered to be unsatisfactory. Therefore deformed bars must be used for flexural resistance in all members that could be affected by seismic loading.

C3.5.4.6 Plain round reinforcement, generally of Grade 275, must be used for stirrups, ties or hoops which are anchored by bending around the longitudinal beam or column reinforcement. In deformed bars, the forming of such hooks may lead to cracking in the bar at the roots of ribs at the inside of the bends. When hooks of deformed bars are over-bent and subsequently straightened, a complete fracture may result. Grade 380 plain bars may also be used but a tight engagement of bends around longitudinal bars must be assured without violating the requirements of 5.3.4. A simple permanent identification of such bars is also essential.

It is not intended that the use of deformed bars for stirrups in deep foundation beams, shear walls or bridges be prohibited. Bends and hooks at the ends of such bars must be in accordance with table 5.2.

C3.5.5 *Stiffness*

C3.5.5.1 Structural deformations due to seismic loading will generally be associated with high stresses. Consequently extensive cracking in the tension zones of reinforced concrete beams, columns or walls must be expected. The estimation of deflections for the purposes of determining periods of vibration or satisfying requirements for structural separations and limitations on inter-storey drifts, will be more realistic if an allowance for the effect of cracking on the stiffness of members is made. Typically the moment of inertia of a beam section may be based on 50% of the moment of inertia of the gross concrete area, whereas for columns carrying significant axial compression, 100% of the corresponding moment of inertia may be assumed. Similar allowances may be made for the reduction of area of a member if extensional deformations are to be estimated. The allowances for the effects of cracking on stiffness must be consistent through the structure.

C3.5.5.2 For shear wall structures shear deformations and distortions in the anchorages and foundations may also need to be considered and for this purpose Reference [3.18] may be used.

C3.5.6 *Ductile moment resisting space frames*

The required values of the structural type factor S for ductile frames, designed in accordance with capacity design principles and detailed in accordance with the additional seismic principles and requirements of this Code should not be less than 0.8.

C3.5.6.1 The lateral load in such frames is resisted mainly by flexural action of the beams and columns. Those members, which are selected and designed to contribute to the strength, stiffness and energy dissipation by means of plastic hinges of the system, are referred to as primary members. The performance of precast members and in particular their connection in ductile frames has not been explored sufficiently to permit their use without special examination.

C3.5.6.2 The energy dissipation, necessary for a multi-storey frame to survive a severe earthquake, should in general occur by the formation of ductile plastic hinges in beams. This is because the curvature ductility demand of the plastic hinges in columns can be unacceptably high if a column side sway mechanism forms with simultaneous plastic hinges at the top and bottom of the columns of one storey, whereas if plastic hinges develop in the beams up much of the height of the frame the curvature ductility demand in the beams is less severe and can be more easily provided. NZS 4203 permits column side sway mechanisms only in the case of single or two-storey buildings, and in the top storey of multi-storey buildings, because the curvature ductility demand of plastic hinges in columns in these cases is not high and can be met. Also, plastic hinges must be expected to develop in the columns at the base of multi-storey buildings. Therefore, apart from these exceptions, a "strong column – weak beam" concept of design should be adopted.

C3.5.6.3 C3.5.6.4 Because of difficulties in repair and the additional transverse reinforcement required in potential plastic hinge zones, these clauses require that, with the exceptions in 3.5.6.10, plastic hinges in columns should not develop before those in beams. The intent is to minimize the likelihood of plastic hinge formation in columns of intermediate storeys altogether.

There are difficulties in fulfilling such a design aim in multi-storey frames. Large bending moments in columns can be caused by the combined effects of beam overstrength, concurrent earthquake loading, and higher modes of vibration which cause the points of contraflexure to shift well away from the locations indicated by analysis for static code loading. The difficulty of providing sufficiently high column strength to completely protect the columns is such that in a practical design some yielding of columns during a severe earthquake must be considered as a possibility. Note that column yielding resulting from shifts of the position of the points of contraflexure away from the locations indicated from analysis for static code loading, caused by higher mode effects, should occur at only one end of columns in a storey and thus should not lead to a column sidesway mechanism in that storey. Also, significant ductility demand will not occur in a column of a frame if a plastic hinge or plastic hinges have already developed in adjacent beam or beams.

It is evident that the determination of the design column actions in multi-storey frames is uncertain because of the dynamic and random nature of seismic loading and the level of protection necessary. When earthquake effects are determined from equivalent lateral static forces in accordance with NZS 4203 the procedure used in Appendix C3A to this commentary section may be used to determine column actions. This procedure is considered to give large protection against hinging but columns will still need to be detailed for some ductility at potential plastic hinge regions since some yielding can be expected to form there during instants of a severe earthquake.

C3.5.6.5 Columns subjected to small axial compression or to axial tension are very ductile. To alleviate the large demand for flexural reinforcement in such typically exterior columns, the design moments under these circumstances may be reduced, thus permitting relatively early yielding in such columns only, provided that the associated loss of shear resistance in that column is insignificant in terms of the seismic shear resistance of the entire bent. Appendix C3A contains recommended values for such reductions in design moments.

C3.5.6.6 The earthquake induced axial load input into a column at any floor must be based on the overstrength shear forces developed in the beams that frame into that column. The procedure is described in C7.5.1.

C3.5.6.7 A severe design condition may arise, particularly with increasing number of storeys, if the earthquake induced design axial load on the columns is based on the assumption that all beams over the full height of the frame develop plastic hinges simultaneously at flexural overstrength. In such cases a reduction of the maximum feasible earthquake induced axial load is permissible. Similar procedures have been adopted in other codes, which allow a reduction of the total overturning moment resulting from the equivalent lateral static load [3.20]. Unless the column design moments are magnified to allow for dynamic effects, the design axial load due to earthquake should not be less than that computed with an elastic analysis for the equivalent lateral static load of NZS 4203. One procedure for the estimation of axial design load is suggested in Appendix C3A.

C3.5.6.8 Because of uncertainties involved in the assessment of the flexural response of columns (C3.5.6.3) there is no unique procedure available at present by which the column shear forces can be determined. Because of the brittle nature of shear failures, conservatism must prevail in the design of columns for shear. The intent should be to design for the maximum shear that can be placed on the column, whether it is governed by the hinging overstrength of the column itself or by the flexural overstrengths of the beams framing into the column, whichever is less. It is likely that the procedure suggested by SEAOC [3.8] is unnecessarily severe for columns of frames designed in accordance with this Code. Suggested estimates for the column design shear are also made in Appendix C3A.

C3.5.6.9 As the flexural overstrength of a frame with respect to the equivalent lateral load of NZS 4203, is at least 40% in excess of the code load, when Grade 275 flexural reinforcement is used in the beams, the minimum dependable shear capacity of the columns should match this load.

C3.5.6.10 The lateral load resistance of frames, particularly in low rise buildings or in the upper storeys of multistorey structures, may be considerably larger than that required by the earthquake provisions of NZS 4203, because gravity load considerations may have governed the sectional properties chosen. If the "Strong column–weak beam" structural system is used and combined with capacity design procedures, column design moments and shear forces may become unnecessarily excessive. In such cases a partial beam sway mechanism, with at least one plastic hinge in each exterior span, is considered to be acceptable. For a complete sway mechanism to develop in any storey it is necessary that plastic hinges also occur at the top and bottom of some or all interior columns. However, the full development of such a mechanism at any floor should not occur without being accompanied by the development of the combined dependable flexural strength of all potential hinges, equal to or in excess of that which would be required to resist moments resulting from the application of twice the code specified lateral static loading. Hence the lateral static load, corresponding with the dependable strength of such mechanism, should not be less than that obtained with the use of a structural type factor of S = 2 x 0.8 = 1.6. Additionally, exterior columns should be protected against the formation of plastic hinges, with the exception of the column base and the top storey. This may be achieved by magnifying the moments derived for the exterior columns from the code earthquake loading, as described in Appendix C3A. All potential plastic hinges in interior columns must meet fully the additional seismic requirements of this Code. The ideal shear strength of interior columns, in which simultaneous hinging in a story is designated, should be based on the development of their flexural overstrength at both ends with the corresponding critical axial load acting on the column. These provisions are principally intended for three to four storey ductile frames with large column spacings.

These provisions are not intended to apply to the design of frames in which deformations are controlled by shear walls or substantial earthquake load-dominated external frames. For these frames the use of S = 0.8 would be adequate, provided the internal columns were fully confined as required for yielding members. Nor are the provisions intended to apply to frames designed as secondary elements in accordance with 3.5.14.

Where external columns are weak, such as may result when the contribution of adjacent beam hinges to the total overturning moment is small, the provisions of this clause may not be adequate for tall frames because a mechanism involving the columns of one storey only may result. In this case special study, leading to larger lateral load resistance of the frame, will be required.

C3.5.6.11 It was pointed out in 3.5.6.2 that column sway mechanisms in one or two storey frames are permitted because the curvature ductility demands in such columns can be met if they are designed in accordance with 6.5. If the flexural strength of beams which frame into such hinging columns is comparable with that of the columns, so that yielding in beams is also possible, the beams must also satisfy the additional seismic requirements of Sections 6, 7 and 9. However, if, with the rigorous application of capacity design, the possibility of beam yielding under the most adverse seismic load conditions is eliminated, then such beams are exempted from the additional seismic requirements of this Code of practice. In this case the earthquake induced shear forces in the beams must be based on the moment input from the adjacent plastic hinges of the columns at their overstrength. Beam-column joints, however, should be designed in accordance with Section 9.

C3.5.6.12 Components, which are designed so as not to contribute to resistance against seismic loading or displacements, must be provided with ductile end-connections of sufficiently low resistance to ensure that no significant damage occurs due to the largest expected inelastic deformations in the ductile frame. In particular, such members must not interfere with the proper functioning of the primary lateral load resisting members to which they may be connected. Secondary members must also sustain satisfactorily at all times the gravity load assigned to them. Cast-in-place or precast mullions or lintels, the flexural stiffness of which are neglected in the lateral load analysis, are typical examples. Cast-in-place secondary columns, intended to carry axial compression only may be assumed, for example, to develop plastic hinges simultaneously at the top and the bottom of a storey. However, the confined core of such a plastic hinge zone should be designed to carry the factored dead and live load with or without earthquake induced axial forces. Moreover, no yielding should occur in the primary beams at or near the connection as a result of moment transfer from such a secondary column. The deformation of such prop-columns must be controlled by the primary lateral load resisting system which should therefore contain much stiffer vertical elements. The ideal shear strength of such columns should be based, however, on the flexural overstrength of the end hinges. Clause 3.5.14 sets out detailed requirements.

C3.5.7 *Ductile shear wall structures*

C3.5.7.1 The important rôle of shear wall structures in resisting lateral seismic forces and in controlling seismic deformations in buildings is discussed in the commentary on NZS 4203. The required structural type factors S are as follows:

(a) Two or more cantilever shear walls

$$S = 1.0Z \leqslant 2.0$$

(b) Two or more ductile coupled shear walls

when $A \geqslant 0.67$ $\quad S = 0.8Z \leqslant 2.0$

$A \leqslant 0.33$ $\quad S = 1.0Z \leqslant 2.0$

with linear interpolation between these values,

Where $A = T\ell/M_o$ . . . . . . (Eq. 3-A)

(c) Single cantilever shear wall

$$S = 1.2Z$$

For items (a) to (c)

$1.0 \leqslant Z = 2.5 - 0.5\, h_w/\ell_w \leqslant 2.0$ . . . . . . (Eq. 3-B)

Detailed design requirements for walls are contained in Section 10. In the light of recent studies [3.18, 3.21], it is considered that allowances should be made for the magnification of bending moments and shear forces derived from equivalent static loading, due to the dynamic response of shear wall structures. Therefore corresponding recommendations are made in the subsequent clauses.

C3.5.7.2 The principal aim of the designer should be to ensure that energy dissipation in all types of shear walls will be predominantly by flexural yielding. This should be the overriding concern in assessing likely performance rather than geometrical limitations such as height to horizontal length ratio limitations. Even squat shear walls can be made ductile [3.10]. Controlled diagonal cracking may contribute to energy dissipation and damping but as yet there are no reliable design techniques by which such responses could be quantified.

C3.5.7.3 Capacity design procedures with respect to shear wall structures relate to the estimation of maximum shear forces that could be generated when the chosen flexural energy dissipating mechanisms are operating at overstrength.

The potential plastic hinge will generally be located at the base of the walls. In evaluating its overstrength, the contribution of all Grade 380 or Grade 275 longitudinal reinforcement should be considered. Studies [3.21] have indicated that the shear forces induced during the dynamic response of cantilever shear walls may be considerably higher than the values obtained from the equivalent lateral static loads even when these are scaled up to correspond with the flexural overstrength of the base hinge. It is therefore recommended that the shear forces at all levels of shear walls be obtained from

$V_{wall} = \omega_v \phi_o V_{code} < 4\, V_{code}/S$ . . . . . . (Eq. 3-C)

where the value of dynamic shear magnification factor ω_v is given in table C3.1, and the flexural overstrength factor is

$$\phi_o = \frac{\text{Overstrength moment of resistance}}{\text{Moment resulting from code loading}}$$

where both moments refer to the base section of the wall.

Table C3.1 DYNAMIC SHEAR MAGNIFICATION

Number of storeys	ω_v
1 to 5	$0.1N + 0.9$
6 to 9	1.5
10 to 14	1.7
15 and over	1.8

NOTE – N = number of storeys

Based on recent studies[3.18], particularly those of the Portland Cement Association in Skokie, U.S., it is also recommended that the vertical flexural reinforcement in walls of shear wall structures should be curtailed so that the wall's ideal moment of resistance reduces linearly from the end of the potential plastic hinge zone to the value of the design moment at the top of the structure. This procedure is likely to eliminate the possibility that extensive wall yielding would occur due to higher mode dynamic response at any level above the potential plastic hinge at the wall base. The interpretation of this recommendation is shown in fig. C3.2 for an example shear wall structure

C3.5.7.4 To ensure that the large ductilities that may be required in coupling beams can develop without excessive overstrength, preferably Grade 275 steel, should be used when reinforcing them, as required in Section 10.

The desired energy dissipation in coupled shear walls can be expected if the axial forces, derived from an elastic analysis for the seismic loading required by NZS 4203, resist at least two-thirds of the over-turning moment as expressed by eq. 3-A, at the base of the structure. This is only possible if the coupling beams are sufficiently stiff in relation to the walls. In this case the structural type factor $S = 0.8Z$ is appropriate (see C3.5.7.1). Such a system is likely to ensure that during a severe earthquake most of the beams will yield before the walls, thereby minimizing wall damage. To maintain this primary energy dissipating system, it is necessary that the walls sustain the axial loads introduced by the coupling beams at their flexural over-strength together with the moments at the base, so as to resist at least 1.4 times the overturning moment at the base due to the lateral static loading required by NZS 4203. The details of the design of coupled shear walls are discussed in reference [3.18].

Design moments may be reduced by redistribution but only from tension walls to compression walls and within the limits permitted in 3.5.3.4 (f). In the estimation of the design shear, recommended in C3.5.7.3, such redistribution of moments of resistance should also be considered.

When the coupling system is not efficient in transmitting shear forces from one wall to another, the walls may become the primary energy dissipating elements during seismic inelastic displacements. Walls coupled by floor slabs only are typical examples. For such structures the higher structural type factors, S, listed in C3.5.7.1 for one or more ductile cantilever walls are appropriate.

C3.5.8 *Ductile hybrid structures*

It is not possible as yet to give detailed guidance for structures consisting of different sub-assemblages, such as ductile frames and cantilever or coupled shear walls, to be

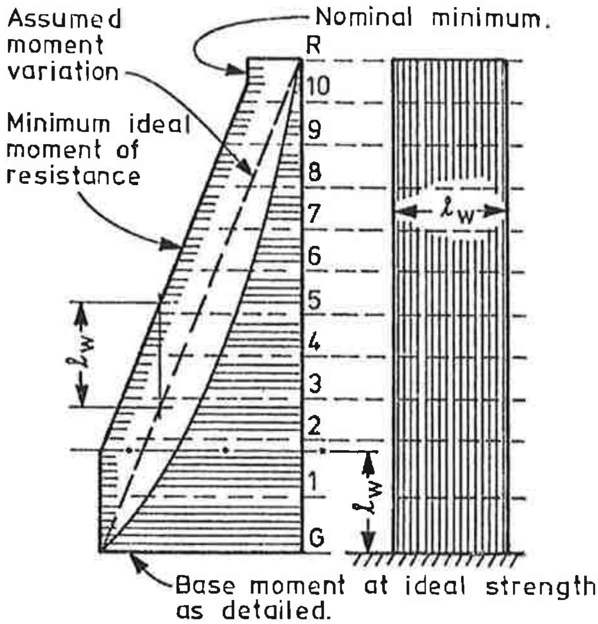


Fig. C3.2 RECOMMENDED DESIGN BENDING MOMENT ENVELOPE FOR CANTILEVER SHEAR WALLS

designed for earthquake resistance. The designer must use established engineering principles and the general guidelines of this Code. The design forces should be allocated to each element of a hybrid structure in proportion to the relative stiffness of that element, taking also torsional or concurrency effects into account. However, in cognizance of the inelastic response of ductile structures, some plastic redistribution from potentially weaker to potentially stronger elements may be considered. It is important that the primary energy dissipating elements be clearly identified. Capacity design procedures should be used in respect of all elements as well as the structure as a whole, whenever the design shear forces are evaluated. The flexural strength requirements for components may be satisfied by either strength design or capacity design procedures, depending on the consequences of flexural yielding or plastic hinge formation in that element on the overall structural response during severe seismic excitation.

When shear walls dominate the deformed shape of buildings that are subject to seismic loading, the columns of ductile frames, which interact through the diaphragm action of floors or roofs, should be subject to capacity design. However, no dynamic magnification of column moments, as suggested in Appendix C3A, should need to be considered in such structures.

C3.5.9 *Ductile bridge structures*

C3.5.9.1 Design loadings for highway bridges are specified in the MWD Highway Bridge Design Brief [3.22].

Design loadings for railway bridges are normally specified by the New Zealand Railways Department, by specific instruction for the project concerned.

In the context of this Code "ductile" bridge structures are those which form a plastic hinge mechanism with a long plateau in the force–displacement relationship, which can be sustained over several cycles of reversed loading. A structural system which does not fall either into the "ductile" category or the "limited ductility" category of 3.5.10 may be called "partially ductile". For such a system there is a significant upward slope in the force–displacement relationship in the post-yield range. This characteristic would exist when, considering displacements in the longitudinal direction, piers yield in flexure and elastomeric bearings at the abutments remain elastic. For such a system the provisions of this Part of the Code should be applied with discretion, taking into account the relationship between cost and benefit gained in seismic protection.

The capacity design requirements of this clause will often result in a structural strength which exceeds the minimum design requirements. In some cases the margin will be large because of other design considerations – for example, the required strength along another axis, or another loading condition such as eccentric live loading, governing the pier design. In all cases the design value for seismic loading need not exceed that equivalent to elastic response acceleration of the structure.

The current Highway Bridge Design Brief [3.22] specifies seismic loading assuming a structure ductility of 6. Other curves are also included to provide upper limits of seismic loading which need be applied. It is probable that the next revision of this loading code will include loading curves presented in a similar form to those in Reference [3.23]. These curves allow a better understanding of the possibilities of strength/ductility relationships, and relate more realistically to the likely reduced structure response accelerations of longer period structures.

C3.5.9.2 Members that are selected and designed to contribute to the energy dissipation, necessary to survive a severe earthquake, are referred to as primary energy dissipating members. In general these members will be the piers rather than the members forming the foundations, because of the greater accessibility for inspection and repair that the piers offer. Other structural elements, referred to as resisting members, should be provided with sufficient strength to ensure that the chosen energy dissipating mechanism is maintained throughout the deformations expected to occur during a severe earthquake.

Recommended margins of flexural strength between yielding and resisting members are included in Section 3 of Reference [3.23].

C3.5.9.3 The most adverse axial load, including the effects of frame action and vertical accelerations, should be considered in calculating the flexural reinforcement required in all members. Vertical acceleration effects are allowed for in the loading combinations of Reference [3.22].

C3.5.9.4 Recommendations for a maximum design coefficient of friction for sliding bearings and a suitable margin of strength are included in Section 3 of Reference [3.23].

C3.5.9.5 Where shear forces in elastomeric bearings induce moments in primary energy dissipating members, the design requirements are covered by 3.5.9.2. The structural system applicable in 3.5.9.5 corresponds to one with significant post-yield stiffness as described in C3.5.9.1. Suitable margins of strength are included in Section 3 of Reference [3.23].

C3.5.9.6 In calculating the maximum shear forces likely to develop in structural members during a severe earthquake, the most adverse axial load, including the effects of frame action and vertical accelerations, should be used for deriving overstrength member flexural strengths. Vertical acceleration effects are allowed for in the loading combinations of Reference [3.22]. Because of the brittle and therefore serious nature of shear failures, the possibilty of shear forces in some members exceeding those derived from the plastic analysis of 3.5.9.2 should be considered where there are uncertainties in the assessment of structural responses. This is most likely to occur in foundation members such as piles, where properties and deformations of the soil are less reliably predicted. Considerations should be given to assuming that overstrength plastic hinging might occur within such a member instead of in the intended location. The ideal shear strength of the member should be made to equal or exceed the resulting shear force. Design judgement is necessary to balance structural benefits against economic consequences in such cases.

C3.5.9.7 The required design structure ductility factor is an integral part of the design procedure and the design loading. This is set out in Reference [3.22]. Conclusions from recent tests on reinforced concrete columns to investigate available member ducility, are summarized in Section 5 of Reference [3.23]. It is important for the designer to consider factors which may increase ductility demand on plastic hinges (for example, flexible foundations) and to ensure that the necessary curvature ductility is available. This may simply be a matter of checking that demand is less than that known to be available, or it may require more detailed evaluation of available ductility. Background material and a simple design office procedure are contained in reference [3.24]. A revision of this reference, based on more recent test data, is in progress.

C3.5.9.8 Recommended clearances between major structural components and around items such as holding down bolts and linkage bolts, designed for relative movement, are given in Reference [3.22].

It is accepted that damage caused by a severe earthquake may occur in elements that are not main structural members. In this case it may be desirable to introduce a plane of weakness to allow damage to occur in a predetermined and limited manner. This should permit early use of the bridge after a major earthquake. For longer period structures the values specified are excessive because of the characteristics of the loading curves. The curves in Section 2 of reference [3.23] are considered to result in more realistic calculated displacements.

C3.5.9.9 It is important to ensure that structural integrity is maintained during a severe earthquake. Recommended design forces for linkage bolts and holding down devices are given in reference [3.22]. Other options, such as the linkage slab shown in Section 8 of Reference [3.23], should also be considered.

C3.5.10 *Structures of limited ductility*

The configuration of structural components, the interaction of different lateral load resisting structural systems and functional requirements may result in a structure which may possess only limited ductility. This may arise particularly in low rise buildings. For such buildings the added complexity in the design, involved when complying with the additional seismic requirements of this Code, may not be warranted. Therefore, while providing guidance for special problems and design aspects, the Code endeavours to establish simple and yet not unduly conservative rules for structures classified in 3.5.1.1 (b) the details of which are given in Section 14. However, the more important general principles and requirements are given as follows:

(a) In recognition of the limited ability of this class of structures or components to be sufficiently ductile, greater lateral load carrying capacity must be provided. This is achieved with the use of larger structural type factors S, as follows:

(1) Framed concrete structures not exceeding four storeys or five storeys with a roof in light construction 2.0

(2) Shear wall structures 2.0

(3) Elastically responding concrete structures 4.6

It was considered to be impractical to define more precisely appropriate S factors for all types of structures of this class that will occur in practice. Therefore in many situations engineering judgement will be required. For example, for deep membered frames or shear walls which do not lend themselves to rational analysis because of significant and irregularly arranged openings, a structural type factor larger than 2 may be appropriate. Framed buildings taller than four storeys or five storeys when a light roof is used, must be designed as fully ductile structures as required by 3.5.6

(b) It is to be noted that selected members of this class of structures must still be ductile. Therefore the additional requirements for detailing to achieve ductility, must be complied with. However, the requirements for fully ductile structures are somewhat relaxed in Section 14 in recognition of the lower ductility demand

(c) Because ductility and consequent detailing is provided, where specifically selected by the designer, capacity design procedures are also relevant to structures of limited ductility. This is likely to lead also to the most economical solution. Because in certain members the potential overstrength may exceed the load demand that would arise during the fully elastic response of the structure to an expected earthquake, Section 14 allows certain maximum values for actions to be used. Generally, actions derived from capacity procedures need not exceed values corresponding with $S = 6$

(d) To simplify design procedures, the code also allows the strength design method, together with the appropriate strength reduction factors, to be used. To ensure that no premature brittle failures can occur, Section 14 specifies, however, certain restrictions particularly relevant to shear strength. This simplified design procedure is necessarily more conservative than that of capacity design.

C3.5.12 *Foundations*

C3.5.12.1 The concrete foundation structure should be capable of sustaining the gravity loads even after the largest expected earthquake. Therefore it should be protected against earthquake actions that, as a result of damage, could reduce its strength [3.18, 3.29].

C3.5.12.2 *Elastic foundations supporting ductile superstructures.* When the superstructure, consisting of ductile components, is assigned to dissipate energy by selected mechanisms, at a load level exceeding that which corresponds to the appropriate value of SM, it is necessary that it be supported on foundations which can transmit the ensuing actions without deterioration of strength. This may be achieved by ensuring that the foundations remain elastic. This system is particularly applicable to ductile frames. The aim may be achieved by:

(a) Determining the maximum actions which can be transmitted to the foundation structure by elements of the superstructure. Capacity design procedures, used to complete the design of the superstructure, reveal these earthquake induced actions. Appropriate factored gravity loads must be included in determining the overstrength load on the foundation and the corresponding earth pressures or reactions in piled foundations

(b) Providing ideal strength in each member of the foundation structure, equal to or larger than the strength demand that results from the loading determined in accordance with (a)

(c) Detailing the foundation structure so as to resist the design actions at or just below yield strength of the reinforcement. As inelastic deformations are not expected to occur, the Code does not require the additional seismic requirements to be met. Hence in members of the foundation structure so designed,only the general requirements of 3.3 and corresponding clauses of Sections 6, 8, 9 and 12 need be satisfied.

C3.5.12.3 *Elastic foundations supporting elastic superstructures.* When the entire structure is designed to respond elastically, then capacity design procedures are not required for the foundation system. Hence no additional seismic requirements need be considered in the detailing of the components of such a foundation structure.

C3.5.12.4 *Ductile foundation structures.* Provision is made for the possibility that the designer might choose a structural system in which the foundation structure is assigned to dissipate seismic energy in a ductile manner. The superstructure may be chosen to remain elastic at all times. When the potential stregth of the superstructure is in excess of the overstrength of its ductile foundation structure, the superstructure should not need to meet the additional principles and requirements for the design of structures subjected to seismic loading:

(a) When the load that could be imposed by a ductile superstructure could not exceed the load that would ensue from the use of SM = 1.6, all potentially yielding members of the foundation structure must be subject to capacity design principle, and be detailed for ductility

(b) When yielding in the foundation could only occur when the load that could be transmitted by the superstructure is equal to or more than that which would result from the use of SM = 1.6, the foundation system may be considered as a structure of limited ductility. Accordingly only the principles of Section 14 need be satisfied.

Typical structures to which these provisions may be relevant are cantilever shear walls for which excessive lateral loads and correspondingly excessive foundations would be required to develop plastic wall hinges above the foundations.

If energy dissipation is to take place in components of the foundation structure, then the designer must clearly define these areas of yielding. The ductilities imposed on potential plastic hinges in the foundation system that are different from those normally encountered in superstructures should be checked.

Energy dissipation in foundation structures is expected to be assigned to footing pads, foundation beams and piles, but not to short columns which are subjected to significant axial compression or the yielding of which may lead to a storey sway mechanism. The consequences of damage in the foundation structure during moderate earthquakes, such as reduced protection of reinforcement against corrosion when the surrounding cracked concrete is in contact with the ground, and the feasibility of access for inspection or of repair should be carefully weighed when considering this structural system.

C3.5.12.5 *Rocking foundations.* The dimensions and locations of structural walls resisting earthquake induced lateral forces, may be such that, even when satisfying minimum strength requirements, they would develop overturning moment capacities that would be difficult or impossible to resist at foundation level. In such situations a structural system may be chosen in which rocking of the superstructure and its foundation provides the principal mechanism of earthquake response. Rocking response is not intended for framed structures. Because of the complete absence of experience, the design should be based on special studies, including appropriate dynamic analyses [3.30] based on realistic mathematical modelling of both structure and soil, to verify suitability of the chosen system. Special requirements that should be satisfied are outlined in the following paragraphs.

Gravity loads with adverse load factors must be considered when assessing restoring forces. In this the three dimensional behaviour of the structure must be considered. For example shear walls might collect additional gravity loads through yielding transverse beams and slabs which connect the wall to adjacent non-rocking frames.

The lateral load that would cause a structural wall to rock must be based on the contribution to gravity load on the rocking wall by all elements considered. The total lateral load on the entire structure is derived from the summation of the lateral load on all rocking walls and non-rocking frames which are effectively interconnected by rigid floor diaphragms.

The foundation structure should be so proportioned as to protect the supporting soil against excessive plastic deformations that might lead to misalignment of the entire structure. This consideration may lead to independent footings that distribute the loading to the soil at points of rocking of walls.

When all actions on potentially rocking walls and their foundations are derived from capacity design procedures, including the effects of ductile non-rocking adjacent frames and other components, the rocking system could be considered to be sufficiently protected against overload to permit the exemptions from the relevant additional seismic design and detailing requirements of this Code to be made in their design.

C3.5.12.6 The increase of earth pressure against retaining walls during ground shaking, particularly on sloping sites, should be considered. The transmission of base shear forces during seismic excitation from superstructure or from the foundation structure to the supporting soil should be taken into account in the design of piles[3.22].

C3.5.13 *Structures incorporating mechanical energy dissipating devices.* An alternative approach from the conventional seismic design procedures, on which this Code is based, is that of the "base isolation". Earthquake generated forces are reduced by supporting the structure on a flexible mounting, usually in the form of elastomeric rubber bearings, which will isolate the structure from the greatest disturbing motions at the likely predominant earthquake ground motion frequencies[3.19]. Damping, in the form of hysteretic energy dissipating devices, is introduced to prevent a quasi-resonant build-up of vibration. Although this approach is still in its developmental stages, potential advantages over the conventional ductile design approach appear to be: simpler design procedures; use of non-ductile forms or components; construction economies; and greater protection against earthquake induced damage, both structural and non-structural. The greatest potential advantages are for stiff structures fixed rigidly to the ground, such as low-rise buildings or nuclear power plants. Because these structures are commonly constructed in reinforced concrete, these provisions have been included in this Code, although the principles may be applicable to other materials. Bridges often already incorporate elastomeric rubber bearings, and the greatest benefits for such structures may derive from the potential for more economic seismic resistant structural forms[3.23, 3.25].

The design and detailing of structures designed for base isolation and incorporating mechanical energy dissipating devices should satisfy the criteria set out in the following paragraphs.

Moderate earthquake. For a moderate earthquake, such as may be expected two or three times during the life of a structure, energy dissipation is to be confined to the devices, and there is to be no damage to structural members.

"Design" earthquake. For a "design" earthquake, for example one with a return period one or two times the anticipated life of the structure, the designer may adjust the strength levels in the structural members to achieve an

optimum solution between construction economies and anticipated frequency of earthquake induced damage. However, the Code requires that the degree of protection against yielding of the structural members is to be at least as great as that implied for the conventional seismic design approach without dissipators. (In many cases this could be achieved with substantial construction cost savings. That is, the lower structural member strength requirements more than compensate for the extra costs of the devices.) It is recommended that the extent to which the degree of protection is increased above that minimum, to reduce the anticipated frequency of earthquake induced damage, should be resolved with regard to the client's wishes.

Extreme earthquake. For an extreme earthquake there is to be a suitable hierarchy of failure of the structural and foundation members that will preclude a brittle collapse. This may be achieved by appropriate margins of strength between non-ductile and ductile members and with attention to detail.

Although the design criteria outlined above encompass three earthquake levels, the design practice need only be based on the "design" earthquake. In the course of that design, the implications of yield levels on response to the "moderate" earthquake would have to be considered, as would also the implications of strength margins and detailing for an "extreme" earthquake. In general, the lower ductility demand on the structure means that the simplified detailing procedures of Section 14 would be satisfactory.

Because application of these devices to seismic resistant structures is still in its infancy, dynamic inelastic analyses should generally be undertaken for design purposes. Such analyses should consider acceleration records appropriate for the site, in particular taking account of any possibility of long period motions. As experience is accumulated, there is potential for development of standardized design procedures for common applications.

C3.5.14 *Secondary structural elements*

C3.5.14.1 The definition of a secondary element is more particular than that in NZS 4203, and includes such primary gravity-load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads. Caution must however be exercised in assumptions made as to the significance of participation. Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members. For convenience of reference and specification of requirements, secondary elements have been subdivided into groups, that is, Group 1 and Group 2 elements.

C3.5.14.2 To avoid any form of deformation – induced loading, in Group 1 elements, separations must be meticulously detailed. Similarly close attention must be given to details of supports, and to their positioning. Reference[3.26] discusses separation, while reference[3.27] discusses such aspects as the conflict between these separation requirements and the requirements of sound attenuation, fire protection and the like. The loading is specified as an equivalent static load. Since these loads are already scaled to account for amplification of accelerations within the structure, no additional scaling of deflections and element actions is required. Often Group 1 elements are geometrically complex, and where appropriate the yield line method, for instance, of Section 11 would be appropriate to their analysis.

Ductile behaviour remains the prime objective of adequate detailing and must be sought by the detailer. The details however need not be elaborate to allow such behaviour. Wall panels, for instance, may be reinforced with a single layer of reinforcement without any additional confinement, and still provide adequate ductility.

C3.5.14.3 In the consideration of Group 2 elements:

(a) The additional seismic requirements of the relevant sections of the Code need not be complied with when the elastic deformation-induced actions on the element are derived from elastic analysis using deformation not less than $\nu\Delta$

(b) Where ductile action is relied on to produce adequate inelastic deformation capacity, all additional seismic detailing requirements of relevant sections must be met

(c) NZS 4203 sets out the requirements to be met in regard to inertia loading and to amplified deformations, and the commentary to that code provides guidance on methods of calculation

(d) The deformation calculated in accordance with NZS 4203 may be exceeded in some structures and in localized areas. Furthermore the pattern of deformation will usually vary significantly from the first mode pattern assumed in calculation. These variations should be taken into account in assessing member actions when they might have a marked effect on element performance

(e) In certain cases elastic response may not be desirable, as forces may become excessive and even lead to inferior performance of the primary structure. Therefore inelastic action is permissible. However, elements must be designed for at least the elastic fraction of the total deformation of the primary elements, to prevent excessive damage in moderate earthquakes. Normally elastic actions will be selected. In most instances achievement of this will not prove to be unduly onerous. In many cases design will be controlled by

$U = 1.4D + 1.7L_R$

For the seismic load to predominate would require in this event $\nu E^* > 0.25\ (1.4D + 1.7L_R)$ in which E^* is the action induced by Δ, and in which $0.75\ (1.4D + 1.7L_R)$ is an approximation for $D + 1.3L_R$ (for a definition of terms refer to NZS 4203). Therefore with $\nu = 2.5$:

$E^* < 0.1\ (1.4D + 1.7L_R)$ would require no additional demand on strength

(f) Inelastic action may only be assumed when detailing allows adequate ductility. Where strength is derived from loadings consistent with one-half of the amplified deformations, ductility demand is likely to be met by detailing for limited ductility.

Where strength has been determined from deformations less than this, design and detailing must be such as to allow fully ductile behaviour. Therefore, limited capacity design might be appropriate for shear force determination and in determining whether or not adjacent members yield, but it is not considered necessary to amplify column moments for higher mode effects to prevent yielding of columns because column hinging is not of particular significance.

C3.5.15 *Fixings of precast non-structural elements*

C3.5.15.1 A number of problems relevant to the fixing and separation of and damage to non-structural components in buildings are discussed in reference [3.26] and reference [3.28].

C3.5.15.2 This Clause simplifies design in cases where the calculated movements are small.

C3.5.15.3 The intention must be to minimize the risk of breakage in fixings if seismic movements exceed the clearances provided. Such failure could result in the falling of elements and a possible blockage of means of egress. The requirement that anchorages of the fixings to the elements shall allow fixings to deform in a ductile manner, usually necessitates the anchorage being welded to the reinforcing mesh of the element, or connected to the reinforcement by lapped splices or by mechanical connections. Special care must be taken to ensure that embrittlement is avoided, such as may occur where steel components are cold formed and then galvanized. For heavy exterior panels it is preferable to avoid bolted connections, which transfer the weight of the panel entirely by shear, whenever the same bolts are subjected also to shear during large seismic movements. Because of the serious consequences of failures, testing of unusual fixings for elements, such as heavy concrete cladding panels in multi-storey buildings, should be carried out. The tests should check the performance in both resisting seismic loads, and permitting seismic deflections greater than the clearance limits.

REFERENCES

3.1 Muto, K., "Aseismic Design Analysis of Buildings", Maruzen, Tokyo, 1974, 361 pp.

3.2 "Continuity in Concrete Building Frames", Portland Cement Association, Skokie, EB033D, 1959, 56 pp.

3.3 Pauw, Adrian, "Static Modulus of Elasticity of Concrete as Affected by Density", ACI Journal, Proceedings V. 32, No. 6, Dec. 1960, pp. 679–687.

3.4 Okamura, H., et al., "Elastic Frame Analysis – Corrections Necessary for Design of Short Concrete Columns in Braced Frames", ACI Journal, Proceedings V. 67, Nov. 1970, pp. 894–897.

3.5 MacGregor, J.G., Discussion of "Determination of Effective Length Factors for Slender Concrete Columns", ACI Journal, Proceedings V. 70, May 1973, p. 370.

3.6 "Handbook of Frame Constants", Portland Cement Association, Skokie, EB034D, 1958, 32 pp.

3.7 "Continuous Concrete Bridges", Portland Cement Association, Skokie, EB041E, 1941, 106 pp.

3.8 Seismology Committee, "Recommended Lateral Force Requirements and Commentary", Structural Engineers Association of California, 1975, 119 pp.

3.9 Blume, J.A., Newmark, N.M. and Corning, L.H., "Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions", Portland Cement Association, Skokie, 1961, 318 pp.

3.10 Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley & Sons, 1975, 769 pp.

3.11 Sozen, M.A., "Structural Damage Caused by the Skopje Earthquake of 1963", Bulletin 279, Civil Engineering Studies, Sructural Research Series, University of Illinois, 1964, 39 pp.

3.12 "The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks", U.S. Department of Commerce, Coast and Geodatic Survey, Vol. II, Part A, Washington, 1967, 392 pp.

3.13 "Caracas Earthquake Damage Reported by the Portland Cement Association Team", ACI Journal Proceedings V. 65, No. 4, April 1969, pp. 292–294.

3.14 Sozen, M.A., "The Caracas Earthquake of July 29, 1967", ACI Journal, Proceedings V. 65, No. 5, May 1968, pp. 394–401.

3.15 Jennings, P.C., "Engineering Features of the San Fernando Earthquake, February 9, 1971", Earthquake Engineering Research Laboratory, California Institute of Technology, EERL 71-02, Pasadena, June 1971, 512 pp.

3.16 Bulletin of the New Zealand National Society for Earthquake Engineering "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames", Sections A, E, F, G and H, Vol. 10, No. 2, June 1977, pp. 69–105.

3.17 Bulletin of the New Zealand National Society for Earthquake Engineering, "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames", Sections C and J, Vol. 10, No. 4, Dec. 1977, pp. 219–237.

3.18 "Papers Resulting from Deliberations of the Society's Discussion Group on Seismic Design of Reinforced Concrete Walls and Diaphragms", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, June 1980, pp. 182–193.

3.19 Blakeley, R.W.G., Charleson, A.W., Hitchcock, H.C., Megget, L.H., Priestley, M.J.N., Sharpe, R.D. and Skinner, R.I., "Recommendations for the Design and Construction of Base Isolated Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, June 1979, pp. 136–157.

3.20 "National Building Code of Canada 1980, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, NRCC No. 17303, 547 pp.

3.21 Blakeley, R.W.G., Cooney, R.C. and Megget, L.M., "Seismic Shear Loading at Flexural Capacity in Cantilever Wall Structures", Bulletin of the New Zealand National Society of Earthquake Engineering, Vol. 8, No. 4, Dec. 1975, pp. 278–290.

3.22 "Highway Bridge Design Brief", Civil Division Publication CDP 701/D September 1978, Ministry of Works and Development, Wellington, New Zealand, and amendment of November 1978, 52 pp. or its later revision when published.

3.23 "Papers Resulting from Deliberations of the Society's Discussion Group on the Seismic Design of Bridges", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 3, Sept. 1980, pp. 226–307.

3.24 "Ductility of Bridges with Reinforced Concrete Piers", Ministry of Works and Development, Wellington, New Zealand, Civil Division Publication CDP 810/A, April 1975, 109 pp.

3.25 Park, R. and Blakeley, R.W.G., "Seismic Design of Bridges", Summary Volume 3, Road Research Unit Structures Committee Seminar, Auckland, November 1978.

3.26 Glogau, O.A., "Separation of Non-Structural Components in Buildings", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 3, Sept. 1976, pp. 141–158.

3.27 Allardice, N.W., "Seismic Design of Ductile Moment-Resisting Reinforced Concrete Frames", Section K: Parts, Portions and Secondary Elements", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 2, June 1977, pp. 102–105.

3.28 McKenzie, G.H.F., "Problem of Damage to Non-Structural Components and Equipment – Walls and Stairs", Proceedings of the Workshop on Earthquake Resistant Rinforced Concrete Building Construction, University of California, Berkeley, 1977, Vol. 3, pp. 1128–1139.

3.29 Taylor, P.W. and Williams, R.L., "Foundations for Capacity Designed Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, June 1979, pp. 101–113.

3.30 Priestly, M.J.N., Evison, R.J. and Carr, A.J., "Seismic Response of Structures Free to Rock on Their Foundations", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 11, No. 3, Sept. 1978, pp. 141–150.

APPENDIX TO COMMENTARY ON SECTION 3

C3A A METHOD FOR THE EVALUATION OF COLUMN ACTIONS IN DUCTILE MULTISTOREY FRAMES

C3.A1 Notation

A_g gross area of section, mm^2

E_o load derived from earthquake induced overstrength of structural members

f'_c specified compressive strength of concrete, MPa

f_y specified yeild strength of non-prestressed reinforcement, MPa

h_b overall depth of beam, mm

H height of centre line of top floor beam above base of first storey columns

I moment of inertia about centroidal axis of gross section

k relative flexural stiffness

ℓ length of member between centre lines of supports

ℓ_c storey height

ℓ_n clear length of column between beam faces

M_{code} column bending moment at the centre line of a beam, derived with an elastic analysis for the code specified lateral seismic loading without any moment redistribution.

$M_{code, top}$ the value of M_{code} for first storey columns at first floor level

M_{col} design moment at the critical sections of a column

$M_{col, reduced}$ reduced column design moment at the critical section

$M_{o, col}$ the flexural overstrength of a first storey column, as built, at its restrained base with the design axial load acting

n the number of floors above the column section considered

P_e maximum total design axial load on a column during an earthquake

P_{eq} maximum axial design load on a column due to earthquake only

P_g appropriately factored design gravity load on the column

R_m moment reduction factor

R_v axial load reduction factor

T_1 the computed period of the structure in its first mode of vibration, seconds

V_{code} column shear force derived from code specified lateral seismic loading

V_{col} column design shear force

V_{oe} maximum earthquake induced beam shear force at the development of beam flexural overstrengths

ϕ strength reduction factor

ϕ_o beam overstrength factor

ρ_t ratio of total reinforcement in a column section

ω dynamic magnification factor

ω_r ω applicable to roof level

C3.A2 Scope and limitations

This Appendix presents a procedure for the evaluation of the magnitudes of the axial loads, bending moments and shear forces that may be used in the determination of the required ideal strength at the critical sections of columns of regular framed buildings three storeys and higher, subject to design earthquake loading. The procedure is intended to satisfy the requirements of NZS 4203. It uses the results of the equivalent static force analysis of that code of practice*.

* Subsequent references to code loading imply equivalent static forces specified by Section 3.4 of NZS 4203.

With the introduction of capacity design philosophy and its detailed requirements, as specified at various parts of this Code, it has become necessary to modify traditional frame analyses to yield the required design actions for various members. The method given here is a modified form of the procedure given in Section G of reference [3.16].

Traditional elastic frame analyses resulted in member actions which, when suitably factored, could be used directly in the design of each member. In capacity design, however, it is necessary to estimate the maximum actions that could be generated in certain members, such as beams, before the design actions on other members, such as columns, can be assessed. This way the desirable formation and hierarchy of the chosen energy dissipating mechanisms in the frame can be expected.

A summary of suggested design steps is given in C3.A9. The procedure intends to provide a high degree of protection against yielding in columns, which could possibly occur before any beam has yielded, and it aims to eliminate the possibility of the development of storey sway mechanisms in any storey of regular building frames during the most severe seismic motions. The procedure is likely to lead also to adequate reserve shear strength.

The following limitations apply:

(a) The procedure is intended to apply principally to regular frames in which the relationship between relative stiffness, k, of the columns in a storey and that of the beams framing into them is such that

$$\frac{\Sigma k_{\text{upper beams}} + \Sigma k_{\text{lower beams}}}{2k_{\text{column}}} > 0.2$$

where $k = I/\ell$. Columns of such frames, when subject to code load, will exhibit a point of contraflexure in each storey[3.1].

(b) When the above stiffness criterion is not met, cantilever action may dominate the moment pattern of columns and an approach applicable to shear wall structures may be more appropriate.

(c) Certain numerical values, particularly those of the dynamic magnification factor are based only on a limited study of the inelastic response of ductile frames. These may change as more information comes to hand.

(d) The procedure does not apply to small frames in which column hinges are intended to form the primary energy dissipating mechanisms, or to frames in which column displacements are controlled by shear walls, or to columns which are acting as props and are not required or intended to contribute to lateral load resistance.

(e) When gravity load considerations govern the strength of beams, capacity design will require columns of ductile frames to be designed for moments that may be much larger than those resulting from code earthquake loading. In such cases the acceptance of column hinging before the development of full beam sway mechanisms, at a load considerably in excess of code loading, may be more appropriate.

C3.A3 Strength requirements

The method conforms with the requirements of the Code for capacity design procedures. Accordingly the ideal flexural and shear strength of columns to be provided should be at least equal to the loads determined by this method. Strength reduction factors, ϕ, need not be used, or where they appear in equations, $\phi = 1.0$ should be used.

C3.A4 Beam overstrength factors

C3.A4.1 One of the basic requirements of the design of ductile frames is that the possibility of column hinging during inelastic displacements of a frame be minimized. To this end the maximum load input from the beams into the columns, developed during large inelastic deformations, needs to be determined. This is achieved with the use of the beam overstrength factor, ϕ_o, at each column centre line at each floor and in each direction of loading.

The beam overstrength factor, ϕ_o, at a column is the ratio of the sum of the flexural overstrengths developed by the beams, as detailed, to the sum of the flexural strengths required in the given direction by the Code specified lateral seismic loading alone, both sets of values being taken at the centre line of the relevant column.

The evaluation of flexural overstrength, referred to in C3.5.1.3, C6.5.1 and C7.5.1, must take into account all the reinforcement that is likely to particpate in the flexural resistance at potential plastic hinges that may be formed during inelastic displacements of the frame. Plastic hinges in a beam may occur at the faces of the columns or somewhere else within the clear span of the beam. The beam flexural overstrength input with reference to the centre line of an interior column must be determined from the appropriate beam moment patterns, consistent with the direction of the lateral load, with two plastic hinges developing the flexural overstrengths of the relevant sections in each of the two adjacent beams. At an exterior column only one beam is considered.

The beam overstrength factor needs to be determined at each floor and at each column at the centre line of the beam-column joints separately for both directions of the lateral displacements. The reference to member centre lines is preferable to avoid additional calculations when checking joint equilibrium.

The overstrength factor enables the total moment input from beams into the columns during inelastic lateral interstorey displacements to be considered. Therefore, in this method, moments in columns due to gravity load need not be considered separately.

When Grade 275 flexural reinforcement is used the ideal value of ϕ_o is $1.25/0.9 = 1.39$. (See C3.5.1.3.) However, when gravity load dominates the required beam flexural strength, this value may be greatly exceeded. When beam moment redistribution is applied the value of θ_o may be locally less than 1.39.

C3.A4.2 At the ground floor or at foundation level, where the full column flexural capacity could be developed, no beams may frame into the column. Consequently, a ϕ_o factor applicable to that locality would appear to be irrelevant. It is considered that the potential ideal lateral load strength of first storey columns at their base should be comparable with the strength of the remainder of the frame. Consequently the value of ϕ_o, applicable at a column base, should not be taken less than 1.4. The ideal moment of resistance so obtained for the column section is comparable with that derived from unmagnified moments due to code load and the strength design method, using a strength reduction factor of $\phi = 0.7$.

In exceptional cases at ground floor level, factored gravity load with or without factored wind load may result in more critical moments.

C3.A4.3 At roof level generally gravity load will govern the design of beams. At a roof column-beam joint, it is not necessary to increase the column capacity to match or exceed the beam flexural capacity because column hinging at this level is acceptable. The value of $\phi_o = 1.1$ should be used to compensate for the capacity reduction factor of $\phi = 0.9$ that would normally be used in this situation. The adequacy of such columns for gravity loads only must however be checked.

C3.A4.4 When a column is considerably stiffer than the beams which frame into it, cantilever action will dominate its behaviour in the lower floors. In such cases the column moment at a floor may be larger than the total beam moment input at that floor. Therefore at the floor below which a column point of contraflexure is not indicated by the elastic analysis for code loading, and at all floors in the lower soreys below that level, the value of ϕ_o need not be taken more than 1.4.

C3.A5 Dynamic magnification

C3.A5.1 The equivalent static load specified by the Code is considered to lead to satisfactory distribution of potential beam strengths throughout the frame. To give columns a high degree of protection against premature yielding, approximate allowance is made for the departure of the column moment pattern from that which was obtained with an elastic frame analysis for the code static load distribution. This is because of dynamic effects, in particular the higher mode dynamic responses of the structure. The phenomenon may be gauged, for example, by the movement of the column point of contraflexure away from its location that was determined in the static code load elastic analysis. The higher mode dynamic responses are assumed to become more prominent in the upper storeys and as the fundamental period of vibration of the structure increases. Therefore the dynamic magnification factor is introduced, which for columns of one-way frames is

$$\omega = 0.6\,T_1 + 0.85 \qquad \text{(Eq. C3.A-1)}$$

but not less than 1.3 nor more than 1.8.

The lower limit of $\omega = 1.3$ has been set to minimize the possibility of a storey sway mechanism forming in columns that are part of a one-way frame, that is, in which beams frame into a column in one plane only. The columns of two-way frames, in which the lateral load in one direction is resisted entirely by shear walls, may be designed similarly. For convenience the values of ω are given in table C3.A1.

C3.A5.2 For columns of two-way frames the effects of concurrent earthquake attack along both principal axes of the structure need be considered. This involves analysis of column sections for biaxial bending and axial load. The concurrent development of plastic hinging in all beams framing into a column should also be taken into account. However, if the dynamic magnification factor is suitably increased, the design process may be simplified by allowing each column section to be designed only for unidirectional earthquake attack, separately in each of the two principal directions of the structure. A column so designed may then be considered to possess adequate flexural strength to resist various combinations of biaxial flexural demands, satisfying the requirements of NZS 4203.

Accordingly for columns of two-way frames

$$\omega = 0.5\,T_1 + 1.0 \qquad \text{(Eq. C3.A-2)}$$

but not less than 1.5 nor more than 1.9. (See table C3.A1.)

The minimum value results from the consideration that the columns should be capable of sustaining simultaneous beam hinge moment inputs from two directions, corresponding with the elastic moment pattern that resulted from the analysis for code load. When it is considered that a square column section subjected to moment along its diagonal is approximately 90% as efficient as for moment action along its principal axes, the approximate minimum value of ω is obtained thus: $\sqrt{2/0.9} \simeq 1.5$.

The likely concurrency of very large orthogonal moments at any one column section due to the occurrence of higher mode shapes is, however, considered to diminish with lengthening of the fundamental period of the building. Therefore, the allowance for concurrent moment attack gradually reduces with increased fundamental period, that is with the increase in number of storeys, as may be seen in table C3.A1.

C3.A5.3 Higher mode responses are not affecting the required strength of the bottom storey columns where base restraint exists. At this section hinging is to be expected and the column is to be detailed accordingly. To ensure that the flexural capacity of column sections in two-way frames is adequate to sustain at any angle an attack of code load intensity, the unidirectional moment demand should be increased by approximtely 10%. Similar considerations apply to column sections at roof level. Accordingly to allow for this at the roof and at ground floor or foundation level, the value of ω may be reduced to:

(a) 1.0 for columns of one-way frames

(b) 1.1 for columns of two-way frames

C3.A5.4 It is considered that higher mode dynamic responses are affecting the moments in columns of the upper storeys of a frame more significantly than in the lower storeys. Accordingly the values of the dynamic magnification factor, ω, as given by eq. C3.A-1 and eq. C3.A-2 are applicable only to levels at and above 0.3 times the height of the structure, measured from the level at which elastic first storey columns are considered to be effectively restrained against rotations. Below this specified level a linear variation of ω should be assumed. However in no case should the value of ω at first floor level be taken less than the minimum specified in C3.A5.1 or C3.A5.2.

An example for the relevant values of ω, corresponding with the requirement of C3.A5.at all levels, for a 15 storey one-way and a two-way frame, with an assumed fundamental period of 1.5 s and a given column moment pattern for M_{code}, is shown in fig. C3.A1.

C3.A5.5 In many cases, where columns are stiff relative to the beams, the column moment pattern in the lower storeys of frames, obtained from the analysis of code loading, may be such that no point of contraflexure appears in several storeys above the base. As pointed out in C3.A4.4 this indicates increased cantilever action of the column which, at these levels, is not likely to be affected significantly by the higher modes of dyanamic response. Therefore in the lower storeys in which, according to the analysis for code earthquake loading, no point of contraflexure along a column is indicated, the value of ω at the first floor may be taken not less than 1.3 for one-way and 1.5 for two-way frames, and then linearly increased to the value obtained from eq. C3.A-1 or eq. C3.A-2, as appropriate, at the floor above which the first point of contraflexure above ground floor is identified. The interpretation of this interpolation is the same as shown in fig. C3.A1. This provision is less stringent than C3.A5.4, when the first point of contraflexure along the column is situated higher than 0.3 H above ground floor level.

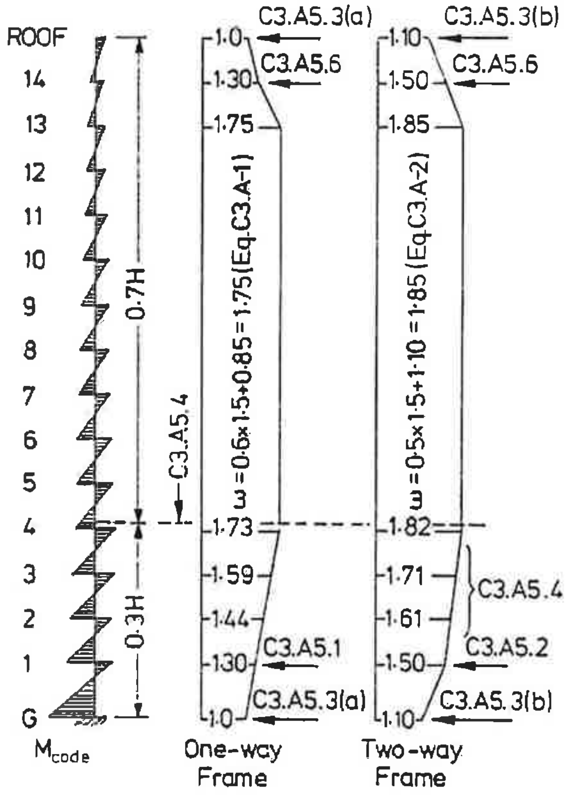


Fig. C3.A1
THE EVALUATION OF THE DYNAMIC MAGNIFICATION FACTOR ω FOR A 15-STOREY EXAMPLE COLUMN

C3.A5.6 At the floor immediately below roof, the value of ω may be taken as 1.3 for one-way frames and 1.5 for two-way frames. At the top storey of multi-storey frames the development of storey mechanisms is considered to be acceptable, and hence there is no need for a high degree of protection against column yielding. Correspondingly at the floor immediately below roof level these minimum values of the dynamic magnification factor have been stipulated. The interpretation of these requirements is illustrated in fig. C3.A1.

C3.A6 Design axial forces

The derivation of the earthquake induced axial forces is based on the assumption that, with increasing number of storeys above the specified level, the relative number of beam hinges at which the flexural overstrength may be developed simultaneously is reduced. To allow for this, 1.5% reduction per floor in the maximum feasible earthquake induced column axial load at any level has been considered, up to a maximum of 30% for 20 floors or more above the level to be considered. Hence the axial forces induced at any level by earthquake loading only, and used together with the appropriately factored gravity forces and the moments derived in accordance with C3. A.7, to determine the column section strength, should not be less than:

$$P_{eq} = R_v \Sigma V_{oe} \qquad \text{(Eq. C3.A-3)}$$

where ΣV_{oe} is the sum of the earthquake induced beam shear forces at all floors above the level considered, developed at all sides of the column, taking into account the beam overstrengths and the appropriate sense of the forces. The value of the reduction factor R_v is given in table C3.A.2.

In obtaining R_v the dynamic magnification factor, ω, given in table C3.A2 should be as appropriate to the floor considered (see fig. C3.A1). It is considered that the maximum earthquake induced axial forces are not likely to coincide with the maximum column design moments that result from magnification, in accordance with C3.A7, to allow for higher mode responses. Consequently larger axial load reductions are considered to be appropriate when ω is larger than 1.4. The maximum reduction, when $\omega = 1.9$, is an additional 0.83% per floor, as is seen in table C3.A.2.

In summing the shear forces at the column faces all beams in both directions need to be considered. In general this procedure will not significantly affect the axial load on interior columns. However, for outer columns and corner columns in particular, significant increases in axial load will result and this should be considered as a consequence of a skew earthquake attack. When the dynamic magnification in the two principal directions of the structure is different, the larger of the ω values, relevant to the level under consideration, may be taken when determining R_v to evaluate the axial load due to concurrent actions.

C3.A7 Design moments

C3.A7.1 The design column moments at the intersections of the reference axes of beams and columns are obtained by multiplying at each end of a column the corresponding moments, obtained for the code loading, M_{code}, by the product of the ϕ_o and ω appropriate to that floor. The moment magnification applies to the end moments only and not to the moment pattern. The two end design moments so obtained for a column in a storey are not expected to occur simultaneously.

The critical column section is assumed to be at the top or the soffit of the beams and accordingly the centre line column moment $\phi_o \omega M_{code}$ may be reduced. In this only 60% of the moment gradient, used for the determination of the column shear, is considered. Hence the centre line column moment, as shown in fig. C3.A2, is reduced by $0.6 \times 0.5 h_b\ V_{col}$, where the column design shear force, V_{col}, is in accordance with C3.A8. Consequently the design moment, M_{col}, to be used, together with the appropriate axial load, P_e, for the determination of the ideal strengths of the critical column section, separately in each of the two principal directions of the structure, should not be less than:

$$M_{col} = \phi_o\, \omega M_{code} - 0.3 h_b V_{col} \qquad \text{(Eq. C3.A-4)}$$

where the value of V_{col} is that given by eq. C3.A-6 or eq. C3.A-7.

The application of moment magnification for the moment pattern in the lower storeys of a column of a two-way example frame, shown in fig. C3.A1, with specific values of ϕ_o and ω, is illustrated in fig. C3.A2.

C3.A7.2 When a column is subjected to small axial compression or to net tension, yielding is more acceptable. For such cases the design moment may be reduced. The larger the axial tension load and the value of the dynamic magnification factor ω, given by eq. C3.A-1 or eq. C3.A-2, the larger the acceptable moment reduction. Accordingly when the total design axial compression, P_e, on a column does not exceed $0.1\, f_c' A_g$, the design column moment may be reduced as follows:

$$M_{col,\,reduced} = R_m\, [\phi_o\, \omega M_{code} - 0.3 h_b\, V_{col}] \qquad \text{(Eq. C3.A-5)}$$

where R_m is given in table C3.A.3 and where P_e is to be taken as negative if causing tension and provided that:

(a) The value of $P_e / f_c' A_g$ should not be taken less than -0.15 nor less than $-0.5\, \rho_t f_y / f_c'$

(b) The value of R_m taken for any one column should not be less than 0.3

(c) The moment reduction used for columns of a bent should not be more than 10% of the sum of the unreduced design moments as obtained from eq. C3.A-4, for all columns of the bent, taken at the same level.

The requirements of C3.A7.2 (a) and (b) are intended to ensure that the reduction of the magnified moments is not excessive. The reduction of column design moments in a

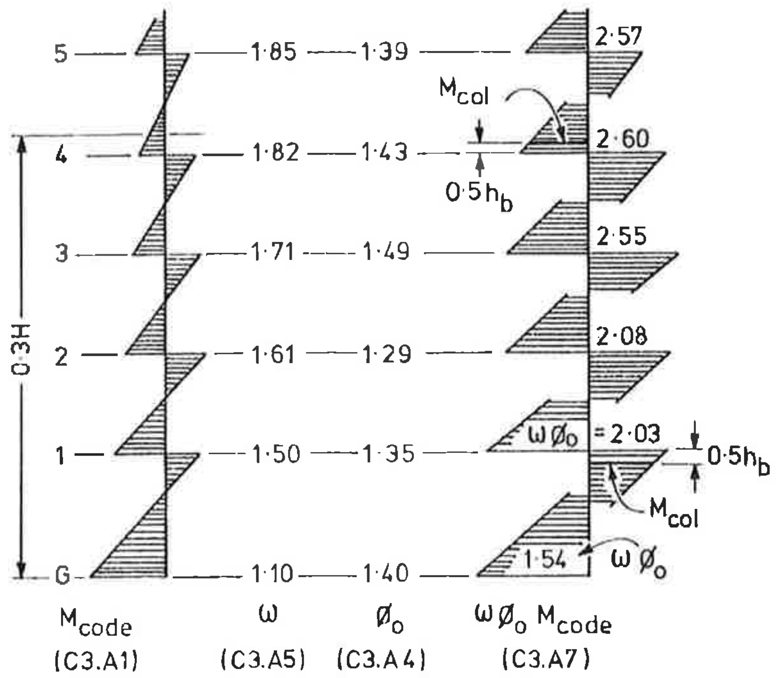


Fig. C3.A2 THE DETERMINATION OF THE DESIGN MOMENTS IN THE LOWER STOREYS OF AN EXAMPLE COLUMN

bent may result in loss of lateral load carrying capacity in that bent. Usually there will be only one column in the bent that will qualify for reduction of design moment. Because of the possible strength loss, when hinge over-strength capacities are being developed, according to C3.A7.2 (c) the moment reduction allowed should not exceed 10% of the sum of the column design moments, taken before the application of moment reduction at the same level for all columns, that is, immediately above or below the beam of the affected bent.

C3.A8 Design shear forces

C3.A8.1 The design shear forces across a column of one-way frames, V_{col}, given below are evaluated from the column end moments:

(a) In upper storey columns of one-way frames

$$V_{col} = 1.3\,\phi_o V_{code} \qquad \text{Eq. (C3.A-6)}$$

should be used.

The shear force is estimated from a probable and critical moment gradient along the column. For upper storey columns it is assumed that a shear force 20% in excess of that generated by ϕ_o times the column end moments due to code load may be developed. To provide for additional reserve strength, a reduction factor of 0.85 is incorporated, and hence the ideal column shear strength required will be close to that given by eq. C3.A-6. In estimating the maximum possible value for the design shear, it has been assumed that at the development of all beam hinges above and below a storey, increase in storey shear in excess of 20% is not possible during the earthquake.

(b) In first storey columns, in addition to satisfying the requirement of C3.A8.1 (a), the shear forces given by

$$V_{col} = (M_{o,\,col} + 1.3\,\phi_o M_{code,\,top}) / (\ell_n + 0.5h_b) \qquad \text{(Eq. C3.A-7)}$$

should also be considered.

At the base of a first storey column, hinging with considerable plastic rotations is to be expected. Consequently for these columns $M_{o,\,col}$, the flexural overstrength capacity of the base section, with allowance for the axial load on the column that is consistent with the direction of the loading considered, must be evaluated.

C3.A8.2 The design shear forces across columns of two-way frames should be not less than the following values:

(a) In columns of upper storeys

$$V_{col} = 1.6\,\phi_o\,V_{code} \qquad \text{(Eq. C3.A-8)}$$

(b) In first storey columns of two-way frames

$$V_{col} = (M_{o,\,col} + 1.5\,\phi_o M_{code,\,min}) / (\ell_n + 0.5\,h_b) \qquad \text{(Eq. C3.A-9)}$$

but not less than that given by eq. C3.A-8.

C3.A8.3 In columns of the top storey,where columns may hinge before the top floor beams would, the column shear should be evaluated the same way as in the first storey columns.

C3.A9 The design steps to determine column actions – Summary

Step 1 – Derive the bending moments for all members of the frame for the specified lateral earthquake load only, using an appropriate elastic analysis. M_{code} refers to the column moments so obtained.

Step 2 – Superimpose the beam bending moments resulting from the lateral load upon the appropriately factored gravity load moments. Subsequently carry out a moment redistribution in accordance with 3.5.3.4 for all spans in each bent.

Step 3 – Design all critical beam sections so as to provide the required dependable flexural strengths and determine and detail the reinforcement for all beams of the frame.

Step 4 – Compute the flexural overstrength of each potential plastic hinge, as detailed, in each span of each continuous beam for both directions of the applied lateral load. Using bending moment diagrams or otherwise determine the corresponding beam overstrength moments at each column centre line (fig. C6.6) and determine the associated moment induced shear forces (C7.5.1), V_{oe}, in each span.

Step 5 – Determine the beam overstrength factor, ϕ_o, at the centre line of each column for both directions of the loading on the frame (C3.A4.1). Fixed values of ϕ_o are as follows:

(a) At ground floor level $\phi_o = 1.4$

(b) At roof level $\phi_o = 1.1$

(c) At any level at which the lateral load analysis does not indicate a point of contraflexure on the column the value of ϕ_o need not be taken larger than 1.4.

Step 6 – From the fundamental period of vibration of the structure and using table C3.A1 determine the value of the dynamic magnification factor, ω, for the frame (C3.A5.1, C3.A.5.2). At certain floors exceptions apply as follows:

(a) At ground floor and at roof level $\omega = 1.0$ or $\omega = 1.1$ for one-way and two-way frames respectively (C3.A5.3)

(b) At the floor immediately below roof level $\omega = 1.3$ or $\omega = 1.5$ for one-way or two-way frames respectively (C3.A5.6)

(c) At floors situated at the lower 30% of the height, H, of the frame, the value of ω is interpolated between the minimum values (1.3 and 1.5) at first floor level and its values obtained from table C3.A1, which are applicable at and above the level of $0.3H$ above ground floor

(d) For frames in which the analysis for lateral load does not indicate a point of contraflexure the minimum values of ω (that is, 1.3 or 1.5) may be taken at first floor level and then ω may be linearly increased to its full values, obtained from table C3.A1, at the floor immediately above the level at which the first column point of contraflexure is indicated.

Step 7 – Sum up all the earthquake induced overstrength beam shear forces, V_{oe}, for all floors from roof level down to ground floor, and determine at each floor $P_{eq} = R_v \Sigma V_{oe}$, (C3.A6) where value R_v is obtained from table C3.A2.

Step 8 – Determine the design axial loads, P_e at each floor for the load combinations of $(D + L_R + E_o)$ or $(0.9\,D + E_o)$ paying attention to the sense of the forces.

Step 9 – The magnified column moments below and above the beam centre lines are now found from $\phi_o\, \omega M_{\text{code}}$ (fig. C3.A2).

Step 10 – For columns under low axial compression or axial tension the moments obtained in Step 9 may be reduced by the factor R_m where the value of R_m is obtained from table C3.A3 (clause C3.A7.2).

Step 11 – The critical design moments at the top or the soffit of beams, to be considered together with the design axial loads P_e, (Step 8) are finally found (C3.A7.1) from

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

and the longitudinal reinforcement may now be determined.

Step 12 – The column design shear force at a typical upper storey is generally computed from eq. C3.A-6 or eq. C3.A-8 depending whether the column is part of a one-way or two-way frame. For first storey columns equations C3.A-7 or C3.A-9 need also be considered.

Table C3.A1

DYNAMIC MAGNIFICATION FACTOR ω

Type of frame	*Period of structure in seconds,* T_1									
	<0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	>1.6
One-way frames	1.30	1.33	1.39	1.45	1.51	1.57	1.63	1.69	1.75	1.80
Two-way frames	1.50	1.50	1.55	1.60	1.65	1.70	1.75	1.80	1.85	1.90

Table C3.A2

AXIAL LOAD REDUCTION FACTOR R_v

Number of floors above the level considered	*Dynamic magnification factor* ω					
	1.4 or less	*1.5*	*1.6*	*1.7*	*1.8*	*1.9*
2	0.97	0.97	0.96	0.96	0.96	0.95
4	0.94	0.94	0.93	0.92	0.91	0.91
6	0.91	0.90	0.89	0.88	0.87	0.86
8	0.88	0.87	0.86	0.84	0.83	0.81
10	0.86	0.84	0.82	0.80	0.79	0.77
12	0.83	0.81	0.78	0.76	0.74	0.72
14	0.80	0.77	0.75	0.72	0.70	0.67
16	0.77	0.74	0.71	0.68	0.66	0.63
18	0.74	0.71	0.68	0.64	0.61	0.58
20 or more	0.71	0.68	0.64	0.61	0.57	0.54

Table C3.A3

MOMENT REDUCTION FACTOR R_m

ω	P_e/f'_cA_g										
	-0.150	-0.125	-0.100	-0.075	-0.050	-0.025	0.00	0.025	0.050	0.075	0.100
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.1	0.85	0.86	0.88	0.89	0.91	0.92	0.94	0.95	0.97	0.98	1.00
1.2	0.72	0.75	0.78	0.81	0.83	0.86	0.89	0.92	0.94	0.97	1.00
1.3	0.62	0.65	0.69	0.73	0.77	0.81	0.85	0.88	0.92	0.96	1.00
1.4	0.52	0.57	0.62	0.67	0.71	0.76	0.81	0.86	0.90	0.95	1.00
1.5	0.44	0.50	0.56	0.61	0.67	0.72	0.78	0.83	0.89	0.94	1.00
1.6	0.37	0.44	0.50	0.56	0.62	0.69	0.75	0.81	0.88	0.94	1.00
1.7	0.31	0.38	0.45	0.52	0.59	0.66	0.73	0.79	0.86	0.93	1.00
1.8	0.30	0.33	0.41	0.48	0.56	0.63	0.70	0.78	0.85	0.93	1.00
1.9	0.30	0.30	0.37	0.45	0.53	0.61	0.68	0.76	0.84	0.92	1.00
	Tension							Compression			

COMMENTARY

C4 STRENGTH AND SERVICEABILITY

C4.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in Section 4 of the Code:

a depth of equivalent rectangular stress block as defined in 6.3.1.7, mm

a_1 coefficient used in eq. 4-A for crack spacing (see C4.4.2.2)

a_2 coefficient used in eq. 4-A for crack spacing (see C4.4.2.2)

A_v area of shear reinforcement, within a distance s, mm²

b width of beam section, mm

b_1 coefficient used in eq. 4-C relating to average crack width (see C4.4.2.2)

b_2 coefficient used in eq. 4-C relating to average crack width (see C4.4.2.2)

b_w web width of beam, mm

c concrete cover to reinforcment, mm

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement, mm

d_b reinforcement bar diameter, mm

D dead load as defined by NZS 4203

E design earthquake load as defined by NZS 4203

E general term for modulus of elasticity, MPa

$f_{max.}$ $f_{min.}$ plus the additional compressive stress due to live load plus impact, MPa

$f_{min.}$ minimum compressive stress level in the concrete due to dead load, creep, shrinkage, temperature, etc. MPa

f_{sr} stress range $= f_{max.} - f_{min.}$, MPa

h height of rolled-on transverse deformation on reinforcing bar, mm

I general term for the moment of inertia of section about the centroidal axis, mm⁴

L_R reduced live load as defined by NZS 4203

M_D moment due to dead load, N mm

M_i ideal flexural strength of section, N mm

M_{LR} moment due to the reduced live load L_R, N mm

M_u required moment strength due to factored loads, N mm

p reinforcement area ratio with respect to concrete surround within 15 bar diameters (see eq. 4-A, C4.4.2.2)

P_b axial load at balanced condition, N

P_i ideal axial load strength at given eccentricity, N

r base radius of rolled-on transverse deformation on reinforcing bar, mm

r_s ratio of steel stress at first cracking to steel stress at design load

s spacing of shear or torsion reinforcement along a member, mm

s spacing of longitudinal reinforcing bars, mm

s_r crack spacing, mm

v_c nominal permissible shear stress carried by concrete, MPa

V_c ideal shear strength provided by concrete, N

V_D shear force due to dead load, N

V_i ideal shear strength of section, N

V_{LR} shear force due to the reduced live load, L_R, N

V_s ideal shear strength provided by reinforcement, N

V_u factored shear force for applied loads, N

w_k characteristic crack width, mm

w_m average crack width, mm

γ ratio of distance between centroids of tensile and compressive reinforcement to overall depth of the member

ϵ_m parameter used in eq. 4-B for average crack width (see C4.4.2.2)

ϵ_s steel tensile strain

C4.2 General

The Code requires that strength be adequate to support the factored loads and applied actions and that serviceability of the structure at the service load level be assured. For actions other than flexure, the strength and detailing provisions are considered to provide satisfactory performance for both strength and serviceability at service loads. For flexure, special serviceability provisions include those for deflection, cracking, vibration and fatigue.

C4.3 Strength

C4.3.1 *General requirements*

C4.3.1.1 The basic requirement for strength design may be expressed as follows [4.2, 4.3]:

Required strength $\leqslant$ design strength

$U \leqslant \phi$, ideal strength

In the strength design procedure, the margin of structural safety is provided in the following two ways:

(a) The "required strength" U is computed by multiplying the service loads by load factors [4.1] greater than one to provide for excess load effects from such possible sources as overloads and simplified assumptions in structural analysis [4.1, 4.2]. The required load factors for buildings are given in NZS 4203 and for highway bridges are given in the MWD Highway

Bridge Design Brief [4.4]. Thus, for example, the factored moment M_u or "required moment strength" for dead and live load on a building is computed as:

$$U = 1.4D + 1.7L_R$$

or $$M_u = 1.4M_D + 1.7M_{LR}$$

where M_D and M_{LR} are the moments due to service dead and reduced live loads.

(b) The "design strength" of a structural element is computed by multiplying the ideal strength by a strength reduction factor ϕ which is less than one. The strength reduction factor accounts for uncertainties in design computations and relative importance of various types of members, and provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits, may combine to result in understrength [4.1, 4.2]. The ideal strength is computed by the Code procedures assuming that the member will have the exact dimensions and material properties used in the computations [4.5]. Thus, for example, the design strength in flexure of a cross-section (without compression reinforcement) may be expressed as:

$$\phi M_i = \phi \left[A_s f_y \left(d - \frac{a}{2}\right)\right]$$

When the above two safety provisions are combined, the basic requirement for the design of a beam cross-section can be stated as:

Required strength $\leqslant$ design strength

$$M_u \leqslant \phi M_i$$

$$1.4M_D + 1.7M_{LR} \leqslant \phi \left[A_s f_y \left(d - \frac{a}{2}\right)\right]$$

Similarly, for shear strength of a beam (normal density concrete) the basic requirement for strength design can be stated as:

Required strength $\leqslant$ design strength

$$V_u \leqslant \phi V_i$$

$$\leqslant \phi(V_c + V_s)$$

$$1.4\,V_D + 1.7\,V_{LR} \leqslant \phi \left[v_c b_w d + \frac{A_v f_y d}{s}\right]$$

The notations and definitions presented above for the strength design procedure were introduced in ACI 318-77 to alleviate confusion expressed by code users in applying the terminology of ACI 318-71, which used a single subscript "u" to denote both required strength and design strength of a member.

For this Code, all notations with the subscript "u" such as M_u, P_u, and V_u refer only to the required strength values. The design strength values are denoted by (ϕ x ideal strength), such as ϕM_i, ϕP_i, and ϕV_i.

C4.3.1.2 The design strength of a member, as used in this Code, is the ideal strength calculated in accordance with the provisions and assumptions stipulated in the Code multiplied by a strength reduction factor ϕ, which is always less than one except as allowed in 4.3.2. The rules for computing the ideal strength of a member are based on chosen limits of stress, strain, cracking or crushing, and conform to research data for each type of structural action.

The purpose of the strength reduction factor ϕ is:

(a) To allow for the probability of understrength members due to variations in material strengths and dimensions;

(b) To allow for inaccuracies in the design equations;

(c) To reflect the degree of ductility, and required reliability of the member under the load effects considered; and

(d) To reflect the importance of the member in the structure [4.1, 4.2].

For members subject to flexure and relatively small axial loads, failure is initiated by yielding of the tension reinforcement and takes place in an increasingly ductile manner as the ratio of axial load to moment decreases. At the same time, the variability of the strength also decreases. Hence, for small axial loads it is reasonable to permit an increase in the ϕ factor from that required for compression members. Thus when the axial load is zero and the member is subjected to pure flexure, the strength reduction factor ϕ becomes 0.90. To account for the increased ductile behaviour a varying ϕ factor is permitted. The ϕ may be increased from that for compression members to the 0.90 value permitted for flexure as the required axial load strength decreases from $0.10f'_cA_g$ to zero.

The value of required axial load strength $0.10f'_cA_g$, below which ϕ may be increased, corresponds to the provisions of ACI 318-77 for sections with symmetrical reinforcement provided f_y does not exceed 415 MPa and the distance γh (distance between A_s and A'_s) is not less than 0.7 h. When these conditions are not satisfied, ACI 318-77 requires that the axial load at balanced condition P_b be calculated to determine the upper value of design axial load strength ϕP_i (lesser of $0.10f'_cA_g$ or P_b) below which an increase in ϕ can be made. This latter requirement has not been included in this Code because for columns with steel which does not have a definite yield point, such as prestressing steel, or with steel distributed around the perimeter of the section, the definition of P_b loses its meaning.

For columns with confining reinforcement complying with 6.5.4.3 (a) or (b), recent large-scale testing has shown that the confinement leads to increased effective concrete strength in the confined core, at strains less than .004. Providing the column strength is calculated in accordance with 6.3.1 an 'overcapacity' of at least 15%, and considerably more in some cases, will exist. Furthermore, the ductility of such members has been demonstrated, with member displacement ductility factors of 6 being readily attained. Consequently the ϕ value for such members is set at 0.9 in recognition of this reserve strength. The approximation inherent in this approach justifies omitting any adjustment of ϕ with axial stress, particularly as there are indications that overstrength effect increases with increased axial stress (and consequent increased confining steel quantity). The advantage of a constant ϕ value for design purposes is obvious.

If design assumptions other than those set out in 6.3.1.7 are used for strength calculations to more nearly represent the stress-strain curve for confined concrete, a ϕ value for 'other members' in accordance with 4.3.1.2 (c) is recommended.

Figure C4.1 illustrates the variation in ϕ for columns or piers satisfying different Code requirements for lateral reinforcement. These provisions differ from those of ACI 318-77 in that ϕ is related to the required axial load strength P_u rather than to the ideal strength P_i. The latter requires an extra complication in that neither ϕ nor P_i are initially known whereas P_u ($\leqslant \phi P_i$) is known from factored loads. Furthermore, it appears logical to base ϕ on P_u since it is the applied load rather than the ideal strength which will govern reliability of strength and ductility.

In all cases in which axial tension force occurs (with or without flexure), $\phi = 0.9$ is permitted.

The ϕ factor for bearing on concrete in this Section does not apply to post-tensioning anchorage bearing plates. Refer instead to 13.3.9.

C4.3.1.3 Development lengths for reinforcement do not require a strength reduction modification.

The specified development lengths in Section 5 already include an allowance for understrength.

Likewise, ϕ factors are not required for splices since splice lengths are expressed in multiples of the development lengths.

C4.3.1.4 The upper limit of 550 MPa on the yield strength of reinforcing steels other than prestressing tendons was chosen for the 1977 and previous ACI Codes on the basis that the yield strain for 550 MPa steel is about equal to the maximum usable strain in unconfined concrete in compression.

The maximum reinforcing steel yield strength covered by NZS 3402 is 380 MPa. Also, no ASTM specification for deformed bars with a yield strength, f_y, exceeding 415 MPa is currently available. Before using steels with greater strengths than the above, the designer should ascertain their properties to ensure that they are suitable for the intended application.

Note that in other sections of the Code yield strengths are limited to 415 MPa; namely, 7.3.6 for shear and torsion reinforcement, and 3.5.4.3 for seismic force resisting members.

The deflection provisions of 4.4.1 and the crack width criteria of 4.4.2 become increasingly critical as f_y increases.

C4.3.2 *Additional requirements for members designed for seismic loading*

Where members are designed on the basis of load combinations such as $D + L_R + E$ or $0.9D + E$ and for overstrengths of adjacent members, the maximum likely actions that could ever develop on the section are being considered. Examples are where beam shears are derived from beam flexural overstrengths at plastic hinges, where joints are designed from overstrength input into the joint and where

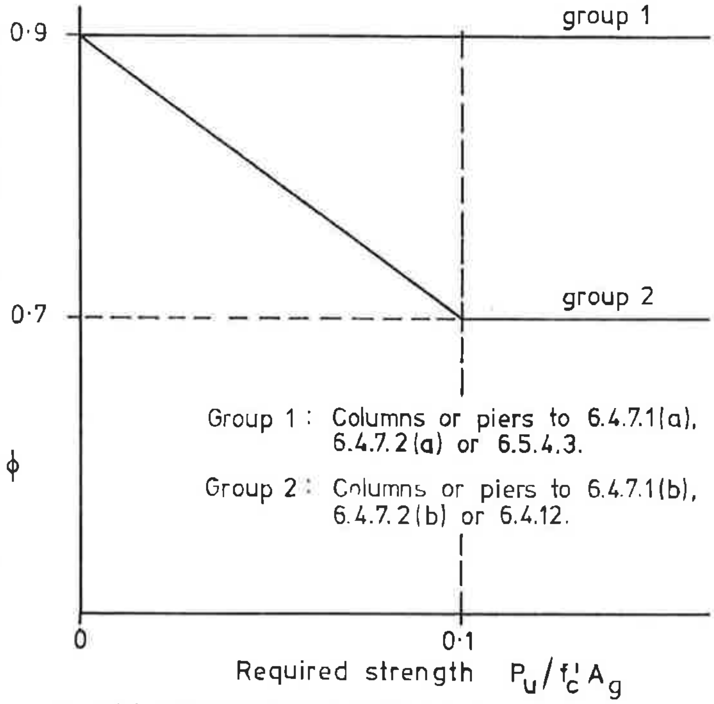


Fig. C4.1 VARIATION OF ϕ FOR COLUMNS OR PIERS

column actions are derived in accordance with the Appendix to Commentary Section 3. For this extreme loading case additional reserve strength as will usually be provided by adopting a ϕ of less than 1 is not considered to be necessary. Consequently in capacity design, strength reduction factors need not be used, that is it may be assumed that $\phi = 1.0$.

C4.4 Serviceability

C4.4.1 *Deflection*

C4.4.1.1 *General.* The provisions of 4.4.1 are concerned only with the deflections or deformations which may occur at service load levels. Where long-time deflections are computed, only the dead load and that portion of the live load which is sustained need be considered.

With the increased use of high strength materials and more sophisticated methods of design that provide depths of sections somewhat less than those used in the past, it becomes increasingly important to give attention to control of deflections at service loads.

Two methods are given for controlling deflections. For non-prestressed beams and one-way slabs, and for composite members, provision of a minimum overall thickness as required by table 4.1 will satisfy the requirements of the Code for members not supporting or attached to partitions or other construction likely to be damaged by large deflections. For non-prestressed two-way construction, minimum thicknesses as required by 4.4.1.2 (b) will satisfy the requirements of the Code. Also, for flexural members of bridge structures, minimum member thicknesses as required by table 4.2 will satisfy the requirements of the Code.

For non-prestressed members which do not meet these minimum thickness requirements or which support or are attached to partitions or other construction likely to be damaged by large deflections, and for all prestressed concrete flexural members, deflections must be calculated by the procedures described or referred to in 4.4.1.3 and must be limited to the values nominated in 4.4.1.4.

C4.4.1.2 *Minimum thicknesses*

One-way construction (non-prestressed) for buildings

The minimum thicknesses of table 4.1 apply for non-prestressed beams and one-way slabs and for composite members. The minimum thickness values may be used in lieu of calculation of deflections only for these types of members and only if these members do not support and are not attached to partitions or other construction likely to be damaged by larger deflections.

The values of table 4.1 must be modified as indicated in the footnotes if other than normal density concrete and Grades 275 or 380 reinforcement are used. If both of these variations exist, both sets of corrections should be applied. The modification for lightweight concrete is based on studies of the results and discussions in reference [4.6]. No correction is given for concretes with density between 1850 and 2400 kg/m^3 since the correction term would be close to unity in this range. The modification for yield strength is based on judgement, experience, and studies of the results of tests and of unpublished analyses. The simple expression given is approximate but should give conservative results for the types of members considered in the table, for typical reinforcement ratios, and for values of f_y between 275 and 550 MPa.

If the minimum thickness obtained using this table is considered excessive, the designer has the option of computing deflections in accordance with 4.4.1.3 and 4.4.1.4.

Two-way construction (non-prestressed) for buildings

Deflections of two-way systems of construction of the types considered in Section 11 need not be computed if the minimum overall thickness requirements of this section are satisfied. Equations 4-1, 4-2 and 4-3 will provide an overall thickness consistent with that found from experience to provide satisfactory control of deflections for flat slabs, flat plates and conventional two-way slabs supported on stiff beams. The equations provide for a transition from slabs on stiff beams to slabs without beams, and involve a term to adjust the thickness as a function of the design yield strength of the reinforcement. The effect of reinforcement yield strength in these equations is different from in the footnote to table 4.1 because the degree of cracking has been observed to be less in two-way slabs than in beams and one-way slabs, with a consequent smaller effect of steel stress or strain on the stiffness of the element. This conclusion was reached and the form of the expression involving yield strength in equations 4-1, 4-2 and 4-3 was chosen after study of the results of the extensive tests on floor slabs described in the references listed in commentary for Section 11. For the benefit of designers the limits on thickness of two-way slabs corresponding to equations 4-1, 4-2 and 4-3 have been plotted in fig. C4.2. It may be seen that where α_m exceeds 2, such as may be expected where the flexural stiffness of the beams on the edges of a panel are governed by seismic considerations, the minimum slab thickness corresponding to eq. 4-2 will govern.

Composite members for buildings

In terms of this Code composite members refer to members comprising a combination of concrete elements, precast or cast *in situ*.

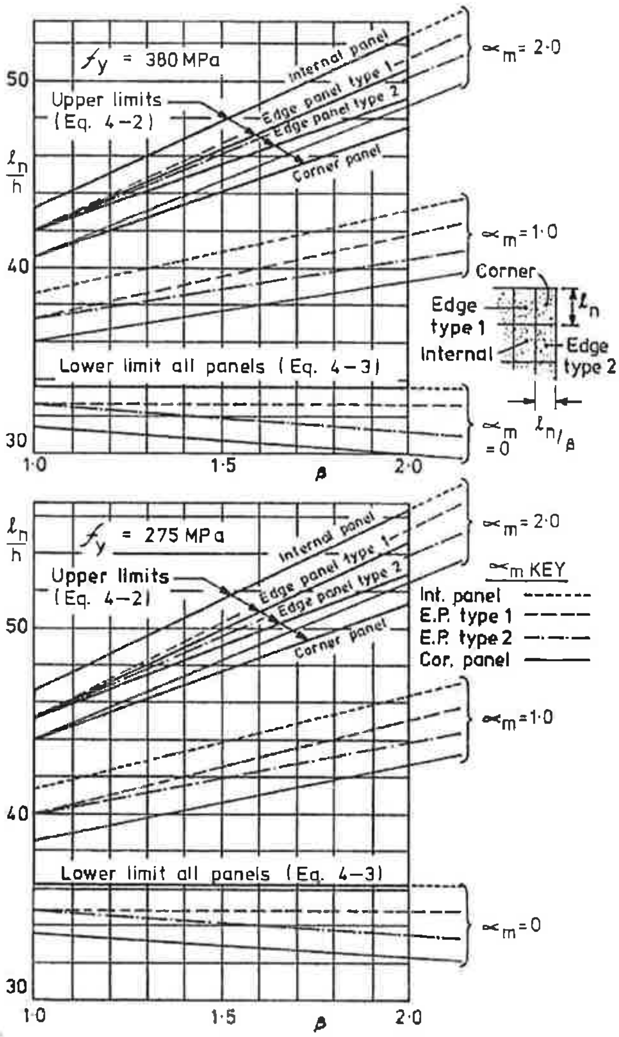


Fig. C4.2 MINIMUM THICKNESS OF TWO-WAY SLABS

Composite structural steel – concrete members are covered in NZS 3404.

Bridge structures

Deflection of bridges is not usually critical. The superstructure will often have a built-in camber to account for deflection under dead load. In some cases it may be important to limit deflections under long-time loadings or live loads for aesthetic or other reasons. The minimum thicknesses specified in table 4.2 are based on AASHTO [4.7] requirements. They are introduced primarily to guard against excessive traffic-induced vibrations giving concern to pedestrians or occupants of stationary vehicles. These requirements may be waived if special consideration is given to design for vibration [4.4].

The recommended values of table 4.2 are for continuous spans; simple spans should have about 10% greater thickness.

C4.4.1.3 *Computation of deflection*

One-way construction (non-prestressed)

For the calculation of immediate deflections of uncracked prismatic beams, the usual methods or formulae for elastic deflections may be used with a constant value of $E_c I_g$ along the length of the beam. However, if the beam is cracked at one or more sections or if its depth varies along the span, a more exact calculation becomes necessary. The I_e procedure described in the Code and developed in reference [4.8] was selected as being relatively simple and sufficiently accurate for use with limiting values in 4.4.1.4 to control deflections [4.9, 4.10, 4.11].

It is noted that for additional load increments, such as live load, I_e must be computed for total moment M_a, and the deflection increment computed from the total deflection, as indicated by equations 29 to 32 in references [4.11] and [4.12]. For simplicity in the case of continuous beams, the Code procedure suggests a simple averaging of positive and negative moment values for I_e. In certain cases, a weighted average relative to the moments may be preferable, such as the methods suggested in reference [4.12].

For normal density concrete, the value of f_r required for the calculation of the cracking moment is given as $0.6\sqrt{f_c'}$. Modifying factors based on the splitting tensile strength f_{ct} are given for "all-lightweight" and "sand-lightweight" concretes. For a lightweight aggregate from a given source, it is intended that appropriate values of f_{ct} should be obtained in advance of design, but tests for f_{ct} are not required for subsequent acceptance of concrete during construction. Indirect control will be maintained through the normal compressive strength test requirements.

Shrinkage and creep due to sustained loads cause additional deflections over and above those which occur when loads are first placed on the structure. The additional deflections are called "long-time deflections". Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, magnitude of the sustained load, and other factors.

The expression given for K_{cp} (eq. 4-7) should be taken as giving minimum likely creep values. Creep studies show a variation of results of up to 400% depending on the aggregate used. Current (1980) research on creep characteristics of test specimens using different New Zealand aggregates is investigating a more accurate expression. In the meantime available information includes reports of ACI Committees 435 [4.9, 4.13] and 209 [4.11], and results of New Zealand Research [4.14, 4.15].

It should be noted that the deflection computed in accordance with this Clause is the *additional* long-time deflection due to the dead load and that portion of the live load which will be sustained for a sufficient period to cause significant time-dependent deflections. Figure C4.3 [4.16] may be used to estimate the *additional* long-time deflection at various time periods. A net multiplier can be obtained from fig. C4.3 when there is a time lag between removing of shores and placing of non-structural elements.

Two-way construction (non-prestressed)

The calculation of deflections for slabs is complicated even if linear elastic behaviour can be assumed. For immed-

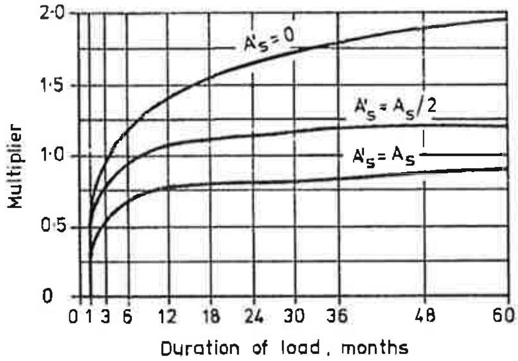


Fig. C4.3 MULTIPLIERS FOR LONG-TIME DEFLECTIONS

iate deflections, the values of E_c and I_e specified in 4.4.1.3 may be used [4.11]. However, other procedures and other values of the stiffness, EI, may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests. Such a procedure, for example, is described in reference [4.17].

Since the available data on long-time deflections of slabs are too limited to justify more elaborate procedures, the additional long-time deflection for two-way construction is required to be computed using the multiplier given in 4.4.1.3 (a) (2).

Prestressed concrete construction

The Code requires deflections for all prestressed concrete flexural members to be computed and compared with the allowable values nominated in 4.4.1.4.

Immediate deflections of prestressed concrete members may be calculated by the usual methods or formulae for elastic deflections using the moment of inertia of the gross (uncracked) concrete section and the modulus of elasticity for concrete specified in 3.3.4.1.

It has also been shown in reference [4.9] that the I_e method can be used to compute deflections of partially prestressed members loaded above the cracking load. In this case, the cracking moment must, of course, take into account the effect of prestress. The requirement in 13.4.1.3 for calculation of deflection of partially prestressed concrete members may be satisfied by this method or by consideration of the transformed cracked section.

A method for predicting the effect of non-prestressed tension steel in reducing creep camber is given in reference [4.9] with approximate forms referred to in references [4.11] and [4.12].

The calculation of long-time deflections of prestressed concrete flexural members is complicated. The calculations must consider not only the increased deflections due to flexural stresses, but also the additional long-time deflections resulting from time dependent shortening of the flexural member.

Prestressed concrete generally shortens more with time than similar non-prestressed members. This is due to the pre-compression in the slab, beam or girder which causes axial creep. This creep together with shrinkage of the concrete, results in significant shortening of the flexural members which continues for several years after construction and must be considered in design. The shortening tends to reduce the tension in the prestressing tendons, thus reducing the pre-compression in the member and thereby causing increased long-time deflections.

Another factor that can influence long-time deflections of prestressed concrete flexural members is adjacent concrete or masonry non-prestressed in the direction of the prestressed member. This can be a slab non-prestressed in the beam direction adjacent to a prestressed beam, or a non-prestressed slab system. As the prestressed member tends to shrink and creep more than the adjacent non-prestressed concrete, the structure will tend to reach a compatibility of the shortening effects. This results in a reduction of the pre-compression in the prestressed member as the adjacent concrete absorbs the compression. This reduction in pre-compression of the prestressed member can occur over a period of years and will result in additional long-time deflections and in increased stresses in the prestressed member.

Any suitable method for calculating long-time deflections of prestressed concrete members may be used, provided all effects are considered. Guidance may be found in references [4.11, 4.13, 4.18] and [4.19].

Composite members

Since few tests have been made to study the immediate and long-time deflections of composite members, the rules given in 4.4.1.3 (d) were based on the judgement of ACI Committee 318 and on experience.

If any portion of a composite member is prestressed or if the member is prestressed after the components have been cast, the provisions of 4.4.1.3 (c) apply and deflections must be calculated. For non-prestressed composite members, deflections need to be calculated and compared with the limiting values in 4.4.1.4 only when the thickness of the member is less than the minimum thickness given in table 4.1 or 4.2. In unshored construction the relevant thickness depends on whether the deflection before or after the attainment of effective composite action is being considered. In 8.3.1.4 it is stated that distinction need not be made between shored and unshored members. This refers to strength calculations, not to deflections.

C4.4.1.4 *Allowable deflection.* It is recommended that the limits in table 1 of NZS 4203 be applied for buildings. AASHTO [4.7] recommended the following values for bridges:

"Members having simple or continuous spans shall be designed so that the deflection due to live load plus impact shall not exceed 1/800 of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall be 1/1000.

The deflection of cantilever arms due to live load plus impact shall be limited to 1/300 of the cantilever arm except for the case including pedestrian use, where the ratio shall preferably be 1/375."

C4.4.2 *Cracking*

C4.4.2.1 *General.* The occurrence of cracks in reinforced concrete structures is inevitable because of the low tensile strength of concrete. Wide cracks may occur with high service load steel stresses, particularly as a result of the use of high-strength steel. The cracking should not be such as to spoil the appearance of the structure or to lead to corrosion of the reinforcement. The conditions nominated in 4.4.2.1 are considered to be those in which there is a possibility of cracking being critical and only in these cases must the requirements of 4.4.2.2 and 4.4.2.3 be followed. Although calculation of crack widths is required where the environment is aggressive, this obviously would not apply to prestressed concrete sections designed on the basis of zero tensile stress, that is a no-cracking condition.

Although the cracking provisions in this clause are based on calculation of surface crack widths, it must be recognized that other factors may be just as important, for example the width of the crack at the level of the reinforcing steel, the length of the crack, concrete quality and compaction may all be as significant with respect to corrosion potential.

The provisions make no distinction between members of different depth. For thin members such as slabs and shells, the crack width at the tensile reinforcement will be much less than at the level of the reinforcement for deeper members, for a particular surface crack width. However, it was not considered warranted to attempt a more sophisticated assessment in view of the approximations inherent in the approach. Also, the crack width restrictions serve to avoid unacceptable visual effects and, in the case of reservoirs, inadequate watertightness. In these instances surface crack width may, in fact, be the most appropriate parameter.

C4.4.2.2 Despite considerable research on cracking of reinforced concrete members, there is still no satisfactory theory available to accurately predict such behaviour. A statistical analysis of available data by Gergely and Lutz[4.21] showed the significance of the variables involved, the most important being the effective area of concrete in tension, the number of bars, the side or bottom cover, the strain gradient from the level of the steel to the tension face, and the steel stress. Equation 4-8 is derived from this study. In view of the scatter of test results it was not considered warranted to require use of any of the more complicated formulae available.[4.22]

The effective tension area of concrete surrounding the main reinforcement, A in eq. 4-8, is defined as having the same centroid as that reinforcement. This area is to be bounded by the surfaces of the cross-section and a straight line parallel to the neutral axis. Computation of the effective area per bar, A, is illustrated by the example shown in fig. C4.4. The centroid of the main reinforcement is located 75 mm from the bottom of the beam. The effective tension area is then taken as twice 75 mm times the beam width, b. Divided by the number of bars, this gives 9 000 mm^2 in the example shown.

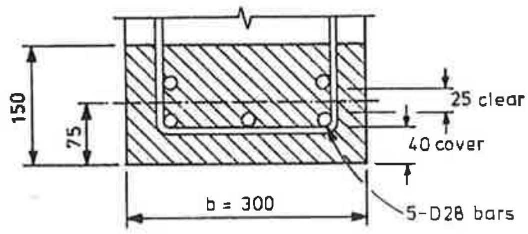


Fig. C4.4 CALCULATION OF EFFECTIVE AREA OF CONCRETE IN TENSION

When the main reinforcement consists of several bar sizes, the number of bars is to be calculated as the total steel area divided by the area of the largest bar used.

Laboratory tests have shown that the Gergely–Lutz expression of eq. 4-8 applies reasonably well to one-way slabs. An alternative expression for calculation of crack widths in two-way slabs is given in the Commentary Section 10.6.5 of the ACI 318-71 Code.

The Code allows calculation of crack widths at the surface of prestressed concrete members "by suitable methods". A review of available methods is given in references [4.23] and [4.24]. Crack width calculation methods fall within three general groups, based on:

(a) Statistical analysis of experimental results

(b) A physical model of the cracking mechanism

(c) A fictitious tensile stress approach.

Of the first group the Gergely–Lutz [4.21] equation is typical and widely applied to reinforced concrete. Testing of partially prestressed concrete beams [4.25] with deformed reinforcement between the prestressing tendon and the tensile concrete surface showed the Gergely–Lutz equation to be a realistic design expression, with measured crack widths under static and repeated loading generally not exceeding 80% of the predicted.

For the second group, equations have been proposed on the basis of physically defined cracking mechanisms. The two common models are, first, the cover controlled or no-slip mechanism in which wedge-shaped cracks between the tension surface of the concrete and the reinforcement develop with spacing and width proportional to the concrete cover, and second, the classical bond-slip approach in which the crack dimensions depend on a bar bond parameter. The effects of both mechanisms are summed in the latest CEB–FIP [4.26] approach. The design assumptions for reinforced concrete sections may be applied to prestressed concrete by calculating the excess tension in the tendons relative to tension at zero strain in the concrete at the level of the tendon. The CEB–FIP approach is:

Crack spacing s_r is first calculated according to:

$$s_r = 2\,(c + s/10) + a_1\,a_2\,d_b/p \qquad \text{(Eq. 4-A)}$$

where

c = concrete cover

s = reinforcement bar spacing

a_1 = 0.4 for deformed, 0.8 for plain bars

a_2 = 0.125, 0.250 for bending and axial tension respectively

d_b = bar diameter

p = reinforcement area ratio with respect to concrete surround within 15 bar diameters.

Average crack width, w_m, is then determined allowing for tension stiffening reduction progressively to a zero asymptote as the moment increases beyond the first cracking moment:

$$w_m = s_r\,\epsilon_m \qquad \text{(Eq. 4-B)}$$

where

$$\epsilon_m = \epsilon_s\,(1 - b_1\,b_2\,r_s^2) \geqslant 0.4\,\epsilon_s \qquad \text{(Eq. 4-C)}$$

r_s = ratio of steel stress ($0 < r_s < 1$, cracked section) at first cracking to steel stress at design load.

b_1 = $0.4/a_1$

b_2 = 1 for a single loading, 0.5 for sustained or repeated loading

ϵ_s = steel tensile strain

The characteristic crack-width is then determined from the calculated average from $w_k = 1.7\,w_m$ and limiting values for w_k for different exposure conditions lie between 0.1 mm and 0.4 mm. The 1978 CEB–FIP [4.26] prediction of crack widths substantially exceeds that in accordance with the 1970 CEB–FIP code but is quite similar to the Gergely–Lutz prediction of eq. 4-8 when applied to partially prestressed concrete sections.

The third of the groups, the fictitious tensile stress approach was introduced by Abeles [4.27] specifically for partially prestressed beams. Crack widths are related to nominal fictitious stresses at the concrete tension surface calculated assuming an uncracked concrete section. The fictitious concrete tensile stress approach has the advantage of simplicity and avoids the inaccuracies inherent in calculating the magnitude and statistical distribution of crack widths and relating these to acceptable serviceability criteria. It is, however, unlikely that the procedure can be developed to accommodate all cases. For the CP110 [4.22] fictitious tensile stress approach, the moment in excess of the decompression moment is dependent on fictitious concrete tensile stresses specified for 0.1 mm and 0.2 mm crack widths for different concrete grades. Each stress is increased by an amount proportional to the quantity of non-prestressed reinforcement in the tensile zone. The fictitious tensile stresses are independent of concrete cover. Section 13 of the Code allows, as an alternative to crack width calculation, fictitious tensile stress limits of up to $0.5\sqrt{f_c'}$ MPa depending on load category.

The equations developed for prediction of crack widths in reinforced concrete members can be used for prestressed members where strands are used, with incremental steel stresses taken relative to zero concrete strain conditions at the level of the tendons. Hence the Gergely–Lutz formula of eq. 4-8 may be used. The behaviour of smooth-drawn wire prestressed tendons will be somewhat different, with internal failure more commonly by slip rather than internal cracking. The formulae of Beeby [4.28, 4.29] may be used with appropriate values of the coefficient reflecting the ratio of cover to crack height where cracking is uncontrolled by reinforcement.

C4.4.2.3 *Allowable crack widths.* The nominated allowable crack widths in table 4.3 are in line with those recommended in the CEB–FIP recommendations [4.26]. The difference between crack width limits for reinforced and prestressed concrete members is not as great as required in some codes. Although prestressing steel is generally regarded as being more sensitive to corrosion than reinforcing bars because of its higher stress and smaller diameter, prestressed members have an advantage of a greater tendency for crack closure after overload.

Account has been taken of the values presented by the

US Department of Transportation [4.20] for highway bridge members. Since those recommendations relate to crack width at the level of the reinforcement, the values of table 4.3 are higher. In view of the relaxation of allowable live load stress range in 4.5.1.2, and the crack width limits of table 4.3, substantial economies are possible for highway bridge deck slabs using grade 380 MPa rather than grade 275 MPa reinforcing steel. For the special requirements of concrete reservoirs reference should be made to DZ 3106 [4.30].

The appropriate loadings codes will give direction on where the loading categories of table 4.3 are to apply. In general, frequently repetitive loads may be considered as those which will occur more than 50 000 times in the life of the structure, and typically relate to highway bridges. Building loads will not, in general, be considered as frequently repetitive loads. The effects of repetitive loads cause difficulties regarding crack widths in loss of bond strength of the concrete.

For application of table 4.3 an "aggressive environment" may be regarded as one where the member is exposed to sea water or a marine atmosphere or other corrosive atmosphere or to aggressive groundwater. A member exposed to the weather but not in the above types of aggressive environment may be regarded as being in an "external environment". The interior cells of box girder bridges are regarded as being in an external environment because usually provisions are not made to exclude dampness.

It should be noted that the control of cracking depends not only on the quantity of reinforcement provided but also on its distribution in the tension zone. Limitations of spacing of tendons and reinforcement are given in 5.3.5.

Where unbonded tendons are used without supplementary mild steel, the design criterion is to avoid cracking under all loading categories of table 4.3.

C4.4.3 *Vibration*

Guidance for reducing resonance in buildings containing machinery is given in NZS 4203, clause C1.4.3.

Recommendations for the design of bridges to avoid unacceptable traffic induced vibrations are given in the MWD Highway Bridge Design Brief. [4.4]

C4.5 Other considerations

C4.5.1 *Fatigue*

C4.5.1.1 Members in some structures, for example deck slabs of bridges, will be subject to large fluctuations of stress under repeated cycles of live loading.

C4.5.1.2 The limitations on range of stress of 150 MPa under live load, irrespective of the grade of reinforcing used, is based on AASHTO standards [4.7] and was considered necessary to avoid the possibility of fatigue failure in the reinforcing steel. The range of stress of 150 MPa is allowed for reinforcing steel or for adequately bonded prestressing steel. The lower value of 100 MPa in table 13.2 Load Category II is to make allowance for possible bond deficiencies of grouted prestressing tendons. The effect of the 150 MPa range is usually to limit crack widths to approximately 0.25 mm. This requirement supersedes, where appropriate, the allowable stress limits of Appendix B, Clause B4.2.

The allowed relaxation of the requirements of this Clause, if a "special study" is made, is in recognition that some opinion [4.24, 4.31] believes these requirements to be too conservative. The requirements of a "special study" may be deemed to be satisfied if the following revised AASHTO procedures are followed:

Concrete

The range between the maximum compressive stress ($f_{max.}$), and the minimum compressive stress ($f_{min.}$), in the concrete at points of contraflexure and at sections where stress reversals occur, shall not exceed $0.5 f'_c$.

$f_{sr} = f_{max.} - f_{min.}$ = stress range (MPa)

$f_{min.}$ = minimum compressive stress level in the concrete due to dead load, creep, shrinkage, temperature, etc. (MPa)

$f_{max.}$ = $f_{min.}$ plus the additional compressive stress due to live load plus impact (MPa)

The points of contraflexure in bridge deck slabs are excluded from this requirement.

Reinforcement

The range between the maximum tension stress ($f_{max.}$) and the minimum stress ($f_{min.}$) in straight reinforcement at service load level, shall not exceed:

$$f_{sr} = f_{max.} - f_{min.} = [145 - 0.33 f_{min.} + 55 (r/h)]$$

f_{sr} = stress range (MPa)

$f_{min.}$ = algebraic minimum stress level due to dead load, creep, shrinkage, temperature etc. (MPa) (tension positive, compression negative)

$f_{max.}$ = $f_{min.}$ plus the additional tension stress due to live load plus impact (MPa)

r/h = ratio of base radius to height of rolled-on transverse deformation; when actual value is not known use 0.3.

Bends in primary reinforcement shall be avoided in regions of high stress range.

Fatigue shall be checked for normal service live loads only. Overloads are specifically excluded from the requirements of this Clause.

C4.5.1.3 Bends in primary reinforcement should be avoided in regions of high stress range. The minimum diameter of bend of slab reinforcing steel, for example of cranked transverse reinforcing steel in bridge deck slabs, is increased to 20 bar diameters, because localized areas of high stress concentration under tight radius bends cause fatigue failure to propagate from these locations.

Reference [4.32] indicates the diminution of bond strength under repeated loads.

C4.5.2 *Fire resistance*

Fire is a primary accidental hazard during which it is necessary to ensure the carrying capacity and integrity of the structure for a defined period of time in order to permit the evacuation of occupants, to afford appropriate protection for fire-fighting services, to prevent the spread of fire and to protect adjoining property. NZS 1900 Chapter 5 gives the legal requirements. Other information is given in reference [4.33].

REFERENCES

4.1 MacGregor, J.G., "Safety and Limit States Design for Reinforced Concrete", *Canadian Journal of Civil Engineering*, Vol 3, No 4, December 1976, pp. 484–513.

4.2 Winter, G., "Safety and Serviceability Provisions in the ACI Building Code", ACI–CEB–FIP–PCI Symposium, *ACI Special Publication.*

4.3 Cornell, C.A., "A Probability Based Structural Code", *Proceedings ACI,* Vol 66, No 12, December 1969, pp. 974 985.

44. ———— , "Highway Bridge Design Brief", Ministry of Works and Development, Wellington, New Zealand, CDP 701/D, September 1978, 52 pp.

4.5 Allen, D.E., "Probabilistic Study of Reinforced Concrete in Bending", *Proceedings ACI,* Vol 67, 1970, pp. 989–993.

4.6 ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete", *Journal American Concrete Institute, Proceedings,* Vol 64, No 8, August 1967, pp. 443–464. Also Discussion by McManas, Ralph N. and Committee Closure, *Journal American Concrete Institute, Proceedings,* Vol 65, No 2, February 1968, pp. 151–155. Also *ACI Manual of Concrete Practice,* American Concrete Institute, Detroit.

4.7 ————, "Standard Specifications for Highway Bridges", The American Association of State Highway and Transportation Officials, 1977, 496 pp.

4.8 Branson, Dan E., "Instantaneous and Time-Dependent Deflections on Simple and Continuous Reinforced Concrete Beams", *HPR Report,* No. 7, Part 1, Alabama Highway Department, Bureau of Public Roads, August 1963 (1965), pp. 1–78.

4.9 ACI Committee 435, "Deflections of Reinforced Concrete Flexural Members, *Journal American Concrete Institute, Proceedings,* Vol 63, No 6, June 1966, pp. 637–674. Also *ACI Manual of Concrete Practice,* Part 2, 1968, American Concrete Institute, Detroit.

4.10 Subcommittee 1, ACI Committee 435, "Allowable Deflections, Journal American Concrete Institute, Proceedings, Vol. 65, No. 6, June 1968, pp. 433–444.

4.11 Subcommittee 2, ACI Committee 209, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures", Designing for the Effects of Creep, Shrinkage and Temperature in Concrete Structures, pp. 51–93, SP-27. ACI Publication, American Concrete Institute, Detroit, 1971.

4.12 Branson, D.E., Discussions of "Proposed Revision of ACI 318-63: Building Code Requirements for Reinforced Concrete" by ACI Committee 318, Journal American Concrete Institute, Proceedings, Vol. 67, No. 9, September 1970, pp. 692–695.

4.13 Subcommittee 5, ACI Committee 435, "Deflections of Prestressed Concrete Members, Journal American Concrete Institute, Proceedings, Vol. 60, No. 12, December 1963, pp. 1697–1728. Also ACI Manual of Concrete Practice, Part 2, 1968, American Concrete Institute, Detroit.

4.14 Fenwick, R.C., "Creep and Shrinkage of Concrete Bridges", Road Research Unit Bulletin No. 28, National Roads Board, New Zealand, 1974.

4.15 Bryant, A.H., "Effect of Environment on Creep and Shrinkage", University of Auckland, School of Engineering, Report No. 150, February 1977, 24 pp. Also published as Road Research Unit Bulletin No. 57, National Roads Board, New Zealand, 1981.

4.16 Yu, W.W., and Winter, G., "Instantaneous and Long-Time Deflections of Reinforced Concrete Beams under Working Loads", Journal American Concrete Institute, Proceedings, Vol. 57, No. 1, July 1960, pp. 29–50.

4.17 Vanderbilt., M.D.; Sozen, M.A. and Siess, C.P., "Deflections of Multiple-Panel Reinforced Concrete Floor Slabs, Proceedings, ASCE, Vol. 91, ST4, August 1965, pp. 77–101.

4.18 Branson, D.E.; Meyers, B.I. and Kripanarayanan, K.M., "Time-Dependent Deformation of Non-composite and Composite Prestressed Concrete Structures", Symposium on Concrete Deformation, Highway Research Record 432, Highway Research Board, 1970, pp. 15–43.

4.19 Shaikh, A.F. and Branson, D.E., "Non-Tensioned Steel in Prestressed Concrete Beams", Journal Prestressed Concrete Institute, Vol. 15, No. 1, February 1970, pp. 14–36.

4.20 ————, "Strength and Serviceability Criteria Reinforced Concrete Bridge Members", U.S. Department of Transportation, 1969, 97 pp.

4.21 Gergely, P. and Lutz, L.A., "Maximum Crack Width in Reinforced Flexural Members", Causes, Mechanism and Control of Cracking in Concrete, SP-20, American Concrete Institute, Detroit, 1968, pp. 88–117.

4.22 ————, "The Structural use of Concrete", British Standards Institution, CP 110 : Part 1 : 1972, p. 15, Appendix A.

4.23 Blaikie, E.L., "Partial Prestressing", ME Report, Department of Civil Engineering, University of Canterbury, 1973, 151 pp.

4.24 Dean, J.A.; Park, R.; Tian, L.C.; "Serviceability and Strength Criteria for Partially Prestressed Concrete Members". Proceedings of FIP Symposia on Partial Prestressing and Practical Construction, Bucharest, Romania, September 1980.

4.25 Tian, L.C., "Flexural Cracking in a Partially Prestressed Concrete Beam", ME Report, Department of Civil Engineering, University of Canterbury, February 1979, 88 pp.

4.26 CEB–FIP, "Model Code for Concrete Structures", Comité Euro-International du Béton, Fédération Internationale de la Précontrainte, 3rd Edition 1978, 348 pp.

4.27 Abeles, P.W., "Studies of Crack Widths and Deformations Under Sustained and Fatigue Loading", PCI Journal, Vol. 10, No. 6, December 1965.

4.28 Beeby, A.W.; Eyder, E.K. and Taylor, H.P.J., "Cracking and Deformation of Partially Prestressed Concrete Beams", Cement and Concrete Association Technical Report No. 42-465, January 1972.

4.29 Beeby, A.W., "The Prediction of Crack Widths in Hardened Concrete", The Structural Engineer, Vol. 57A, No. 1, January 1979, pp. 9–17.

4.30 ———, "Supplementary Draft New Zealand Standard Model Building Bylaw (NZS 1900) Chapter 11 – Special Structures, Division 11.1 Code for Design of Concrete Structures for the Storage of Liquids (to be NZS 3106)", Standards Association of New Zealand, 1978, 59 pp.

4.31 Hawkins, N.M., "Fatigue Design Considerations for Reinforcement in Concrete Bridge Decks", Journal American Concrete Institute, Proceedings, Vol. 73, No. 2, February 1976, pp. 104–114.

4.32 Perry, E.S. and Jundi, N., "Pullout Bond Stress Distribution Under Static and Dynamic Repeated Loadings", Journal American Concrete Institute, Proceedings, Vol. 66, No. 5, May 1969, pp. 377–380.

4.33 ———, "Fire properties of building materials and elements of structure", MP9, parts 1–10, Standards Association of New Zealand, 1980.

COMMENTARY

C5 REINFORCEMENT – DETAILS, ANCHORAGE AND DEVELOPMENT

C5.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 5 of the Code.

f_{sb} tensile stress at the start of a bend, MPa

ℓ_s splice length, mm

m modification factor for development length

C5.2 Scope

The provisions of Section 5 apply to detailing of reinforcement and design of anchorage, development and splices. Provisions cover minimum bend radii, covers, minimum reinforcement in walls, shrinkage and temperature reinforcement in slabs. Requirements are also given for spiral, circular and rectangular hoop reinforcement in columns and for stirrups and ties in flexural members.

The provisions for development include deformed and plain bars and wire, bundled bars, welded and smooth wire fabric and prestressing strand.

They also cover standard hooks in tension, mechanical anchorage and anchorage of transverse reinforcement. A comprehensive set of requirements is given to govern development of flexural reinforcement. Provisions for splices deal with lap splices, welded splices and mechanical connections.

C5.3 General principles and requirements

C5.3.1 *Steel reinforcement*

C5.3.1.1 In general, plain round bars are preferable for ties and stirrups because the small radius bends which are required have undesirable metallurgical and mechanical effects on deformed bars. Also, in most situations ties and stirrups do not rely on high bond strengths along their straight legs for their action. However, there are some cases, such as deep wall beams, where it may be necessary for stirrups and ties to develop high bond values along their straight portions. In such cases it is acceptable to use deformed bars, provided that the radii satisfy 5.3.4.1.

Where fatigue criteria govern, it may be considered desirable to use plain round bars for flexural reinforcement.

C5.3.2 *Hooks*

The standard hooks defined in this Section are shown in fig. C5.1.

C5.3.3 *Minimum bend diameters*

The minimum bend diameters given in table 5.1 are generally twice the bend test diameters specified in NZS 3402P.

Recent tests[5.1] indicate that for large diameter bars manufactured in New Zealand it may be necessary to adopt bend radii larger than those permitted in 5.3.3 because of strain embrittlement of the steel within the bend.

When large steel stresses need to be developed in the bend, radial bearing stresses in the concrete may become excessive. Equation 5-1 controls the diameter of the bend when there is a combination of high tensile stress in the bend, large bar diameter and low concrete strength. It has

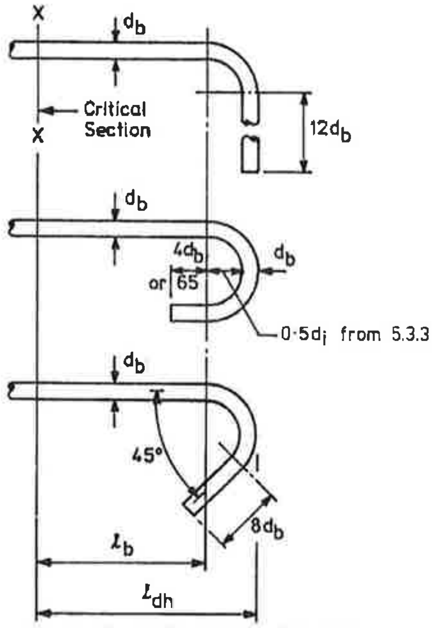


Fig. C5.1 STANDARD HOOKS

been adopted from the CEB-FIP recommendations[5.2]. Values of d_i for the extreme condition when $f_{sb} = f_y$ are given in table C5.1 for Grade 275 and Grade 380 reinforcing bars, for the typical value of s_b = 40 mm + d_b and two concrete strengths.

f_{sb} is calculated by eq. C5-1

$$f_{sb} = (1 - \frac{\ell_b}{\ell_d}) f_y \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(Eq. C5-1)}$$

ℓ_b is shown on fig. C5.1

In table C5.1, all cases except where f'_c = 30 MPa, f_{sb} = 380 MPa and d_b = 6 mm require values of d_i greater than those given in table 5.1. For these cases, if it is desired to use the values of table 5.1, the designer can do so providing a short length, ℓ_b between the critical section and the start of the bend, in order to reduce f_{sb} as calculated by eq. C5.1. The values of ℓ_b/ℓ_d required to allow the use of table 5.1 diameters are given in table C5.2.

Table C5.1 MINIMUM BEND DIAMETERS BY EQ. 5-1

Nominal bar dia.	*Minimum bend dia.* d_i *to inside of bar*			
	f'_c = *20 MPa*		f'_c = *30 MPa*	
d_b	f_{sb} = *275*	f_{sb} = *380*	f_{sb} = *275*	f_{sb} = *380*
6	49	67	32	45
10	90	124	60	83
12	113	156	75	104
16	161	223	108	149
20	214	296	143	197
24	270	372	180	248
28	328	453	218	302
32	388	536	259	357
40	513	709	342	473

Based on: s_b = 40 mm + d_b $\quad f_{sb} = f_y$

Clause 5.3.3 also permits the addition of transverse bars to allow the use of table 5.1 values for d_i in cases where eq. 5-1 would call for larger values.

Table C5.2 MINIMUM VALUES OF ℓ_b/ℓ_d TO ALLOW USE OF BEND DIAMETERS GIVEN IN TABLE 5.1

Nominal bar dia. d_b	ℓ_b/ℓ_d f'_c = *20 MPa* f_y = *275*	f'_c = *20 MPa* f_y = *380*	f'_c = *30 MPa* f_y = *275*	f'_c = *30 MPa* f_y = *380*
6	0.38	0.28	0.07	0
10	0.44	0.35	0.17	0.04
12	0.47	0.38	0.20	0.08
16	0.51	0.43	0.26	0.14
20	0.53	0.46	0.30	0.19
24	0.56	0.36	0.34	0.03
28	0.58	0.38	0.36	0.07
32	0.51	0.40	0.26	0.10
40	0.53	0.44	0.30	0.15

The arrangement of the transverse bars is shown in fig. C5.2, and is based on the fact that excessive bearing stresses will not extend past the first 60° of bend that is closest to the critical section.

The transverse bars should extend for a distance of at least d_b beyond the centreline of the outermost bars in each layer.

C5.3.4 *Stirrup and tie bends*

These minimum diameters of bends are the test diameters as specified in NZS 3402P. It is not intended that a tie have different bend diameters should it pass round longitudinal bars of different diameters.

Welded wire fabric, of plain or deformed wire, can be used for ties and stirrups. The wire at welded intersections does not have the same uniform ductility and ease of bending as in areas which were not heated. These effects of the welding temperature are usually dissipated in a distance of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the bend test for wire material. The requirements of 5.3.4.2 for welded wire fabric are shown in fig. C5.3.

C5.3.5 *Spacing of reinforcement*

C5.3.5.1 The spacing limits of this Clause have been developed from successful practice over many years, remaining essentially unchanged through many codes. The minimum limits were established to permit concrete to flow readily into spaces between bars and forms without honeycomb, and to ensure against concentration of bars on a line that might result in shear or shrinkage cracking.

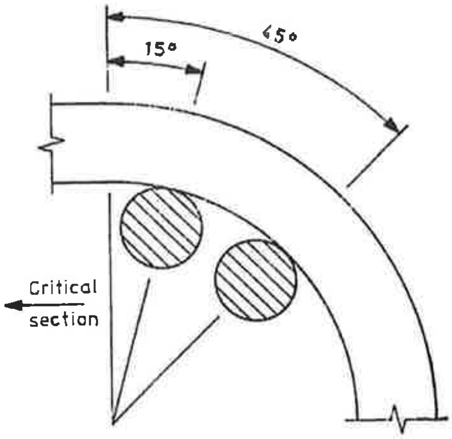


Fig. C5.2 ARRANGEMENT OF ADDITIONAL TRANSVERSE BARS TO REDUCE BEARING STRESS

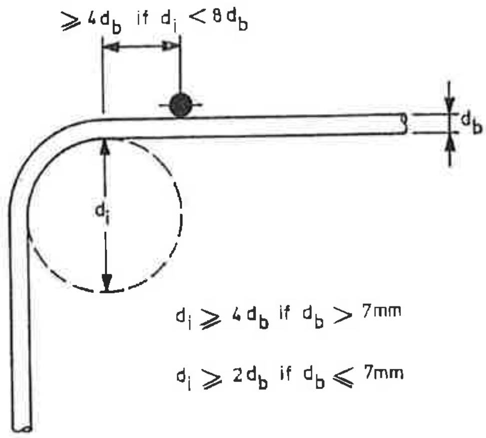


Fig. C5.3 BENDS IN WELDED WIRE FABRIC FOR STIRRUPS AND TIES

C5.3.5.3 Bond research [5.3] showed that bar cut-offs for girders and splices for columns should be staggered. Bundled bars should be tied, wired or otherwise fastened together to ensure remaining in position.

A limitation that bars larger than 35 mm be not bundled in beams or girders of buildings has been added, taken from the 1977 ACI Code which applies primarily to buildings. The American Association of State Highway and Transportation Officials (A.A.S.H.O.) design criteria [5.4] for reinforced concrete bridges by strength design permits two-bar bundles of bars up to 57 mm in bridge girders or columns, usually more massive than those in buildings. Conformity with crack control requirements in the Code will effectively preclude bundling of bars larger than 35 mm as tensile reinforcement. The Code phrasing "bundled in contact, assumed to act as a unit", is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes are triangular, square, or L-shaped patterns for three or four-bar bundles. As a practical provision, bundles more than one bar deep in the plane of bending may not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger them. Bending and hooking of bundles must be established in this manner, even at supports.

C5.3.5.4 These maximum spacing limits have remained essentially unchanged for many years. They have been extended to large sections with 57 mm bars.

C5.3.5.5 Experience has shown that the spacing of reinforcement at greater than 300 mm centres in the exposed surfaces of bridge decks or abutment walls is likely to result in long term maintenance problems, due to shrinkage effects of direct exposure to the weather and the fatigue effects of live loading. Two cases are given which would permit the spacing to be increased to a maximum of 450 mm. An example of (a) would be the soffits of cantilever slabs, while an example of (b) would be the earth face of abutment retaining walls.

C5.3.5.6 These requirements for minimum bar spacing, like those in 5.3.5.1, were developed originally to provide access for concrete placing in columns. Use of the bar diameter, as a factor in establishing the minimum spacing, permitted extension of the original provision to larger bars.

C5.3.5.7 Commentary Clauses C5.3.5.1 and C5.3.5.6 are applicable here. See also 5.3.5.4.

C5.3.5.8 These requirements are provided to prevent weakened planes for splitting bond failure developing in the cover concrete in the anchorage zones.

C5.3.5.9 When ducts for post-tensioning steel in a beam are arranged closely together vertically, provision must be made to prevent the steel, when tensioned, from breaking through the duct. Horizontal disposition of ducts must allow proper placement of concrete. Generally a clear spacing of one and one-third times the nominal maximum size of the coarse aggregate, but not less than 25 mm, proves satisfactory. Where concentration of steel or ducts tends to create a weakened plane in the concrete cover, reinforcement should be provided to control cracking.

C5.3.6 *Development of reinforcement – General*

The development length concept for anchorage of reinforcement was first introduced in the 1971 ACI Building Code, to replace the dual requirements for flexural bond and anchorage bond contained in earlier editions of the ACI Building Code. It is no longer necessary to consider the flexural bond concept which placed emphasis on the computation of nominal peak bond stresses. Consideration of an average bond resistance over a full development length of the reinforcement is more meaningful, partially because all bond tests consider an average bond resistance over a length of embedment of the reinforcement, and partially because uncalculated extreme variations in local bond stresses exist near flexural cracks[5.5].

The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement. In application the development length concept requires the specified minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. In flexural members such peak stresses generally occur at the points specified in 5.3.24.2. This development length or anchorage is necessary on both sides of such peak stress points; on one side to transfer stress into and on the other to transfer stress out of the reinforcement. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span.

C5.3.7 *Development length of deformed bars and deformed wire in tension* [5.6, 5.7, 5.8]

From a statistical evaluation of test results[5.9, 5.10], ACI Committee 408 prepared recommendations that include allowances for the effects of cover, bar spacing and presence of transverse reinforcement.

The recommendations of ACI Committee 308 have been adopted for this Code with only small modifications. A strength reduction factor of 0.8 has been incorporated into the equations of this Section so that, in accordance with 4.3.1.3, the designer need not use a ϕ factor when determining development or splice lengths.

C5.3.7.1 Concrete cover, distance between bars in a layer and bar diameter are the principal quantities which determine the basic development length of a bar. Transverse reinforcement, crossing splitting cracks, will, to a certain extent, improve anchorage. Accordingly, empirical expressions have been derived to determine the basic development length ℓ_{db}. Additional parameters, which may influence development, are then taken into account separately in the form of modification factors m. Consequently the required development length for a bar is obtained from the relationship

$$\ell_d = m\ell_{db} > 300 \qquad \text{(Eq. C5-2)}$$

C5.3.7.2 The basic development length ℓ_{db} in this Clause is given only for Grade 275 steel.To simplify computation, the designer may use equations 5-2 and 5-3 provided that the cover is not less than 40 mm, that is, in beams and columns, and the centre-to-centre spacing is not less than 100 mm. In these equations the beneficial effect of transverse reinforcement, which may be present, is neglected. If the spacing between bars is less than 100 mm the basic development length obtained from equations 5-2 and 5-3 may be multiplied by $50/c_s$, where the dimension c_s is shown in fig. C5.4. Note that equations 5-2 and 5-3 may not be used in slabs or walls where the cover is less than 40 mm.

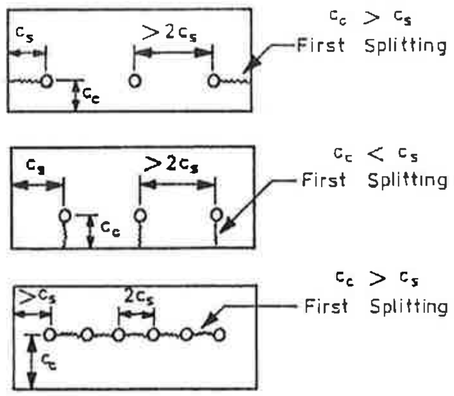


Fig. C5.4 DEFINITION AND SIGNIFICANCE OF DISTANCE c_c AND c_s

When the conditions set out for equations 5-2 and 5-3 are not met, equations 5-4 and 5-5 may be used. This is a simplified form of that recommended by ACI Committee 308. The parameter c which takes into account the cover, needs careful interpretation. It is the smaller of c_c and c_s and its value indicates the resistance to splitting. In a beam c_c is measured vertically, as shown in fig. C5.4, from the extreme tension fibre to the centre of the outermost layer of bars. It is thus equal to the vertical cover plus $0.5\ d_b$. The distance c_s on the other hand is measured horizontally, and in a beam it is relevant to the horizontal length of a splitting crack.

Equations 5-4 and 5-5 are simplified alternatives to eq. 5-6 which may be used for the respective spacing limitations commonly used in columns and beams.

C5.3.7.3 The modification factors, represented by m in eq. C5-2, are specified in this Section. The factors are multiplied together when more than one are applicable. For Grade 380 steel, $m = 1.38$. The factor 1.3 for top reinforcement recognizes the reduction in the quality of bond because the excess water used in the mix for workability and air entrapped during the mixing and placing operations rise toward the top of the finished concrete before setting is complete. Entrapped below bars, this water and air leaves the bar less bonded to the concrete on the underside. For horizontal top bars in a structural member, bond resistance reflects this weakened underside restraint because the loss can be of the order of 50% in extreme cases.

Because the development length required is proportional to the tensile stress to be developed, the full development length may be reduced proportionally when the stress is lower than the yield strength. This is achieved by the modification factor A_{sr}/A_{sp}. It should be noted, however, that this reduction must not be used at or near critical sections of members subjected to earthquake loads, nor should the area of reinforcement required only to control shrinkage and temperature effects in restrained members be omitted when computing A_{sr}.

The beneficial effect of transverse reinforcement, which may control the opening of splitting cracks as shown in fig. C5.4, is expressed by the parameter A_{tr}. The effectiveness of ties, stirrups, hoops or spirals in crossing a potential splitting crack is illustrated in fig. C5.5. For such reinforcement to be effective, at least three bars must cross a potential crack over the development length. However, transverse steel used for any other purpose, such as shear resistance or to provide confinement of compression bars or concrete, may be included for this purpose.

In the case of fig. C5.5 (a) A_{tr} is effective only for the outer bars. The designer would have several choices in this case. A different ℓ_{db} could be calculated for the inner and outer longitudinal bars, or the effect of transverse reinforcement could be ignored, or A_{tr} could be taken into account as an average for the bars, using total area of transverse steel crossing the plane of splitting divided by n. The last approach was checked[5.10] using data reported by Untrauer and Warren[5.6] and it was found to give a reasonable estimate of measured values. This approach is incorporated in the definition of A_{tr} in this Section.

Where a number, n_s, of longitudinal bars are enclosed by a spiral the value of A_{tr} is given by

$$A_{tr} = A_t \qquad \text{(Eq. C5-3)}$$

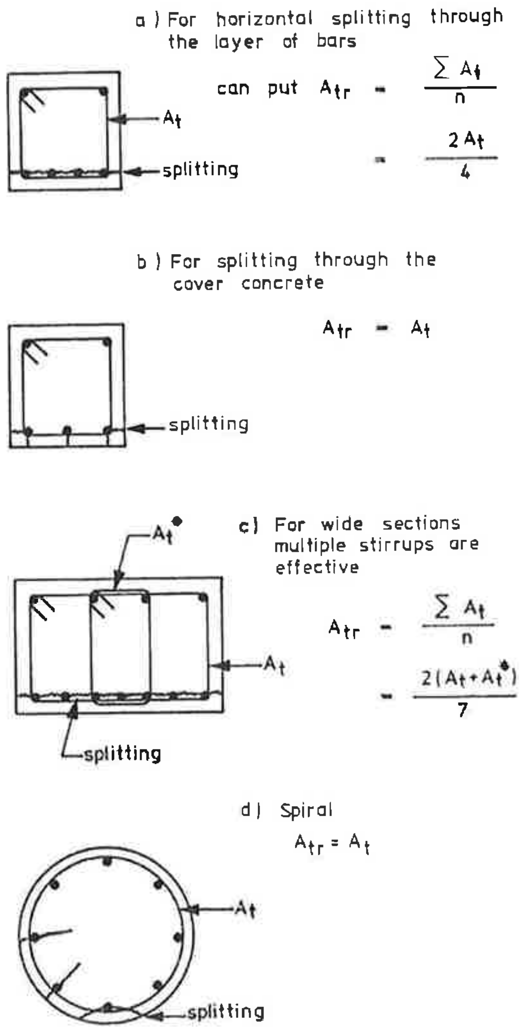


Fig. C5.5 BASIS FOR CALCULATION OF A_{tr}

The 300 mm minimum development length should not be multiplied by the modification factors in 5.3.7.3.

Because the multiplier in 5.3.7.3 (d), which allows for the beneficial effects of transverse reinforcement, involves additional calculations, the designer may always assume that A_{tr} and hence k_{tr} is zero, so that the multiplier becomes unity.

A comprehensive set of tables for development lengths is given in the New Zealand Reinforced Concrete Design Handbook[5.22].

C5.3.9 *Development length of deformed bars in compression*

These provisions are the same as those of the 1977 ACI Code.

The weakening effect of flexural tension cracks is not present for bars in compression and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths have been specified for compression than for tension. The development length may be reduced by up to 25% according to 5.3.9.3 (b) when the compression reinforcement is enclosed within a column by spiral or rectangular ties, hoops or supplementary ties, or an individual spiral around each bar or group of bars is used. The interpretation of the effective transverse steel area in the calculation of A_{tr}/s for 5.3.9.3 (b) is defined in the Notation of the Code.

C5.3.11 *Development of bundled bars*

An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilise bond resistance from the "core" between the bars.

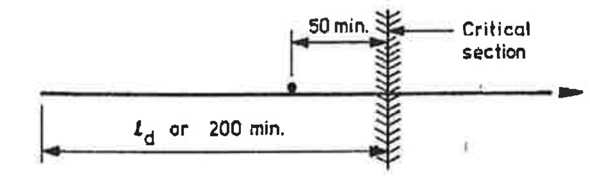
The designer should also note 5.3.5.3 relating to the cut-off points of individual bars within a bundle and 5.3.17.4 relating to splices of bundled bars.

C5.3.12 *Development of welded deformed wire fabric in tension*

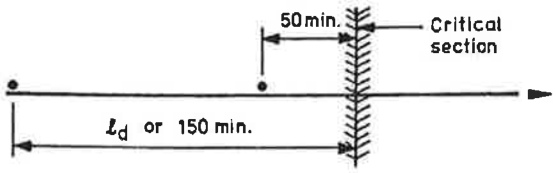
Figure C5.6 (a) shows the development requirements for deformed wire fabric with one cross wire within the development length. In the ASTM A497 deformed wire fabric specifications, welds are not required to be as strong as those required for the smooth wire fabric (NZS 3422). Hence, some of the development is assigned to welds and some assigned to the length of deformed wire. The development computations are simplified from the previous code provisions by assuming that one cross wire is contained in the development length. The factors in 5.3.7.3 for modifying the basic development length are made applicable. However the minimum development length is 200 mm.

C5.3.13 *Development of welded smooth wire fabric in tension*

Figure C5.6 (b) shows the development requirements for smooth wire fabric with development primarily dependent on the location of cross wire. An embedment of at least two cross wires 50 mm or more beyond the point of critical section is adequate to develop the yield strength of the anchored wires [5.11].



(a) Development of welded deformed wire fabric — C5.3.12.



(b) Development of welded smooth wire fabric — C5.3.13.

Fig. C5.6 DEVELOPMENT OF WELDED WIRE FABRIC

C5.3.14 *Development of prestressing strand*

The development requirements for prestressing strand are intended to provide bond integrity for the strength of the member. The provisions are based on tests performed on normal weight concrete members with a minimum cover of 50 mm. These tests may not represent the behaviour of strand in low water-cement ratio, no-slump concrete. Fabrication methods should insure consolidation of concrete around the strand with complete contact between the steel and concrete. Extra precautions should be exercised when low water-cement ratio, no-slump concrete is used. In general, this Clause will control only for the design of cantilever and short-span members. The requirement of doubled development length for strand not bonded through to the end of the member is also based on test data [5.12].

The expression for development length ℓ_d may be re-written as:

$$\ell_d = [\frac{f_{se}}{3} d_b + (f_{ps} - f_{se}) d_b] / 7 \quad \ldots\ldots \text{(Eq. C5-3)}$$

where ℓ_d and d_b are in mm, and f_{ps} and f_{se} are in MPa. The first term represents the transfer length of the strand, that is, the distance over which the strand must be bonded to the concrete to develop the prestress f_{se} in the strand. The second term represents the additional length over which the strand must be bonded so that a stress f_{ps} may develop in the strand at nominal strength of the member.

The variation of strand stress along the development length of the strand is shown in fig. C5.7.

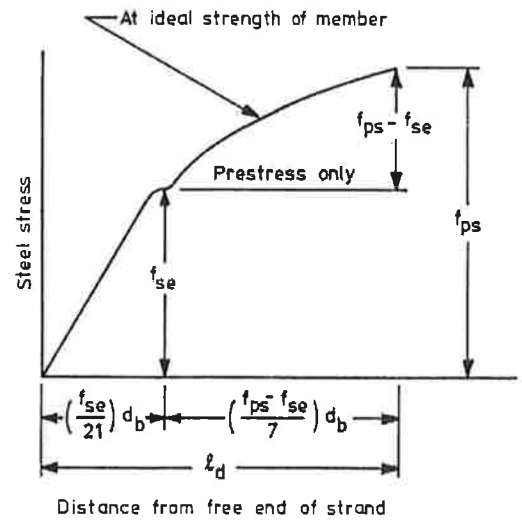


Fig. C5.7 VARIATION OF STEEL STRESS WITH DISTANCE FROM FREE END OF STRAND

The expressions for transfer length and for the additional bonded length necessary to develop an increase in stress of $(f_{ps} - f_{se})$ are based on tests of members prestressed with clean 8 mm, 9 mm and 12 mm diameter strands for which the maximum value of f_{ps} was 1980 MPa [5.12, 5.13, 5.14].

The transfer length of strand is a function of the perimeter configuration area and surface condition of the steel, the stress in the steel and the method used to transfer the steel force to the concrete. Strand with a slightly rusted surface can have an appreciably shorter transfer length than clean strand. Gentle release of the strand will permit a shorter transfer length than abruptly cutting the strands.

The provisions of 5.3.14 do not apply to plain wires nor to end anchored tendons. The length for smooth wire could be expected to be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred.

C5.3.15 *Standard hooks in tension*

Clause 5.3.15 is based on the recommendations of ACI Committee 408[5.9, 5.10].

The approach is a major departure from ACI 318-77 in that it uncouples hooked-bar anchorages from straight bar development provisions and gives the total embedment length ℓ_{dh} as indicated in fig. C5.1.

A study of the failures of hooked bars indicates that splitting of the cover parallel to the plane of the hook is the primary cause of failure and that the splitting originates at the inside of the hook where stress concentrations are very high. For this reason, eq.5-10 is a function of d_b which governs the magnitude of compressive stresses on the inside of the hook. Only standard ACI hooked bars were tested and the influence of larger radius of bend was not evaluated. The test results indicate that as the straight lead length increases, the lateral splitting force which can be developed in the side cover increases; this is reflected in an improvement in hook capacity.

The recommended provisions include adjustments to reflect the resistance to splitting provided by enclosure in transverse reinforcement. If the side cover is large so that side splitting is effectively eliminated, as in mass concrete, the product of the factors as given in 5.3.15.3 may be used. Minimum values of ℓ_{dh} are indicated to prevent failure by direct pullout in cases where the standard hook may be located very near the critical section. No distinction is made between top bars and other bars.

In many cases where the value of ℓ_{hb} given by eq. 5-10 is used, the value of d_i required will be greater than that given in table 5.1 as it will be governed by eq. 5-1. In such cases, if it is desired to reduce the value of d_i to that given in table 5.1, the value of ℓ_{dh} will have to be increased above that given by eq. 5-10 in order to give a value of ℓ_b which will give this reduced value of d_i from eq. 5-1.

C5.3.16 *Mechanical anchorages*

Mechanical end anchorages can be made adequate for strength both for prestressing tendons and for reinforcing bars.

C5.3.17 *Splices in reinforcement – General*

For ductility, lap splices should be adequate to develop more than the yield strength of the steel; otherwise a member may be subject to sudden splice failure when the yield strength of the steel is reached and no "toughness" is obtainable in the member. The lap splice lengths specified in the Code satisfy this ductility requirement.

Splices should, if possible, be located away from points of maximum tensile stress.

C5.3.17.2 It is feasible to produce welds in Grade 380 bars with the required mechanical and metallurgical properties. However this requires a high level of competence and technical knowledge on the part of the welding fabricator and on the part of the inspection staff, together with special quality control facilities.

C5.3.17.3 Research on lap splices on 40 mm and 57 mm bars is limited. There are insufficient data to establish lap lengths for either tensile or compressive lap splices for 40 mm or 57 mm bars.

C5.3.17.4 The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Where the factors in this Clause are applied it is not intended that the factors in 5.3.11 should also be applied.

C5.3.17.5 If individual bars in non-contact lap splices are too widely spaced, an unreinforced section is created. Forcing the potential crack to follow a zigzag line (5 to 1 slope) is considered a minimum precaution. The requirement of 150 mm maximum spacing is made because most research available on the lap splicing of modern deformed bars was conducted with reinforcement which was within this spacing.

C5.3.17.6 Welds complying with 5.3.17.6 (a) can withstand the most severe strain or stress cycles. Hence they are acceptable also in plastic hinge zones (see 5.5.1.1). The appropriate weld quality in the NZS 4702 classification[5.15] would be S or A, depending on the consequences of failure of a small proportion of such welds.

In 5.3.17.6 (b) and (c), the limit of $1.6f_y$ or breaking strength of the bar, whichever is less, will ensure that the strength of the connection will be greater than the maximum design force in the bar. These categories of splices will be adequate for large bars in main members outside plastic hinge zones. The appropriate weld quality in the NZS 4702 classification would be A. The value of 1.6 is derived from multiplication of a factor of 1.3, representing statistical assessment of the likely upper limit of the ratio of the actual yield strength to minimum specified yield strength for reinforcing bars produced in New Zealand, and 1.25 representing the anticipated maximum hardening at strains corresponding to plastic hinging under severe seismic loading[5.18].

The limit of $1.6f_y$ will generally govern because most New Zealand manufactured reinforcing bars have a breaking strength greater than $1.6f_y$.

To ensure that large premature cracks are not produced by slack mechanical splicing devices or excessive extensions in splicing devices, a stiffness criterion is imposed. Accordingly the displacement of the spliced bars relative to each other and measured in a test over the length of the connector, should not exceed twice the elongation of the same size of unspliced bar over the same measured distance when subjected to $0.7f_y$.

In 5.3.17.6 (d), the use of welded splices or mechanical connections with capacity less than the actual breaking strength is permitted if the design criteria of 5.3.19.2 are met. Therefore, lap welds of reinforcing bars, either with or without backup material, welds to plate connections, and end-bearing splices are allowed under certain conditions.

C5.3.18 *Lap splices of bars and wire in tension*

This Clause follows the recommendations of ACI Committee 408 [5.9, 5.10]. It is a major departure from ACI 318-77 because splice and development lengths are the same. Statistical studies have shown that no additional factors are necessary for splices.

In determining the required splice length, ℓ_s, the distance c_s to be used is illustrated in fig. C5.8. Where all bars are spliced at the same location, c_s is half the centre-to-centre distance between bars. Where the splices are staggered and the overlap is less than $\ell_s/2$, c_s reflects this improvement. With staggered splices, the spacing between bars generally will not be as critical as is the cover to the centre of the bar.

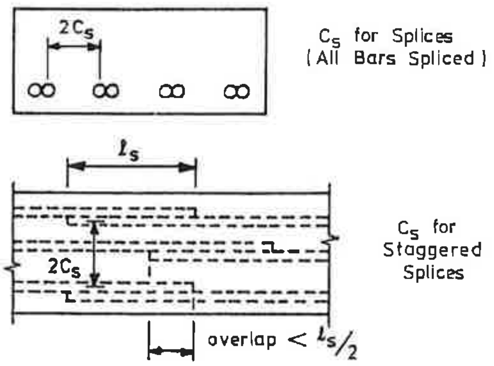


Fig. C5.8 DEFINITION OF c_s FOR SPLICES

C5.3.19 *Welded splices and mechanical connections in tension*

C5.3.19.2 See Commentary on 5.3.17.6 (d). This Clause describes the situation where welded splices or mechanical connections of capacity less than the actual breaking strength of the bar may be used. It provides a relaxation in the splice requirements where the splices or connections are staggered and excess reinforcement area is available. The criterion of twice the computed tensile stress is used to cover sections containing partial tensile splices with various percentages of total steel continuous.

C5.3.20 *Lap splices in compression*

Recent bond research has been primarily related to bars in tension. Bond behaviour of compression bars is not complicated by the problem of transverse tension cracking and thus compression splices do not require provisions as strict as those specified for tension splices.

C5.3.20.2 Effective tie legs included in the evaluation of A_{tr} are those which cross a potential splitting crack which develops in the plane at which two spliced bars might be in contact with each other. An example is shown in fig. C10.5.

C5.3.20.3 Compression lap lengths may be reduced when the lap splice is enclosed throughout its length by spirals because there is increased splitting resistance. Spirals should meet requirements of 5.3.29 and 6.4.7.1. Because spirals do not cross a potential splitting crack when spliced bars in contact are aligned radially, they are less efficient in confining a splice. Therefore the area of the spiral is required to be 67% more than that of a tie crossing a crack at 90°. Potential radial splitting cracks, developing when all spliced bars touch a circular spiral, are, however, controlled with full efficiency by such a spiral.

C5.3.22 *Splices of welded smooth wire fabric in tension*

The strength of lap splices of welded smooth wire fabric is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason the lap is specified in terms of overlap of cross wires rather than in wire diameters or millimetres. The 50 mm additional lap required is to assure over-lapping of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research [5.16] has shown that increased splice length is required when fabric of large, closely spaced wires, is lapped and as a consequence additonal splice length requirements are provided for these fabrics. The development length ℓ_d, is that computed in accordance with the provisions of 5.3.13 without regard to the 150 mm minimum. Splice requirements are illustrated in fig. C5.9.

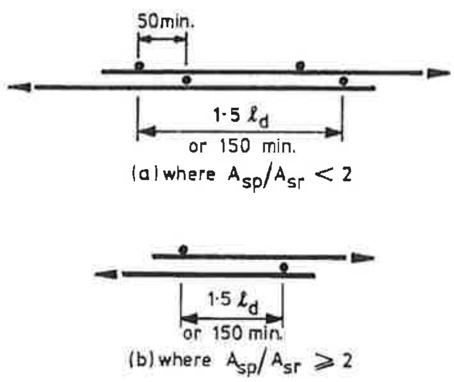


Fig. C5.9 LAP SPLICES OF SMOOTH WIRE FABRIC

C5.3.23 *Splices of welded deformed wire fabric in tension*

The splice formulae stated are based on available tests. Since there is no New Zealand Standard for welded deformed wire fabric, the formulae and values stated refer to deformed wire fabric to ASTM A497, which has lower shear strength requirements for the welds of the cross wires than NZS 3422, the New Zealand Standard for welded smooth wire fabric.

C5.3.24 *Development of flexural reinforcement – General*

C5.3.24.2 Critical sections for a typical continuous beam are indicated with a "Y" or an "X" in fig. C5.10. For uniform loading, the positive reinforcement extending into the support is more apt to be governed by the requirements of 5.3.25.3 rather than by development length measured from a point of maximum moment or bar cut-off.

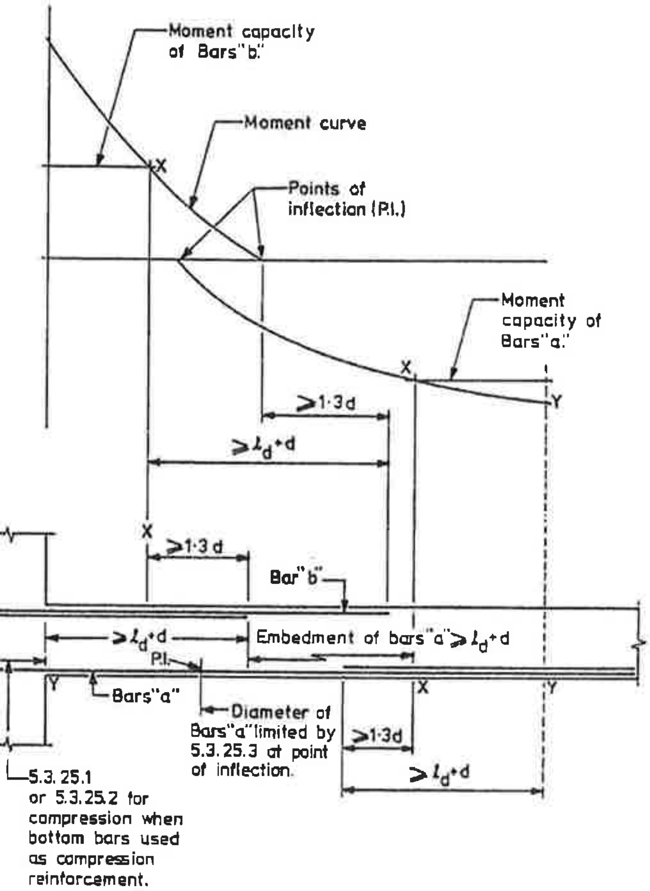


Fig. C5.10 DEVELOPMENT OF FLEXURAL REINFORCEMENT IN A TYPICAL CONTINUOUS BEAM

C5.3.24.3 The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d towards a point of zero moment. When stirrups are provided, this effect is less severe, although still present to some extent.

To provide for shifts in the moment demand the code requires the extension of reinforcement by a distance 1.3 d beyond the point at which it is theoretically no longer required to resist flexure, and by d beyond the length ℓ_d.

Cut-off points of bars to meet this requirement are illustrated in fig. C5.10.

When bars of different sizes are used, the extension should be in accordance with the diameter of bar being terminated. A bar bent to the opposite face of a beam and continued there may logically be considered effective, in satisfying this Clause, to the point where the bar crosses the mid-depth of the member.

Peak stresses exist in the remaining bars wherever adjacent bars are cut off, or bent in tension regions. In fig. C5.10 an "X" mark is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cut off. If bars are cut off as short as the moment diagrams allow, these peak stresses become the full f_y, which requires a full $\ell_d + d$ extension as indicated. This extension may exceed the length required for flexure.

C5.3.24.4 Evidence of reduced shear strength and loss of ductility when bars are cut off in a tension zone, as in fig. C5.10, has been reported by several investigators[5.17]. As a result, the code does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexural cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the steel stress in the continuing reinforcement and the shear strength are each near their limiting value, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely to form where shear stress is low (see 5.3.24.4 (a)). Diagonal cracks can be restrained by closely spaced stirrups (see 5.3.24.4 (b)). A lower steel stress reduces the probability of such diagonal cracking (see 5.3.24.4 (c)). Tension bars bent into the web at an angle not exceeding 45° and terminating at a distance of at least $d/2$ away from the tension face may be considered exempt from the requirements of this Clause, because such bars do not terminate in a tension zone. These requirements are not intended to apply to tension splices which are covered by 5.3.18 and the related 5.3.7.

C5.3.24.5 Members, such as brackets, members of variable depth and others where steel stress f_s does not decrease linearly in proportion to a decreasing moment require special consideration for proper development of the flexural reinforcement. For the bracket shown in fig. C5.11 the stress in the reinforcement at ideal strength is almost constant at approximately f_y from the face of support to the load point. In such a case, development of the flexural reinforcement depends largely on the anchorage provided at the loaded end. Figure C7.10 suggests a welded cross bar of equal diameter as a means of providing effective end anchorage. An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because an unreinforced concrete corner may exist near loads applied close to the corner. For wide brackets (perpendicular to the plane of the figure) and loads not applied close to the corners, U-shaped bars in a horizontal plane provide effective end hooks.

C5.3.25 *Development of positive moment reinforcement*

C5.3.25.1 Special amounts of the positive moment reinforcement are required to be carried into the support to provide for some shifting of the moment due to changes in loading, settlement of supports, lateral loads and other causes.

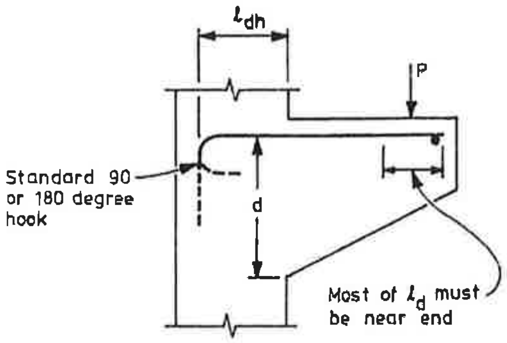
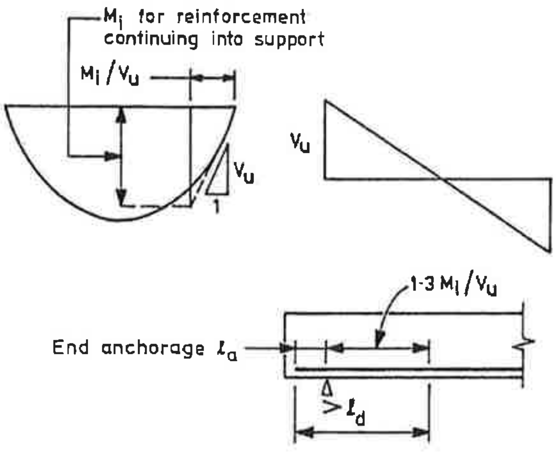


Fig. C5.11 EXAMPLE OF A SPECIAL MEMBER LARGELY DEPENDENT ON END ANCHORAGE

C5.3.25.2 When a flexural member is part of a primary lateral load resisting system, loads greater than those anticipated in design may cause reversal of moment at supports; therefore the required positive reinforcement should be well anchored into the support. This anchorage is to assure ductility of response in the event of unexpected overstress, such as from blast or earthquake. It is not sufficient to use more reinforcement at lower stresses. The full anchorage requirement does not apply to any excess reinforcement provided at the support.

C5.3.25.3 At simple supports and points of inflection (P.I. in fig. C5.10) the diameter of positive reinforcement must be small enough to satisfy the requirement $M_i/V_u \geq \ell_d - \ell_a$. Figure C5.12 illustrates the use of this provision.



Note : The 1·3 factor is usable only if the reaction confines the ends of the reinforcement.

Fig. C5.12 PROCEDURE FOR DETERMINING MAXIMUM SIZE BAR AT SIMPLE SUPPORT

In routine design it may often be found that $M_i/V_u > \ell_d$ and hence no further check need then be made. When the requirement of eq. 5-11 is not satisfied, the designer should either reduce the diameter of bars, whereby ℓ_d is reduced, or increase the area of positive reinforcement at the section considered, whereby M_i is increased, or undertake both of these steps.

This method of checking the local bond stress, induced by the rate of change of the tension force along a bottom bar of a beam, was introduced in the 1977 ACI code where background information is also given.

The distance ℓ_a is an additional embedment length which is considered to relieve the local bond stress. In its evaluation for use in eq. 5-11 the following points should be noted:

(a) Length ℓ_a at points of inflection is limited to the effective depth d or 12 d_b, whichever is greater. This is illustrated in fig. C5.13

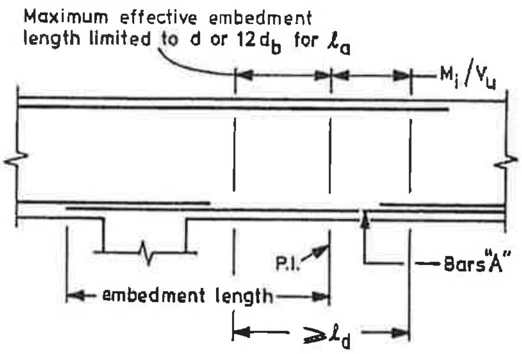


Fig. C5.13 PROCEDURE FOR DETERMINING MAXIMUM SIZE OF BOTTOM BAR "A" AT POINT OF INFLECTION

(b) The value of ℓ_a used must not exceed the actual bar extension beyond the point of zero moment, as shown on the structural drawings.

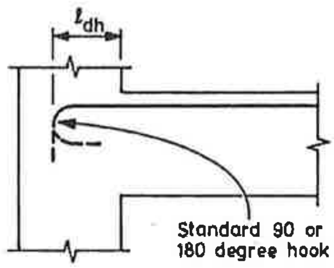


Fig. C5.14 ANCHORAGE INTO EXTERIOR COLUMN

C5.3.26 *Development of negative moment reinforcement*

Figures C5.14 and C5.15 illustrate two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, see commentary discussion C5.3.15.

Clause 5.3.26.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under C5.3.24.3. This requirement may exceed that of 5.3.24.3 and the more restrictive of the two provisions governs.

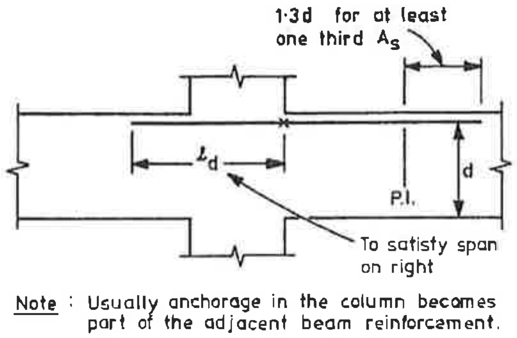


Fig. C5.15 ANCHORAGE INTO ADJACENT BEAM

C5.3.27 *Special details for columns and piers*

C5.3.27.1 Offset bending of bundled bars is prohibited for practical reasons.

C5.3.27.2 This requirement for lap spliced dowels with column faces offset 75 mm or more, together with 5.3.17.3, precludes offsetting 75 mm or more in columns reinforced with bars larger than 35 mm since lap splices are prohibited for such bars.

C5.3.27.4 This Clause provides an effective maximum of 50% transmission of load by end bearing on ends of metal core. The Clause encourages, thereby, provision of some tensile capacity at such splices (up to 50%) since the remainder of the compression stress must be transmitted by welds, dowels, splice plates, and the like. This should ensure that splices in composite columns meet requirements for tensile capacity similar to those for reinforced concrete columns.

For seismic members, NZS 3404 requires the development of full yield capacity of the steel section in axial load, moment and shear at column splices.

C5.3.28 *Connections*

Confinement is needed at connections to assure that the flexural capacity of the members can be developed without deterioration of the joint. A rational analysis is required when shear stresses are induced in joints.

C5.3.31 *Stirrup and tie reinforcement in beams*

C5.3.31.2 Compression reinforcement in beams or girders must be enclosed to prevent buckling. It is considered good practice to enclose all longitudinal bars where practicable.

C5.3.32 *Shrinkage and temperature reinforcement*

So-called shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to prevent excessive cracking and to tie the structure together to assure its acting as assumed in the design. The amounts specified are empirical but have been used satisfactorily for many years.

Deformed bars of 380 MPa steel are recognized on the same basis as welded wire fabric.

The provisions of this Section apply to "structural floor and roof slabs" only and not to slabs on ground.

It should be kept in mind that the reinforcement ratios given in this Clause are minimum values and apply to the situation where restraint against shrinkage has been minimized. It is well known that if the slabs are fully restrained against shrinkage and temperature movement, much higher reinforcement ratios are required to avoid severe cracking. In most cases it is possible to select structural form, construction joint positions and pouring sequences to minimize restraint in suspended slabs, and this Code has followed the practice of most leading national codes in giving reinforcement ratios appropriate to this situation.

For the condition of full restraint, first principles require that the yield strength of the steel passing through any potential crack position should be greater than the ultimate tensile strength of the corresponding cross-sectional area of concrete during the period after initial setting. This would require, for example, a steel percentage of the order of 0.45% for the case of a specified 28 day concrete compressive strength f_c' of 25 MPa and yield strength of reinforcement f_y of 275 MPa.

Provisions for use of steel with yield strength up to 550 MPa have been added. Splices and end anchorages must be designed for the full specified yield strength.

C5.3.33 *Concrete protection for reinforcement*

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where minimum cover is prescribed for a class of structural member, it is measured to the outer edge of the stirrups, ties or spirals if transverse reinforcement encloses main bars; to the outermost layers of bars if more than one layer is used without stirrups or ties; to the metal end fitting or duct on post-tensioned prestressing steel. These minimum concrete covers are essentially the same as the covers specified in NZS 3101P:1970 with the exception of 20 mm bars and less in walls not exposed to weather or in contact with the ground. The cover requirements are set out in table C5.3.

In Item (b) the lesser thicknesses for precast construction reflect the greater control of proportioning, placing and curing of concrete inherent in precasting.

The phrase "concrete surfaces exposed to the weather" refers to direct exposure to temperature and moisture changes. Slab or thin shell soffits are not usually considered directly "exposed" unless subject to alternate wetting and drying, including that due to condensation or direct leakage from exposed top surface, run-off or similar effects.

Table C5.3 MINIMUM COVERS FROM 5.3.33

<table>
<tr><td colspan="19">These are the minimum covers to be provided for reinforcing bars, prestressing tendons or ducts.</td></tr>
<tr><td colspan="3" rowspan="2">DIAMETER OF BARS d_b</td><td colspan="8">CAST IN SITU</td><td colspan="8">PRECAST</td></tr>
<tr><td>40</td><td>32</td><td>28</td><td>24</td><td>20</td><td>16</td><td>12</td><td>10</td><td>40</td><td>32</td><td>28</td><td>24</td><td>20</td><td>16</td><td>12</td><td>10</td></tr>
<tr><td rowspan="4">Not exposed to weather or in contact with the ground</td><td colspan="2">Walls, slabs and ribs</td><td>40</td><td colspan="3">30</td><td colspan="4">20</td><td>35</td><td colspan="3">25</td><td colspan="4">15</td></tr>
<tr><td rowspan="2">Beams and Columns</td><td>Principal reinforcement</td><td colspan="8">40</td><td colspan="6">35</td><td colspan="2">30</td></tr>
<tr><td>Secondary reinforcement</td><td></td><td></td><td></td><td colspan="4">25</td><td></td><td></td><td></td><td></td><td colspan="3">20</td><td colspan="2">15</td></tr>
<tr><td colspan="2">Shells and folded plate members</td><td colspan="5">20</td><td colspan="3">15</td><td colspan="6">20</td><td colspan="2">15</td></tr>
<tr><td rowspan="4">Exposed to weather or in contact with the ground</td><td colspan="2">Walls, slabs and ribs</td><td colspan="4">45</td><td colspan="4">35</td><td colspan="4">40</td><td colspan="4">30</td></tr>
<tr><td rowspan="2">Beams and Columns</td><td>Principal reinforcement</td><td colspan="8">50</td><td colspan="8">40</td></tr>
<tr><td>Secondary reinforcement</td><td></td><td></td><td></td><td colspan="5">40</td><td></td><td></td><td></td><td colspan="5">30</td></tr>
<tr><td colspan="2">Shells and folded plate members</td><td colspan="4">45</td><td colspan="4">35</td><td colspan="8">30</td></tr>
<tr><td colspan="3">Cast against and permanently exposed to the exposed to the ground</td><td colspan="8">75</td><td colspan="8"></td></tr>
</table>

NOTE

(1) In aggressive environment or severe exposure conditions the above covers shall be suitably increased, and other measures set down in 5.3.33.2 (c) shall be taken.
(2) The above covers can be used for prestressed members only when the stresses are less than or equal to the limits of 13.4.1.1 (a). When tensile stresses exceed this value for members exposed to weather, earth or aggressive environment, cover shall be increased 50%. See also 5.3.33.2 (a) for prestressed pre-tensioned concrete members.
(3) For bar bundles, the minimum cover shall equal the equivalent diameter of the bundle but need not be more than 50 mm or the tabulated minimum, whichever is greater.
(4) These covers are minima and this means that when a surface treatment such as bush hammering cuts into the surface of the concrete, the expected depth of treatment should be added to this minimum cover or the cover required under 5.3.35, whichever is the greater.

C5.4 Principles and requirements additional to 5.3 for members not designed for seismic loading

C5.4.1 *Spiral or circular hoop reinforcement for columns and piers*

Where longitudinal bars are arranged in a circular pattern, only one circular tie per specified spacing is required. This requirement can be satisfied by a continuous circular tie (helix) at larger pitch than permitted for spirals under 6.4.7.1, the maximum pitch being equal to the required tie spacing.

Precast columns with cover less than 35 mm, prestressed columns without longitudinal bars, columns or concrete with small size coarse aggregate, wall-like columns, and other special cases, may require special designs for lateral reinforcement. Smooth or deformed wire, 6 mm or larger, or welded wire fabric consisting of such wire may be utilized for ties or spirals. If such special columns are considered as spiral columns for load capacity in design, the ratio of spiral reinforcement ρ_s must conform to 6.4.7.1.

C5.4.1.3 A provision has been included in this Code requiring ties above the termination of the spirals in a column if enclosure by beams or brackets is not available on all sides of the column. These ties are chosen to enclose the longitudinal column reinforcement and the portion of bars from beams bent into the column for anchorage. The Code allows spirals to be terminated at the level of lowest horizontal reinforcement framing into the column. However, if one or more sides of the column are not enclosed by beams or brackets, ties are required from the termination of the spiral to the bottom of the slab or drop panel. If beams or brackets enclose all sides of the column but are of different depths, the ties should extend from the spiral to the level of the horizontal reinforcement of the shallowest beam or bracket framing into the column.

If concrete cover is lost, ties made of plain bars or wires may not be embedded in concrete and hence a positive means of anchorage is required.

C5.5 Principles and requirements additional to 5.3 for members designed for seismic loading

C5.5.1 *Splices in reinforcement*

C5.5.1.1 Splices other than those required by 5.3.17.6 (a) should not be used in potential plastic hinge regions or in beam-column joints where anchorage conditions may be very critical. Therefore splices should be located away from critical sections of potential plastic hinges by the distances specified. In a column, if plastic hinges form, their location will be at the top and bottom ends of storey heights, adjacent to beams or footings. When plastic hinge development is not expected because columns have considerable reserve flexural strength; for example, they have been designed in accordance with the Appendix to Commentary, Section 3 (C3A); splices may be located also in the top and bottom ends of storey heights and the preferred position will usually be immediately above a floor.

C5.5.1.2 Transverse reinforcement provided around splices with Grade 380 bars, in accordance with the equation given, was found to ensure that at least 85% of the ideal strength of a column section with all bars spliced, could be sustained in at least 20 cycles of reversed loading without distress. Such splices were found to sustain even a few limited excursions beyond yield. In determining the splice length from 5.3.7, the beneficial effect of this transverse reinforcement may be utilized by substituting in 5.3.7.3(d), $k_{tr} = 0.8\,d_b$.

C5.5.1.3 In determining the criteria for welded splices and mechanical connections, standard of workmanship, difficulty of inspection, and the final reliability of the splice in service has been taken into account. Even so designers should be aware of the necessity for a site testing programme to ensure that these splices meet the requirements of 5.3.17.6 (b) and 5.3.17.6 (c). The requirement for staggering positive connections, by at least 900 mm if the strain measured over the full length of the connector (at 0.9 bar yield) exceeds that of an unspliced bar by more than 10%, is because of increased strains at the point of maximum moment in a splice region. A series of tests recently carried out in New Zealand on a number of mechanical reinforcing bar splice systems[5.18] indicated that several systems could meet this strain limitation requirement.

C5.5.2 *Development of flexural reinforcement*

C5.5.2.1 The bending moment envelope to be used is that corresponding to the formation of two plastic hinges in each span under the combined effects of seismic and gravity loadings. The moments at the plastic hinges are to be based on the flexural overstrengths of the sections as detailed. To ensure that the curtailed reinforcement is adequate for the moment demand between plastic hinges the envelope should also take into account the possibility of overstrength being developed at one plastic hinge of the beam while only the ideal moment is developed at the other plastic hinge.

In some circumstances, when flexural overstrength is developed at the critical section of a plastic hinge region, some sections outside the plastic hinge region may develop greater than ideal flexural strength. The reinforcement should not be increased beyond the hinge to meet such a condition. However, reinforcement provided at the critical sections of the plastic hinges should not be terminated unless the continuing bars provide an ideal flexural strength at least as great as the moment demand resulting when flexural overstrength is attained at either or both of the critical sections in the plastic hinge regions.

C5.5.2.2 Because of yield penetration from the face of a column toward its core, length available for the development of the strength of beam bars is gradually reduced during cyclic reversals of earthquake loading. To ensure that the beam capacity is maintained after several excursions of the structure into the inelastic range half the column depth or $10\,d_b$, whichever is less, is required to be disregarded for the purpose of anchorage. This ineffective development length is illustrated in fig. C5.16. Tests have indicated that top and bottom bars anchored separately, as in fig. C5.16, perform better than those having a continuous hoop. This does not apply when a sufficient column stub, such as shown in fig. C5.19, is used.

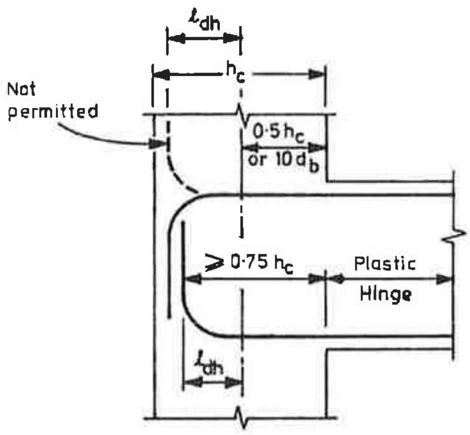


Fig. C5.16 ANCHORAGE OF BEAM BARS WHEN THE CRITICAL SECTION OF THE PLASTIC HINGE FORMS AT THE COLUMN FACE

When the flexural steel is curtailed in such a way that the critical section of a potential plastic hinge is at a distance from the column face of at least the beam depth or 500mm, whichever is less, progressive yield penetration into the column is not expected. Only in this case may the development length for the beam bar be assumed to commence at the column face where the beam bar enters. This case is shown in fig. C5.17.

The provisions of 5.3.7.3 (c) which reduce the anchorage and hence lap splice length to less than that required for a bar at yield stress is unsafe for seismic loading.

C5.5.2.4 These requirements at exterior columns are illustrated in examples of figures C5.16 and C5.17.

When the moment demands, particularly those involving bottom steel, are different at opposite faces of an interior column, some of the beam bars may be terminated at the interior column. This will enable the unnecessary boosting of flexural capacity to be avoided. To avoid anchorage in the usually congested area of the joint core such bars may

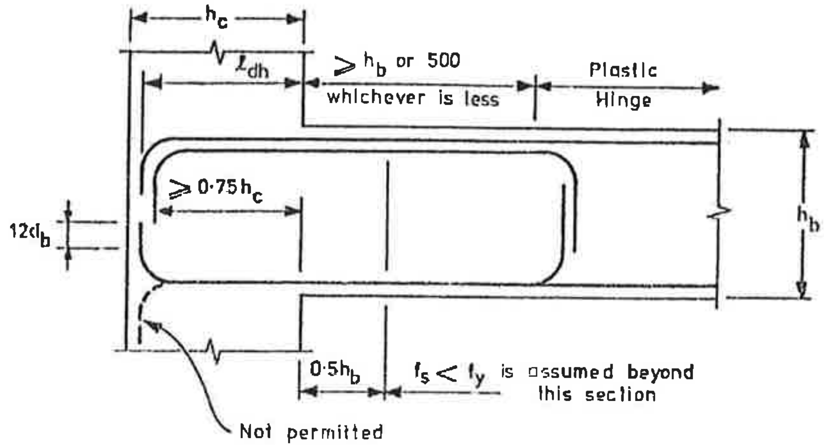


Fig. C5.17 ANCHORAGE OF BEAM BARS WHEN THE CRITICAL SECTION OF THE PLASTIC HINGE IS AT A DISTANCE FROM THE COLUMN FACE OF AT LEAST THE BEAM DEPTH OR 500 mm, WHICHEVER IS LESS

be bent just past the column core as shown in fig. C5.18 and thus anchored with a standard 90° hook.

When bars are anchored in or near a column core the bearing stress developed in the bend is required to be directed towards the core to ensure sufficient force transfer within the joint. Therefore, the bending of bars away from the core, as illustrated by dashed lines in figures C5.16 and C5.17 is not permitted.

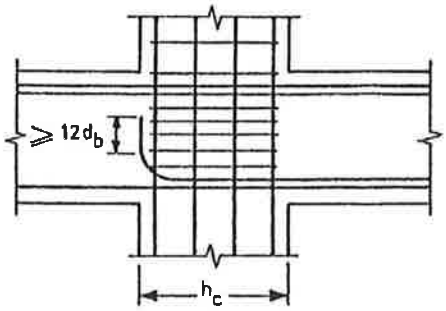


Fig. C5.18 TERMINATION OF BEAM BARS AT AN INTERIOR JOINT

C5.5.2.5 At interior beam-column joints, such as shown in fig. C5.18, extremely high bond stresses can develop when a frame sustains large inelastic deformations due to seismic motions. Beam bars may be forced to yield in tension at one column face and have a large compressive stress at the opposite column face. Also, yield penetration along a beam bar from either face of an interior column may considerably reduce the effective anchorage length of the bar.

Thus the limit for the ratio of bar diameters to the column depth (h_c in fig. C5.18), is intended to ensure that a beam bar will not slip prematurely through the joint core during cyclic reversed inelastic displacements[5.19, 5.20]. However, when potential plastic hinges are designed so that yielding in the beam bars cannot develop nearer than half a beam depth to the column face, as shown in fig. C5.17, better bond conditions exist and consequently larger diameter beam bars may be used[5.21]. For paired or bundled bars, the diameter should be taken as the diameter of a single bar of equivalent area.

In low-rise structures in which column sidesway mechanisms are permitted, shallow columns are common. Since the beam reinforcement may be controlled by gravity loading considerations, a large excess of strength under seismic loading may exist, and beam bar stresses at the moment capacity of the columns may be of one sign (for example tensile) through the full width of the joint.

The limitations set in 5.5.2.5 (a) and (b) are derived for the condition of beams hinging, at flexural overstrength, at both faces of the column, producing bar stresses ranging from tensile yield at one face of the column to compressive yield at the other. Where such conditions do not exist, such as where the bar force remains tensile through the joint, lower bond stresses will result, and consequently increased bar sizes are permitted. In addition, any loss of anchorage caused by deteriorating bond conditions within the joint may, under these conditions, be accommodated in the opposite beam without detriment to the structural performance. The relaxation permitted will alleviate the congestion caused by the need for abnormally small bar diametres otherwise required by the shallow columns. Equation 5-14 is based on the assumption that bond stresses are directly proportional to the moment gradient across the column. In the absence of axial load, the ratio d_b/h_c will vary from 1/25 when ΣM_c equals ΣM_b, and will approach 2/25 as ΣM_b becomes very large in comparison with ΣM_c.

Clause 5.5.2.5 (a) refers to bars that are situated in layers adjacent to columns, that is, in the shaded areas shown in fig. C6.9. These bars are limited to one-fifth of the slab thickness because it would be difficult to prevent the inelastic buckling of larger size bars. Moreover, it is more difficult to ensure the force transfer from larger bars in the slab to the column core under predominantly earthquake loadings. In any case sufficient transverse steel should be present in such slabs to ensure effective transfer of bond forces to the column core.

C5.5.2.6 The sloping bars, secondary reinforcement, in fig. C5.19, indicate one way by which anchorage of the beam bars can be boosted when in compression. Mechanical anchorage devices, such as plates welded to the end of the beam bars, while performing well when the bar to be anchored is in tension, should be tied back into the column core where the development length is inadequate to develop the strength of the bars in compression without the anchorage device.

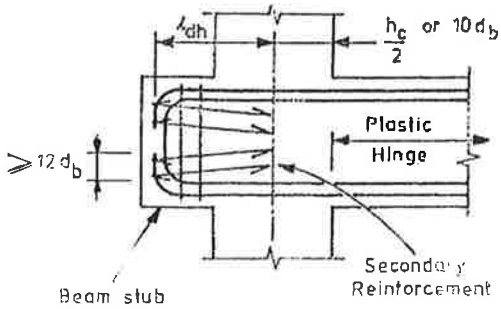


Fig. C5.19 ANCHORAGE OF BEAM BARS IN A BEAM STUB

C5.4.2.7 The necessary straight development length ℓ_{dh} for hooked bars, shown in fig. C5.16, may be larger than what might be available in a column when the requirements of 5.5.2.2 are to be satisfied. In such situations it is better to improve the bearing conditions in the bend than to provide extra straight anchorage length beyond the 90° bend. When transverse bars, as shown in fig. C5.2, are provided, a 20% reduction in the development length ℓ_{dh} of fig. C5.1 may be made. When beam bars are anchored within column bars in the core of a beam-column joint, the application of the multiplier 0.7 in 5.3.15.3 (b) is appropriate.

When the same bar is required to develop yield strength in compression, the bent portion of the anchorage must be disregarded in satisfying the requirements of 5.3.9.1. However, when bars are anchored in column cores, as described above, the confinement may be considered to be equivalent to that implied in 5.3.9.3 (b).

C5.5.3 *Development of column reinforcement*

C5.5.3.1 Plastic hinges may form at the base of the lower storey column, the top of the upper storey column and, due to higher mode effects and the like, in other storeys. The possibility of the column being in single curvature should be taken into account when determining bending moment envelope to be used for curtailment and splice locations.

The reasons for the provisions in this clause are the same as those given in C5.5.2.2 for the anchorage of beam bars.

C5.5.3.2 The reasons for the provisions in this Clause are the same as those given in C5.5.2.4 for the anchorage of beam bars.

In some cases of exterior beam to column joints, where the column terminates at the top surface of the beam, as in knee joints in portal frames and in joints between beams and exterior columns in top storeys of buildings, a significant portion of the transmission of the negative moment from the end of the beam to the column is obtained from lap splice action between the horizontal leg of the standard hook in the column bars and the adjacent top beam bars. In such cases the designer should consider whether he needs to increase the length of this horizontal leg beyond the normal minimum of $12d_b$ for a standard hook.

C5.5.3.3 At all beam-column joints, in the event of hinge formation in the column adjacent to the joint, extremely high bond stresses can develop due to a vertical bar being forced to yield in tension at one beam face and also having a large compressive stress at the other beam face. However, because the bond conditions are better for vertical column bars than for horizontal beam bars, the maximum values of d_b/h_b are greater than the corresponding values of d_b/h_c given in 5.5.2.5 (a)

C5.5.3.4 When plastic hinges are not intended to develop in columns, bond conditions in the beam-column joint area are better than those which would exist if plastic hinges occurred adjacent to the beam faces. Bond conditions are likely to be better than those corresponding to 5.5.2.5 (c), because maximum column moments are unlikely to occur simultaneously on both sides of the joint and bond conditions for vertical column bars are better than those for horizontal beam bars in the joint area. Hence, the values of d_b/h_b are greater than the corresponding values of d_b/h_c given in 5.5.2.5 (c).

C5.5.3.5 The provisions of this Clause are intended to prevent the possibility of lateral buckling of bars at bends for offsets where the bars are in compression, or the spalling of the cover concrete when bars are in tension.

C5.5.5 *Rectangular stirrup and tie reinforcement for columns and piers*

C5.5.5.3 The requirements of this Clause ensure some anchorage of the tie inside the core even after the cover concrete has spalled off.

C5.5.6 *Stirrup and tie reinforcement in beams*

C5.5.6.2 See C5.5.5.3.

REFERENCES

5.1 Erasmus, L.A. and Pussegoda, N., "Safe Bend Radii for Deformed Reinforcing Bar to Avoid Failure by Strain Age Embrittlement", New Zealand Engineering, V.33, No. 8, August 1978, pp. 170-177.

5.2 CEB-FIP Model Code for Concrete Structures, Third Edition, 1978.

5.3 ACI Committee 408, "Bond Stress — The State of the Art", ACI Journal, Proceedings V.63, No. 11, Nov. 1966, pp. 1161-1188. Also ACI Manual of Concrete Practice, American Concrete Institute, Detroit, 1979, Part 2.

5.4 "Standard Specifications for Highway Bridges", American Association of State Highway and Transportation Officials, Washington, D.C., 11th edition, 1974, 284 p. Interim Specifications, 1973.

5.5 ACI Committee 408, "Bond Stress – The State of the Art", ACI Journal, Proceedings V. 63, No. 11, Nov. 1966, pp. 1161-1188.

5.6 Untrauer, Raymond E. and Warren, George E., "Stress Development of Tension Steel in Beams", ACI Journal, Proceedings V.74, No. 8, Aug. 1977, pp. 368-372.

5.7 Orangun, C.O., Jirsa, J.O. and Breen, J.E., "A Re-evaluation of Test Data on Development Length and Splices"ACI Journal, Proceedings V.74, March 1977, pp. 114-122.

5.8 Ferguson, Philip M., "Small Bar Spacing or Cover – A Bond Problem for the Designer", ACI Journal, Proceedings V.74, No. 9, Sept. 1977, pp. 435-439.

5.9 ACI Committee 408, "Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension", Concrete International, V.1, No. 7, July 1979, pp. 45-46.

5.10 Jirsa, J.O., Lutz, L.A. and Gergely, P., "Rationale for Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension", Concrete International, V.1, No.7, July 1979, pp. 47-61.

5.11 Anderson, A.R., "Bond Properties of Welded Wire Fabric", ACI Journal, Proceedings V.50, No. 4, April 1952, pp. 681-692.

5.12 Kaar, P. and Magura, D., "Effect of Strand Blanketing on Performance of Pretensioned Girders", Journal, Prestressed Concrete Institute, V. 10, No. 6, Dec. 1965, 15p.

5.13 Hanson, N.W. and Kaar, P.H., "Flexural Bond Tests Pretensioned Beams", ACI Journal, Proceedings V.55, No. 7, Jan. 1959, pp. 783-802. Also, Development Department Bulletin D28, Portland Cement Association, 1959, 20p.

5.14 Kaar, P.H., La Fraugh, R.W. and Mass, M.A., "Influence of Concrete Strength on Strand Transfer Length", Journal, Prestressed Concrete Institute, V.8, No. 5, Oct. 1963, pp. 47-67. Also, Development Department Bulletin D71, Portland Cement Association, Oct. 1963, 21p.

5.15 NZS 4702 New Zealand Standard Metal-Arc Welding Grade 275 Reinforcing Bar.

5.16 Lloyd, John P., "Splice Requirements for One-Way Slab Reinforced with Smooth Welded Wire Fabric", Publication No. R(S)4, Civil Engineering, Oklahoma State University, June 1971, 37p.

5.17 Ferguson, Philip M. and Matloob, Farid N., "Effect of Bar Cut-off on Bond and Shear Strength of Reinforced Concrete Beams", ACI Journal, Proceedings V.56, No. 1, July 1959, pp. 5-24.

5.18 Raper, A.F., "Evaluation of Mechanical Reinforcing Bar Splice Systems for New Zealand Conditions", Research and Development Report No. 77-5 of the Office of the Chief Structural Engineer, Ministry of Works and Development, New Zealand, November 1977.

5.19 Blakeley, R.W.G., Megget, L.M. and Priestley, M.J.N., "Seismic Performance of Two Full-Size Reinforced Concrete Beam-Column Joint Units", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 8, No. 1, March 1975, pp. 38-69.

5.20 Blakeley, R.W.G., Edmonds, F.D., Megget, L.M. and Wood, J.H., "Cyclic Load Testing of Two Refined Reinforced Concrete Joints", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1979, pp. 238-255.

5.21 Birss, G.R., "The Elastic Behaviour of Earthquake Resistant Reinforced Concrete Interior Beam-Column Joints", Department of Civil Engineering, University of Canterbury, New Zealand, Report No. 78-13, 1978, 96 p.

5.22 New Zealand Reinforced Concrete Design Handbook (new section to be published 1982), N.Z. Portland Cement Association.

COMMENTARY

C6 FLEXURE WITH OR WITHOUT AXIAL LOAD

C6.2 Scope. Section 6 covers the design of members for flexure with or without axial load. Clause 6.3 gives general principles and requirements, 6.4 gives additional principles and requirements for members not designed for seismic loading, and 6.5 gives additional principles and requirements for ductile members designed for seismic loading. The design should comply with 6.3 and with either 6.4 or 6.5.

The commentary to follow is taken largely from the Commentary on the Building Code of the American Concrete Institute (ACI 318-77), supplemented where necessary.

C6.3 General principles and requirements

C6.3.1 *General design assumptions*

C6.3.1.1 The flexural strength of a member computed by the strength design method of the code requires that two basic conditions be satisfied: (1) static equilibrium and (2) compatibility of strains. Equilibrium between the compressive and tensile forces acting on the cross-section at ideal strength must be satisfied. Compatibility between the stress and strain for the concrete and the reinforcement at ideal strength conditions must also be established within the design assumptions allowed by 6.3.1.[6.1]

C6.3.1.2 Many tests have confirmed that the distribution of strain is essentially linear across a reinforced concrete cross-section, even near flexural strength.

Both the strain in reinforcement and in concrete are assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.

C6.3.1.3 The maximum concrete compressive strain at crushing of the concrete has been observed in tests to vary from 0.003 to much higher values under special conditions.[6.1] However, the strain at which ultimate (maximum) moments are developed is usually about 0.003 to 0.004 for members of normal proportions and materials. Nevertheless, the use of a strain of 0.003 may lead to a significant underestimate of the flexural strength of members with large quantities of transverse confining steel since large quantities of confining steel lead to an increase in the strength and ductility of the core concrete and hence of the member. The use of a higher concrete strain than 0.003 may be justified in cases where the concrete is heavily confined.

C6.3.1.4 For deformed bar reinforcement, it is sufficiently accurate to assume that the stress in reinforcement is proportional to strain below the yield strength f_y. The increase in strength due to the effect of strain hardening of reinforcement is neglected for strength computations. In strength computations, the force developed in tensile or compressive reinforcement is computed as:

$$A_s f_s = A_s E_s \epsilon_s, \quad \text{if } \epsilon_s < \epsilon_y$$

$$\text{and } A_s f_s = A_s f_y, \quad \text{if } \epsilon_s \geqslant \epsilon_y$$

where ϵ_s = steel strain, ϵ_y = steel strain at first yield $= f_y/E_s$, A_s = steel area, f_s = steel stress and f_y = steel yield strength.

For design, the modulus of elasticity of steel reinforcement E_s may be taken as 200 GPa.

C6.3.1.5 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is about 10 to 15% of the compressive strength. Tensile strength of concrete in flexure is neglected in strength design. For members with normal percentages of reinforcement, this assumption agrees well with tests.

The strength of concrete in tension, however, is important in cracking and deflection considerations at service loads.

C6.3.1.6 This assumption recognizes the inelastic stress distribution of concrete at high stress. As maximum stress is approached, the stress-strain relationship for concrete is not a straight line but some form of a curve (stress is not proportional to strain). The general shape of the curve for unconfined concrete is a function of concrete strength and consists of a rising curve from zero to a maximum at a compressive strain between 0.0015 and 0.002 followed by a descending curve to an ultimate strain (crushing of the concrete) of generally greater than 0.003. As discussed under C6.3.1.3, the code sets the maximum usable strain at 0.003 for design.

The actual distribution of concrete compressive stress in a practical case is complex and usually not known explicitly. However, research has shown that the important properties of the concrete stress distribution can be approximated closely by using any one of several different assumptions as to the form of stress distribution. The code permits any particular stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Many stress distributions have been proposed. The three most common are the parabola, trapezoid, and rectangle.

C6.3.1.7 For practical design, the code allows the use of a rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distributions. In the equivalent rectangular stress block, an average stress of $0.85\,f'_c$ is used with a rectangle of depth $a = \beta_1 c$. The value of $\beta = 0.85$ for concrete with $f'_c \leqslant 30$ MPa and 0.04 less for each 5 MPa of f'_c in excess of 30 MPa was determined experimentally.

In 1975 a lower limit of β_1 equal to 0.65 was adopted by the ACI Code for concrete strengths greater than 55 MPa. Research data from tests with high strength concretes[6.2, 6.3] supported use of the equivalent rectangular stress block for concrete strengths exceeding 55 MPa with a β_1 equal to 0.65. Use of the equivalent rectangular stress distribution specified in ACI 318-71, with no lower limit on β_1, resulted in inconsistent designs for high strength concrete for members subject to combined flexure and axial load.

The rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but does provide essentially the same results as those obtained in tests.[6.4] All strength equations presented in the references listed in C6.4.1 are based on the rectangular "stress block".

C6.3.2 *Distribution of flexural reinforcement in beams and slabs*

C6.3.2.1 Many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high strength reinforcing steels are used at high service load stresses, however, visible cracks must be expected, and steps must be taken in detailing of the reinforcement to control cracking. To assure protection of reinforcement against corrosion, and for aesthetic reasons, many fine hairline cracks are preferable to a few wide cracks.

C6.3.2.2 It is intended that distribution of reinforcement required by 11.4 should provide adequate control of cracking. Although not derived for two-way slabs, there is increasing evidence that eq. 4-8 for calculating crack widths (see 4.4.2.2) is also applicable to two-way slabs. Hence eq. 4-8 could be used in those cases where a check of crack widths in two-way slabs is considered to be advisable.

C6.3.2.3 An explanation of 6.3.2.3 is given in C4.4.2.2. Several bars at moderate spacing are more effective in controlling cracking than one or two bars of equivalent area.

C6.3.2.4 In major T-beams, the distribution of the negative moment tension reinforcement for control of cracking at service load should be considered. If all the reinforcement is concentrated in the flange directly over the beam web the outer region of the flange may develop wide cracks. Some reinforcement can be placed in the outer regions of the flange to help control such cracking. Note, however, the requirements of 6.5.3.2 (e) (5) for at least 75% of the beam steel to pass through the column core in seismic design.

C6.3.2.5 For relatively deep flexural members, some reinforcement should be placed near the vertical faces in the tension zone to control cracking in the web. Without such auxiliary steel, the width of the cracks in the web at service load may greatly exceed the crack widths at the level of the flexural tension reinforcement.

C6.3.3 *Deep beams.* The code does not contain detailed requirements for designing deep beams for flexure except that non-linearity of strain distribution and lateral buckling must be considered.

Suggestions for the design of deep beams for flexure are given in references [6.1, 6.5] and [6.6].

C6.3.4 *Transmission of column loads through floor systems.* The requirements of this clause are based on a paper on the effect of floor concrete strength on column strength.[6.7] The provisions mean that where the column concrete strength does not exceed the floor concrete strength by more than 40%, no special precautions need be taken. For higher column concrete strengths, methods in 6.3.4.2 or 6.3.4.3 must be used for corner or edge columns and methods in 6.3.4.2, 6.3.4.3 or 6.3.4.4 for interior columns with adequate restraint on all four sides.

C6.3.5 *Bearing strength.* This clause deals with bearing strength on concrete supports. The permissible bearing strength of 0.85 f'_c is based on tests reported in reference[6.8]. (See also 4.3.1.2 (f).)

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength.No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 7.3.15.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Figure C6.1 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support. Such a load path would have steeper sides. However, the frustum described has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

Post-tensioning anchorages are normally laterally reinforced, in accordance with 13.3.9.

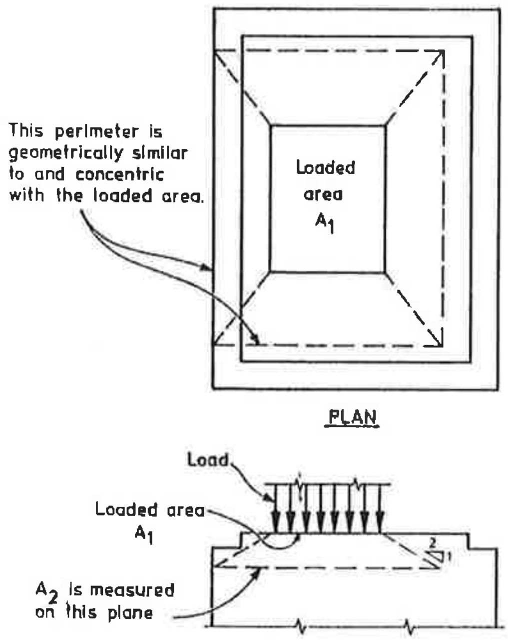


Fig. C6.1 APPLICATION OF FRUSTUM TO FIND A_2 IN STEPPED OR SLOPED SUPPORTS

C6.4 Principles and requirements additional to 6.3 for members not designed for seismic loading

C6.4.1 *Strength calculations*

C6.4.1.1 Design strength equations for members subject to flexure or combined flexure and axial load are derived in publications such as references [6.1] and [6.4]. Reference [6.9] gives design aids calculated using the yield strengths of New Zealand reinforcing steels. Design equations are not given here because they have become part of standard theory.

C6.4.1.2 A balanced strain condition exists at a beam cross-section when the maximum strain at the extreme compression fibre just reaches 0.003 simultaneously with the first yield strain f_y/E_s in the tension reinforcement. The reinforcement ratio ρ_b which produces balanced conditions under flexure, depends on the shape of the cross-section and the location of the reinforcement.

C6.4.1.3 For the additional strength due to compression reinforcement see references such as [6.1] and [6.4].

C6.4.1.4 Corner and other columns exposed to known moments about each axis simultaneously should be designed for biaxial bending and axial load[6.1, 6.10] to [6.13].

C6.4.1.5 The design axial load strength in compression with or without eccentricity is limited to 85 or 80% of the dependable axial load strength without eccentricity in order to account for accidental eccentricities not considered in the analysis that may occur in a compression member and to recognize that the concrete strength may be less than f'_c at sustained high loads. The 85 and 80% values approximate the axial load strengths at e/h ratios of 0.05 and 0.10 specified in earlier ACI codes for the spirally-reinforced and tied members respectively. The same axial load limitation applies to both cast-in-place and precast compression members.

C6.4.2 *Maximum longitudinal reinforcement in beams and slabs.* The maximum amount of tension reinforcement in flexural members is limited to ensure a level of ductile behaviour. When the flexural strength of a member is reached the concrete in the extreme compression fibre reaches its ultimate strain and the tension reinforcement either just reaches the strain at first yield, or is less than the yield strain (elastic), or exceeds the yield strain (inelastic). Which steel strain condition exists at ultimate concrete strain depends on the relative proportion of steel to concrete and material strengths f'_c and f_y. If $\rho\,(f_y/f'_c)$ is sufficiently low, the strain in the tension steel will greatly exceed the yield strain when the concrete strain reaches its ultimate, with large deflection and ample warning of impending failure (ductile failure condition). With a larger $\rho\,(f_y/f'_c)$, the strain in the tension steel may not reach the yield strain when the concrete strain reaches its ultimate, with consequent small deflection and little warning of impending failure (brittle failure condition). For design it is considered necessary to restrict the steel ratio so that a ductile failure mode can be expected. Unless unusual amounts of ductility are required, the 0.75 ρ_b limitation will provide sufficiently ductile behaviour for most designs.

One condition where greater ductile behaviour is required is in design for redistribution of moments in continuous beams and frames. Moment redistribution is dependent on adequate ductility at plastic hinge regions and in 3.3.3.4 (d) the maximum amount of tension reinforcement is controlled by relating the neutral axis depth (which is a function of the available ultimate curvature) to the amount of moment redistribution permitted.

For ductile behaviour of beams with compression reinforcement, only that portion of the total tension steel balanced by compression in the concrete need be limited; that portion of the total tension steel which is balanced by the compression reinforcement need not be limited by the 0.75 factor.

C6.4.3 *Minimum longitudinal reinforcement in beams and slabs*

C6.4.3.1 The provisions for a minimum amount of reinforcement applies to beams, which for architectural or other reasons, are much larger in cross-section than required by strength considerations. With a very small amount of tensile reinforcement, the computed moment strength as a reinforced concrete section becomes less than that of the corresponding plain concrete section computed from its modulus of rupture. Failure in such a case can be quite sudden.

To prevent such failure the minimum steel ratio is limited to $\rho = 1.4/f_y$. For Grade 275 and 380 steels, the minimum steel ratio required becomes 0.51 and 0.37%, respectively. The $1.4/f_y$ value was originally derived to provide the same 0.5% minimum (for mild steel) as required in earlier editions of the ACI 318.

C6.4.3.2 The minimum reinforcement required by eq. 6-2 must be provided wherever positive reinforcement is needed, except where both positive and negative reinforcement are one-third greater than required by analysis. This exception provides sufficient additional reinforcement in large members where an area of 1.4 bd/f_y would be excessive.

C6.4.3.3 The minimum reinforcement required for slabs is somewhat less than that required for beams, since an overload would be distributed laterally and a sudden failure would be less likely. The structural reinforcement should, however, be at least equal to the shrinkage and temperature reinforcement, as required by 5.3.32.

Soil supported slabs, such as slabs on grade, are not considered to be structural slabs in the context of this clause, unless they transmit vertical loads from other parts of the structure to the soil. Reinforcement, if any, in soil-supported slabs should be proportioned with due consideration of all design forces. Mat foundations and other slabs which help support the structure vertically should meet the requirements of this clause.

C6.4.4 *Limits for transverse reinforcement in beams and slabs.* All longitudinal bars in compression must be enclosed within lateral stirrups or ties in order to provide some restraint against lateral buckling, and to hold the bars adequately while the concrete is being placed.

C6.4.5 *Distance between lateral supports of beams.* Tests have shown that laterally unbraced reinforced concrete beams of any reasonable dimensions, even when very deep and narrow, will not fail prematurely by lateral buckling provided the beams are loaded without lateral eccentricity that could cause torsion [6.14, 6.15].

Laterally unbraced beams are frequently loaded off-centre (lateral eccentricity) or with slight inclination. Stresses and deformations set up by such loading become detrimental for narrow, deep beams, the more so as the unsupported length increases. Lateral supports spaced closer than 50 b may be required by actual loading conditions.

C6.4.6 *Limits for longitudinal reinforcement in columns and piers*

C6.4.6.1 This clause prescribes the limits on the amount of longitudinal reinforcement for non-composite compression members. If the use of high reinforcement ratios would involve practical difficulties in the placing of concrete, a lower percentage and hence a larger column, or higher strength concrete or reinforcement should be considered. The percentage of reinforcement in columns should usually not exceed 4% if the column bars are required to be lap spliced.

Minimum reinforcement: Since the design methods for columns incorporate separate terms for the load carried by concrete and by reinforcement, it is necessary to specify some minimum amount of reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may exist whether or not computations show that bending exists, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Tests have shown that creep and shrinkage tend to transfer load from the concrete to the reinforcement, with a consequent increase in stress in the reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless a lower limit is placed on this ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. This phenomenon was emphasized in the report of ACI Committee 105 [6.16] and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively. A minimum reinforcement ratio of 0.008 for both types of laterally reinforced columns is recommended in this Code.

Maximum reinforcement: Extensive tests of the ACI column investigation [6.16] included reinforcement ratios no greater than 0.06. Although other tests with as much as 17% reinforcement in the form of bars produced results similar to those obtained previously, it is necessary to note that the loads in these tests were applied through bearing plates on the ends of the columns and the problem of transferring a proportional amount of the load to the bars was thus minimized or avoided. It is required that bending be considered in the design of all columns, and the maximum ratio of 0.08 can be considered a practical maximum for reinforcement in terms of economy and requirements for placing.

C6.4.6.2 This clause requires a minimum of six bars for circular compression members and four for rectangular compression members. For other shapes, one bar should be provided at each apex or corner, and proper lateral reinforcement provided. For example, tied triangular columns should contain at least three bars.

C6.4.7 *Limits for transverse reinforcement in columns and piers.* Where columns contain a large amount of transverse confining reinforcement they exhibit significant ductility at high axial strains after ultimate load when the concrete shell outside the core concrete spalls off. This ductility is due to the increased strength and ductility of the concrete core, and to the restraint against buckling of the longitudinal reinforcement, provided by the transverse confining reinforcement.[6.1]

Columns may be designed using a strength reduction factor ϕ of 0.9 (see 4.3.1.2 (c)) if the quantity and arrangement of transverse reinforcement is adequate to ensure ductile behaviour. Clause 6.4.7.1 (a) specifies the required spiral or circular hoop reinforcement, and 6.4.7.2 (a) the required rectangular hoop or tie reinforcement, considered necessary for using $\phi = 0.9$. The amount of spiral reinforcement required by eq. 6-3 is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. This principle was recommended by ACI Committee 105 [6.16] and has been part of the ACI Code since 1963. The derivation of eq. 6-3 is given in the ACI Committee 105 report and elsewhere.[6.1]. Tests and experience show that columns containing the amount of spiral reinforcement required by this clause exhibit considerable ductility. Equation 6-4 was derived in the same manner as eq. 6-3 except that it was assumed that the efficiency of rectangular hoops or ties as confining reinforcement is approximately 50% of that of spirals or circular hoops. The maximum centre-to-centre spacing of the transverse reinforcement permitted in 6.4.7.1 (a) and 6.4.7.2 (a) is that considered necessary to restrain buckling of longitudinal steel and for adequate confinement of the concrete. Too great a spacing would not provide adequate lateral restraint or confinement; too small a spacing would not allow aggregate particles to pass through the spiral when concrete is being placed. Note that the confining steel is required without reduction up the full height of the column, since failure could occur away from the ends of the column, as the bending moment may be more critical elsewhere when gravity load effects dominate.

If the transverse steel present is less than that specified in 6.4.7.1 (a) or 6.4.7.2 (a), the column needs to be designed using a strength reduction factor ϕ of 0.7 if $P_u > 0.1 f'_c A_g$, or between 0.7 and 0.9 depending on the axial load level if $P_u \leqslant 0.1 f'_c A_g$, as permitted in 4.3.1.2 (c). This is because for columns with a small transverse steel content, the failure can be quite brittle once the concrete reaches its compressive strength, since the concrete is not well confined. In this case the transverse reinforcement provides some restraint against lateral buckling of longitudinal bars and also holds the bars while the concrete is being placed. Clauses 6.4.7.1 (b) and 6.4.7.2 (b) give the transverse steel requirements for this low ductility case. The quantity of transverse steel necessary is only governed by the specified spacing and the allowable minimum bar diameter.

Note that when the axial load on the column is low (that is, $P_u/f'_c A_g$ is relatively small), the dependable strength of the column with transverse steel for low ductility ($\phi < 0.9$) may be adequate. However the ductile design case ($\phi = 0.9$)

is a useful means of increasing the dependable strength of the column when the axial load on the column is relatively heavy.

C6.4.9 *Design dimensions for columns and piers.* The minimum sizes for compression members are not specified and thus reinforced concrete compression members of small cross-section may be used in lightly loaded structures, such as low rise residential and light office buildings. The engineer should recognize the need for careful workmanship, as well as the increased significance of shrinkage stresses with small sections.

For column design[6.17], the Code provisions for quantity of reinforcement, both vertical and spiral, are based on the gross column area and core area, and the design strength of the column is based on the gross area of the column section. In some cases, however, the gross area is larger than necessary to carry the factored load. The basis of 6.4.9.2 to 6.4.9.4 is that it is satisfactory to design a column of sufficient size to carry the factored load and then simply add concrete around the designed section without increasing the reinforcement to meet the minimum percentages required by 6.4.6.1. The additional concrete must not be considered as carrying load; however, the effects of the additional concrete on member stiffness must be included in the structural analysis. The effects of the additional concrete must also be considered in design of the other parts of the structure that interact with the oversize member.

C6.4.10 *Slenderness effects in columns and piers braced against sidesway.* The provisions for slenderness evaluation of reinforced concrete not designed for seismic loading are based on the recommendations of ACI Committee 441[6.18]. These recommendations call for an improved analysis procedure wherever possible or practical (see 6.4.10.1) but in lieu of such improved analysis the Code provides an approximate method (see 6.4.10.2 and 6.4.11) based on a moment magnifier principle and similar to the procedure used as part of the American Institute of Steel Construction Specification[6.19].

Procedures for carrying out second order analyses for members not designed for seismic loadings are given in references [6.20, 6.21] and [6.22]. Flexural stiffness values that could be used in such analyses are given in reference [6.23].

C6.4.11 *Approximate evaluation of slenderness effects for columns and piers braced against sidesway.* This clause describes an approximate slenderness-effect design procedure based on the moment magnifier concept. The moments computed by an ordinary frame analysis are multiplied by a "moment magnifier" which is a function of factored axial load P_u and the critical buckling load P_c for the column. The design procedure embodies some of the similar design provisions of the working stress procedure for steel beam-columns included in the AISC specifications for structural steel for buildings [6.19].

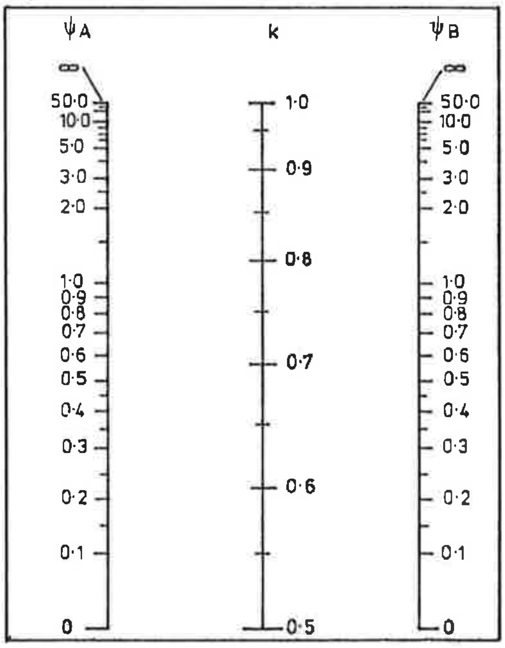
Only columns and piers braced against sidesway are considered, since columns and piers in unbraced frames will be subject to seismic loading and must not be designed using clause 6.4. The method given here allows only for elastic deformations with an allowance for concrete creep and does not account for plastic hinging which is deliberately introduced in earthquake frame design. Hence the method should not be used for members in which plastic hinging is expected to occur [6.24, 6.25]. Examples of compression members braced against sidesway are those located in a storey in which bracing elements such as shear walls or shear trusses carry almost all of the lateral seismic loading. With these stiff elements the lateral deflection of the storey, even when the stiff elements yield and deflect into the inelastic range, should not be great enough to cause the braced compression members to yield. The provisions of this clause are only applicable if the members braced against sidesway are not expected to yield under seismic loading.

Reference [6.9] gives some design aids for slender columns.

C6.4.11.2 The Code requires the use of effective length factors in computing slenderness effects. The fundamental equations for the design of slenderness compression members were derived for hinged ends and must be modified to account for the effect of end restraints. This is done by using an "effective length", $k\ell_u$, in the computation of slenderness effects, as is used for beam-column design in the AISC specifications [6.19].

The primary design aid available to designers for estimating the effective length factor k is the Jackson and Moreland Alignment Charts (fig. C6.2) which allow a graphical determination of k for a column of constant cross-section in a multi-bayframe [6.26]. The use of these charts for computing the effective length is recommended by ACI Committee 441.

The effective length is a function of the relative stiffness at each end of the compression member and studies have indicated that the effects of widely varying beam and column reinforcement percentages and beam cracking should be considered in determining the relative end stiffnesses. In determining the effective length factor k by fig. C6.2 the rigidity of the beams may be calculated on the basis of the moment of inertia of the cracked transformed section and the rigidity of the compression members on the basis of EI from eq. 6-8 with $\beta_d = 0$ [6.27].



where ψ = Ratio of $\Sigma\,(EI/\ell_c)$ of columns to $\Sigma\,(EI/\ell)$ of beams in a plane at one end of a column

k = Effective length factor

Fig. C6.2 EFFECTIVE LENGTH FACTORS FOR BRACED FRAMES

Alternatively, using a value of $0.5\,I_g$ for beams (to account for the effect of cracking and reinforcement on relative stiffness) and I_g for compression members when computing ψ will result in reasonable member sizes for columns with $k\ell_u/r$ less than 60.

Simplified equations for computing the effective length factors for braced compression members have since been recommended by the 1972 British Standard Code of Practice [6.28, 6.29]. An upper bound to the effective length for braced compression members may be taken as the smaller of the following two expressions according to the British Code:

$$k = 0.7 + 0.05\,(\psi_A + \psi_B) \leqslant 1.0 \quad \ldots\ldots \quad \text{(Eq. A)}$$

$$k = 0.85 + 0.05\,\psi_{min.} \leqslant 1.0 \quad \ldots\ldots \quad \text{(Eq. B)}$$

where ψ_A and ψ_B are the values of ψ at the two ends of the column and $\psi_{min.}$ is the smaller of the two values.

The use of the charts in fig. C6.2 or eq. A or B may be considered as satisfying the requirements of the code to justify k less than 1.0.

C6.4.11.4 The slenderness ratio limit, below which slenderness effects need not be considered in design, indicate that many stocky and sufficiently restrained compression members can essentially develop the full cross-sectional strength. The lower limit was determined from a study of a wide range of columns and corresponds to lengths for which a slender member strength of at least 95% of the cross-sectional strength can be developed. While elimination of slenderness considerations for these members may result in strength inaccuracies of up to 5%, the designer's job is considerably simplified, since studies [6.18] of a series of actual structures indicate that slenderness effects could be neglected for about 90% of the columns in braced frames. In most braced frames it is sufficiently accurate to evaluate the limit on slenderness in 6.4.11.4 by using estimated values of the effective length factor k. For columns hinged at both ends, k of 1.0 should be used. For stocky columns restrained by flat slab floors, k ranges from about 0.95 to 1.0, and conservatively can be estimated as 1.0 for a preliminary slenderness evaluation. For columns in beam-column frames, k ranges from about 0.75 to 0.9, and conservatively can be estimated as 0.90. If the initial computation of slenderness ratio based on estimated values of k indicates that effects of slenderness must be considered in the design, a more accurate value of k should be calculated and slenderness re-evaluated.

An upper limit is imposed on the slenderness ratio of columns designed by the moment magnification method of 6.4.11. No similar limit is imposed if design is carried out according to 6.4.10. The limit of $k\ell_u/r = 100$ represents the upper range of actual tests of slender compression members in frames.

C6.4.11.5 The approximate slender column design equations are based on the concept of a moment magnifier δ which amplifies the column moments to account for the effect of axial loads on these moments. The column cross-section is then designed for the axial load and the magnified moment. In application, δ is a function of the ratio of the axial load in the column to the assumed critical load of the column, the ratio of column end moments, and the deflected shape of the columns.

In the case of compression members which are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in eq. 6-5 and C_m must be taken as 1.0 for this case.

In defining the critical load, the main problem is the choice of a stiffness parameter EI, which reasonably approximates the stiffness variations due to cracking, creep, and the non-linearity of the concrete stress-strain curve. ACI Committee 441 [6.18] recommended that where more precise values are not available, EI be defined by eq. 6-8 and eq. 6-9. These expressions approximate the lower limits of EI for practical cross-sections and hence are conservative for secondary moment calculations. They were derived for small e/h values and high P_i/P_o values, where the effect of axial load is most pronounced. P_o is the ideal axial load strength at zero eccentricity and P_i is the ideal axial load strength at given eccentricity. The approximate nature of the EI expressions is shown in fig. C6.3 where they are compared with values derived from load-moment-curvature diagrams for the case of no sustained load ($\beta_d = 0$). Equation 6-8 represents the lower limit of the practical range of stiffness values. This is especially true for the heavily reinforced columns. Equation 6-9 is simpler to use but greatly underestimates the effect of reinforcement in heavily reinforced columns. However, in many cases, when reinforcement percentages are low, or slenderness effects not very substantial, its relative simplicity may be desirable. Creep due to sustained loads tends to reduce the effective value of EI. This is taken into account by dividing the EI term by $(1 + \beta_d)$ where β_d is the ratio of dead load moment to total load moment. This factor gives a correct trend when compared to both analyses and tests of columns under sustained loads.

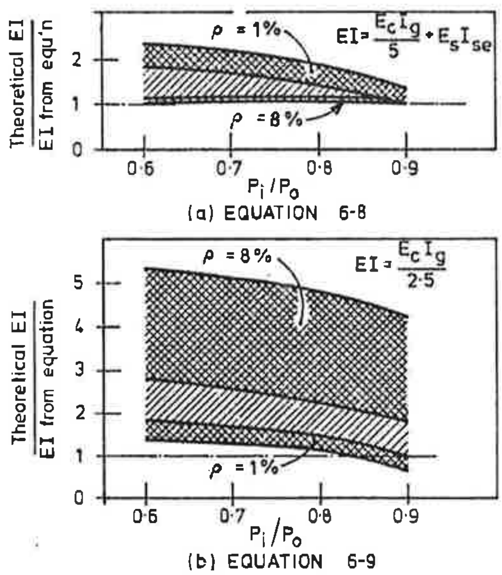


Fig. C6.3 COMPARISON OF EQUATIONS FOR *EI* WITH *EI* VALUES FROM MOMENT-CURVATURE DIAGRAMS

Note that the Code states that EI in eq. 6-7 may be taken as either value obtained from eq. 6-8 or 6-9 in lieu of a more precise calculation. In this respect, the code refers to a more accurate value of EI as obtained from moment-curvature relationships,based on the integration of acceptable non-linear stress-strain diagrams for concrete in flexure. Any stress-strain function which provides agreement with test data may be used. The more accurate values of EI may be used for designing columns or walls under the provisions stated in Section 6.

In computing δ, the factor C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near mid-height of the column. If the maximum applied moment occurs at one end of the column, design must be based on an "equivalent uniform moment", $C_m M_2$, which would lead to the same maximum moment when magnified [6.18].

In this Code, slenderness is accounted for by magnifying the column end moments. If the factored column moments are very small or zero, the design of slender columns must be based on the minimum eccentricity given in this clause. It is not intended that the minimum eccentricity be applied about both axes simultaneously. The factored column end moments are used in eq. 6-10 in determining the ratio M_1/M_2 for the column when the design must be based on minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with computed eccentricities less than the minimum eccentricity and columns with computed eccentricities equal to or greater than the minimum eccentricity. If computations show that there is no moment at both ends of a column due to the greater relative flexibility of the restraining members at the column ends, the ratio M_1/M_2 should be taken equal to 1.0.

C6.4.11.6 When biaxial bending occurs in a compression member, the computed moments about each of the principal axes must be magnified. The magnification factors δ are computed considering the buckling load P_c about each axis separately, based on the appropriate effective lengths ($k\ell_u$) and the related stiffness (EI). The clear column height may differ in each direction, and the stiffness ratios $\Sigma\,(EI/\ell_c)$ of columns to $\Sigma\,(EI/\ell)$ of flexural members may also differ. Thus, the different buckling capacities about the two axes are reflected in different magnification factors. The moments about each of the two axes are magnified separately, and the cross-section is then proportioned. References [6.1] and [6.10] to [6.13] provide guidance in this respect. Note that the moment, $M_c = \delta M_2$, refers to the "larger end moment" with respect to bending about one axis. It will usually be necessary, therefore, to magnify the moments at both ends of a column subjected to biaxial bending, and to investigate both conditions at both ends.

C6.4.12 *Composite columns and piers*

C6.4.12.1 Composite columns are defined without reference to classifications of combination, composite, or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used with concrete in construction.

C6.4.12.2 The same rules used for computing the load-moment interaction strength for reinforced concrete sections can be applied to composite sections.

C6.4.12.3 and C6.4.12.4 The requirement that loads assigned to concrete must be developed by direct bearing against the concrete effectively eliminates the old combination column as a composite column under the Code definition. Direct bearing can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. Simply wrapping concrete around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

C6.4.12.5 The rules of 6.4.11.3 for estimating the radius of gyration are over-conservative for concrete-filled tubing, and an alternate procedure is provided in this Clause. The EI formula suggested is consistent with 6.4.11.5 and provides a conservative estimate of the concrete stiffness. It leads to conservative moment magnification and estimates of strength.

C6.4.12.6 Steel encased, concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

C6.4.12.7 Concrete that is laterally contained by a substantial spiral has increased load carrying strength, and the size of spiral required can be regulated on the basis of the strength of the concrete outside the spiral by means of the same reasoning that applies for columns reinforced only with longitudinal bars. The radial pressure provided by the spiral insures interaction between concrete, reinforcing bars, and steel core such that longitudinal bars will both stiffen and strengthen the cross-section.

C6.4.12.8 Concrete that is laterally contained by light tie bars is likely to be rather thin along at least one face of a steel core section, and complete interaction between the core, the concrete, and any longitudinal reinforcement should not be assumed. Concrete will probably separate from smooth faces of the steel core. To maintain the concrete around the structural steel core, it is reasonable to require more lateral ties than needed for ordinary reinforced concrete columns. As a result of probable separation at high strains between the steel core and the concrete, longitudinal bars will be ineffective in stiffening cross-sections even though they would be useful in sustaining compression forces. Finally, the yield strength of the steel core should be limited to that which exists at strains below those that can be sustained without spalling of the concrete. It has been assumed that axially-compressed concrete will not spall at strains less than 0.0018. The yield strength of 0.0018 x 200 000 = 350 MPa represents an upper limit of the useful maximum steel stress.

C6.5 Principles and requirements additional to 6.3 for members designed for seismic loading

C6.5.1 *Strength calculations*

C6.5.1.1 The same assumptions apply for the strength design of cross-sections as are used for members not designed for seismic loading. References [6.1, 6.4, 6.9] and others give theory and design aids. The use of an extreme fibre concrete compressive strain of 0.003, as specified in 6.3.1.3, will generally result in a satisfactory calculation for the flexural strength of a beam but may lead to a significant underestimate of the flexural strength of a column if high strength steel is used or large quantity of confining steel is present. This is because, for beams with steel only in the extreme fibres, the maximum moment may not show much variation over a large range of high strains, but for columns the strain level may have a significant effect on the stresses in steel around the column perimeter and on the strength of confined concrete. Hence it may be advantageous to use a higher extreme fibre concrete compressive strain (say up to 0.004) in assessing the strength of columns, but that strain should only be taken as greater than 0.004 if the concrete is confined as in 6.5.4 and if the contribution of the concrete cover is neglected. Also, use of an extreme fibre concrete compressive strain of 0.003 will result in a very conservative calculation for the ultimate curvature of the member [6.1], particularly if the concrete is confined. It is not intended that an extreme fibre concrete strain of 0.003 be used in ultimate curvature calculations. References, such as [6.1], give information on the stress-strain behaviour of confined concrete at high strains which could be utilized in refined flexural strength and ultimate curvature calculations.

C6.5.1.2 The full member cross-section, including the concrete cover outside the transverse reinforcement, may be assumed to contribute to the flexural strength of the cross-section calculated at an extreme fibre concrete strain of 0.003. At significantly higher concrete strains spalling of the concrete cover will occur but the transverse confining steel present, and strain hardening of the longitudinal steel, will generally allow the core of the member to maintain a substantial moment and axial load. Loss of concrete cover has a more significant effect on the moment capacity of members with small cross-section than on members with large cross-section at high curvatures.

C6.5.1.3 and C6.5.1.4 These Clauses refer both to building frames and to bridge piers, but it should be noted that the philosophy governing earthquake resistant design of buildings and bridges often differs as outlined in Section 3. A procedure for determining the column design actions in multi-storey buildings, which will provide a high degree of protection against plastic hinging of columns during severe seismic motions, is described in the Appendix to the Commentary on Section 3. Note that the flexural strength of members may have to be increased to compensate for *P*-delta effects in accordance with 3.5.6.13.

C6.5.1.5 An upper limit is placed on the axial load in columns and piers because for heavily loaded sections a large amount of confining steel is required to make the section adequately ductile and there comes a stage when the available curvature ductility of very heavily loaded column or pier sections becomes doubtful even with very large amounts of confining steel. The limit of $0.7\ \phi f'_c A_g$ is reasonable for members with moderate longitudinal steel ratios. For members with higher steel ratios the less conservative limit of $0.7\ \phi\ P_o$ can be used. It may be shown that $0.7\ \phi\ P_o$ is greater than $0.7\ \phi f'_c A_g$ when A_{st}/A_g exceeds $0.15 \Big/ \left(\dfrac{f_y}{f'_c} - 0.85 \right)$.

The upper limit of $0.7\ \phi f'_c A_g$ or $0.7\ \phi P_o$ also applies to columns or piers which are designed for deliberate plastic hinging (for example, in the case of one or two storey buildings where NZ4203 allows column sidesway mechanisms, or in bridge piers) since it is considered that columns and piers detailed according to 6.5.4 will have adequate curvature ductility to enable the structure to deform to the required displacement ductility factor. That is, because the amount of transverse steel increases with the axial load level, there is no need to place a more severe limit on axial load level for these cases than the limit given in 6.5.1.5. Where the loads on columns and piers have been derived from a capacity design procedure the value of ϕ in the above expressions should be taken as unity.

When loads on columns and frames are derived from capacity design principles the value of ϕ in the above equations should be taken as unity.

C6.5.2 *Dimensions*

C6.5.2.1 The criteria for the relationship between clear span, depth and breadth of rectangular flexural members are based on dimensional limitations of the British Code of Practice CP 110 (1972). It was recognized, however, that stiffness degradation occurs in a flexural member during reversed cyclic loading in the yield range. Hence only one-half of the maximum slenderness ratios in CP110 have been allowed. It has also been assumed that a continuous beam subjected to end moments due to lateral loading is equivalent to a cantilever with a length equal to two-thirds of that of the continuous beam and having an effective length factor of 0.75. The correspondingly adjusted CP110 recommendations result in equations 6-14 and 6-15.

C6.5.2.2 For cantilevers a similar procedure was used. In this case the true length of the member with an effective length factor of 0.85 was considered, and the free end was not considered to be restrained against lateral movement. For bridge piers the criteria stated in equations 6-16 and 6-17 will not be appropriate if diaphragm action of the superstructure can be relied upon. However, if these equations are not used for bridge piers special studies should be conducted to establish that lateral buckling will not be a problem.

C6.5.2.3 The contribution of flanges, built integrally with a web, to the stability of T- and L-beams has been recognized by allowing the maximum values of the length to breadth ratio, ℓ_n/b_w, for rectangular flexural members to be increased by 50%. Note that the restrictions of equations 6-15 and 6-17 remain the same as for rectangular beams.

The breadth to depth ratios and depth to length ratios are shown in fig. C6.4 as functions of the length to breadth ratio. These rules allow a more uniform design approach to beam, column and wall sections.

C6.5.2.4 The minimum width of the compression face of flexural members is specified as 200 mm. Although not specified in the Code, it is also essential that the beam should not be too wide, so that the bulk of the flexural reinforcement in a beam may be anchored or pass within the column core. It is considered undesirable to make the effective beam section much wider than the width of the column.

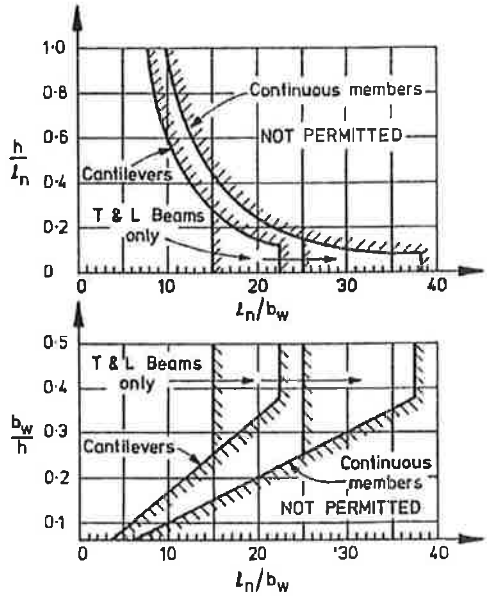


Fig. C6.4 DIMENSIONAL LIMITATIONS FOR MEMBERS

To keep the longitudinal beam steel required for seismic loading reasonably close to the column core, the effective width of a beam should not be more than the width of the supporting column plus a distance on each side of that column equal to one quarter of the overall depth of the column in the relevant direction. Figure C6.5 illustrates the interpretation of these commentary recommendations. Notwithstanding these recommendations, 6.5.3.2 (e) (5) requires that at least 75% of the effective longitudinal beam reinforcement must pass through or be anchored within a column core.

C6.5.2.5 The effective width to be considered for wide columns and the treatment of eccentric beam-column connections are discussed in Commentary Clauses C9.5.7 and C9.5.8. Frame details in which the axes of the beams and columns do not coincide should be avoided.

C6.5.3 *Reinforcement in beams*

C6.5.3.1 The three regions where plastic hinging could occur in beams are discussed below:

(a) Regions adjacent to supporting columns, where both the top and the bottom steel can be subjected to yielding in tension and compression due to reversed flexure. (See fig. C6.6.)

(b) When a plastic hinge is deliberately relocated from a column face it should be designed so that its critical section is at least a distance equal to the member depth h or 500 mm away from the column face. This section will occur where the flexural reinforcement is abruptly terminated by bending it into the beam, or where a significant part of the flexural reinforcement is bent diagonally across the web, or where the narrow end of a haunch occurs. It is considered that under reversed loading yielding can encroach into the zone between the critical section and the column face. Therefore special transverse reinforcement must be placed at least 0.5 h or 250 mm before that section and extended over a distance of 2 h to a point 1.5 h past the critical section. Two examples are given in fig. C6.7.

(c) A plastic hinge may form in the positive moment region of a beam where a negative moment plastic hinge cannot develop. In this region the danger of buckling of the top compression bars is far less, since those bars will never have yielded in tension in a previous load cycle. Moreover such a plastic hinge is likely to be well spread and under yield conditions it will carry very low shear forces.

The two other regions defined in (a) and (c) are shown in fig. C6.6.

C6.5.3.2 (a) Equation 6-18 will ensure that a curvature ductility factor of at least 12 can be attained when Grade 275 steel is used in a rectangular section with a compres-

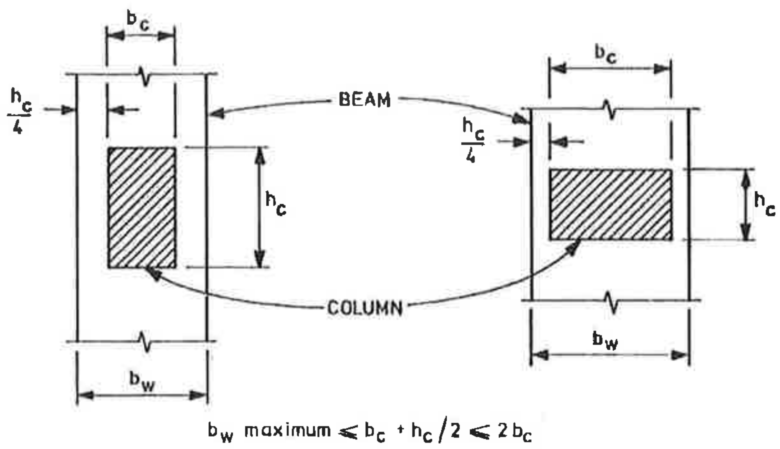


Fig. C6.5 SUGGESTED MAXIMUM WIDTH OF BEAMS

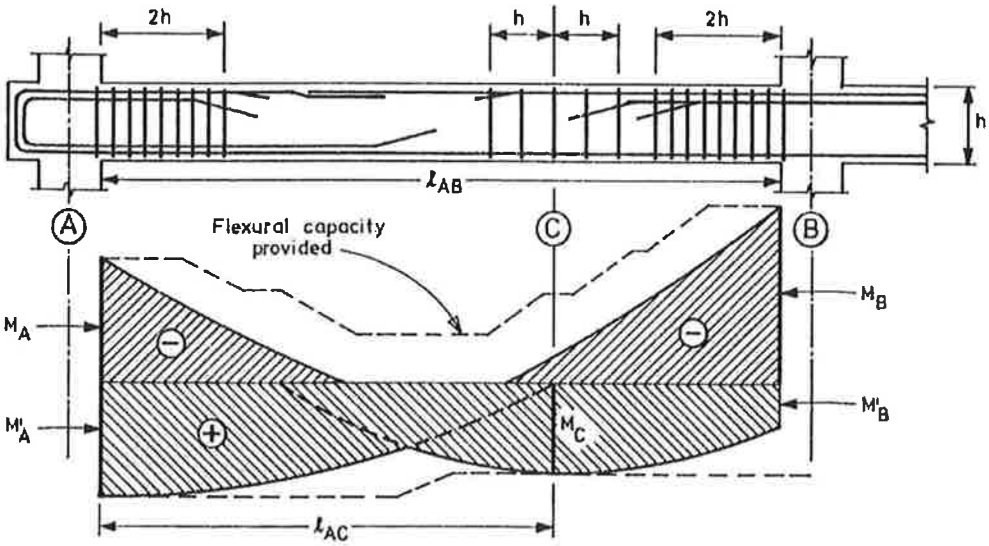


Fig. C6.6 LOCALITIES OF POTENTIAL PLASTIC HINGES WHERE STIRRUP TIES ARE REQUIRED

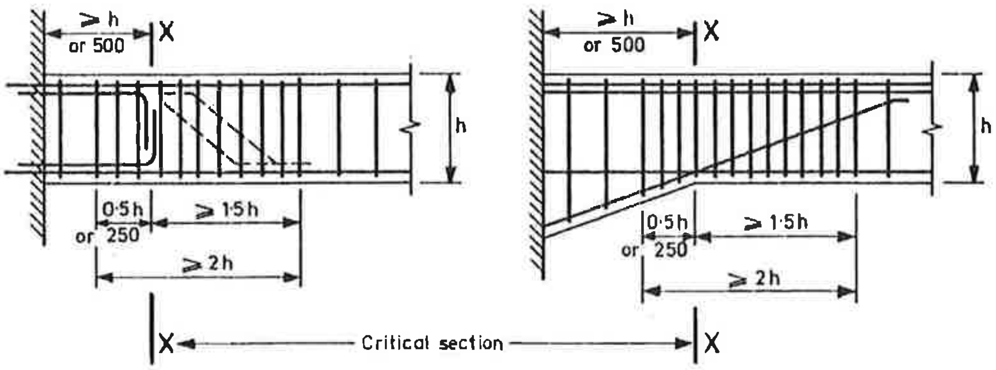


Fig. C6.7 PLASTIC HINGES LOCATED AWAY FROM COLUMN FACES

sion steel area of at least 0.5 of the tension steel area and with an extreme fibre concrete compression strain of 0.004 [6.1]. The available curvature ductility factor from Grade 380 steel using this expression will be rather less. Equation 6-18 recognizes that for the same curvature ductility factor the tension steel ratio can be increased with increased concrete strength; also the equation makes allowance for increased curvature ductility when the case of equal top and bottom steel is being approached. It must be realized, however, that full utilization of the available curvature ductility for the case of high ρ'/ρ ratios is likely to be associated with spalling of the cover concrete, since increasing the compression steel ratio ρ' to greater than 0.5 results in insignificant increases in ductility if the critical concrete strain is not to exceed 0.004. Equation 6-19 imposes a limit on $\rho_{max.}$ as a function of the yield strength in order to limit the magnitude of the maximum tensile force if Grade 380 steel is used. Equation 6-19 gives a maximum steel ratio of 0.0255 for Grade 275 steel and 0.0184 for Grade 380 steel. The maximum steel ratios allowed by equations 6-18 and 6-19 are shown as a function of ρ'/ρ and f'_c for Grade 275 steel in fig. C6.8. Note that the limitations on $\rho_{max.}$ are more severe than the ACI 318-77 or SEAOC limits of 0.025. In any case, for practical reasons, and in particular to control the amount of shear reinforcement in beam-column joints, it is expedient to limit the tension steel ratio in beams to not greater than 0.02.

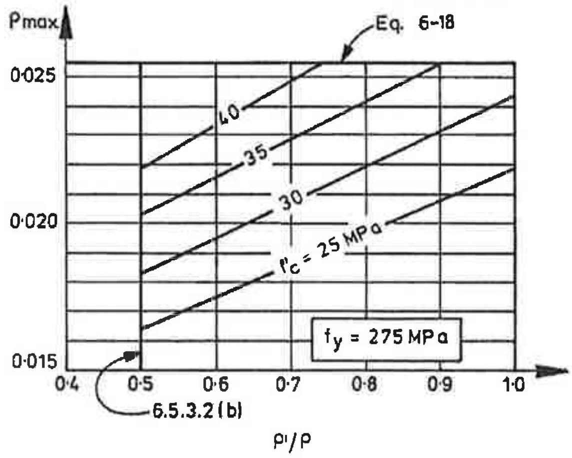


Fig. C6.8 MAXIMUM TENSILE REINFORCEMENT RATIOS FOR BEAMS

C6.5.3.2 (b) It has been widely accepted that the area of compression steel should be at least equal to one-half of the area of tension steel, in order to ensure adequate ductility at potential plastic hinge regions. With less compression steel the tension steel ratio would have to be reduced considerably, as is evident from eq. 6-18 in order to ensure that a reasonable curvature ductility factor is available.

C6.5.3.2 (c) and (d) Equation 6-20 permits a minimum steel ratio of 0.0051 for Grade 275 steel and 0.0037 for Grade 380 steel. It is required that at least two reasonable size bars should exist in both the top and the bottom of the beam throughout its length, and that the top reinforcement along the beam should not be reduced to less than one-quarter of the top reinforcement at either end. This is to ensure continuity of reinforcement and some positive and negative moment capacity throughout the beam to allow for unexpected deformations and moment distributions from severe earthquake loading.

C6.5.3.2 (e) Some of the reinforcement in slabs, parallel and integrally built with a beam, will participate in resisting negative moments at the supports of continuous beams. When earthquake induced moments are to be resisted the tensile and compression forces in the beams must be transferred to the core of the column beam joints. The effectiveness of force transfer to the joint core from slab bars, situated a large distance from the column, is doubtful. On the other hand the moment input capacity of the beams to the columns during large inelastic lateral displacements of the frame must not be grossly underestimated if columns are to be protected against early yielding. The requirements of this Clause represents a compromise till experimental studies furnish more accurate data.

It is the intention in these paragraphs to permit the inclusion of the slab steel, within the prescribed width limits, into the evaluation of the negative moment of resistance of the section and to require it to be considered when the overstrength of the section is being assessed.

Where transverse beams of comparable size to that under consideration, frame into a column, a larger slab width is considered in recognition of a more efficient force transfer to the column beam joint core. The four cases normally encountered are illustrated in fig. C6.9.

Because of the importance of the force transfer within the core of column beam joints under lateral loading, it is stipulated that at least 75% of the beam flexural steel, required for the load combination $D + 1.3\,L_R + E$, must pass through or be anchored in the column core. In frames subjected to earthquake loading it is desirable to place all the principal top and bottom flexural bars within the width of the web, and to carry these into or across the cores of the supporting columns. It is therefore preferable that the width of the web of a beam not be made more than the width of the column plus a distance on each side of the supporting column equal to one quarter of the overall depth of the column in the relevant direction, or more than twice the width of the column, as shown in fig. C6.5. The column core is the volume of concrete contained within hoops or spirals placed around the longitudinal column bars.

C6.5.3.3 Stirrup ties in potential plastic hinge regions of beams serve three main purposes. The first purpose is to prevent buckling of longitudinal bars in compression. The spacing of stirrup ties required to prevent buckling of bars which yield in both compression and tension during reversed flexure is much smaller than for bars which yield only in compression during monotonic flexure, because the Bauschinger effect of steel during reversed yielding causes a reduction in the tangent modulus of the steel at low stress levels. The second purpose of stirrup ties is to provide some

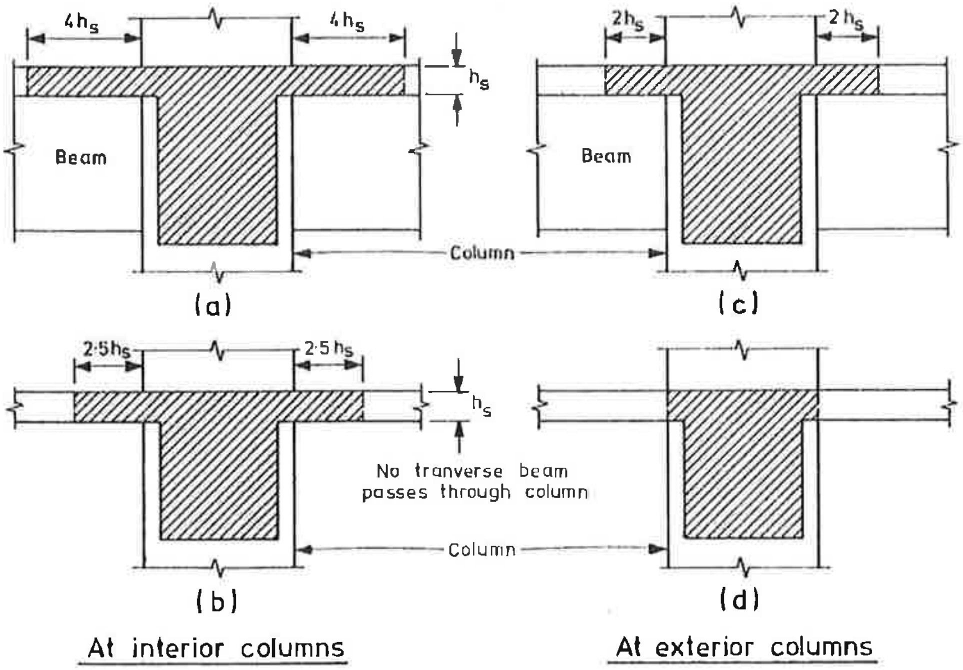


Fig. C6.9 ALL LONGITUDINAL STEEL PLACED WITHIN SHADED AREA TO BE INCLUDED IN FLEXURAL RESISTANCE OF BEAM

confinement of the concrete in the compression zone. Even in beams with equal top and bottom reinforcement it is essential to preserve the concrete in the core of the section, however much it is deformed, otherwise, after the loss of concrete, longitudinal bars could buckle laterally or inwards. The third purpose of stirrup ties is to act as shear reinforcement.

C6.5.3.3 (a) To ensure that compression bars in beams cannot buckle when subjected to yield stress, they must be restrained by a 90° bend of a stirrup tie as shown in fig. C6.10 (a). It is seen that bars numbered 1 and 2 are well restrained. Bar 3 need not be tied because the distance between adjacent tied bars is less than 200 mm. This, however, will affect the size of the ties holding bars numbered 2 as stipulated in ΣA_b in eq. 6-21.

C6.5.3.3 (b) It is considered that the capacity of a tie in tension should not be less than 1/16th of the force at yield in the bar or group of bars it is to restrain at 100 mm centres. For example the area of the tie restraining the corner bars shown in fig. C6.10 (a) should be $A_{te} = A_1/16$ assuming the yield strength for all bars is the same. However, the area of the inner ties must be $A^*_{te} = (A_2 + 0.5A_3)/16$ because they must also give some support to the centrally positioned bar marked 3. In computing the value of ΣA_b the tributary area of the unrestrained bars should be based on their position relative to the two adjacent ties.

Figure C6.10 (b) shows a beam with eight bottom bars of the same size, A_b. Assuming again that $f_y = f_{yt}$, the area of the identical ties will be $A_{te} = 2A_b/16$ because the second layer of bars is centred at less than 75 mm from the horizontal inside legs of the stirrup ties. The inner vertical ties for the bars shown in fig. C6.10 (c), however, need only support one longitudinal bar because the second layer is centred more than 75 mm from the inside of stirrup ties.

C6.5.3.3 (c) The outer bars situated in second or third layers in a beam may buckle outward if they are situated too far from a transverse leg of a stirrup tie. This situation is illustrated in fig. C6.10 (c), which shows a single transverse tie in the third layer, because these outer bars are further than 100 mm from the bottom stirrup ties. The inner four bars need not be considered for restraint because they are situated further than 75 mm from any tie. The outer bars in the second layers shown in fig. C6.10 (b) and (c) are considered satisfactorily restrained against horizontal buckling as long as they are situated no further than 100 mm from the horizontal bottom tie.

C6.5.3.3 (d) and (e) The limitations on maximum spacing are to ensure that longitudinal bars are restrained adequately against buckling and that the concrete has reasonable confinement. The limitations are more severe if longitudinal bar yielding can occur in both tension and compression, for the reasons explained previously.

C6.5.3.3 (f) At potential plastic hinges at the ends of beams, considerable stirrup reinforcement may be required to resist shear. All vertical legs of stirrup ties required according to 6.5.3.3 should be considered to contribute to shear resistance.

C6.5.4 *Reinforcement in columns and piers*

C6.5.4.1 The potential plastic hinge regions in columns and piers are generally smaller than for beams. This is because the bending moment diagrams for columns and piers have a near linear variation in moment between the end moments and the maximum moments occur at the ends of the member. In a beam, due to the presence of gravity loading, the bending moment diagram usually shows a parabolic variation between the end moments and therefore may be near maximum over a substantial length of member; also the region of maximum moment may be away from the ends.

When the axial load on the column or pier is high, the content of confining steel will also be high and will result in an increase in the strength of the confined concrete. Thus the flexural strength of the heavily confined concrete sections in the potential plastic hinge regions at the ends of the member may be significantly greater than the flexural strength of the less heavily confined concrete away from the potential plastic hinge regions. For this reason, the potential plastic hinge region to be confined is longer when the axial load is high. The value of $0.3\,\phi f'_c A_g$ for the load level at which the length of the confined region is extended was determined from the assessment of test results[6.30, 6.31].

The length of the potential plastic hinge region for axial load levels less than $0.3\,\phi f'_c A_g$ is taken as the longer cross-

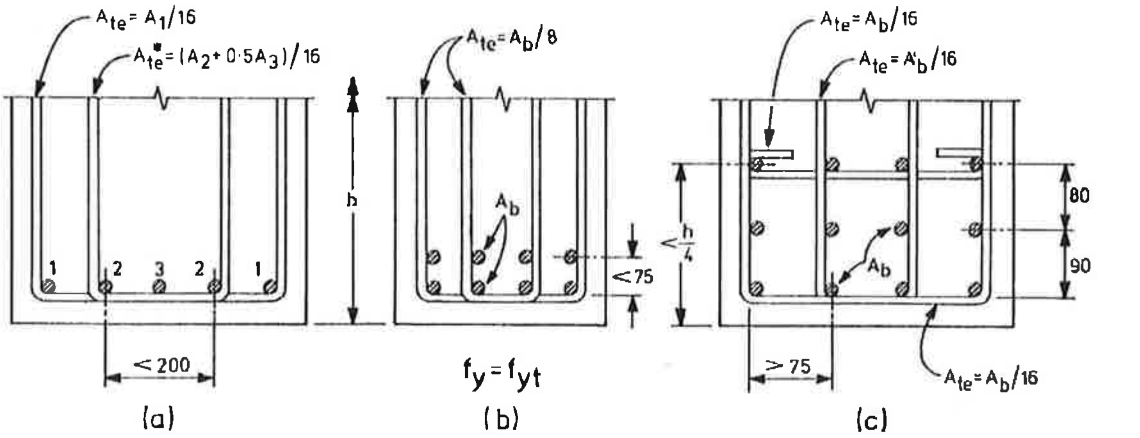


Fig. C6.10 THE ARRANGEMENT AND SIZE OF STIRRUP TIES IN POTENTIAL PLASTIC HINGE ZONES

section dimension, or the diameter, or where the moment exceeds 80% of the maximum moment. For the more heavily loaded column the potential plastic hinge region is 50% greater. The bending moment diagram for a column or pier is known quite accurately in statically determinate cases and in the case of low frames where higher mode effects are not significant. In tall frames where higher mode effects are significant, the moment diagram will be different from that given by Code loading. In lieu of more accurate analysis, the column bending moment diagram used to determine the length of the potential plastic hinge region can be considered to extend linearly from maximum moment at the end under consideration to zero moment at the other end of the column in that storey.

C6.5.4.2 (a) The minimum area of longitudinal steel is similar to that specified for members not designed for seismic loading. The minimum area does not necessarily apply to piles supporting foundation members.

C6.5.4.2 (b) The maximum areas are more specific than for members not designed for seismic loading, and are less for Grade 380 steel in view of the higher yield stress of that steel. The limits placed on maximum steel area at lap splices are such that if the maximum area is used, only one-third of the steel can be lap spliced at one time which ensures that lap splices are staggered when large steel areas are used.

C6.5.4.2 (c) The requirement concerning bar spacing in potential plastic hinge regions is to ensure that bars are distributed reasonably uniformly around the perimeter of the section in order to assist the confinement of concrete. The bars between the corner bars can also act as vertical shear reinforcement in beam column joints if required (see 9.5.5.3). In wide columns with narrow beams some concentration of the effective flexural reinforcement may be required in accordance with 9.5.7.

C6.5.4.3 (a) and (b) The amount of transverse steel in potential plastic hinge zones of columns and piers, specified by equations 6.22 to 6.25 is based on the SEAOC Code [6.32] requirements with the amount of transverse steel modified to take axial load level into account. The amount of spiral steel specified in the SEAOC (and ACI) Code is based on preserving the axial load strength of the section after the cover concrete has spalled rather than aiming to achieve a particular curvature ductility factor for the section. The amount of rectangular hoops specified is also based on the same criterion and assumes that rectangular hoops, because of their shape, are less efficient than spirals in confining the concrete. It should be noted that the philosophy of maintaining the axial load strength of sections after the spalling of 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 required content of transverse steel would be based on ensuring a satisfactory moment-curvature relationship and would include as variables the level of axial load on the section, the longitudinal steel ratio, the proportion of the section confined, the stress-strain characteristics of the longitudinal steel, and the stress-strain curve of the confined concrete as a function of the amount of confining steel. Moment-curvature analyses of columns show a decrease in moment capacity when the cover concrete spalls, but providing adequate confining steel is present the section can maintain substantial moment with further plastic rotation[6.1]. It is to be noted that this type of moment-curvature analysis utilizes the complete stress-strain curve for confined concrete. That is, the analysis does not assume that the ultimate curvature is reached when a particular concrete strain is attained, but rather it assumes that ultimate curvature is reached when the moment capacity decreases to 80%, for example, of the maximum moment capacity. A difficulty with refined moment-curvature analyses of this type is that only limited experimental data has been available to properly establish the complete stress-strain curve for confined concrete including the effect of overlapping hoops and hoops with supplementary cross ties. Only moment-curvature analyses based on stress-strain curves for confined concrete obtained from limited tests are available at present[6.1, 6.33, 6.34]. However such analyses have shown that the 1973 SEAOC Code equations for transverse steel content are generally conservative for moderate axial load levels but are not conservative at high load levels. As a result of these analyses, the SEAOC equations have been modified to take axial load level into account. Equations 6-22 to 6-25 result in the following amounts of transverse steel being placed as a percentage of the SEAOC amounts.

P_e/f'_cA_g	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7
% of SEAOC spiral and rectangular hoops	50	63	75	88	100	113	125	138

The moment-curvature analyses indicate [6.1, 6.33, 6.34] that use of the recommended equations 6-22 to 6-25 will result in significant ultimate curvatures being reached which generally are much greater than five times the yield curvature (defined as when the outermost tension bars first begin to yield), accompanied by a moment capacity which is generally not less than 0.8 times the moment capacity calculated at an extreme fibre concrete strain of 0.003, for columns with longitudinal steel ratio of 0.02 or greater, providing that the axial compression does not exceed an upper limit in the order of $0.6 f'_cA_g$ to $0.7 f'_cA_g$. It should be noted that in no case did the moment-curvature analysis indicate a sudden reduction in moment capacity. The moment-curvature analyses also showed that use of Grade 380 steel as longitudinal reinforcement in sections improves the performance of the sections at high curvatures, because the early strain hardening of that steel helps to compensate for the loss of moment strength due to the reduction of contribution from the concrete. More recent laboratory tests on full scale columns [6.30, 6.31] have demonstrated that the stress-strain curves for confined concrete used for the above moment-curvature analyses were conservative, and that detailing according to equations 6-22 to 6-25 will lead to an available curvature ductility factor of up to 18 in the case of the columns of the test series. Hence the tests confirmed that equations 6-22 to 6-25 will indeed lead to reasonable curvature ductility factors.

The centre-to-centre spacing of transverse steel of not greater than one-fifth of either the least lateral dimension or diameter of the column or pier is to ensure adequate confinement of concrete and lateral restraint against bar buckling. This maximum spacing is kept reasonably small since the concrete is confined mainly by arching between the spiral or hoops. If the spacing is large a significant depth of unconfined concrete will penetrate into the concrete core between the spirals or hoops and thus reduce the effective confined concrete section. This maximum spacing is a function of the column dimension, and hence the spacing is greater for larger sections than for smaller sections, since a greater penetration of unconfined concrete between the transverse steel is less significant for large sections. The requirements that the spacing should not exceed six longitudinal bar diameters is to prevent buckling of longitudinal steel when undergoing yield reversals in tension and compression. It is well known that such stress reversals in the yield range cause a reduction in the tangent modulus of the steel at relatively low stresses, due to the Bauschinger effect, and therefore closely spaced transverse steel providing lateral support is required to prevent buckling of the longitudinal steel.

In most rectangular sections a single rectangular peripheral hoop will be insufficient to properly confine the concrete and to laterally restrain the longitudinal steel against buckling. Therefore an arrangement of overlapping rectangular hoops or supplementary cross ties or both, will be necessary. Supplementary cross ties can only be expected to function effectively if fitted tightly around main bars, a rather difficult requirement in practice. Any cavity left between the inside of the bend of the cross tie and the outside of the hoop bar will mean that outward expansion of the concrete needs to occur before the cross tie becomes fully effective and thus some concrete confinement is lost. It would appear to be better to use a number of overlapping rectangular hoops rather than a single peripheral hoop and supplementary cross ties. An example of alternative details and the preferred arrangement is shown in fig. C6.11. Note from fig. C6.11 (a) and (b) that a supplementary cross tie can engage either the longitudinal bar or the peripheral hoop beside a longitudinal bar. That is, the concrete is confined by arching between hoops, supplementary cross ties and longitudinal bars. In a set of overlapping hoops it is preferable to have one peripheral hoop enclosing all the longitudinal bars together with one or more hoops covering smaller areas of the section. This is because such a detail is easier to construct since the longitudinal bars are held more firmly in place if they are all enclosed by one hoop. Thus the detail in fig. C6.11 (c), which has a hoop enclosing all bars and a smaller hoop enclosing the middle four bars, is to be preferred to the detail in fig. C6.11 (d), which has two hoops each enclosing six bars. Figure C6.12 shows typical details using overlapping hoops for sections with a greater number of longitudinal bars. It is to be noted that the inclined hoop surrounding the four bars at the centre of each face in fig. C6.12 (b) can be counted on making a contribution to A_{sh} in equations 6-24 and 6-25 by determining the equivalent bar area of the component of forces in the required direction. For example, two such hoop legs inclined at 45° to the section sides could be counted as making a contribution of $\sqrt{2}$ times the area of one perpendicular bar in assessing A_{sh}. That is, in fig. C6.12 (b) A_{sh} may be taken as 5.41 A_{te}, where A_{te} is the area of each hoop bar. Note also that the spacing between adjacent hoop legs or supplementary cross ties should not exceed 200 mm, as illustrated in figures C6.11 and C6.12, in order to ensure reasonable confinement from both vertical and transverse steel.

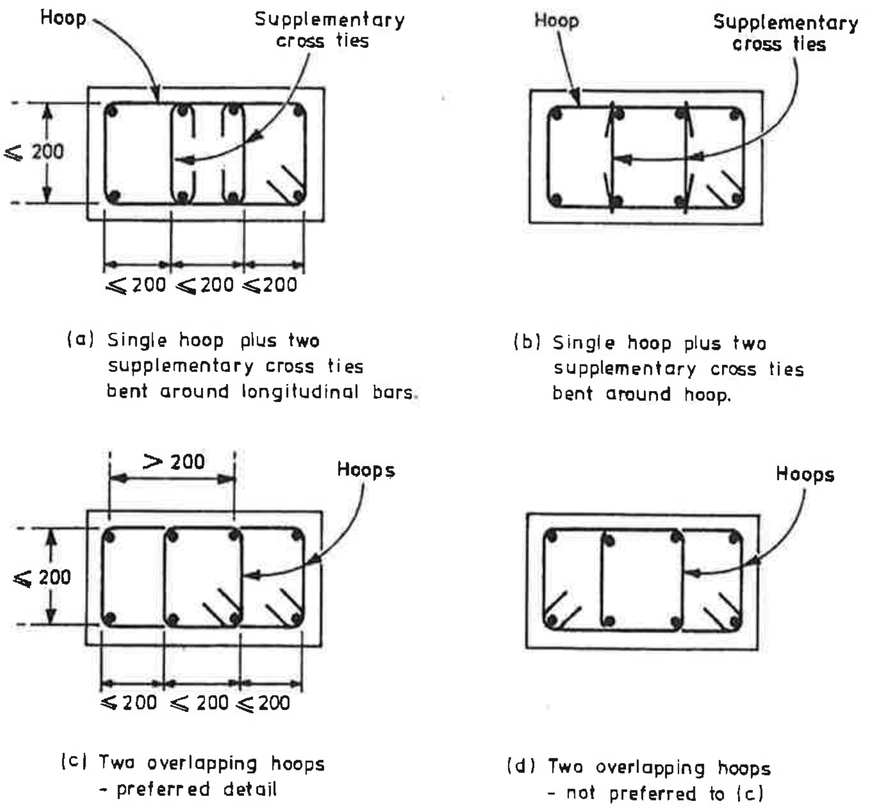
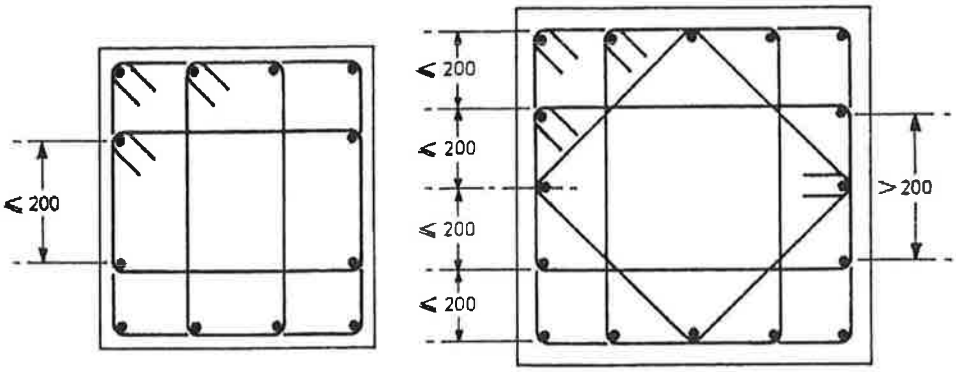


Fig. C6.11 ALTERNATIVE DETAILS USING HOOPS AND SUPPLEMENTARY CROSS-TIES



(a) Three overlapping hoops

(b) Four overlapping hoops

Fig. C6.12 TYPICAL DETAILS USING OVERLAPPING HOOPS

The rectangular hoops and supplementary cross ties in sections are also necessary to provide lateral support to the longitudinal bars to prevent buckling. However, not all bars need to be laterally supported by a bend in a transverse hoop or cross tie. If bars or groups of bars which are laterally supported by bends in the same transverse hoop or cross tie are less than or equal to 200 mm apart, any bar or bundle of bars between them need not have effective lateral support from a bent transverse bar, as is demonstrated in fig. C6.12 (a). Also, bars which lie within the core of the section centred more than 75 mm from the inside face of the peripheral hoop need no special lateral support. When supplementary cross ties are used bent tightly around the hoops (see fig. C6.11 (b)) they also should be secured to the longitudinal bars at each end so as to allow the hoop to give effective lateral support to those longitudinal bars. That is, although the supplementary cross ties do not pass around the four longitudinal bars in fig. C6.11 (b), they can be regarded as providing effective lateral support to those longitudinal bars since they effectively restrain the hoop beside the bars. The requirement that the yield force of the hoop bar or supplementary cross tie should be at least one-sixteenth of the yield force of the bar or bars to be restrained is similar to that for beams and an explanation of this requirement is given in C6.5.3.3.

Details on spiral and hoop anchorage and bends are given in Section 5.

C6.5.4.3 (c) In beams where a capacity design procedure has been used to decrease the possibility of column hinging, the need for transverse steel to confine the concrete is reduced since only limited yielding of columns will occur under extreme conditions. Hence if a capacity design procedure is adopted, which takes into account possible beam overstrength, higher mode effects and concurrent earthquake loading, to determine the design actions for columns, it is considered that the amount of confining steel can be reduced to one-half of that required by equations 6-22 to 6-25. However protection against bar buckling is still required, and some concrete confinement is necessary and hence all the other requirements of detailing of 6.5.4 are still necessary.

This reduction in transverse steel does not apply at the bases of columns (that is, at the bottom of the columns of the lowest storey of frames) since plastic hinging cannot be prevented there. Nor does it apply to columns or piers where plastic hinging is expected to occur, such as in one or two storey frames, or the top storey of multistorey frames, or in bridge piers, which are deliberately designed for plastic hinging.

C6.5.4.3 (d) In regions of columns or piers outside potential plastic hinge regions the spacing of transverse steel can be increased, since yielding of longitudinal steel should not occur there and the concrete does not need as much confinement. However the reduction in quantity of transverse steel should occur reasonably gradually. Hence in the length of member equal to the length of potential plastic hinge region, and immediately adjacent to the potential plastic hinge region, the quantity of transverse steel can be one-half of that in the potential plastic hinge region. Elsewhere between potential plastic hinge regions the requirements of shear will govern the quantity of transverse steel placed.

C6.5.4.3 (e) The transverse reinforcement placed for confinement may be assumed to contribute to the shear strength of the member providing it effectively crosses the section. That is, interior hoops passing around only a few longitudinal bars are not effective as shear reinforcement if they do not extend to near the extreme tension and compression fibres of the section.

REFERENCES

6.1 Park, R. and Paulay, T., "Reinforced Concrete Structures", Wiley Interscience, New York, 1975, 769 pp.

6.2 Nedderman, H., "Flexural Stress Distribution in Extra High Strength Concrete", MS Thesis, University of Texas at Arlington, 1973.

6.3 Kaar, P.H., Hanson, N.W. and Capell, H.T., "Stress-Strain Characteristics of High Strength Concrete", Douglas McHenry International Symposium on Concrete and Concrete Structures, ACI Publication SP-55, 1978.

6.4 Mattock, A.H., Kriz, L.B., and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate

Strength Design", ACI Journal, Proceedings V.57, No. 8, Feb. 1961, pp. 875-928. Also, Development Department Bulletin D49, Portland Cement Association.

6.5 Chow, Li., Conway, Harry, and Winter, George, "Stresses in Deep Beams", Transactions, ASCE, V.118, 1953, pp. 686-708.

6.6 "Design of Deep Girders", Portland Cement Association, ISO79D, 1946, 10 pp.

6.7 Bianchini, Albert C., Wood, Robert E, and Kesler, Clyde E., "Effect of Floor Concrete Strength on Column Strength", ACI Journal, Proceedings V.56, No. 11, May 1960, pp. 1149–1169.

6.8 Hawkins, N.M., "The Bearing Strength of Concrete Loaded Through Rigid Plates", Magazine of Concrete Research (London), V.20, No. 62, Mar. 1968, pp. 31–40.

6.9 "New Zealand Reinforced Concrete Design Handbook", New Zealand Portland Cement Association, Wellington, 1979.

6.10 Weber, D.C., "Ultimate Strength Design Charts for Columns with Biaxial Bending", ACI Journal, Proceedings V.63, No. 11, Nov. 1966, pp. 1205–1230.

6.11 Parme, A.L., Nieves, J.M. and Gouwens, A., "Capacity of Reinforced Rectangular Columns Subjected to Biaxial Bending", ACI Journal, Proceedings V.63, No. 9, Sept. 1966, pp. 911–923.

6.12 Bresler, Boris, "Design Criteria for Reinforced Concrete Columns Under Axial Load and Biaxial Bending", ACI Journal, Proceedings V.57, No. 5, Nov. 1960, pp. 481–490.

6.13 Row, D.G. and Paulay, T., "Biaxial Flexure and Axial Load Interaction in Short Rectangular Reinforced Concrete Columns", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 6, No. 3, September 1973, pp. 110–121.

6.14 Hansell, William and Winter George, "Lateral Stability of Reinforced Concrete Beams", ACI Journal, Proceedings V.56, No. 3, Sept. 1959, pp. 193–214. (Discussion Mar. 1960, pp. 957–967.)

6.15 Sant, Jagadish, K. and Bletzacker, Richard W., "Experimental Study of Lateral Stability of Reinforced Concrete Beams", ACI Journal, Proceedings V.58, No. 6, Dec. 1961, pp. 713–736.

6.16 "Reinforced Concrete Column Investigation – Tentative Final Report of Committee 105", ACI Journal, Proceedings V.29, No. 5, Feb. 1933, pp. 275–282.

6.17 Furlong, Richard W., "Column Slenderness and Charts for Design", ACI Journal, Proceedings V.68, No. 1, Jan. 1971, pp. 9–18.

6.18 MacGregor, James G, Breen, John E. and Pfrang, Edward O., "Design of Slender Concrete Columns", ACI Journal, Proceedings V.67, No. 1, Jan. 1970, pp. 6–28.

6.19 "Commentary and Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings", American Institute of Steel Construction, New York, 1969.

6.20 Wood, B.R., Beaulieu, D. and Adams, P.F., "Column Design by P-Delta Method", Proceedings, ASCE, V.102, No. ST2, Feb. 1976, pp. 411–427.

6.21 Wood, B.R., Beaulieu, D. and Adams, P.F., "Further Aspects of Design by P-Delta Model", Proceedings, ASCE, No. ST3, Mar. 1976, pp. 487–500.

6.22 MacGregor, J.G. and Hage, S.E., "Stability Analysis and Design of Concrete", Proceedings, ASCE, V.103, No. ST10, Oct. 1977.

6.23 MacGregor, J.G., Oelhafen, V.H. and Hage, S.E., "A Re-examination of the EI Value for Slender Columns", Reinforced Concrete Columns, (SP-50), American Concrete Institute, Detroit, 1975, pp. 1–40.

6.24 Andrews, A.L., "Slenderness Effects in Earthquake Resisting Frames", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 3, September 1977, pp. 154–158.

6.25 Paulay, T., "A Consideration of P-Delta Effects in Ductile Reinforced Concrete Frames", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 11, No. 3, Sept. 1978, pp. 151–160.

6.26 "Guide to Design Criteria for Metal Compression Members", Column Research Council, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa. 2nd Edition, 1966.

6.27 Breen, John E., MacGregor, James G. and Pfrang, Edward, O., "Determination of Effective Length Factors for Slender Concrete Columns", ACI Journal, Proceedings V.69, No. 11, Nov. 1972, pp. 669–672.

6.28 "Code of Practice for the Structural Use of Concrete. Part 1, Design Materials and Workmanship", (CP110: Part 1, Nov. 1972) British Standards Institution, London, 1972, 154 pp.

6.29 Cranston, W.B., "Analysis and Design of Reinforced Concrete Columns", Research Report No. 20, Paper 41.020, Cement and Concrete Association, London, 1972, 54pp.

6.30 Potangaroa, R.T., Priestley, M.J.N. and Park, R., "Ductility of Spirally Reinforced Concrete Columns Under Seismic Loading", Research Report 79-8, Department of Civil Engineering, University of Canterbury February 1979.

6.31 Gill, W.D., Park, R. and Priestley, M.J.N., "Ductility of Rectangular Reinforced Concrete Columns with Axial Load", Research Report 79-1, Department of Civil Engineering, University of Canterbury, February 1979.

6.32 "Recommended Lateral Force Requirements and Commentary", Seismology Committee, Structural Engineers Association of California, 1975.

6.33 Park, R. and Sampson, R.A., "Ductility of Reinforced Concrete Column Sections in Seismic Design", Journal ACI, Vol. 69, No. 9, September 1972, pp. 543–551.

6.34 Park, R. and Norton, J.A., "Effects of Confining Reinforcement on the Flexural Ductility of Rectangular Reinforced Concrete Columns with High Strength Steel", Symposium on Design and Safety of Reinforced Concrete Compression Members, Reports of Working Commissions, Vol. 16, International Association for Bridge and Structural Engineering, Quebec, 1974, pp. 267–275.

COMMENTARY

C7 SHEAR AND TORSION

C7.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 7 of the Code.

A_{oc}	area enclosed by the mid-wall perimeter of a thin walled tube section
A_{st}	total vertical reinforcement in wall section
A_{vd}	area of diagonal shear reinforcement, mm^2
E_s	modulus of elasticity for steel
ℓ	span length
ℓ_{AB}, ℓ_{AC}	distance as specified in fig. C6.6
M_A, M_B	plastic hinge moment at A and B
M_{OA}, M_{OB}	negative flexural overstrength at A and B
M'_{OA},	positive flexural overstrength at A
M_{OC}	positive flexural overstrength at C
P_{oc}	perimeter of area A_{oc}
T	torsional moment
v_t	torsional shear stress
V_{DA}, V_{DB}	applied total shear force at A and B due to dead load
V_{L_RA}, V_{L_RB}	applied total shear force at A and B due to reduced live load
V_{iA}, V_{iB}	required ideal shear strength at A and B to resist earthquake and gravity effect
x	distance from beam section to support

C7.2 Scope. This clause includes shear provisions for both non-prestressed and prestressed concrete members with or without axial load effects. The torsion provisions apply only to non-prestressed concrete. Minimum requirements, reinforcement details and the design of reinforcement to resist shear or torsion are set out separately. The shear friction principles of 7.3.11 are particularly applicable to design of reinforcing details in precast structures. Special provisions are made for deep beams (see 7.3.12), brackets and corbels (see 7.3.13), walls (see 7.3.14) and for slabs and footings (see 7.3.15). A design procedure is included for shear head reinforcement at column supports. Special provisions are made for the shear strength of beams, columns, walls and diaphragms in earthquake resisting reinforced concrete structures.

C7.3 General principles and requirements

C7.3.1 *Shear strength.* In the 1977 edition of the ACI Building Code, which is largely followed in this Section, equations for shear and torsion have been rearranged in terms of shear forces and torsional moments because American designers expressed a preference for this. In New Zealand traditionally shear stresses have been used because

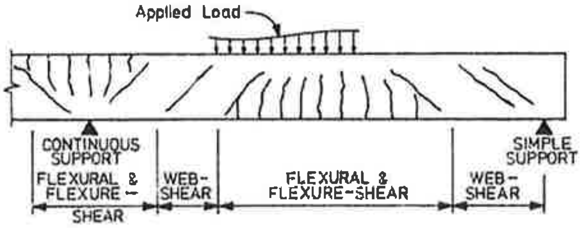


Fig. C7.1 TYPES OF CRACKING IN CONCRETE BEAMS

as an index they immediately conveyed the relative magnitude of actions to the designer and therefore the shear stress equations have been retained. All stress equations refer to ideal stresses emphasising that the strength reduction factor ϕ, leading to the dependable strength, has already been incorporated and need not be considered any further. Equation 7-1 simply relates the relationship between dependable strength required to resist the shear due to factored load on the structure, V_u, and the ideal shear strength of the member. In the routine design process the ideal shear strength $V_i = V_u/\phi$ is established and from eq. 7.2 expressed in terms of the required total shear stress $v_i = V_i/b_w d$, the average shear stress on the full effective cross-section, used in earlier ACI Building Codes. For additional discussion on the concepts and nomenclature for strength design see commentary of Section 4. To account for varied location of the centroid of prestressing tendons, distributed longitudinal bars within the entire depth of columns and walls, the value of d for such members need not be taken less than 0.8 h. Tests [7.1] have indicated that average shear over the full effective section also may be applied for circular sections. The definition of d (see 7.1) as as the distance from the extreme compression fibre to the centroid of the longitudinal reinforcement in the opposite half of the member is intended to cover the case of a circular section subjected only to transverse loads.

In a member without shear reinforcement, shear is assumed carried by the concrete web. In a member with shear reinforcement, shear is assumed carried by the concrete shear resisting mechanisms and the shear reinforcement.

Shear strength provided by concrete v_c is assumed equal in both cases and is taken equal to the shear causing significant inclined cracking. These assumptions are discussed in the ACI-ASCE Committee 426 reports [7.1, 7.2] and in references [7.3, 7.4] and [7.5].

Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in fig. C7.1.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and tensile stress exceeds the tensile strength of the concrete.

When inclined cracking occurs in a non-prestressed concrete member, it is generally of the flexure-shear type. Web-shear cracking generally occurs near the supports of deep flexural members with thin webs, or near the inflection point of bar cut-off points of continuous beams, particularly if the beam is subjected to axial tension.

Both types of inclined cracking may be observed when prestressed concrete beams are subjected to loads greater than the maximum service load. Flexure-shear cracking is the more typical type in prestressed members, particularly those subject to uniform loads. Web-shear cracking may occur in heavily prestressed beams with thin webs, particularly when the beam is subjected to large concentrated loads near a simple support.

Because of the different behaviour of non-prestressed and prestressed members, and because researchers have approached the inclined cracking problem in different ways, it is necessary to calculate the shear stress v_c provided by concrete according to 7.3.2 for non-prestressed members and 7.3.3 for prestressed members.

C7.3.1.2 Once the contribution of the concrete to shear resistance v_c is determined for either a non-prestressed or a prestressed member, the remainder of the ideal shear stress $(v_i - v_c)$ is allocated to resistance by shear reinforcement.

C7.3.1.3 The combination of the concrete to shear strength v_c is increased by axial compression on the member and conversely it is decreased by axial tension. The load producing flexural cracking is decreased should tensile stresses be present as a consequence of effects of creep, shrinkage and temperature. Whether or not these strain induced stresses will reduce the ultimate strength of the member is not clear at the present state of knowledge. It is conservative to assume that they will reduce the shear strength of the member and this is the requirement of this Code.

C7.3.1.4 Shear strength near concentrated loads or reaction is increased if compression is introduced into the member. Accordingly, the Code permits design for a maximum factored shear force V_u at a distance d from the support for non-prestressed members, and at a distance $h/2$ for prestressed members.

Typical support conditions where the factored shear force V_u at a distance d from the support may be used include:

(a) Members supported by bearing at the bottom of the member, such as shown in fig. C7.2 (a) and

(b) Members framing monolithically into another member as illustrated in fig. C7.2 (b).

Support conditions where this provision should not be applied include:

(1) Members framing into a supporting member in tension, such as shown in fig. C7.2 (c).. The critical section for shear should be taken at the face of the support. For this case, shear within the connection should also be investigated, and special corner reinforcement should be provided.

(2) Members loaded such that the shear at sections between the support and a distance d differs radically from the shear at distance d. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in fig. C7.2 (d). In this case the shear at the face of the support should be used.

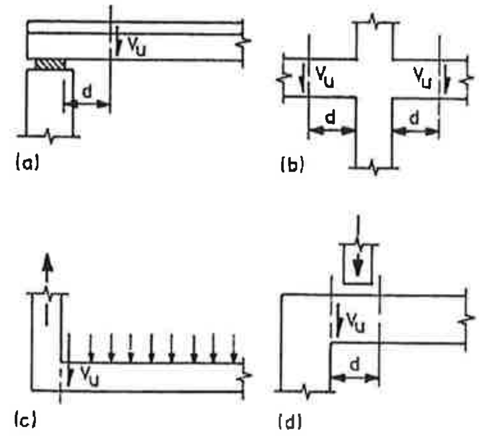


Fig. C7.2 TYPICAL SUPPORT CONDITIONS FOR LOCATING FACTORED SHEAR FORCE V_u

C7.3.1.7 For lightweight concrete two alternate procedures are provided to modify the provisions for shear when lightweight aggregate concrete is used. The lightweight concrete modification applies only to the terms containing $\sqrt{f_c'}$ in the equations of Section 7.

(a) The first alternate is based on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and the compressive strength f_c'. For the lightweight concrete, the splitting tensile strength f_{ct} is approximately equal to $\sqrt{f_c'}/1.8$. Therefore, when f_{ct} is determined for a particular lightweight aggregate concrete (NZS 3152), the value of $1.8 f_{ct}$ may be substituted for values of $\sqrt{f_c'}$ affecting v_c. Tests [7.6, 7.7] have shown this is a valid approach.

However, the calculated shear strength values for lightweight concrete should not exceed those for normal weight concrete; therefore, in calculations, the value of $1.8 f_{ct}$ must not be taken greater than $\sqrt{f_c'}$

(b) As a simplification, the modification may be based on the assumption that, for a given compressive strength of concrete, the tensile strength of lightweight concrete (with or without sand replacement) is a fixed proportion of the tensile strength of normal weight concrete. A factor of 0.75 is applied to the shear strength values for normal weight concrete, if all lightweight aggregate is used. If natural sand is combined with lightweight coarse aggregate (all fine aggregate replaced by sand) the modification factor is 0.85. Linear interpolation is allowed for partial sand replacement of the fine aggregate. Use of the factors 0.75 for "all-light-weight" concrete and 0.85 for "sand-lightweight" concrete imply a ratio of 0.42 and 0.48 for $f_{ct}/\sqrt{f_c'}$ respectively. These values are based on data obtained from tests [7.1, 7.8] on many types of structural lightweight aggregate concrete.

C7.3.1.8 To guard against diagonal compression failure in the web mainly due to truss action, the total shear stress is limited to $0.2 f_c'$ or 6 MPa whichever is less.

C7.3.2 *Shear strength provided by concrete for non-prestressed members*

C7.3.2.1 The shear stress assumed to be carried by the concrete, v_c, is assumed to be a constant for a particular beam regardless of whether the beam has stirrups or not. This shear stress is given by eq. 7-4 which in turn makes use of a basic shear stress, v_b, given by eq. 7-3. Because this same basic shear stress is used in the design equations for v_c for slender and deep reinforced concrete beams and for prestressed concrete beams, it is called v_b rather than v_c.

The use of the basic shear stress v_b has been adopted from an ACI–ASCE Committee 426 report [7.2] and it represents a departure from the provisions of the 1977 ACI Building Code.

The shear carried by the concrete is affected primarily by the concrete strength, included in eq. 7-3 as $\sqrt{f'_c}$, by the ratio of the longitudinal steel, ρ_w; and for short shear spans by the ratio of shear span to depth or M/V_d.

Equation 7-3 indicates that with less than 1% flexural steel the inclined cracking shear stress and the shear v_b carried by the concrete in beams with web reinforcement may reduce significantly below the limiting stresses specified by ACI 318-77. This fact has been observed in tests [7.10] and predicted by analysis [7.11] and is recognized in the design equations for v_c in the Australian, British and European Concrete Committee design codes for reinforced concrete [7.12]. Even with minimum web reinforcement, beams designed to ACI 318-77 can have understrength in shear. Likely positions are where flexural reinforcement terminates outside of maximum moment locations.

To use eq. 7-3 the designer first determines the longitudinal reinforcement required for flexure and then checks the shear capacity taking into account the curtailment of the flexural reinforcement. The ratio ρ_w given in the notation relates the reinforcement ratio to the web width, and it includes conventional reinforcement as well as prestressing tendons. However, only those bars having full development length beyond the section in question may be considered in determining the value of ρ_w. For $\rho_w = 0.013$ or more the value of v_b is limited to $0.2\sqrt{f'_c}$.

For simplicity v_b was made independent of the parameter of M/V_d which was used in previous ACI Codes. The shear stress v_c is increased when axial compression also acts on the member and it is decreased in the presence of axial tension. The second term of eq. 7-6 is always negative.

In a member with variable depth the shear strength of a section is increased or decreased by the vertical components of the inclined flexural stresses. Computation methods are outlined in various textbooks [7.5]. Eq. 7-7 is not intended for axially loaded members and therefore it should not be combined with eq. 7-5 or eq. 7-6.

C7.3.2.2 Special treatment is given to shallow and wide elements because past experience and test data [7.13] show that shear stresses are seldom critical in such members even though ρ_w is usually less than 0.01.

C7.3.3 *Shear strength provided by concrete for prestressed members*

C7.3.3.1 This requirement is necessary to avoid web crushing. Special studies may be made following the CEB Code recommendations [7.48].

C7.3.3.2 Equation 7-8 offers a simplified means of computing v_c for prestressed concrete beams with an effective prestress force at least equal to 40% of the tensile strength of the flexural reinforcement. Thus, eq. 7-8 may be applied to some members reinforced with a combination of prestressed tendons and non-prestressed deformed bars. The equation is discussed in detail in reference [7.4]. It is most applicable to members subject to uniform loading. The equation may give conservative results when applied to members such as composite I-section girders for bridges with concentrated loadings.

In applying eq. 7-8 to simply supported members subject to uniform loads, $V_u d_c/M_{uv}$ becomes a simple function of d_c/ℓ, where ℓ is the span length. If x is the distance from the section being investigated to the support,

$$\frac{V_u d_c}{M_{uv}} = \frac{d_c}{x}\,\frac{(\ell - 2x)}{(\ell - x)}$$

Thus, for concrete with compressive strength of 35 MPa v_c given by eq. 7-8 can be represented as shown in fig. C7.4. Similar curves can be developed for members of other concrete strength. However, eq. 7-8 is quite insensitive to concrete strength and fig. C7.4 could be used for members with concrete strength ranging from 25 to 40 MPa with an error of less than 10%.

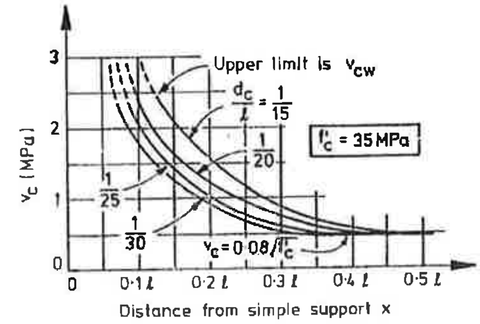


Fig. C7.3 APPLICATION OF EQ. 7-8 TO UNIFORMLY LOADED PRESTRESSED MEMBERS

C7.3.3.3 These sections give the basic design provision for determining v_c for prestressed concrete beams. Except for minor changes, these provisions have remained the same since the 1963 ACI Building Code. Equations 7-9 and 7-11 predict the shear strength at inclined flexure-shear and web-shear cracking, respectively. The ideal shear stress provided by the concrete v_c is assumed to be equal to the lesser of v_{ci} and v_{cw}. The externally applied factored loads V_u and M_u include superimposed dead load, earth pressure, live load, and so on.

Equation 7-9 predicts flexure-shear cracking as the shear force due to dead load causing flexural cracking at the section being investigated, and load required to transform the flexural crack into an inclined crack. It is similar to the equation provided in ACI 318-77 except that the basic shear stress v_b has been introduced. For a reinforced concrete beam with no axial load, eq. 7-9 reduces to eq. 7-4. This allows the same basic design procedure to be used for reinforced concrete beams with and without axial compression, prestressed concrete beams or beams with mixed conventional and prestressed reinforcement.

For beams subjected to point loads eq. 7-9 should not be used but instead the principles implied by 7.3.3.3 (a) should be followed.

For composite beams, when calculating the moment M_O, allowance should be made for the stresses caused by stage construction and differential temperature and for the effects of differential shrinkage and creep.

Equation 7-11 predicts web-shear cracking as the shear causing principal tensile stress of approximately $0.33\sqrt{f_c'}$ at the centroidal axis of the cross-section.

Equation 7-9 is not necessarily appropriate to continuous prestressed concrete members, such as a bridge superstructure. If it is used, the values of shear and moment should be those when the live load is also on the span being considered. A recommended approach for such members, is that proposed by the CEB[7.48]. It is explained by Thürlimann and others [7.14, 7.15, 7.16]. Equation 7-9 may be non-conservative for thin webbed sections, which are common in bridge super-structures, and for these the CEB[7.48] approach is recommended. Where appropriate the effect of differential temperature across the section should be included in eq. 7-11.

C7.3.4 *Shear reinforcement – Minimum requirements.* Shear reinforcement restrains the growth of inclined cracking, and hence increases ductility and provides a warning of failure. Otherwise, in an unreinforced web, the sudden formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a member is subjected to an unexpected tensile force or catastrophic loading. Accordingly, a minimum area of shear reinforcement not less than that given by eq. 7-12 or 7-13 is required wherever the total shear stress v_i is greater than one-half the ideal shear stress provided by concrete v_c. Three types of members are excluded from the minimum shear reinforcement requirement: slabs and footings; floor joists; and wide, shallow beams. Slabs, footings and joists are excluded because there is a possibility of load sharing between weak and strong areas.

In accordance with 7.3.4.2, other members may be excluded if it is shown by appropriate tests that the required strength can be developed when shear reinforcement is omitted.

When repetitive loading might occur on flexural members the possibility of inclined diagonal tension cracks forming at appreciably smaller stresses than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement expressed by eq. 7-12 or 7-13, even though tests or calcualtions based on static loads show that shear reinforcement is not required.

Equation 7-12 may also be applied to prestressed concrete members, but it will generally require greater minimum shear reinforcement in typical building members than eq. 7-13. However, eq. 7-13 may only be used for prestressed members meeting the minimum prestress force requirements of 7.3.4.4.

C7.3.5 *Shear reinforcement details*

C7.3.5.3 It is essential that shear (and torsion) reinforcement be adequately anchored at both ends, to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by 5.3.4 and 5.3.31.

C7.3.6 *Design of shear reinforcement*

C7.3.6.1 Limiting the design yield strength of shear reinforcement to 415 MPa provides a control on diagonal crack width. Higher strength reinforcement also may be brittle near sharp bends.

C7.3.6.2 to C7.3.6.8 Design of shear reinforcement is based on a modified form of the truss analogy. The truss analogy assumes that the total shear is carried by shear reinforcement. However, considerable research on both non-prestressed and prestressed members has indicated that shear reinforcement need be designed to carry only the shear exceeding that which causes inclined cracking [7.17 to 7.20].

The equations giving the required area of vertical or inclined web shear reinforcement are in terms of nominal shear stresses rather than in terms of shear forces. The limitations are those of previous ACI Building Codes. The background to the equations are well established and may be found in standard texts [7.5].

C7.3.6.9 Tests have shown that when a beam is supported by a girder of about the same depth, failure at the joint tends to develop on a surface similar to the one shown in fig. C7.4. To prevent this type of failure, this clause requires hanger stirrups capable of transferring the full reaction across the failure surface. These stirrups should be located in the supporting member (girder) within a distance equal to the width of the beam plus $d/2$ on each side of it. In addition, it is good practice for the longitudinal reinforcement in the supported member to pass over that in the supporting member.

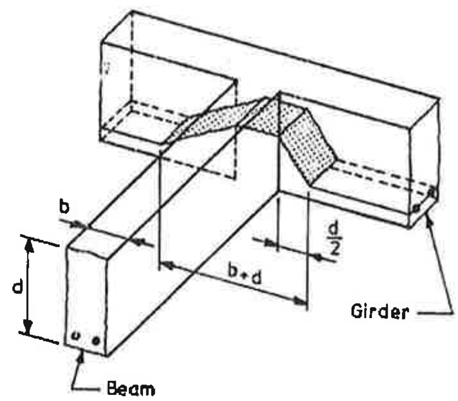


Fig. C7.4 CRACKING AT POINT WHERE A BEAM IS SUPPORTED ON A GIRDER

The requirement for hanger stirrups is waived if the shear stresses at the end of the supported member are low as may be the case for one-way joists. Major inclined cracking in the beam will not occur in that situation, and the shear stresses will be introduced to the girder over the full depth of the beam. Similarly, if the girder is significantly deeper than the beam or if the girder is supported at the joint, the inclined compressive thrust which develops in the beam can be resisted without tearing the bottom of the girder away.

C7.3.7 *Members loaded in torsion*

C7.3.7.2 In the design of reinforced concrete members to resist torsional loads it is necessary to distinguish between two different types of torsion, one arising from equilibrium requirements and the other from the need to satisfy compatibility of deformations.

"Equilibrium torsion" is required to maintain equilibrium in the structure. This is the case in statistically determinate structures. When a member is subjected to "equilibrium torsion" it is necessary to provide adequate reinforcement to ensure that the member is capable of resisting the torsion required by statics. Figure C7.5 shows an example. For the cantilever canopy to be in equilibrium the beam must provide the corresponding torsional as well as flexural and shear strength.

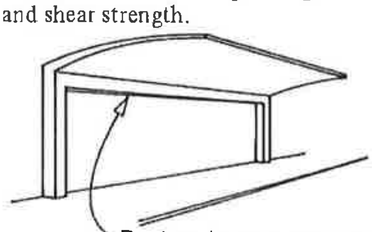


Fig. C7.5 AN EXAMPLE WHERE "EQUILIBRIUM TORSION" IS REQUIRED TO MAINTAIN THE LOAD

"Compatibility torsion" arises when twist is required to maintain compatibility of deformations in the structure. This kind of torsion occurs in statically indeterminate structures if the torsion can be eliminated by releasing relevant restraints. In such situations twist, torsion and torsional stiffness are interrelated. Fig. C7.6 shows an example. The rotation of the ends of the floor beam introduces twist into the spandrels. The resulting bending moment at the ends of the floor beam and the corresponding torsion in the spandrels will depend on the relative values of flexural and torsional stiffness of these members.

Generally the torsional provisions of the Canadian Code for the Design of Concrete Structures for Buildings [7.21] have been adopted rather than those of ACI 318-77. These provisions are similar to those recommended by the European Concrete Committee [7.22]. The torsional strength of prestressed concrete members is considered in references [7.32 and 7.33].

Prior to the formation of diagonal cracks, a reinforced concrete beam in torsion behaves essentially as an elastic beam. The reinforcement at this stage makes no contribution to torsional resistance. The dimensions of the cross-section such as shown in fig. C7.7 (a) and the properties of the concrete alone determine the response [7.5]. It is more convenient, and accurate enough for design purposes, to use the equivalent tube approach [7.5]. This approach replaces the beam's actual cross-section by an equivalent thin walled tube, as shown for example in fig. C7.7 (b). This tube has the same external dimensions as the actual cross-section and has an assumed constant wall thickness of

$$t_c = 0.75 A_{co}/p_c \quad \text{(Eq. 7A)}$$

where p_c is the external perimeter of the actual cross-section and A_{co} is the area enclosed within this perimeter.

Using the well known relationship for the response of a thin-walled tube in torsion, the shear stress, v_t, produced by torsion, T, is given by

$$v_t = \frac{T}{2 A_{oc} t_c} \quad \text{(Eq. 7B)}$$

where A_{oc} is the area enclosed by the mid-wall perimeter of the tube. While A_{oc} can be calculated from the external dimensions and the wall thickness, a reasonble approximation for A_{oc} is $A_{oc} = 0.67 A_{co}$.

When assuming that diagonal cracking will occur when the shear stress reaches $0.33 \sqrt{f'_c}$, the torque required to crack a member will be

$$T_{cr} = 0.44 A_{co} t_c \sqrt{f'_c} \quad \text{(Eq. 7C)}$$

It is implied in 7.3.7.2 that when the required ideal torsional strength $T_i = T_u/\phi$ from eq. 7-17 is less than about one quarter of this cracking torque, the effects of equilibrium torsion may be neglected.

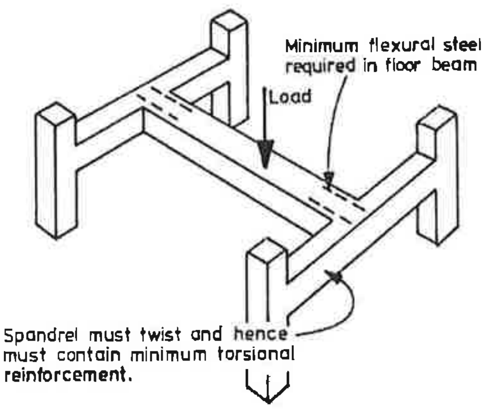
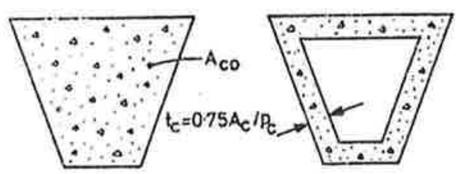


Fig. C7.6 A STRUCTURE IN WHICH TORSION ARISES BECAUSE OF COMPATIBILITY REQUIREMENTS



(a) Actual cross-section **(b) Equivalent tube**

Fig. C7.7 THE EQUIVALENT TUBE CONCEPT FOR UNCRACKED MEMBERS IN TORSION

C7.3.7.3 When compatibility torsion arises it is necessary to evaluate both the flexural and torsional stiffness of the relevant members. The equivalent thin-walled tube may be used to determine the torsional stiffness GK_{gross} of the uncracked member. Tube theory gives:

$$GK_{gross} = G_c \frac{4 A^2_{oc} t_c}{p_{oc}} \quad . \; . \; . \; . \; . \; . \; . \quad \text{(Eq. 7D)}$$

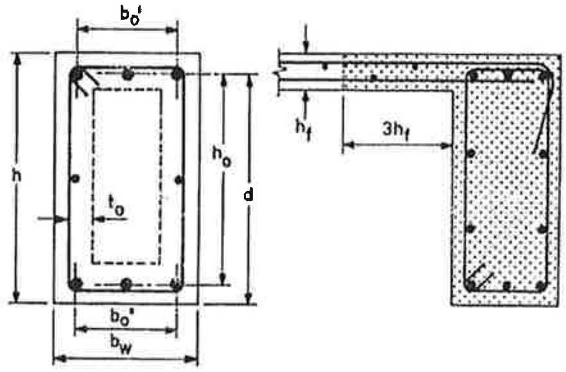
where G_c is the shear modulus of the concrete, which can be taken as 0.4 E_c, and p_{oc} is the perimeter of A_{oc} (p_{oc} can be taken as $p_c - 4\ t_c$). the torsional stiffness of the cracked member is only a small fraction of the torsional stiffness of the uncracked member. The torsional stiffness in the cracked state is primarily determined by the deformations of the reinforcement; its value GK_{cr} can be found from the stiffness of the equivalent thin-walled tube. This is:

$$GK_{cr} = \frac{E_s}{2} \frac{4 A^2{}_o}{p_o} \sqrt{\frac{A_t A_\ell}{s\, p_o}} \quad . \; . \; . \; . \; . \; . \; . \quad \text{(Eq. 7E)}$$

where E_s is the Young's modulus of the reinforcing steel. For typical beams it will be found that the torsional stiffness after cracking as given by eq. 7E, is less than 10% of the torsional stiffness of the uncracked member, as given by eq. 7D.

As torsional reinforcement is only stressed in the cracked state, it is appropriate to design it for the torsional loads the member will experience in the cracked state. For a statically indeterminate structure, these loads could be determined by performing an elastic analysis using stiffness values for the cracked state. In the transition from uncracked to cracked state, the flexural stiffnesses will reduce to about one-half while the torsional stiffnesses will reduce to about one-tenth. Approximately correct stiffness ratios will be obtained if the flexural stiffness values for the uncracked state are used along with the torsional stiffnesses obtained by dividing the torsional stiffnesses for the uncracked member by five [7.23].

Members designed to resist torsions, the magnitudes of which have been determined from the above stiffness values, will certainly behave in a satisfactory manner. The analysis, however, may involve considerable work. If the torsion on the member arises only because the member must twist to maintain compatibility, the magnitude of the



(a) (b)

Fig. C7.8 EFFECTIVE SECTIONS FOR TORSIONAL RESISTANCE

torsion will be almost directly proportional to the torsional stiffness. This is demonstrated in fig. C7.6 where the torsion in the spandrel is caused by the need for the spandrel to rotate with the end of the floor beam. Thus, decreasing the amount of torsional steel will decrease the stiffness and as a result the applied torque will be reduced.

In such cases (that is, where torsion is not needed to maintain equilibrium) 7.3.7.3 allows the designer to provide a minimum amount of properly detailed torsional steel and then assume that the torsional stiffness of the member, and hence the torsion in the member, is zero. In reality, there will be some torsion in the member, and the presence of this torsion must be allowed for when detailing adjacent members. For example, minimum negative flexural reinforcement should be placed in the floor beam shown in fig. C7.6. An additional effect of the small torsion will be to somewhat reduce the shear required to produce first yielding of the web reinforcement. However, if the member has been properly designed for shear (that is, designed to fail in a ductile manner in flexure rather than suffering a brittle failure in shear), the small torsion should have no significant effect on the failure load of the member.

C7.3.7.4 When diagonal cracks develop in a member as a result of torsion the longitudinal steel will be strained and consequently the member becomes longer while the torsional stiffness reduces rapidly. When members are restrained against longitudinal expansion, they may develop significant torsional stiffness. This should be considered when the flexural reinforcement in members, restrained against flexural rotation by the torsion member, is being considered.

C7.3.7.5 By similarity to the requirements of 7.3.1.1 the factored torsion T_u is related to the ideal torsional strength required, which is then used in all subsequent equations.

C7.3.7.6 After diagonal cracks form, the torsional resisting mechanism of the member changes completely. The torsional shear stresses are now provided solely by the diagonal compressive stresses in the diagonally cracked concrete. This diagonally compressed concrete is held in equilibrium by tensile stresses in the longitudinal and the transverse steel. Because the transverse steel cannot "hold" the concrete cover in equilibrium without generating high concrete tensile stresses, particularly at the corners of a section, at higher loads this concrete cover will spall off. Hence in the cracked state it is the dimensions of the reinforcing cage which governs the behaviour and not the exterior dimensions of the concrete.

In the cracked state the beam is again idealized as a thin-walled tube, but this time the mid-wall perimeter, p_o, is assumed to pass through the centres of the longitudinal bars in the corners of the closed stirrups, and the concrete wall thickness, t_o, is assumed to be:

$$t_o = 0.75\, A_o / p_o \quad . \; . \; . \; . \; . \; . \; . \quad \text{(Eq. 7F)}$$

where A_o is the area enclosed by p_o.

Figure C7.8 (a) shows an example section in which $A_o = b_o\ h_o$ where b_o may be taken as 0.5 $(b'_o + b''_o)$. Also $p_o = 2\ (h_o + b_o)$. The assumed thickness of the tube t_o is also indicated.

To check against crushing of the concrete due to the diagonal compression, the ideal torsional shear stress is calcualted from:

$$v_{ti} = \frac{T_i}{2 A_o t_o} \quad \text{. . . . . . .} \quad \text{(Eq. 7.18)}$$

This torsional shear stress is limited to $0.2\, f_c'$ or 6 MPa whichever is less. The value of $0.2\, f_c'$ is a conservative approximation to the values predicted by the compression field theory [7.24].

C7.3.7.7 In determining the effective section for flanged beams, not more than three times the thickness of a flange, as shown in fig. C7.8 should be considered when computing A_o or A_{co}. The designer may neglect the contribution of flanges to torsional strength and stiffness.

C7.3.7.8 To limit the maximum shear stress due to combined torsion and shear, a linear interaction between these stresses, with the maximum set at $0.2\, f_c'$ or 6 MPa whichever is less, has been assumed, and this results in eq. 7-19.

C7.3.8 *Torsional reinforcement – minimum requirements.* For properly designed beams the torsional strength will be governed by yielding of the reinforcement rather than by crushing of the concrete. When checking the capacity of the reinforcement, assuming that the yield strength of the transverse and longitudinal bars is the same, the thin-walled tube concept can again be employed but now the wall thickness of the equivalent tube should be taken as:

$$t_s = \sqrt{\frac{A_t}{s}\frac{A_\ell}{p_o}} \quad \text{. . . . .} \quad \text{(Eq. 7G)}$$

where A_t is the cross-sectional area of one leg of the closed stirrup, s is the spacing of the closed stirrups, and A_ℓ is the area of longitudinal steel assumed to be symmetrically distributed around the cross-section, having the same centroid as p_o.

The ideal torsional strength can thus be taken as [7.5]:

$$T_i = 2 A_o t_s f_y = 2 A_o f_y \sqrt{\frac{A_t}{s}\frac{A_\ell}{p_o}} \quad \text{. . . . . .} \quad \text{(Eq. 7H)}$$

If a member contains too little torsional reinforcement it will fail in a brittle manner upon the formation of the first torsional crack. If more than one torsional crack is to form, the post-cracking torsional strength of the member must be equal to or greater than the cracking torque. Equating the torque from eq. 7H with the cracking torque from eq. 7C gives the following expression for minimum torsional reinforcement:

$$2 A_o f_y \sqrt{\frac{A_t}{s}\frac{A_\ell}{p_o}} \geqslant 0.44\, A_{co}\, t_c \sqrt{f_c'} \quad \text{. . . . . .} \quad \text{(Eq. 7J)}$$

Clause 7.3.8.1 reduces this equation to:

$$\sqrt{\frac{A_t}{s}\frac{A_\ell}{p_o}} \geqslant \frac{1.5}{f_y}\frac{A_{co}\, t_c}{A_o} \quad \text{. . . . . . .} \quad \text{(Eq. 7-20)}$$

C7.3.9 *Torsion reinforcement details.* To ensure the effective response of the torsional steel this Clause requires that the closed stirrups be closely spaced and that reasonably sized longitudinal bars be placed in each corner of the closed stirrups [7.25]. Because of the possibility of the cover concrete spalling, for reasons commented on in C7.3.7.6, closed stirrups for torsion are required to be anchored with 135° hooks.

C7.3.10 *Design of torsion reinforcement*

C7.3.10.1 In this approach to torsion design it is assumed that no interaction exists between torsion and flexure or shear, with or without axial load on the member. Consequently the transverse and longitudinal tension reinforcement is simply additive to that required by other actions. This greatly simplifies design.

C7.3.10.2 The provisions can also be applied to prestressed concrete members provided that minimum shear reinforcement is present. This is achieved when the value of v_c, specified in 7.3.3, is limited to $0.17\sqrt{f_c'}$. This restriction is waived when special studies for torsion in prestressed concrete members is made.

C7.3.10.3 and C7.3.10.4 For design it is convenient to choose the ratio of longitudinal reinforcement to transverse reinforcement in eq. 7G so that the term A_t/s equals the term $A\ell/p_o$. The design equations for the required amount of torsional reinforcement when combined with eq. 7-18 then become:

$$A_t = \frac{T_i s}{2 A_o f_y} = \frac{v_{ti}\, t_o\, s}{f_y} \quad \text{. . . . . . .} \quad \text{(Eq. 7-21)}$$

$$A_\ell = \frac{T_i p_o}{2 A_o f_y} = \frac{v_{ti}\, t_o\, p_o}{f_y} \quad \text{. . . . . . .} \quad \text{(Eq. 7-22)}$$

The second alternative of these expressions, as given in the code, is consistent with the equations used for shear and is also convenient to use as the torsional shear stress v_{ti} needs to be determined from eq. 7-18.

The torsional web steel determined from eq. 7-21 is to be added to the web steel required to resist the shear force acting in combination with the torsion, while the torsional longitudinal steel, from eq. 7-22, is to be added to the longitudinal steel required to resist the flexure and axial force acting in combination with the torsion. In some cases the loading patterns which produce the maximum torsion will be quite different from those which produce the maximum flexure and shear. In these cases the reinforcement provided for the maximum flexure and shear may alone prove to be adequate to resist the maximum torsion in combination with the smaller flexure and shear.

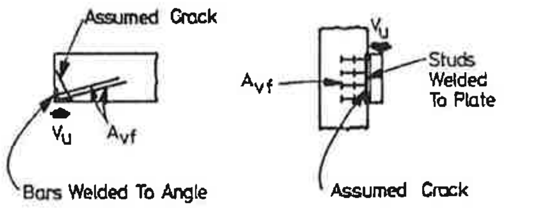
C7.3.11 *Shear-friction*

C7.3.11.1 With the exception of 7.3.11 virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of the provisions of 7.3.11 is to provide a design method [7.26, 7.27, 7.28, 7.29], for conditions where shear transfer must be considered, such as in design of reinforcing details for precast concrete structures. An experimental study of the shear-friction concept is reported in reference [7.29].

C7.3.11.2 Uncracked concrete is relatively strong in direct shear; however, there is always the possibility that a crack will form in an unfavourable location. The design procedure for the shear-friction concept is to assume that a crack will form and then to provide reinforcement that will prevent this crack from causing undesirable consequences.

When shear acts along a crack, slip of one crack face occurs with respect to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimtate, this separation is sufficient to stress the reinforcement crossing the crack to its yield point. This provides a clamping force $A_{vf}f_y$ across the crack faces. The applied shear is resisted by friction between the crack faces, resistance to the shearing off of protrusions on the crack faces and the dowel action of the reinforcement crossing the crack. In the shear-friction method of calculation, it is assumed that all the shear resistance is due to friction between the crack faces. It is, therefore, necessary to use artificial values of the coefficient of friction μ in the shear-friction equation, in order that the calculated shear strength will be in reasonably close agreement with test results.

Successful application of 7.3.11 depends on proper selection of the location of an assumed crack. Some examples are illustrated in fig. C7.9.



(a) Precast beam bearing **(b) Column face plate**

Fig. C7.9 APPLICATION OF SHEAR-FRICTION

Fig. C7.9 (a) is an end-bearing detail for a precast beam. Stirrups or ties may be needed to enclose the shear-friction reinforcement and prevent a secondary failure plane from forming around the shear-friction reinforcement.

Fig. C7.9 (b) illustrates a column face plate. The headed studs function as shear-friction reinforcement, and should be firmly anchored into the confined core of the column.

C7.3.11.5 The provisions for the value of the coefficient of friction μ differ slightly from those of ACI 318-77. It is recognized that if care is exercised to produce the high quality construction joints, for example in earthquake resisting shear walls, shear resistance comparable to that of monolithic concrete can be achieved. An intermediate value of $\mu = 1$ is to be used when intermediate roughness corresponding with a full amplitude of protrusions between 2 and 5 mm is attained. The use of $\mu = 1.4$ necessitates strict site supervision to ensure that the specified minimum surface roughness is attained along a construction joint.

C7.3.11.7 If tensile stresses are present across the assumed crack, reinforcement for the tension must be provided in addition to that provided for shear-friction. Tests [7.30] have shown that the total amount of reinforcement to carry a shear V_u and tension N_u across a crack may be obtained simply by adding together the area of reinforcement required to resist the tensile forces. Unforeseen tension has caused failures particularly in beam bearings. Direct tension across assumed cracks may be induced by temperature, shrinkage, creep, growth in camber due to prestress and creep, and so on.

C7.3.11.8 Since the shear-friction reinforcement acts in tension, it must have full tensile anchorage on both sides of the potential crack. Further, the shear-friction steel anchorage must engage primary steel; otherwise, a potential crack may pass between the shear-friction steel and the body of the concrete. This requirement applies particularly to welded headed studs used with steel inserts for connections in precast concrete. Anchorage may be developed by bond, by a welded mechanical anchorage, or by threaded dowels and screw inserts. Space limitations often require a welded mechanical anchorage.

Vertical construction joints across beams or horizontal construction joints across walls or columns are localities where the formation of cracks should be assumed. All reinforcement crossing such a potential sliding plane which contributes to the flexural strength of the effective section may be included in evaluating A_{vf}. In walls this will include all the vertical reinforcement in the web as well as additional vertical bars that may be placed in the end regions or in effective flanges.

C7.3.12 *Special provisions for deep beams.* A deep beam, with respect to shear, is one in which a significant portion of the shear force is transferred directly from the loads to the support by compressive strut action. Deep beam action can occur if the shear span is less than that defined in this Clause. For such action, the contribution of the concrete to shear strength, as given by eq. 7-4, is increased. In general this condition only exists if the beam is loaded on its top face and supported on its bottom face. If the beam is loaded and supported on its bottom face, a steep compressive thrust line cannot develop and the beam as essentially the same shear strength as a slender beam.

This Clause has been changed with respect to ACI 318-77 provisions in order to extend its range of applicability and to bring it into harmony with 7.3.2 and 7.3.11[7.2] It is intended to apply to members loaded at the top or compressive face and with a ratio of span to depth of less than four. If the loads are applied through the sides or bottom of a member, design for shear should be the same as for ordinary members unless vertical hanger reinforcement with a dependable capacity equal to the factored loads is provided to transfer these to the top face as required by 7.3.6.9.

The longitudinal tensile reinforcement in deep beams should be extended to the support, and adequately anchored by embedment, hooks, or welding to special devices. Special care should be taken in detailing the longitudinal tensile reinforcement since the stresses in that reinforcement after cracking may differ markedly from those calculated using the assumptions of 6.3.1. Detailing recommendations for deep beams are given in references [7.5 and 7.31]. Bearings at load points and supports must satisfy the requirements of 6.3.5.

C7.3.12.1 This Clause is provided to cover two types of deep beams:

(a) Deep beams in which a "principal load" (see 7.3.12.2) acts at a distance 2 d or less from the support. A principal load is one that causes 50% or more of the shear at the support of that span. In such a shear span the thrust line is assumed to be a straight line joining the load and reaction.

(b) Any deep beam without a principal load is assumed to act as a uniformly loaded beam since the major portion of the shear in such a shear span will be due to dead loads or possibly a series of concentrated loads approaching a uniformly loaded condition. The length of such a shear span is defined in terms of the distance ℓ_s from the support to the point of zero shear. This leads to a conservative estimate of v_c.

Reference is made to shear spans. A beam may have both types of shear spans or a beam loaded close to one end may have one deep shear span and one slender shear span.

C7.3.12.5 The basic design approach to the shear strength of deep beams rests on the principle that shear forces can be resisted by the concrete and by vertical and horizontal shear reinforcement. The total shear stress v_i must not exceed $0.2 f_c'$ or 6 MPa.

C7.3.12.6 This Clause makes provisions for the design for shear of deep beams supporting principal loads only.

As the span-depth ratio of a member without web reinforcement decreases, its shear strength increases above the shear causing diagonal tensile cracking. Thus, in eq. 7-24 it is assumed that diagonal cracking occurs at the same nominal shear stress as for ordinary beams, but that the shear stress carried by the concrete will be greater than the shear stress causing diagonal cracking.

Designers should note that shear stresses in excess of the shear stress causing diagonal cracking may result in cracking of unsightly width unless shear reinforcement is provided. Face steel must be provided to control that possibility in very deep beams. Horizontal and vertical web reinforcement will serve this purpose if properly spaced in accordance with 7.3.12.8 and 7.3.12.9.

In addition to carrying shear by direct tension, vertical stirrups in a slender beam restrict the growth of diagonal cracks and thereby insure shear transfer across those cracks by aggregate interlock. Beam action is maintained and arch action and splitting along the flexural steel diminished. As the slenderness ratio decreases, the roles of arch and shearing actions in transferring loads to the supports increase. Proportionately greater amounts of web reinforcement are required in deep shear spans when arch action diminishes and then shearing rather than bending effects dominate. The role of web reinforcement changes from that of carrying significant shears by tension to that of acting primarily as shear-friction reinforcement preventing sliding. As the slenderness ratio decreases, the effectiveness of vertical stirrups diminishes and that of horizontal stirrups increases. If the loads do not act on the top of the beam, arch action is diminished and the need for shear reinforcement is increased.

The relative amounts of horizontal and vertical shear reinforcement that are selected when using eq. 7-25 may vary as long as the limits on the minimum amount and spacing are observed. The design procedures detailed in these Sections are based primarily on shear transfer concepts[7.34].

The quantity A_{vh} is the horizontal reinforcement provided to resist shear and does not include the main longitudinal tensile reinforcement.

Special attention is directed to the importance of adequate anchorage for the shear reinforcement. Horizontal web reinforcement should be extended to the support and anchored in the same manner as the tensile reinforcement. Details are given in reference [7.5].

C7.3.12.7 The principles involved in the design for shear of deep beams not supporting principal load or deep beams loaded with distributed loads in the shear span are the same as outlined in C7.3.12.6.

C7.3.13 *Special provisions for brackets and corbels*

C7.3.13.1 Provisions for the design of brackets and corbels were introduced in the ACI 318-77 code based on tests by Kriz and Raths [7.35] and recommendations by ACI-ASCE Committee 426 [7.1]. However, the limitations contained in Section 11.9 of ACI 318-77 are considered to reflect an arbitrary choice of parameters in the test programme for which satisfactory behaviour was obtained. The shear-friction provisions of Section 11.9 of ACI 318-77 were allowed to be used for the design of corbels providing a/d was less than 0.5. Subsequent testing by Mattock *et al* [7.36, 7.37] indicate that this limit on a/d is unnecessary, provided the corbel is designed for shear according to the shear-friction provisions of 7.3.11 and for flexure using the assumptions of 6.3.1.

C7.3.13.3 Tests [7.36, 7.37] have shown that subject to the provisions of minimum horizontal stirrup reinforcement according to 7.3.13.4 the "useful ultimate strength" of corbels subjected to a combination of vertical and horizontal loads can be calculated with satisfactory accuracy by taking it to be the lesser of:

(a) The shear strength of the corbel-column interface, calculated using the shear-friction provisions of 7.3.11; referred to in 7.3.13.2.

(b) The vertical load corresponding to the development of the flexural ultimate strength of the corbel-column interface, taking into account the additional moment N_u $(h-d)$ about the centroid of the main tension reinforcement, imposed on the corbel by the horizontal tension force N_u, and using the provisions of 6.3.1.

Experience from testing [7.30] indicates that the simultaneous action of a moment less than or equal to the flexural ultimate strength of the cracked section will not reduce the shear which can be transferred across the crack. Thus the shear transfer reinforcement should be concentrated in the flexural tension zone for maximum benefit, in accordance with 7.3.13.3 (d) and as shown in fig. C7.10. Also the total amount of reinforcement needed to carry a factored shear V_u and a factored tension N_u across a crack may be obtained by simply adding together the area of re-

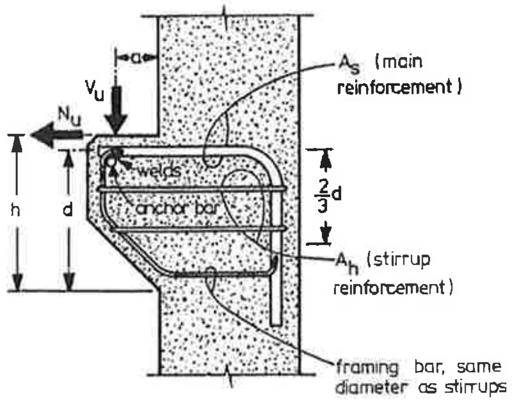


Fig. C7.10 TYPICAL CORBEL REINFORCEMENT

inforcement required to resist the shear, A_{vf}, and the area of reinforcement required to resist the tension force, A_{ct}. Because corbels are relatively small members, details of bond anchorage and bearings are very important. The rules of 7.3.13.3 (f) are based on test experience [7.35]. Also, it is recommended that the outer edge of the bearing area should not be closer than 50 mm to the outer edge of the corbel and not beyond the straight portion of the main tension reinforcement. Various failure modes and detailing requirements are examined in reference [7.5].

C7.3.13.4 The requirements of this Clause are made to avoid premature diagonal tension failure of the corbel. It has been shown [7.36, 7.37] that such a failure will not occur if closed stirrups or ties parallel to the main tension reinforcement are provided having a total yield strength equal to one-half the yield strength of the reinforcement required to resist the factored moment M_u or one-third the yield strength of the reinforcement required to resist the factored shear V_u, whichever is the greater.

C7.3.14 *Special provisions for walls*

C7.3.14.1 Shear in the plane of the wall, as a design consideration, is primarily of importance for shear walls with a small height to length ratio. The design of higher walls, particularly walls with uniformly distributed reinforcement, will probably be controlled by flexural considerations. It is, therefore, essential that the flexural strength of shear walls be computed, along with their shear strength.

C7.3.14.3 Although the width-to-depth ratio of shear walls is less than that for ordinary beams, tests [7.38, 7.39] on shear walls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses up to $\sqrt{f'_c}$ can be obtained. In conformity with the requirement of 7.3.1.8 the maximum shear stress is limited. The thinnest section of the wall, which may occur where horizontal recesses reduce the thickness, must be considered in computing the shear stress.

C7.3.14.5 and C7.3.14.6. Equations 7·32 and 7·33 predict the inclined cracking strength at any section through a shearwall. Equation 7·32 corresponds to the occurrence of a principal tensile stress of approximately $0.33\sqrt{f'_c}$ at the centroid of the shearwall cross-section. Equation 7·33 corresponds approximately to the occurrence of a flexural tensile stress of $0.5\sqrt{f'_c}$ at a section $\ell_w/2$ above the section being investigated. As the term

$$\left(\frac{M_u}{V_u} - \frac{\ell_w}{2}\right)$$

decreases, eq. 7·32 will control before this term becomes negative. Further, the value of v_c obtained when a negative value of

$$\left(\frac{M_u}{V_u} - \frac{\ell_w}{2}\right)$$

is substituted in eq. 7·33 has no physical significance. In this case, eq. 7·32 should be used.

C7.3.14.7 The values of v_c computed from equations 7·32 and 7·33 at a section located a distance $\ell_w/2$ or $h_w/2$ (whichever is less) above the base apply to that and all sections between this section and the base. However, the total shear stress v_i at any section, including the base of the wall, is limited in accordance with 7.3.14.3.

C7.3.14.9 In the design for shear strength of walls, sufficient horizontal shear reinforcement is required to carry the shear exceeding v_c. The minimum horizontal and vertical reinforcement raio should not be less than $0.7/f_y$. This is 0.25% when Grade 275 is used and it is more than that required in beams in accordance with eq. 7·12.

C7.3.15 *Special provisions for slabs and footings*

C7.3.15.1 Differentiation must be made between a long and a narrow slab or footing acting as a beam, and a slab or footing subject to two-way action where failure may occur by 'punching' along a truncated cone or pyramid around a concentrated load or reaction area.

C7.3.15.2 Research studied by ACI-ASCE Committee 426 indicated that the critical section for two directions follows the perimeter at the edge of the loaded area. The ultimate shear stress acting on this section is a function of $\sqrt{f'_c}$ and the ratio of the side dimension of a square column to the effective slab depth. Committee 426 recommended, however, that the effect of this variable be taken into account by assuming a pseudocritical section located at a distance $d/2$ from the periphery of the concentrated load. The ultimate shear stress is then independent of the ratio of column size to slab depth. This method was first adopted for the 1963 ACI Building Code because of its simplicity, especially for irregularly shaped column sections and when slab openings are present near a column.

Previously the permissible shear stress in slabs subjected to bending in two directions was $0.33\sqrt{f'_c}$. Recent tests of punching shear around columns or loads [7.9], as well as obserations in buildings that have suffered distress, indicate that the value of $0.33\sqrt{f'_c}$ is unconservative when the ratio β_c of the lengths of the short and long sides of a rectangular column or loaded area is smaller than 0.5. In such cases, the actual shear stress on the critical section at punching shear failure varies from a maximum of about $0.33\sqrt{f'_c}$ around the ends of the column or loaded area, down to $0.17\sqrt{f'_c}$ or less along the long sides between the two end sections.

The exact shear stress distribution is complex. Equations 7-36 and 7-37 were therefore developed to facilitate calculation of the punching shear strength and to reflect the reduction in shear strength which occurs as the aspect ratio of a column section or loaded area β_c decreases below 0.5.

For shapes other than rectangular, the shear stress on the critical section defined in 7.3.15.1 (b) should be limited to the value given by eq. 7-37, in which case β_c is taken to be the ratio of the shortest overall dimension of the effective loaded area to the longest overall dimension of the effective loaded area measured perpendicular thereto, as illustrated for an L-shaped reaction area in fig. C7.11. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

C7.3.15.3 Research has shown that shear reinforcement consisting of bars or wires can work well in slabs provided that such reinforcement is anchored as described in 5.4.3.2.

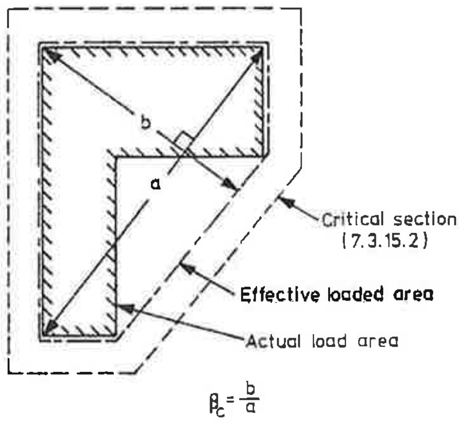


Fig. C7.11 VALUE OF β_c FOR A NON-RECTANGULAR LOADED AREA

The importance of anchorage details for slab shear reinforcement cannot be over-emphasized. Some forms of slab shear reinforcement formerly used, such as those consisting of concentric circles of V-shaped wires may not meet anchorage requirements. Extreme care should be taken to ensure that shear reinforcement is accurately placed, especially in thin slabs.

When bars or wires are provided as shear reinforcement, the shear strength may be increased to a maximum shear stress of $0.5\sqrt{f_c'}$. However, shear reinforcement must be designed to carry all shear in excess of a stress of $0.17\sqrt{f_c'}$. The limit of $0.17\sqrt{f_c'}$ is one-half that permitted by eq. 7-37 for a 1:2 rectangular column support ($\beta_c = 0.5$) when shear reinforcement is not provided.

C7.3.15.4 Based on reported test data [7.40], design procedures are presented for shearhead reinforcement consisting of structural steel shapes in slabs at interior columns. Tests indicate that, due to torsional effects, and other peculiarities, the behaviour of shearheads at a slab edge differs substantially from that at other locations.

There are three basic criteria which must be considered for the design of shearhead reinforcement for connections transferring shear only. First, a minimum flexural strength must be provided to ensure that the required shear strength

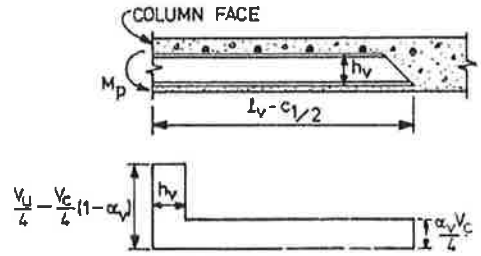


Fig. C7.12 IDEALIZED SHEAR ACTING ON SHEARHEAD

of the slab is reached before the flexural strength of the shearhead is exceeded. Second, the shear stress in the slab at the end of the shearhead reinforcement must be limited. Third, after these two requirements are satisfied, the designer can reduce the negative slab reinforcement in proportion to the moment contribution of the shearhead at the design section. For an interior column connection transferring moment in addition to shear, design considerations are given in Section 9.

The assumed idealized shear distribution along an arm of a shearhead at an interior column is shown in fig. C7.12. The shear along each of the four arms is taken as $\alpha_v v_c b_o d/4$, where α_v is the ratio of the EI value of the shearhead to the EI value of a composite section made up of a portion of the cracked slab, with a width equal to that of the column plus the effective depth of the slab in which the shearhead is embedded, and v_c is the diagonal cracking shear stress for the same portion of the slab. However, the peak shear at the face of the column is taken as the total shear applied per arm, $V_u/4$, minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed in terms of shear stress as

$$(v_c/4)(1-\alpha_v),$$

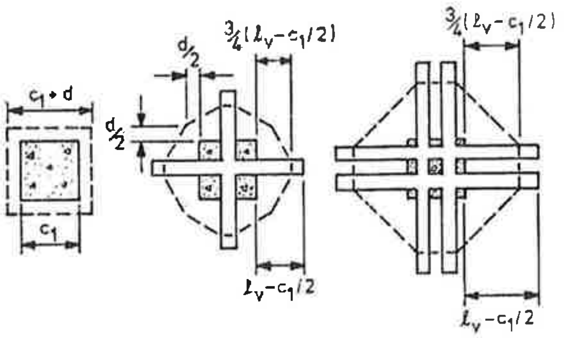
so that it approaches zero for a heavy shearhead and approaches $v_i/4$ where a light shearhead is used. Equation 7-38 then follows for the assumption that the inclined cracking shear stress v_c is about one-half the shear stress v_i. In this equation, ϕ is the strength reduction factor for flexure (0.9) and M_p is the required plastic moment strength of each shearhead arm necessary to ensure that ultimate shear is attained as the moment strength of the shearhead is reached. The quantity ℓ_v is the length from the centre of the column to the point at which the shearhead is no longer required, and the distance $c_1/2$, is one-half the dimension of the column in the direction considered.

The test results indicated that slabs containing "under-reinforcing" shearheads failed at a shear stress on a critical section at the end of the shearhead reinforcement less than $0.33\sqrt{f_c'}$. Although the use of "over-reinforcing" shearheads brought the shear strength back to about the equivalent of $0.33\sqrt{f_c'}$, the limited test data suggest that a conservative deisgn is desirable. Therefore the shear strength is calculated as $0.33\sqrt{f_c'}$ on an assumed critical section located inside the end of the shear head reinforcement.

The design critical section is shown in fig. C7.13. The critical section is taken through the shearhead arms three-fourths of the distance $[\ell_v - (c_1/2)]$ from the face of the column to the end of the shearhead. However, this assumed critical section need not be taken closer than $d/2$ to the column.

For a practical case where the shearhead reinforcement extends beyond the column face a distance equal to the column width, the shear stress on the section at the end of the shearhead becomes $0.27\sqrt{f'_c}$. For a very long shearhead, the minimum shear stress at its end approaches the value of $0.25\sqrt{f'_c}$.

If the peak shear at the face of the column is neglected, and the cracking stress v_c is again assumed to be about one-half of v_i, the moment contribution of the shearhead M_v can be conservatively computed from eq. 7-39, in which ϕ is the factor for flexure (0.9).



(a) No shearhead (b) Small shearhead (c) Large shearhead

Fig. C7.13 LOCATION OF CRITICAL SECTION DEFINED IN 7.3.15.4 (g)

C7.3.15.5 Provisions for design of openings in slabs (and footings) were developed in an early ACI-ASCE Committee 426 report [7.41]. The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in fig. C7.14. Additional research reported by ACI-ASCE Committee 426 [7.9] has confirmed that these provisions are conservative. More recent recommendations of ACI-ASCE Committee 426 are being considered for possible incorporation into the ACI Building Code [7.2].

C7.3.16 *Transfer of moments to columns*

C7.3.16.1 Shear forces induced in joints from adjacent beams or columns must be considered. When only gravity loads are acting, these forces will be generally small and the concessions of 7.4.1 may be used. When earthquake induced forces are to be transferred the special provisions of Section 9 are applicable.

C7.3.16.2 In this clause special provisions for slabs are considered. In reference [7.45] it was found that where moment is transferred between a column and a slab, 60% of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 7.3.15.1 (b), and 40% by eccentricity of shear about the centroid of the critical section. Most of the data in reference [7.45] were obtained from tests of square columns, and little other information is available. Fig. C11.4 shows square supports having the same area as some non-rectangular members. For rectangular columns, it is reasonable to assume that the portion of the moment transferred by shear decreases as the width of the face of the critical section resisting the moment increases. Accordingly, this fraction is taken equal to

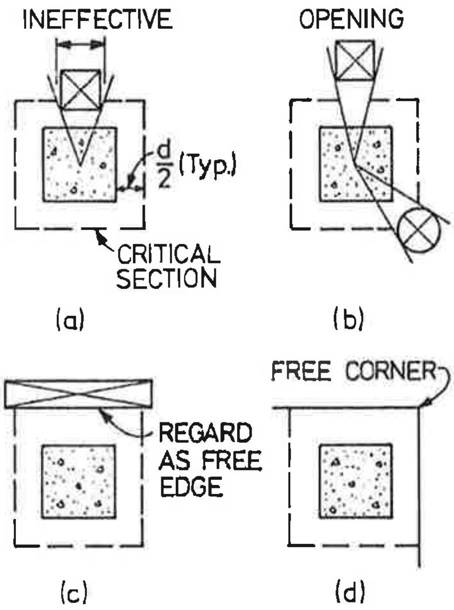
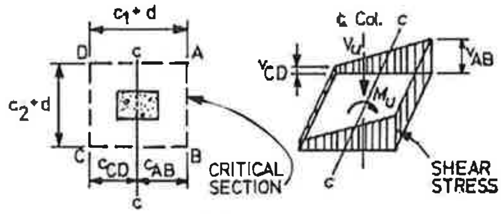
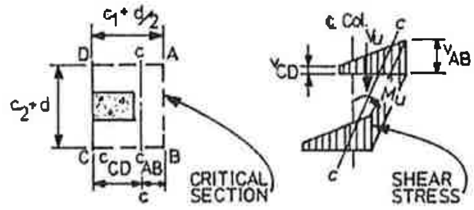


Fig. C7.14 EFFECT OF OPENINGS AND FREE EDGES (EFFECTIVE PERIMETER SHOWN WITH DASHED LINES)



(a) Interior column



(b) Edge column

Fig. C7.15 ASSUMED DISTRIBUTION OF SHEAR STRESS

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{c_1 + d}{c_2 + d}}}$$

where $(c_2 + d)$ is the width of the face of the critical section resisting the moment, and $(c_1 + d)$ is the width of the face at right angles to $(c_2 + d)$. The remainder is taken by flexure in accordance with 11.3.5.

Since the shear stresses shall be taken as varying linearly about the centroid of the critical section, the stress distribution is assumed as illustrated in fig. C7.15 for an interior column. The perimeter of the critical section, *ABCD*, is determined in accordance with 7.3.15.1 (b). The factored shear force V_u and unbalanced moment M_u are determined at the centroidal axis $c - c$ of the critical section. The maximum total shear stress may be calculated from:

$$v_{i(AB)} = \frac{V_u}{\phi A_c} + \frac{\gamma_v M_u c_{AB}}{\phi J_c}$$

$$v_{i(CD)} = \frac{V_u}{\phi A_c} - \frac{\gamma_v M_u c_{CD}}{\phi J_c}$$

where

γ_v = fraction of moment between slab and column that is considered transferred by eccentricity of the shear about the centroid of the assumed critical section and for an interior column, A_c and J_c may be calculated by

A_c = area of concrete of assumed critical section

$= 2d\,(c_1 + c_2 + 2d)$

J_c = property of assumed critical section analogous to polar moment of intertia

$$= \frac{d\,(c_1 + d)^3}{6} + \frac{(c_1 + d)d^3}{6} + \frac{d(c_2 + d)\,(c_1 + d)^2}{2}$$

Similar equations may be developed for A_c and J_c for columns located at the edge or corner of a slab.

The fraction of the unbalanced moment between slab and column not transferred by eccentricity of the shear must be transferred by flexure in accordance with 11.3.5. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 11.3.5. Often designers concentrate column strip reinforcement near the column to accommodate this unbalanced moment. However, available test data seem to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab column junction.

An alternative approach for determining the moment which can be transferred by the connection is to use a beam analogy which is described in more detail for connections with shear reinforcement in C7.3.16.2 (e).

C7.3.16.2 (e) Shear reinforcement in the form of bent up bars, vertical stirrups or structural steel shearheads can be used to increase the moment which can be transferred by the connection [7.46]. A beam analogy for the slab may be used to determine the strength of the connection. In the beam analogy the slab adjacent to the column is assumed to act as beams running in two directions at right angles framing into the column faces. Each beam is assumed to be capable of developing its ultimate bending moment, torsional moment and shear force at the critical sections near the column faces, making due allowance for interaction effects and including the contributions from the shear reinforcement. The strength of the connection is obtained by summing the contributions of the strengths of the beams, calculated using the equations of this code for beams. Note that the maximum shear stress carried by the concrete in the presence of shear reinforcement is $0.17\sqrt{f'_c}$ MPa, rather than the value of $0.33\sqrt{f'_c}$ MPa assumed when shear reinforcement is not present.

C7.4 Principles and requirements additional to 7.3 for members not designed for seismic loading

C7.4.1 Tests have shown that the joint region of a beam to column connection in the interior of a building does not require shear reinforcement if the joint is confined on four sides by beams of approximately equal depth. However, joints without lateral confinement, such as at the exterior of a building, need shear reinforcement to prevent deterioration due to shear cracking [7.47].

C7.5 Principles and requirements additional to 7.3 for members designed for seismic loading

C7.5.1 *Shear strength*

C7.5.1.1 This Clause is intended to deal with beams. Typically two plastic hinges may form in a beam, such as at *A* and *B* in the span shown in fig. C6.6. With the corresponding flexural overstrengths, in accordance with the definitions, denoted as M_{OA} and M_{OB}, the design shear force at *B* will be

$$V_{iB} = \frac{M'_{OA} + M_{OB}}{\ell_{AB}} + V_{DB} + V_{LRB}$$

Similarly the critical shear for the same beam at *A* will be

$$V_{iA} = \frac{M_{OA} + M'_B}{\ell_{AB}} + V_{DA} + V_{LRA}$$

$$= \frac{M_{OA} + M_{OC}}{\ell_{AC}} + V_{DA} + V_{LRA}$$

The value of M'_B must be evaluated from the flexural overstrength in the vicinity of *C*. It will be noted that the shear at *C* for this load combination is zero. The intent is to prevent a shear failure under maximum possible lateral forces. Accordingly the ideal shear strength must be equal to or larger than the shear obtained above. It should be noted that, in accordance with 4.3.2, the strength reduction factor is not used, that is, $\phi = 1.0$, when, as is the case above, earthquake induced shear forces are derived from a capacity design procedure.

C7.5.1.2 This Clause gives guidance in general terms for the determination of shear forces across columns subjected to bending with compression or with tension. With few exceptions the shear results from the applied end moments due to lateral load at the top and the bottom of the column. The determination of these moments is uncertain because of the dynamic and random nature of the loading. The intent of the capacity design, when applied to columns, is to reduce the likelihood of column yielding and to prevent a shear failure under the maximum possible lateral forces. When earthquake effects are determined from equivalent lateral static forces, in accordance with NZS 4203, a procedure given in the Appendix to Commentary Section C3 may be used to determine the design shear forces for columns of ductile frames.

C7.5.2 *Shear strength provided by concrete*

C7.5.2.1 It is assumed that the contribution of the concrete to shear strength, v_c, is negligible in plastic hinge zones of beams and hence web reinforcement is required for the full shear demand. The plastic hinge zones have been defined in 6.5.3.1. In between these regions the value of v_c may be taken as specified in 7.3.2 and the web reinforcement provided according to 7.3.6. If v_c is to be zero in beams because of one seismic load combination it is to remain zero for any other load. For example a positive plastic hinge in a beam away from a column face may be subjected to no shear. However, extensive flexural yielding will reduce its shear capacity for any other load combination that produces shear in this location of the beam without causing significant moments.

C7.5.2.2 In columns subjected to axial load and moment the requirements of 7.3.2.1 (b) and (c) apply except in the end regions adjacent to intersecting members, that is, beams. Yielding or hinging in a column, if it is to occur, is assumed to be confined to the end regions. Therefore the middle portion of a column may be assumed to remain elastic and thus the contribution of the concrete to shear strength, v_c, may be assumed to be sustained under the most adverse seismic conditions. In this portion the stirrup spacing must not exceed 0.4 times the overall depth of the column in accordance with 7.3.5.4 (a).

The end regions, representing the localities of potential plastic hinges, have been defined the same way as the regions at which confining column reinforcement is required in accordance with 6.5.4.1. For small axial compression, that is, $P_e/f'_cA_g < 0.10$ the value of v_c is to be zero as for beams without axial load. For larger axial compression eq. 7-5 has been modified to give a gradual rather than an abrupt increase in v_c. For values of $P_e/f'_cA_g > 0.4$ the values of v_c given by eq. 7-41 are very close to those given by eq. 7-5.

When computing v_i, v_c and P_e, it is necessary to ensure that they correspond to the same seismic load combination. For the magnitude of P_e in eq. 7-41 the minimum probable value consistent with the acting shear force, should be assumed.

C7.5.3 *Shear reinforcement details*

C7.5.3.1 When determining the quantity and spacing of stirrups it is necessary to check that other requirements for transverse reinforcement, particularly in columns, are also satisfied.

C7.5.4 *Design of shear reinforcement*

C7.5.4.2 These provisions have been made to safeguard beams, subjected to reversed cyclic loading, against sliding shear failure and to reduce the loss of energy dissipation due to transverse slip in the plastic hinge zones. When the top and bottom flexural reinforcement progressively yield, wide full depth cracks will develop. This may significantly reduce the interface shear transfer capacity of the concrete and a detrimental overloading of the dowel mechanism of the longitiduinal flexural reinforcement may ensue. Therefore diagonal reinforcement needs to be provided at every section, taken at right angles across the plastic hinge zone, effectively crossing potential full depth cracks, when the total shear stress in both directions exceeds $0.3\sqrt{f'_c}$. When the shear stress due to gravity load on a beam is significant, the combined gravity and seismic shear at a positive moment hinge near a column face may be zero or very small. Such is the case at support A of the beam shown in fig. C6.6. In this case shear reversal at that plastic hinge does not occur and hence no grinding of the concrete along wide full depth cracks is to be expected. Consequently the algebraic value of the ratio r of the shear forces developed during positive and negative hinge formation is to be taken as zero and the total shear stress will be $0.6\sqrt{f'_c}$, before diagonal web reinforcement needs to be considered to control sliding.

The total shear stress under such conditions is limited to $0.9\sqrt{f'_c}$ and this limit, when compared with that set by 7.3.1.8, will control beams when f'_c is larger than 20 MPa. When shear stresses in plastic hinges approach this limit the placing of transverse shear reinforcement is likely to become difficult.

The value of r to be used is always negative.

When the reversible total shear stress v_i at the plastic hinge zone is large, according to eq. 7-42 a large proportion of the total shear force must be resisted by diagonal reinforcement. Tests [7.42] have shown that diagonal reinforcement arranged as shown in fig. C7.16 (b) is very efficient in improving the hysteresis response of the plastic hinge zone. The application of the requirements of 7.5.4.2 is shown in fig. C7.16 for two values of r and in each case for two values of the total shear stress v_i.

According to 7.5.4.2 (d) the diagonal web steel across the potential plastic hinge zone does not resist the total shear. To control diagonal tension the available stirrup ties must have a minimum capacity of resisting the remainder of the shear force. The necessary area of stirrups for each case is also shown in fig. C7.16.

These provisions do not affect plastic hinge zones in the positive moment area, away from the face of the beam support where, as at C in fig. C6.6, shear stresses will be low and the top reinforcement is not likely to be subject to tensile yielding.

Further research may indicate that large diameter short longitudinal bars, placed near the mid-depth of the plastic hinge region of flexural members, may also effectively reduce the loss of energy dissipation due to reversed cyclic sliding displacements.

C7.5.4.3 The primary purpose of diagonal web reinforcement in this case is not to form part of the traditional truss mechanism, but to effectively cross every potential full depth crack after the flexural reinforcement in both faces of a member has yielded. A rational analysis is required to show that the vertical component of the diagonal web reinforcement across each section of the potential plastic hinge within a distance d away from the theoretical section of maximum moment, such as a column face, is equal or larger than the shear force to be resisted.

The area of the diagonal reinforcement A_{vd} required to resist a shear force V_{di} at a potential full depth crack can be computed from

$$A_{vd} = \frac{V_{di}}{n f_y \sin \alpha}$$

where V_{di} can be derived from eq. 7-42 and where $n = 1$ or 2 depending whether diagonal reinforcement with area A_{vd}

is provided in one or two directions, as illustrated in fig. C7.16. It will be more expedient to utilize an existing bar or a suitable bar diameter in the locality and to determine the inclination of the bar, α, to the longitudinal axis of the member.

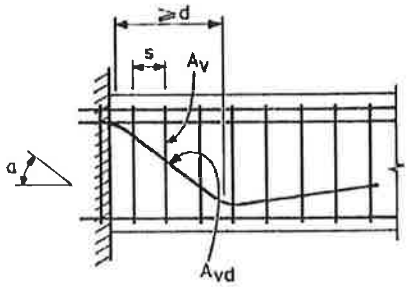
The inclination of bent bars with respect to the longitudinal axis of the member should not be less than 30° nor should it be more than 60°. Such bars must be adequately anchored so as to develop their strength at every part of their inclined length.

When inclined bars are required to resist the shear in both directions in a beam subjected to large earthquake induced shear forces and relatively small gravity shears, the vertical components of both the inclined tension and compression bars may be assumed to contribute to the total shear resistance across every cross-section of a potential plastic hinge zone.

When computing the shear strength of the plastic hinge region along a potential 45° failure plane, only the contribution of the diagonal tension reinforcement may be added to the resistance of stirrups. The interpretation of the requirements, intended to prevent diagonal tension and sliding shear failure in a potential plastic hinge region, that is removed from a column face, is shown in fig. C7.16 (b) for the case when reversed shear forces of equal intensity are expected.

In the evaluation of the flexural overstrength of a plastic hinge, the contribution of the diagonal reinforcement to the development of moment should not be overlooked.

C7.5.4.4 Plastic hinges, if any, that may develop in columns which are designed in accordance with these recommendations, are not expected to be exposed to large ductility demands and hence to reversed cyclic tensile yielding. Moreover, at least for one direction considerable axial compression will be present that would close wide cracks. Also the dowel action of a considerable number of the longitudinal column bars is likely to be more effective in the confined end regions. For this reason the requirements of 7.5.4.2 need not be applied to members referred to in 7.5.4.4. However, if the axial compression is low and it does not exceed an average stress of $P_e/A_g = 0.10 f'_c$ for a



$r = -0.5$

$v_i = 0.4\sqrt{f'_c}$ | $v_i = 0.8\sqrt{f'_c}$

$v_i < 0.45\sqrt{f'_c}$ | $v_i > 0.45\sqrt{f'_c}$

(i) For sliding shear

$V_{di} = 0$ | $V_{di} = 0.42\, V_i$

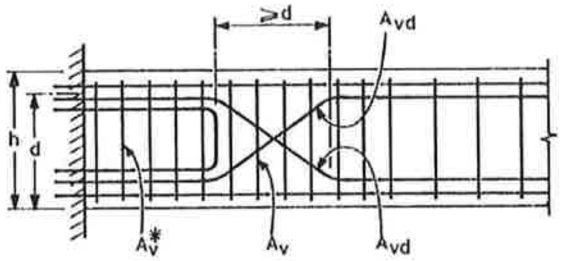
A_{vd} not required | $A_{vd} \geqslant \dfrac{0.42\, V_i}{f_y \sin\alpha}$

(ii) For diagonal tension

$A_v \geqslant 1.00 \dfrac{v_i b_w s}{f_y}$ | $A_v \geqslant 0.58 \dfrac{v_i b_w s}{f_y}$

CASE (a)

The upward shear is 50% of the downward shear and 7.5.4.2 (b) applies.



$r = -1.0$

$v_i = 0.4\sqrt{f'_c}$ | $v_i = 0.8\sqrt{f'_c}$

$v_i > 0.3\sqrt{f'_c}$ | $v_i > 0.3\sqrt{f'_c}$

(i) For sliding shear

$V_{di} = 0.56\, V_i$ | $V_{di} = 0.84\, V_i$

$A_{vd} \geqslant \dfrac{0.28\, V_i}{f_y \sin\alpha}$ | $A_{vd} \geqslant \dfrac{0.42\, V_i}{f_y \sin\alpha}$

(ii) For diagonal tension

$A_v \geqslant 0.72 \dfrac{v_i b_w s}{f_y}$ | $A_v \geqslant 0.58 \dfrac{v_i b_w s}{f_y}$

CASE (b)

Reversed shear of equal magnitude acts across a plastic hinge away from a column face. The relevant design equations corresponding with different shear stress levels in accordance with 7.5.4.2 (b) are given.

Fig. C7.16 EXAMPLE FOR THE DESIGN OF DIAGONAL SHEAR REINFORCEMENT AND STIRRUPS IN POTENTIAL PLASTIC HINGE ZONES TO CONTROL SLIDING AND DIAGONAL TENSION FAILURES

particular direction of lateral load, the column should be treated as a beam. Such columns may occur in low buildings or in the upper storeys of multi-storey frames. In most cases it is likely that the total shear stress in such columns will not exceed $0.3\sqrt{f'_c}$ MPa.

Columns subjected to moment and axial tension should be treated as beams in accordance with 7.5.2.1. Thus diagonal shear reinforcement should be provided across the critical end region sections of such a column when the total shear stress exceeds $0.3\sqrt{f'_c}$ MPa. It should be noted that for this situation it is intended to allow a hinge formation and hence considerable yielding of the column steel. With large axial tension the neutral axis depth may be so small that very little if any concrete will be in contact at the critical section. The recommendation implies that the equivalent of up to $0.3\sqrt{f'_c}$ stress can be carried by dowel action only.

C7.5.4.5 When diagonal reinforcement is provided in such a way that the entire moment and shear force at every section can be resisted only by steel forces in tension and compression, as in the case of diagonally reinforced coupling beams, (see 10.5.7.2) there appears to be no need to limit the nominal shear stress v_i, which under these circumstances is an irrelevant quantity.

C7.5.5 *Special provisions for earthquake resisting walls and diaphragms*

C7.5.5.2 The provisions for shear strength of walls are those of 7.3.14. Additional restrictions are required, however, in potential plastic hinge zones, that is, end regions, where shear strength is affected by yielding of the vertical wall reinforcement during reversed cyclic loading in a major earthquake. The restriction on shear stress v_c is similar to that given for columns in 7.5.2.2 except that the contribution of the concrete v_c to shear strength may be assumed even for very small axial compression loads. This has been established in recent tests [7.43].

Because of the distribution of the vertical reinforcement throughout the depth of a shear wall section, better control of diagonal crack width is expected than in beams.

Tests [7.43, 7.44] have shown that web crushing in the plastic hinge zone, at the base of cantilever shear walls, may occur after only a few cycles of reversed loading involving displacement ductilities of four or more. When the imposed displacement ductilities in these tests were only three or less, the shear stress levels specified by 7.5.4.5 could be repeatedly attained. Web crushing may lead to apparent sliding shear failure. To prevent web crushing due to excessive shear load, eq. 7-44 makes the maximum total shear stress dependent on the ductility demand, as measured by the inverse of the structural type factor S, and the flexural strength that may have been provided in excess of that required by NZS 4203, as measured by the flexural overstrength factor for walls, ϕ_o. This results for a coupled shear wall without excess strength, that is, $\phi_o = 1.39$ (see C3.5.1.3) and $S = 0.8$ in $v_{i\ \max.} = 0.49\sqrt{f'_c}$. On the other hand for a cantilever wall with limited ductility, that is, $S = 1.6$, the value of $v_{i\ \max.}$ will approach the maximum permissible in plastic hinge regions, that is, $0.9\sqrt{f'_c}$.

C7.5.5.3 In the design of shear reinforcement for walls, sufficient horizontal shear reinforcement is required to carry the shear exceeding v_c. The additional requirement of eq. 7-45 for the ratio of horizontal reinforcement is to ensure that yielding of the vertical reinforcement due to flexure will occur before diagonal tension failure is likely to develop. The steel ratio ρ_n includes all the vertical wall reinforcement distributed over the entire effective wall section, A_{st}. The steel ratio is then $\rho_n = A_{st}/b_w \ell_w$. Where the content of the vertical wall reinforcement, ρ_n, is controlled by loading other than seismic, for example by face loading on the wall, the in-plane flexural capacity and the corresponding shear strength of the wall may never need to be developed. For such cases horizontal shear reinforcement need not be provided for an ideal shear force in excess of that corresponding with a structural type factor of $S = 4$. Equation 7-45 may control the quantity of horizontal reinforcement in walls with aspect ratios, h_w/ℓ_w, less than one. When the shear stress is large or when flanges are used in squat shear walls, it will be advisable to use diagonal wall reinforcement to resist at least some of the shear force developed with the flexural strength. This is to reduce the loss in energy dissipation due to sliding shear displacements at the base of ductile squat shear walls. When diagonal bars are used, their contribution to flexure should also be considered and accordingly eq. 7-45 needs to be modified.

C7.5.6 *Openings in the web.* These recommendations have been largely restricted to the restatement of general principles in "good engineering practice". There is relatively little in the literature that is relevant to seismic load conditions, for which, in principle, more stringent rules should apply.

C7.5.6.1 Openings must be located in such a way that no potential failure planes, passing through several openings, can develop. In considering this the possible reversal of the shear forces, associated with the development of the flexural overstrength of the members, should be taken into account.

C7.5.6.2 Small openings with areas not exceeding those specified are considered not to interfere with the development of the strength of the member. However, such openings must not encroach into the flexural compression zone of the member. Therefore the edge of a small opening should be no closer than 0.33 d to the compression face of the member, as required by 7.5.6.4. When two or more small openings are placed transversely in the web the distance between the outermost edges of the small openings should be considered as being equivalent to the height of one large opening and the member should be designed accordingly.

C7.5.6.3 Parts of the web adjacent to an opening, larger than that permitted by 7.5.6.2, should be subjected to rational analysis to ensure that failure of the member at the opening cannot occur under the most adverse load conditions. This will require the design of orthogonal or diagonal reinforcement around such openings.

C7.5.6.4 More severe restrictions apply when the largest dimension of an opening in the web exceeds 0.25 d. Openings of this size are not permitted in areas of the member where the ideal shear stress exceeds $0.4\sqrt{f'_c}$ nor in plastic hinge zones. To ensure that the moments and shear forces can be effectively transmitted by the compression zone of the member, the opening must not encroach into the flexural compression zone. The dimension of the opening at right angles to the axis of the member must not exceed 0.4 d. The horizontal clear distance between adjacent large openings in a beam should not be less than twice the length of the opening or the depth of the member, whichever is the greater.

C7.5.6.5 Only the part of the web above or below an opening which is in compression should be considered to transmit shear. The stiffness or the tension part, which accommodates the longitudinal flexural reinforcement, is considered to be negligible because of extensive cracking. The amount, location and anchorage of the longitudinal reinforcement in the compression part of the web above the opening must be determined from first principles so as to resist one and one-half times the moment induced by the shear force across the opening. Similarly, shear reinforcement in the compression chord adjacent to the opening must resist 150% of the design shear force. This is to ensure that no failure should occur as a result of the local weakening of the member due to the opening. Effective diagonal reinforcement above or below the opening, resisting one and one-half times the shear and moment, is also acceptable.

C7.5.6.6 At either side of an opening where the moments and shear forces are introduced to the full section of a beam, horizontal splitting or diagonal tension cracks are to be expected. To control these cracks transverse reinforcement, resisting at least twice the design shear force, must be provided on both sides of the opening. Such stirrups can be distributed over a length not exceeding 0.5 d at either side immediately adjacent to the opening.

Typical details of reinforcement around a large opening in the web of a beam, complying with these requirements, are shown in fig. C7.17.

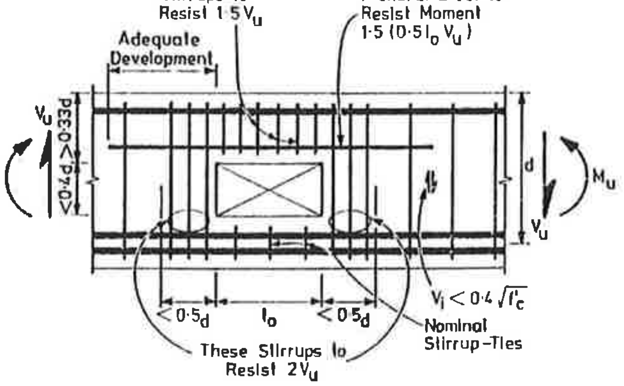


Fig. C7.17
DETAILS OF REQUIREMENTS AT A LARGE OPENING IN THE WEB OF A BEAM

REFERENCES

7.1 ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members", Chapters 1 to 4, Proceedings, ASCE, V. 99, No. ST6, June 1973, pp. 1148–1157. Also ACI Manual of Concrete Practice, American Concrete Institute, Detroit. See also Reference 7.9.

7.2 ACI-ASCE Committee 426, "Suggested Revisions to Shear Provisions for Buildings Codes", ACI Committee Report ACI 426:1R-77, 1979, 82 pp.

7.3 Bresler, B. and MacGregor, J.G., "Review of Concrete Beams Failing in Shear", Proceedings, ASCE, V. 93, No. STI, Feb. 1967, pp. 343–372.

7.4 MacGregor, James G. and Hanson, John M., "Proposed Changes in Shear Provisions for Reinforced and Prestressed Concrete Beams", ACI Journal, Proceedings V. 66, No. 4, April 1969, pp. 276–288.

7.5 Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley & Sons, 1975, 769 pp.

7.6 Hognestad, E., Elstner, R.C. and Hanson, J.A., "Shear Strength of Reinforced Structural Lightweight Aggregate Concrete Slabs, ACI Journal, Proceedings V. 61, No. 6, June 1964, pp. 643–656.

7.7 Ivey, D.L. and Buth, E., "Shear Capacity of Lightweight Concrete Beams", ACI Journal, Proceedings V. 64, No. 10, Oct. 1967, pp. 634–643.

7.8 Hanson, J.A., "Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete", ACI Journal, Proceedings V. 58, No. 1, July 1961, pp. 1–40. (See also Reference 7.7.)

7.9 ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members", Proceedings, ASCE, V. 100, No. ST8, Aug. 1974, pp. 1543–1591.

7.10 Rajagopalan, K.S. and Ferguson, P.M., "Exploratory Shear Tests Emphasizing Percentage of Longitudinal Steel", ACI Journal, Proceedings, V. 65, No. 8, Aug. 1968, pp. 634–638.

7.11 Zsutty, T.C., "Beam Shear Strength Prediction by Analysis of Existing Data", ACI Journal, Proceedings, V. 65, No. 11, Nov. 1968, pp. 943-951.

7.12 MacGregor, J.G. and Gergely, P., "Suggested Revisions to ACI Building Code Clauses Dealing with Shear in Beams", ACI Journal, Proceedings, V. 74, No. 10, Oct. 1977, pp. 493–500.

7.13 Hawkins, N.M. and Mitchell, D., "Progressive Collapse of Flat Plate Structures", ACI Journal, Proceedings, V. 76, No. 7, July 1979, pp. 775–808.

7.14 Thürlimann, B., "Shear Strength of Reinforced and Prestressed Concrete Beams – C.E.B. Approach", : ACI – Symposium 1976, Philadelphia, U.S.A.

7.15 Grob, J. and Thürlimann, B., "Ultimate Strength and Design of Reinforced Concrete Beams under Bending and Shear", Bulletin of the International Association of Bridge and Structural Engineering, Vol. 36-II, 1976, pp. 105–120.

7.16 Müller, P., "Failure Mechanisms for Reinforced Concrete Beams in Torsion and Bending", Bulletin of the International Association of Bridge and Structural Engineering, Vol. 36-II, 1976, pp. 147–163.

7.17 Kani, G.N.J., "Basic Facts Concerning Shear Failure", ACI Journal, Proceedings, V. 63, No. 6, June 1966, pp. 675–692.

7.18 Diaz de Cossio, R. and Loera, S., Discussion of "Basic Facts Concerning Shear Failure", by G.N.J. Kani, ACI Journal, Proceedings V. 63, No. 12, Dec. 1966, pp. 1511–1514.

7.19 Kani, G.N.J., "How Safe Are Our Large Reinforced Concrete Beams?", ACI Journal, Proceedings, V. 64, No. 3, Mar. 1967, pp. 128–141.

7.20 MacGregor, James G., Discussion of "How Safe Are Our Large Reinforced Concrete Beams?", by G.N.J. Kani, ACI Journal, Proceedings, V. 64, No. 9, Sept. 1967, pp. 603–604.

7.21 "Code for the Design of Concrete Structures for Buildings", CAN-A23.3-M77, National Standard of Canada, Canadian Standards Association, 1977, 131 pp.

7.22 Comité Européen du Béton, Commission V, Manual de calcul, "Effort Tranchant – Torsion", CEB Bulletin No. 92, June 1973, 278 pp.

7.23 Collins, Michael, P., "The Torque–Twist Characteristics of Reinforced Concrete Beams", Inelasticity and Non-Linearity in Structural Concrete, University of Waterloo Press, 1972, pp. 211–232.

7.24 Mitchell, Denis, and Collins, Michael, P., "Diagonal Compression Field Theory – A Rational Model for Structural Concrete in Pure Tension", ACI Journal, Proceedings, V. 71, Aug. 1974, pp. 396–408.

7.25 Mitchell, D. and Collins, M.P., "Detailing for Torsion", ACI Journal, Proceedings, V. 73, No. 9, Sept. 1976, pp. 506–511.

7.26 Birkeland, P.W. and Birkeland, H.W., "Connections in Precast Concrete Construction", ACI Journal, Proceedings, V. 63, No. 3, March 1966, pp. 345–368.

7.27 Mast, R.F., "Auxiliary Reinforcement in Precast Concrete Connections", Proceedings, ASCE, V. 94, No. ST6, June 1968, pp. 1485–1504.

7.28 Mattock, A.H., "Design Proposals for Reinforced Concrete Corbels", Journal, Prestressed Concrete Institute, V. 21, No. 3, May/June 1976, pp. 2–26.

7.29 Mattock, A.H. and Hawkins, N.M., "Shear Transfer in Reinforced Concrete – Recent Research", Journal, Prestressed Concrete Institute, V. 17, No. 2, Mar./Apr. 1972, pp. 55–75.

7.30 Mattock, A.H., Johal, L.P. and Chow, C.H., "Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane", PCI Journal, Vol. 20, No. 4, July/Aug. 1975, pp. 76–93.

7.31 Comité Européen du Béton, "International Recommendations for the Design and Construction of Deep Beams", Information Bulletin, No. 73, Paris, June 1970, pp. 17–24.

7.32 Thürlimann, B., "Torsional Strength of Reinforced and Prestressed Concrete Beams – CEB Approach", ACI – Symposium 1976, Philadelphia, U.S.A.

7.33 Zia, P. and McGee, W.D., "Torsion Design of Prestressed Concrete", PCI Journal, Vol. 19, No. 2, March/April 1974, pp. 46–65, and No. 6, Nov./Dec. 1974, pp. 67–82.

7.34 MacGregor, J.G. and Hawkins, N.M., "Suggested Revisions to ACI Building Code Clauses Dealing with Shear Friction and Shear in Deep Beams and Corbels", ACI Journal, Proceedings, V. 74, No. 11, Nov. 1977, pp. 537–545.

7.35 Kriz, L.B. and Raths, C.H., "Connections in Precast Concrete Structures – Strength of Corbels", Journal, Prestressed Concrete Institute, V. 10, No. 1, Feb. 1965, pp. 16–47.

7.36 Mattock, A.H., Chen, K.C. and Soongswang, K., "The Behaviour of Reinforced Concrete Corbels", PCI Journal, Vol. 21, No. 2, March/April 1976, pp. 52–77.

7.37 Mattock, A.H., "Design Proposals for Reinforced Concrete Corbels", PCI Journal, V. 21, No. 3, May/June 1976, pp. 18 42.

7.38 Cardenas, Alex E., Hanson, John M., Corley, W. Gene and Hognestad, Eivind, "Design Provisions for Shear Walls", ACI Journal, Proceedings, V. 70, No. 3, March 1973, pp. 221–230. Also Research and Development Bulletin RD028D, Portland Cement Association, Skokie, Ill.

7.39 Cardenas, Alex E. and Magura, Donald D., "Strength of High-Rise Shear Walls – Rectangular Cross-Section", Response of Multi-storey Concrete Structures to Lateral Forces. SP-36, American Concrete Institute, Detroit, 1973, pp. 119–150. Also Research and Development Bulletin RDO29D, Portland Cement Association, Skokie, Ill.

7.40 Corley, W.G. and Hawkins, N.M., "Shearhead Reinforcement for Slabs", ACI Journal, Proceedings, V. 65, No. 10, Oct. 1968, pp. 811–824.

7.41 ACI-ASCE Committee 326 (later 426), "Shear and Diagonal Tension", ACI Journal, Proceedings, V. 59, No. 1, Jan. 1962, pp. 1–30, No. 2, Feb. 1962, pp. 277–334, and No. 3, March 1962, pp. 352–396.

7.42 Paulay, T. and Bull, I.N., "Shear Effects on Plastic Hinges of Earthquake Resisting Reinforced Concrete Frames", Comité Euro-International du Béton, Bulletin D'Information No. 132, April 1979, pp. 165–172.

7.43 Oesterle, R.G., Fiorato, A.E., Johal, L.S., Carpenter, J.E., Russell, H.G. and Gorley, W.G., "Earthquake Resistant Structural Walls – Tests of Isolated Walls", Report to National Science Foundation, Portland Cement Association, Skokie, Nov. 1976, 44 pp. (Appendix A, 38 pp., Appendix B, 233 pp).

7.44 Bertero, V.V., Popov, E.P., Wang, T.Y. and Vallenas, J., "Seismic Design Implications of Hysteretic Behaviour of Reinforced Concrete Structural Walls", 6th World Conference on Earthquake Engineering, New Delhi, 1977, Preprints 5, pp. 159–165.

7.45 Hanson, N.W. and Hanson, J.M., "Shear and Moment Transfer Between Concrete Slabs and Columns", Journal, PCA Research and Development Laboratories, V. 10, No. 1, Jan. 1968, pp. 2–16. Also, Development Department Bulletin D129, Portland Cement Association, 1968, 16 pp.

7.46 Park, R. and Islam, S., "Strength of Salb-Column Connections with Shear and Unbalanced Flexure", Journal of the Structural Division, ASCE, Vol. 102, No. ST9, Sept. 1976, pp. 1879–1901.

7.47 ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures", ACI Journal, Proceedings, V. 73, No. 7, July 1976, pp. 375–393.

7.48 CEB-FIP Model Code for Concrete Structures, 3rd Edition, 1978.

COMMENTARY

C8 COMPOSITE CONCRETE FLEXURAL MEMBERS

C8.2 Scope

C8.2.1 The scope of this Section is intended to include all types of composite concrete flexural members including composite single-T or double-T members, box sections, folded plates, lift slabs, and other structural elements, all of which should conform to the provisions of this Section. In some cases with fully cast-in-place concrete, it may be necessary to design the interface of consecutive placements of concrete as required for composite members. Composite structural steel-concrete members are not covered in this Section, since such sections are covered in NZS 3404.

C8.2.2 The Code in its entirety applies to composite concrete flexural members except as specifically modified in Section 8. For instance, deep composite beams shall be designed in accord with 7.3.12.

C8.3 General principles and requirements

C8.3.1 *General considerations*

C8.3.1.1 This Clause permits the designers to use any or all of the various components in supporting the load in the most expeditious manner.

C8.3.1.4 Tests to destruction indicate no difference in strength of shored and unshored members.

C8.3.1.6 The extent of cracking permitted is dependent on such factors as environment, aesthetics, and occupancy. In addition, composite action must not be impaired.

C8.3.1.7 The premature loading of precast elements can cause excessive deflections as the result of creep and shrinkage. This is especially so at early ages when the moisture content is high and the strength low.

The transfer of shear by direct bond is essential if excessive deflection from slippage is to be prevented. A shear key is an added mechanical factor of safety but it cannot operate until slippage occurs.

C8.3.2 *Shoring.* The provisions of 4.4.1.3 (d) must be considered with regard to deflections of shored and unshored members. Before shoring is removed it should be ascertained that the strength and serviceability characteristics of the structure will not be impaired.

C8.3.4 *Horizontal shear*

C8.3.4.1 The full transfer of horizontal shear between segments must be ensured by contact stresses or properly anchored ties, or both.

C8.4 Principles and requirements additional to 8.3 for members not designed for seismic loading

C8.4.1 *Horizontal shear*

C8.4.1.1 Tests[8.1] indicate that horizontal shear does not present a problem in T-beams when the portion below the flange is designed to resist the vertical shear, the interfaces of the components are rough and minimum ties are provided according to 8.4.2.1. The ties must be extended across the joint and fully anchored on both sides of the joint in accord with 5.4.3 and 5.5.6. These considerations may be used with other segmental shapes.

C8.4.1.3 The calculated horizontal shear stress represents the force per unit area of interface. When the design is developed using the alternate method of Appendix B, V_u is to be taken as the shear due to dead and live load calculated using the Alternative Method design load combinations of NZS 4203 or other appropriate loadings code.

C8.4.1.4 The permissible horizontal shear stresses, v_h, apply when the design is based on the strength requirements and the ϕ factor of Section 4. When the alternative design method of Appendix B is used, the value of v_h should be reduced in accordance with the provisions for shear stresses in Appendix B, Clause B7.2.

In reviewing composite concrete flexural members for serviceability at service loads and for handling and construction loads, V_u may be replaced by the service load shear or handling load shear in eq. 8-1. The resulting service load or handling load horizontal shear stress should be compared with the allowable stresses ($0.55\, v_h$) to ensure that an adequate factor of safety results.

C8.4.1.5 Proper anchorage of bars extending across joints is required to ensure that contact of the interfaces is maintained.

C8.4.1.7 The provisions of this Clause are necessary in bridges, to resist any tendency for progressive breakdown of horizontal shear strength under traffic vibrations. Compared with buildings, bridges have the following characteristics which influence the provisions of this Clause:

(a) Greater possibility of overloads

(b) Dynamic effects associated with highway loads

(c) Potentially greater life

(d) Greater probability of presence of an aggressive environment.

C8.4.2 *Ties for horizontal shear.* The minimum areas and maximum spacings are based on test data given in references [8.1] to [8.5] inclusive.

C8.4.3 *Intentional roughness.* This Clause conforms to the provisions of 7.3.11.9 and is based on tests discussed in reference [8.1].

REFERENCES

8.1 Saemann J.C., and Washa, George W., "Horizontal Shear Connections Between Precast Beams and Cast-in-Place Slabs"' ACI Journal, Proceedings V.61, No. 11, Nov. 1964, pp. 1383–1409. See also discussion ACI Journal June 1965.

8.2 Hanson, N.W., "Precast — Prestressed Concrete Bridges: (2) Horizontal Shear Connections", Journal PCA Research and Development Laboratories, V. 2, No. 2 May 1960, pp. 38–58. Also, Development Department Bulletin D35, Portland Cement Association. 21pp.

8.3 Mattock, A.H. and Kear P.H., "Precast — Prestressed Concrete Bridges: (4) Shear Tests of Continuous Girders", Journal, PCA Research and Development Laboratories, V.3, No. 1, Jan. 1961, pp. 47–56. Also, Development Department Bulletin D45, Portland Cement Association. 29pp.

8.4 Grossfield, B., and Brinstiel, C., "Tests of T-Beams with Precast Webs and Cast-in-Place Flanges", ACI Journal, Proceedings V.59, No. 6, June 1962, pp. 843–851.

8.5 Mast, Robert F., "Auxiliary Reinforcement in Concrete Connections," Proceedings, ASCE V. 94, No. ST6, June 1968, pp. 1485–1504.

COMMENTARY

C9 BEAM – COLUMN JOINTS

C9.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 9 of the Code.

A_{s1}, A_{s2}	area of beam flexural reinforcement
C_c'	compression force in the flexural compression zone of a beam
C_s'	compression force in the compression reinforcement of a beam
f_{oy}	overstrength of Grade 275 longitudinal reinforcement, generally $1.25 f_y$
ℓ_1, ℓ_2	span of beam between centre-to-centre of supports
ℓ_{1n}, ℓ_{2n}	length of clear span of beam, measured face-to-face of supports
ℓ_c, ℓ_c'	height of column, centre-to-centre of floors or roof
M_{o1}, M_{o2}	flexural overstrength of beam section at faces of column
T	tension force in tension reinforcement
T_p, T_p'	prestressing force at flexural capacity of section

C9.2 Scope

C9.2.1 Section 9 covers the design of beam-column joints. Clause 9.3 gives general principles and requirements, 9.4 gives additional principles and requirements for members not designed for seismic loading and 9.5 gives additional principles and requirements for members designed for seismic loading. For beam-column joints in most structural frames designed for New Zealand conditions, seismic loading is the critical condition and the provision of 9.5 will govern. In applications such as knee joints in gravity dominated portal frames and particularly frames in which seismic load resistance is provided by other elements such as shear walls, the provisions of 9.4 may apply.

Design of slab/column connections including provisions for shearhead reinforcement is covered in 7.3.15.

C9.3 General principles and requirements

C9.3.1 The basic requirements of a beam-column joint are that it must perform satisfactorily under service loads, that its strength should not normally govern the strength of the structure, and that its behaviour should not impede the development of the full strength of the adjoining members. Other important requirements are ease of construction and access for depositing and compacting concrete.

The structural demand on joints is greatly dependent on the type of loading, and therefore design procedures are necessary appropriate to the severity of each type of loading. Where static gravity loading governs, strength under monotonic loading without stress reversals will be the design criterion. Seismic loading is more severe, because strength degradation of the concrete in the joint may occur under repeated reversed loading, and a large amount of joint reinforcement is therefore required.

C9.4 Principles and requirements additional to 9.3 for joints not designed for seismic loadings

C9.4.2 *Design forces.* The joint must be designed to resist the forces considered in designing the members and in those combinations producing the most severe force distribution at the joint. Forces produced by deformations resulting from time-dependent effects such as creep, shrinkage or settlement should be considered.

Forces in the joint should be determined by considering a free body of the joint with forces on the joint-member boundaries properly represented.

The serviceability requirements of the code are intended primarily for the members meeting at a joint. However, joint behaviour could be significant if bars slipped within the joint core leading to cracking and member rotation at the face of the joint core.

C9.4.3 *Strength reduction factor.* The dependable shear capacity of the joint, using $\phi = 0.85$, is to be equated to the shear demand corresponding to the most adverse combination of factored loads on the joined members.

C9.4.4 *Maximum permissible horizontal stress.* An upper limit on the nominal horizontal shear stress across the joint area is specified to safeguard the core concrete against excessive diagonal compressive stresses.

C9.4.5 *Design principles.* Joints subjected to non-seismic loading may be designed using the relevant principles of force equilibrium. A rational analysis may be used to show the extent to which a principal diagonal compression strut can carry a proportion of the joint shear, the remainder being carried by horizontal and vertical or diagonal joint shear reinforcement. Alternatively equations 9-2 or 9-3 and 9-5 and 9-6 may be used to evaluate this contribution.

The corner joint of a portal frame is a common example that will not necessarily require orthogonal reinforcement. Recommendations for design and detailing are given in reference[9.1]. Design requirements for a knee joint will differ for a moment that tends to close the right angle and for a moment that tends to open it as indicated in fig. C9.1.

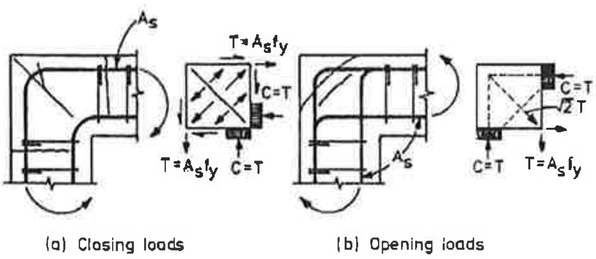


Fig. C9.1 FORCES IN KNEE JOINT OF PORTAL FRAME

Under "closing" moments, knee joints of small members, slabs and walls in particular, are considered to require for adequate strength, tension steel continuous around the corner and with sufficient radius to prevent bearing or splitting failure, and the amount of tension reinforcement (conservatively) limited to $\rho < 0.5 \sqrt{f_c'} / f_y$. When using larger structural members having substantial reinforcing content, secondary reinforcement is required to preserve the integrity of the concrete within the joint by controlling splitting cracks and by providing confinement for the inner corner. A right angle corner joint is more severely affected when the applied moments tend to "open" the angle. Compression forces near the outer corner tend to "push off" the triangular corner portion of the joint. The use of secondary reinforcement to resist diagonal tension cannot be avoided in structural member of major frames, a recommended solution being to provide radial hoops to resist the whole of the diagonal tension across the corner [9.1].

C9.4.6 *Horizontal joint shear reinforcement.* These provisions apply to the behaviour of a joint subject to shear due to unbalanced gravity load moments in horizontal members of the joint but not subject to seismic loading. Such a joint is therefore not subject to yield incursion along beam bars passing through the joint, nor to degradation under repeated inelastic load cycles and may be expected to be similar to those of "elastic" joints in 9.5.4.2 (c). Accordingly, the provisions of that clause for A_s'/A_s of unity may be used for this case. These provisions make due allowance for the considerable contribution of the diagonal compressive strut in the concrete to joint shear transfer. The allowable proportion of the joint shear resisted by the concrete strut for a particular column increases with axial load. The factor C_j is introduced to allocate the effect of axial compression to the two principal horizontal directions x and z of a space frame where a joint is required to transfer joint shears V_{jx} and V_{jz} concurrently in each direction. For unidirectional joint loading C_j is unity.

The alternative equation 9-3 for calculation of V_{ch} allows a simplified calculation procedure for squat joints where the applied horizontal joint shear stress will not be high.

The provisions of 9.5.4.3 for horizontal shear reinforcement are based on providing reinforcement to resist that portion of the joint shear not resisted by the diagonal concrete strut corner-to-corner across the joint.

C9.4.7 *Vertical joint shear reinforcement.* To sustain a diagonal compression field by a truss mechanism vertical joint shear reinforcement is also required. This can be computed the same way as the horizontal joint shear reinforcement for "elastic" behaviour. The allowance for vertical shear carried on the concrete is greater than that for horizontal shear because of the beneficial effects of column axial load.

C9.4.8 *Confinement.* The minimum transverse reinforcement required in the joint is the same as the confinement reinforcement specified for the column ends immediately above or below the joint, except that where the joint is confined by beams on all four sides these requirements may be relaxed.

C9.5 Principles and requirements additional to 9.3 for joints designed for seismic loading

C9.5.1 *General.* Provisions are made for beam-column joints that are subjected to forces consistent with lateral loading on frames causing inelastic displacements. Particularly severe conditions can arise with respect to shear strength and anchorage of the reinforcement passing through or terminating in a joint. The basic requirement of the design is that joints must be somewhat stronger than adjacent hinging members, which are normally the beams. Because shear strength and the anchorage of the reinforcement controls joint design, energy dissipation within the joint core is undesirable. It can lead to rapid loss of strength under seismic load conditions and is therefore, to be avoided.

C9.5.2 *Design forces*

C9.5.2.1 To ensure that a joint possesses adequate reserve strength, the flexural over-strength of the adjacent beams and the corresponding internal forces must be evaluated. The simultaneous forces in the column that maintain joint equilibrium must also be determined. These must correspond with plastic hinges in the beams that may form either at the column face or at a distance away from the column where the beam overstrengths are developed. In frames where inelastic inter-storey displacements can occur in both principal directions, generally at right angles to each other, development of beam over-strengths from both of those directions should be considered separately [9.2]. When stiff structural systems, such as shear walls, preclude the

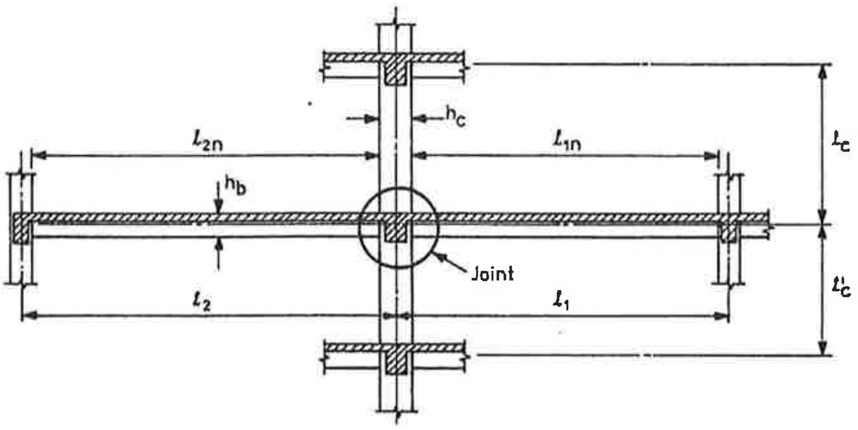


Fig. C9.2 AN INTERIOR BEAM-COLUMN JOINT

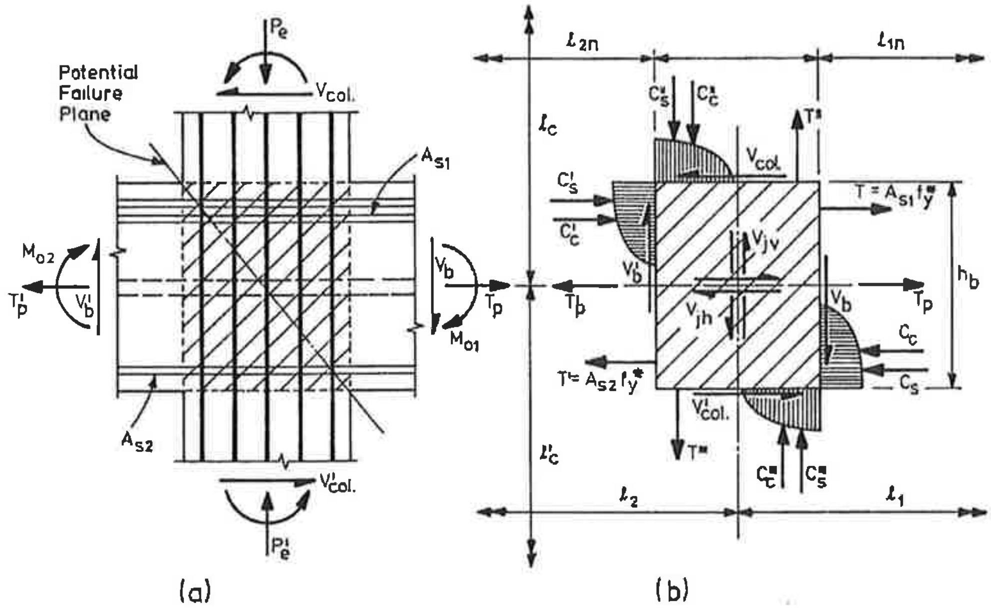


Fig. C9.3 EXTERNAL ACTIONS AND INTERNAL FORCES OF A TYPICAL INTERIOR BEAM-COLUMN JOINT

possibility of yielding in beams and columns in one or both principal directions of the building, a rational analysis must show that the elastic joint possesses adequate strength.

The same procedure applies to one or two storey frames or the top storey of multi-storey frames where columns may be designed to hinge. For the purpose of joint design, the role of beams and columns is simply reversed and the relevant clauses should be applied in a rational manner.

C9.5.2.2 For the purpose of evaluating the forces within a joint, such as fig. C9.2, the stress resultants in the adjacent beams, normally at the development of overstrengths of the members, may be used. With reference to fig. C9.3 the horizontal shear force V_{jh} across a typical interior joint is

$$V_{jh} = T + C'_c + C'_s + T_p - T'_p - V_{col}$$

For conventionally reinforced concrete members without prestressing this simplifies to

$$V_{jh} = A_{s1} f_{oy} + A_{s2} f_{oy} - V_{col}$$

Similar expressions are obtained for external joints where only one beam frames into a column.

The value of the column shear V_{col} will depend on the column moment gradients above and below the joint. However, from figures C9.2 and C9.3 its value may be estimated using a mean moment gradient, thus

$$V_{col} = 2\left(\frac{\ell_1}{\ell_{1n}} M_{o1} + \frac{\ell_2}{\ell_{2n}} M_{o2}\right) \Big/ (\ell_c + \ell'_c)$$

When necessary the value of the vertical joint shear force, V_{jv}, may be derived from similar considerations. Alternatively, the vertical joint shear force may be approximated as follows:

$$V_{jv} \approx V_{jh} \frac{h_b}{h_c}$$

C9.5.3 *Design assumptions*

C9.5.3.1 The observed failure plane due to shear in joints of one-way frames bisects the joint along a diagonal from one beam-column edge to another. The reinforcement provided must ensure that the shear force responsible for this failure plane is transmitted without unrestrained yielding of the reinforcement[9.3]. In accordance with 3.5.3.3 and 4.3.2, where joint shear forces are derived from overstrength member input, ϕ may be taken as 1.0.

C9.5.3.2 An upper limit on the nominal shear stress across the joint area is specified to safeguard the core concrete against excessive diagonal compression stresses. The horizontal nominal stress corresponding with the critical shear force, V_{jh}, is based on the nominal gross horizontal area of the joint $b_j h_c$, as defined in fig. C9.4 (a) and (b).

C9.5.3.3 There are two loading cases that need to be considered. The first loading case occurs when plastic hinges form in the beams at column faces where the beam flexural overstrengths may be developed. This will involve yield penetration along the reinforcing bars into the joint core. Also, under reversed loading less and less concrete will participate in the moment transfer from beams to the joints, that is, the moment will tend to be transferred by a steel couple. Under these circumstances the entire computed joint shear must be resisted by a truss mechanism which would normally consist of a diagonal concrete compression field, horizontal joint stirrup ties and intermediate vertical column bars passing through the joint. For the same load conditions other types of mechanisms may also be used, provided that analysis or tests show that the proposed alternative solution is satisfactory. Typically, bars bent up diagonally within the joint in both directions in the plane of the beams and columns may be used. When axial compression is transferred across the joint from the columns, or due to prestressing of beams, some allowance may be made for shear transfer by one principal concrete compression strut.

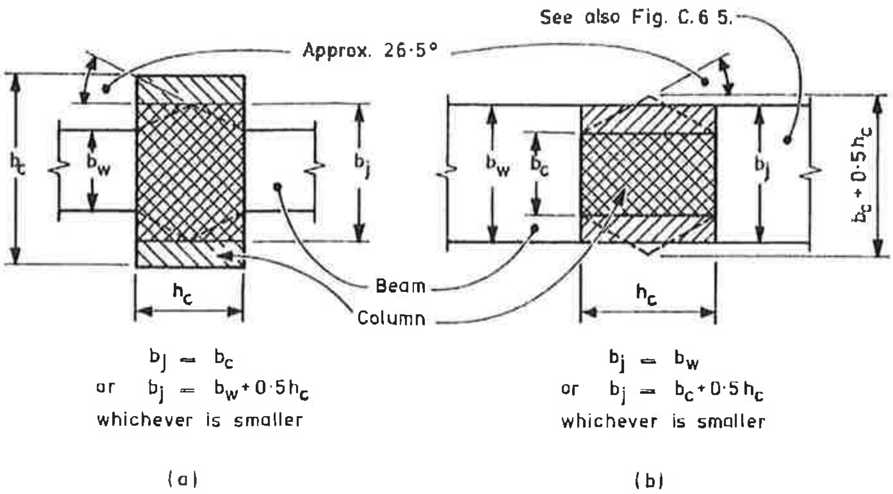


Fig. C9.4 EFFECTIVE JOINT AREAS

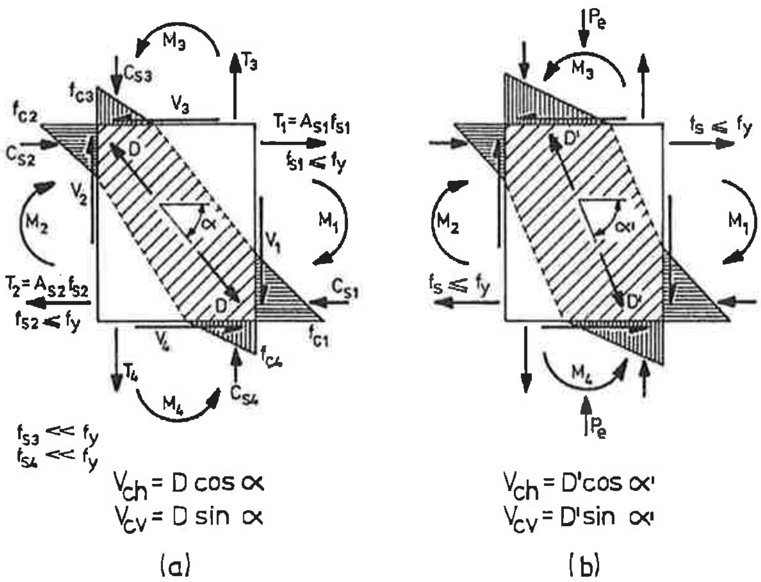


Fig. C9.5 PARTIAL SHEAR TRANSFER BY A CONCRETE DIAGONAL COMPRESSIVE STRUT IN "ELASTIC" BEAM-COLUMN JOINTS

The second loading case refers to the situation when potential plastic hinges are deliberately located so that yielding in the longitudinal beam bars is not expected to occur at or near the joint. In this case the joint may be designed using the relevant principles of equilibrium. A rational analysis should show the extent to which a principal diagonal concrete strut, such as shown in fig. C9.5, can carry components of the joint shear so that adequate horizontal and vertical or diagonal joint shear reinforcement can be provided for the excess shear. Alternatively the equations 9-11 and 9-14 may be used to evaluate this contribution.

C9.5.3.4 As required by 9.5.3.2, only a part of the joint width should be considered as being effective, as shown in fig. C9.4 for a one-way frame. Any horizontal and vertical reinforcement that is present in the column but is placed outside the effective joint area should not be considered to contribute to the joint shear strength.

C9.5.4 *Horizontal joint shear*

C9.5.4.1 The horizontal joint shear force in a joint may be assumed to be transferred from the level of the top beam flexural reinforcement to the level of the bottom beam bars by diagonal strut action in the concrete core, V_{ch}, and by a truss mechanism, V_{sh}, consisting of diagonal concrete struts, parallel to the potential failure plane, and horizontal stirrup ties and vertical shear reinforcement. With the value of V_{ch}, given by equations 9-9 or 9-10 or 9-11, the shear force to be resisted by horizontal shear reinforcement, V_{sh}, is calculated from eq. 9-8.

C9.5.4.2 When plastic hinges in the beams develop immediately adjacent to the joint, for reasons explained in 9.5.3.3 the entire horizontal shear must be resisted by reinforcement. However, certain exceptions exist:

(a) When the column of a one-way frame is subjected to average compression stresses in excess of $0.1 f'_c$ some of the bond forces along the beam flexural reinforcement are assumed to combine with the total compression force in the column to form a diagonal strut and hence some shear is assumed to be transmitted this way [9.3].

The factor C_j is introduced to allocate the effect of axial compression to the two principal directions x and z of the earthquake loading when joint shears V_{jx} and V_{jz} are concurrently developed. For unidirection joint loading C_j is unity, for a symmetrical two-way frame $C_j = 0.5$. Note that the nominal shear stress refers to the effective area $b_j h_c$ of the joints. The shear obtained from eq. 9-9 is similar to that given by eq. 7-41. Equation 9-9 is considered as being conservative. Further research may lead to its relaxation.

(b) Horizontal prestressing is considered to increase the diagonal strut action in the joint. However, prestressing steel that is present near the extreme fibres of the section must be assumed to have sustained permanent set strains and therefore to have lost its prestress after the formation of plastic hinges. Prestressing steel at the central third of the beam depth is considered to remain effective and only this prestress force after all losses may be considered for horizontal shear resistance in the joint. As well as promoting diagonal strut action in the joint the prestressing steel in the central depth of the beam serves to restrain diagonal tension cracking. Because the contributions to shear strength from column axial load and from prestress involve different mechanisms, the contributions towards V_{ch} may be added.

(c) The considerable contribution of the concrete strut within an "elastic" joint to shear transfer is recognized by eq. 9-11. It is essential to ensure, however, that no yielding of the beam flexural reinforcement is likely to occur at the joint so that the concrete compression stresses in the beams and columns, such as shown in fig. C9.5 (a), are maintained under reversed loading. Case studies indicated that approximately one half of the horizontal joint shear V_{jh} can be transferred by the diagonal concrete strut in beams with equal top and bottom flexural reinforcement when there is no axial compression on a column. The model that can be used to calculate the value of V_{ch} is shown in fig. C9.5 (a) where the tensile forces T_1 to T_4 and the steel compression forces C_{s1} to C_{s4} can be identified. The shear forces from the beams and columns V_1 to V_4, may be assumed as being transferred in the corresponding compression zones only. As V_{ch} is the horizontal component of the diagonal thrust D that is introduced to the concrete at the diagonally opposite compression corners only, this force will pass through the centre of the joint core. Figure C9.5 (b) shows the effect of axial compression on the joint.

A portion of the longitudinal beam and column reinforcement may be assumed to be effectively anchored in the shaded zones shown in fig. C9.5. Therefore shear, requiring shear reinforcement, is introduced by bond forces approximately in the area outside the shaded zone. The proportion of the joint shear resisted by the concrete strut increases with increasing axial load and with the ratio of the beam compression steel to the tension steel, A'_s/A_s. These trends simplified from case studies, are represented in eq. 9-11 and fig. C9.6 (a). Recent testing[9.4, 9.5] indicates that this equation is satisfactory.

In external beam-column joints, favourable diagonal strut action is developed provided that the anchorage of flexural bars is assured, particularly with the use of external beam stubs. Testing of such joints indicates that eq. 9-11 represents satisfactorily the contribution of the concrete strut[9.6, 9.7]. Where beams are haunched so that the beam plastic hinge is deliberately relocated from the column face in accordance with 6.5.3.1 (b), eq. 9-11 will apply for calculation of joint shear contribution of the concrete strut.

The provisions of 9.5.4.2 (c) are also applicable to joints when yielding of the beam reinforcement within the joint core is prevented and the formation of a diagonal strut, shown in fig. C9.5, is made possible by means other than relocating the plastic hinge away from column faces. Increased beam flexural reinforcement within the joint together with the use of suitably welded steel bond plates, when practicable, have been shown to result in satisfactory joint behaviour[9.9].

Testing[9.8] has shown that in haunched beam column connections even where plastic hinging may occur up to the column face, the joint behaviour may be considerably better than for a conventional connection. That is, the beneficial effects of the haunch in reducing concentrated rotations at the column face have been evident. A suitable concession for this case would be to dèsign for joint shear derived from probable yield strength of the beam reinforcing steel rather than overstrength. Clause C3.5.1.3 suggests a relationship of probable strength equal to 1.15 times ideal strength.

When columns are subjected to axial tension the assumption is made that the shear resistance due to the diagonal concrete strut action within the joint diminishes from the values given by eq. 9-11 when $P_e = 0$, to $V_{ch} = 0$ when the axial tension stress on the gross column section area is $0.2 f'_c$. Thus for large axial tension full shear reinforcement is required even in an "elastic" joint. This is also shown in fig. C9.6.

(d) Recent tests[9.10] have shown that exterior beam-column joint assemblages behaved satisfactorily with reduced horizontal joint shear reinforcement, provided that top and bottom beam bars were anchored near the outer face of the column with at least a 90° standard hook, as shown in fig. C5.16, satisfying the requirements of 5.5.2, and when intermediate column bars passing through the joint and meeting at least the requirements of 9.5.5, were used. The ratio h_c/h_b in the multiplier recognizes the reduced efficiency of the diagonal strut, such as shown in fig. C9.5, when

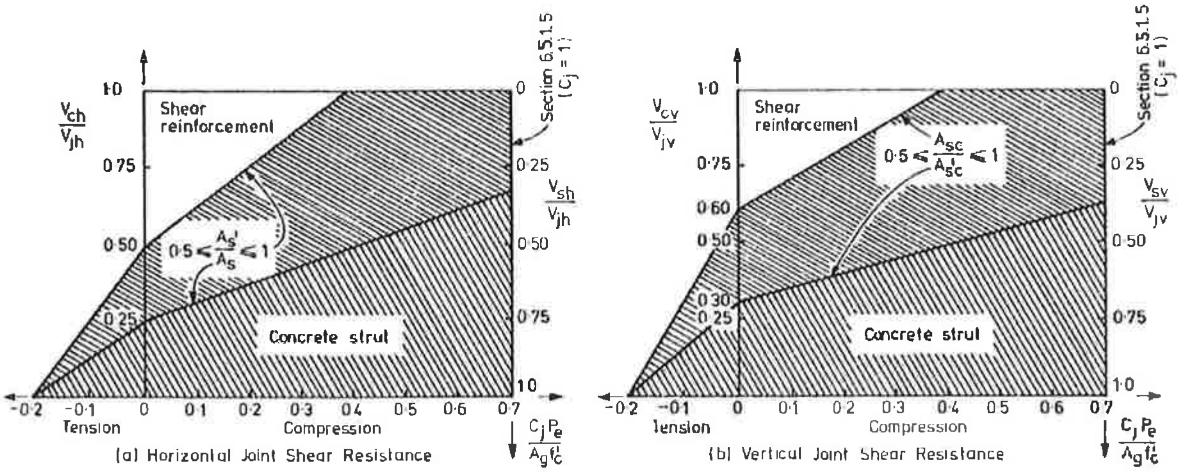


Fig. C9.6 THE ALLOCATION OF THE JOINT SHEAR TO THE CONCRETE STRUT AND TO STIRRUP TIES

shallow exterior columns support relatively deep beams.

(e) For squat joints where the column is wide, beam bar bond stress transfer is favourable and the concrete shear resisting mechanism may be expected to contribute significantly. The nominal shear stress v_c of $0.2\sqrt{f'_c}$ is allowed according to eq. 9-12.

C9.5.4.3 Stirrup ties anchored around column bars that pass through the joint may cross the potential failure plane at different angles depending on the shape of these ties. The failure planes of each tie leg in the direction of the joint shear force needs to be considered. Any tie leg between bends around column bars that does not cross the potential failure plane or is shorter than one third of the depth of the column section, h_c, should be neglected. Only those tie legs should be relied on which are situated within the effective joint core shown in fig. C9.4.

The maximum spacing of stirrups is controlled by the requirements relevant to the end regions of columns set out in 6.5.4 and which are referred to in 9.5.6.1.

C9.5.5 *Vertical joint shear*

C9.5.5.1 To sustain a diagonal compression field by a truss mechanism, vertical joint shear reinforcement is also required. This can be computed the same way as the horizontal joint shear reinforcement. However, the vertical joint shear force, V_{jv}, may be approximated as suggested in C9.5.2.2.

C9.5.5.2 Generally columns will not yield while the overstrength of the beams, adjacent to the joint, is being developed. Therefore the stresses in the compression zone of the column section can be assumed to provide partial vertical restraint to the joint truss mechanism, thereby reducing the required vertical joint steel requirements [9.5]. This stress condition is similar to that generated in an elastic joint as given by eq. 9-11. Ideally equations 9.11 and 9-15 should be of the same form. However, recent testing [9.4, 9.5, 9.6], indicates that eq. 9-11 is appropriate for horizontal shear contribution from the diagonal strut, but a greater contribution may be allowed from the concrete for vertical shear resistance because of the beneficial effects of column axial load. The effect of eq. 9-15 is shown in fig. C9.6 (b). The area of reinforcement at each face of the column section, A_{sc} and A'_{sc}, will normally be equal.

The joint at which hinging in a column rather than in the beam is expected, is an exception. In this case the vertical joint shear reinforcement should be designed on the same basis as the horizontal joint shear reinforcement for hinging beams. An exception is where column plastic hinging can occur only on one side of a joint and anchorage of the column flexural steel is outside of the joint core, for example where column steel is anchored in a column stub, or where the storey above or below a joint will remain elastic. The column-beam joint is then analogous to an external beam-column joint, for which the provisions of 9.5.4.2 (c) apply.

C9.5.5.3 When only four corner bars make up the longitudinal column reinforcement, intermediate vertical bars, placed between the corner bars, as shown in fig. C9.3 (a), need to be provided. These need not extend over the full length of a column but they need to be adequately anchored in the column above and below the joint.

C9.5.5.4 The most expedient solution for the vertical joint reinforcement is to use existing column bars within the joint core. Such intermediate bars are not expected to be fully stressed due to column load alone. It is important that at least one bar, but for larger columns two or more intermediate vertical column bars, situated between corner bars, should pass through the joint, as shown in fig. C9.3(a). Therefore the column bar spacing in the relevant column faces should not exceed 200 mm.

C9.5.6 *Confinement.* When plastic hinges could form in the beams, adjacent to the column faces, the minimum transverse reinforcement required in the joint is the same as the confinement reinforcement specified for the column ends immediately above or below the joint. However, when the adjacent beam regions around the joint in all four directions have been designed to remain elastic, these regions are assumed to provide adequate transverse confinement. Consequently the confining steel may be reduced to one half of that normally required. To safeguard column bars against buckling, particularly those at the corners of rectangular column sections which may be outside the joint core, the tie spacing is limited.

C9.5.7 *Joints with wide columns and narrow beams.* When, due to seismic actions, a narrow beam transmits moments to a wide column, it may be unsafe to assume that the longitudinal column reinforcement located away from the joint area will effectively participate in transferring moments between column and beam. Therefore the longitudinal column reinforcement which is required to interact at a particular level with a narrow beam should be placed within the effective joint width. The cross-shaded area of the column section, shown in fig. C9.4 (a), should accommodate such column bars. To resist loads from floors above, or from beams framing into the column from the other direction, and to satisfy minimum requirements for the distribution of longitudinal reinforcement, in accordance with 6.5.4.2, longitudinal bars must also be placed outside the effective joint area, $b_j h_c$, such as shown in fig. C9.4 (a).

An example of reinforcing details is shown in fig. C9.7. Requirements for longitudinal and transverse reinforcement outside the effective joint area, $b_j h_c$, are to provide for torsional resistance where required and confinement.

C9.5.8 *Eccentric beam-column joints.* To avoid the necessity of having to estimate torsional effects in a column or a joint, as a result of eccentric location of a beam which transfers earthquake induced moments, the effective joint width is artificially reduced and, as a concession, the normal design procedure for the joint and the column, as specified in the previous sections, is allowed. It is considered that this restriction will allow sufficient reserve strength from outside the specified effective joint area of the column, to safely absorb torsional effects. However, some conservatism in design is warranted because the behaviour of eccentric joints is as yet not fully understood.

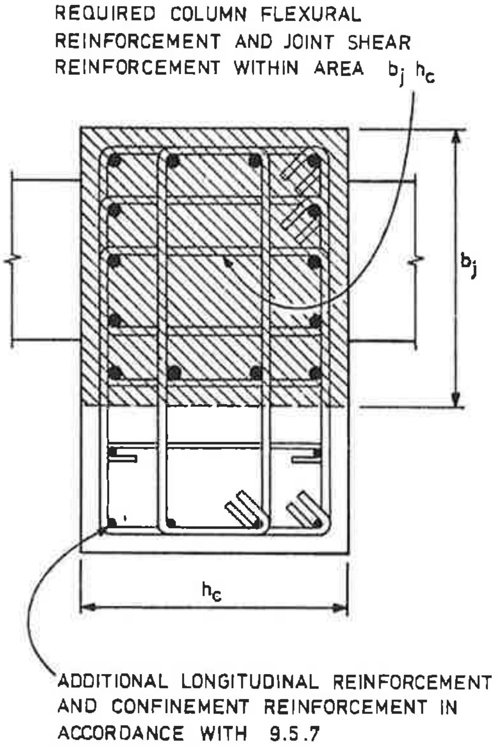


Fig. C9.7 REINFORCING DETAILS FOR JOINTS WITH WIDE COLUMNS AND NARROW BEAMS

REFERENCES

9.1 Park, R. and Paulay, T. "Reinforced Concrete Structures", John Wiley and Sons, 1975, 769 pp.

9.2 Blakeley, R.W.G., "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames", Section J, Design of Beam-Column Joints. Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 4, Dec. 1977, pp. 226–237.

9.3 Paulay, T., Park, R. and Priestley, M.N.J., "Reinforced Concrete Beam-Column Joints Under Seismic Actions", ACI Journal Proceedings, Vol. 75, No. 11, November 1978.

9.4 Birss, G.R., "The Elastic Behaviour of Earthquake Resistant Reinforced Concrete Interior Beam-Column Joints", Department of Civil Engineering, University of Canterbury, New Zealand, Report No. 78-13, 1978, 96 pp.

9.5 Park, R. and Yeoh Sik Keong, "Tests on Structural Concrete Beam-Column Joints with Intermediate Column Bars", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1978.

9.6 Blakeley, R.W.G., Megget, L.M. and Priestley, M.J.N., "Seismic Performance of Two Full Size Reinforced Concrete Beam-Column Joint Units", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 1, March 1975, pp. 38-69.

9.7 Park, R. and Paulay, T., "Behaviour of Reinforced Concrete External Beam-Column Joints under Cyclic Loading", Proceedings of 5th World Conference on Earthquake Engineering, Rome 1973, Vol. 1, Paper 88, pp. 772–781.

9.8 Blakeley, R.W.G., Edmonds, F.D., Megget, L.M. and Wood, J.H., "Cyclic Load Testing of Two Refined Reinforced Concrete Joints", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 3, September 1979, pp. 238–255.

9.9 Fenwick R.C. and Nguyen, H.T., "Reinforced Concrete Beam-Column Joints for Seismic Loading", Report No. 220, Department of Civil Engineering, University of Auckland, 1981, 62 p.

9.10 Paulay, T. and Scarpas, A., "The Behaviour of Exterior Beam-Column Joints", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14, No. 3, September 1981, pp. 131–144.

COMMENTARY

C10 WALLS AND DIAPHRAGMS

C10.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 10 of the Code.

d_b diameter of reinforcing bar

h_w height of wall to top of the top floor

M_{code} moment resulting from lateral loading specified by NZS 4203, at the base of a cantilever wall

M^o overstrength moment of resistance of section at the base of a cantilever wall

P_u factored axial load on a bearing wall

C10.3 General principles and requirements

This Clause requires that walls be designed to resist loads to which they are subjected, including eccentric gravity loads and lateral loads due to wind or earthquake, which may result in forces acting in the plane of or transversely to the wall.[10.1] In general this Clause applies to walls spanning vertically between horizontal supports.

Walls must be designed for combined flexure and axial load according to 6.3, considering the wall to be a compression member with flexure, while also satisfying the requirements of 5.3.36 with respect to vertical and horizontal reinforcement, unless meeting the empirical requirements of 10.3.2.

C10.4 Principles and requirements additional to 10.3 for walls not designed for seismic loading

Where the resultant gravity load falls within the middle third of the wall thickness, walls not considered to resist seismic loading may be designed by the empirical method prescribed. However, the empirical dimensional limits of 10.3.2 must also be satisfied.

Eccentric vertical loads and moments induced by face loading on the wall are used to determine the total eccentricity of the factored axial load P_u. If this eccentricity does not exceed $b/6$, the empirical design of 10.4 may be used. The design is then performed considering P_u as a concentric load.

The square function of the member slenderness in eq. 10-1 makes the resulting capacities for Sections 6 and 10 relatively compatible for braced members loaded within the middle third of the thickness.[10.2] The factored axial load P_u must be less than ϕP_{iw}, where P_{iw} is computed from eq. 10-1 and $\phi = 0.7$ as for tied columns. Equation 10-1 is applicable for walls that are restrained against rotation at one or both ends (top or bottom, or both) and are braced top and bottom against lateral translation. For other end conditions, an effective length $k\ell_n$ should be substituted for ℓ_n.

These provisions, which are identical with those of ACI 318-77, are likely to be applicable only rarely because in most walls seismic loading will also need to be considered.

C10.5 Principles and requirements additional to 10.3 for walls and diaphragms designed for seismic loading

C10.5.1 *General seismic design requirements*

C10.5.1.1 The accepted principles of monolithic structural action are expected to be used in the design of cantilever or coupled shear walls and diaphragms. The shear and flexural reinforcement must be allocated to each part of the cross-section in accordance with established engineering principles. The designer must ensure, by using appropriate detailing, that the required interaction between components can take place when the overstrength and required ductility of walls are developed[10.3] to [10.6].

C10.5.1.2 Clause 10.5 makes provisions only for the design of ductile shear walls in buildings and for the design of diaphragms. Therefore the general requirements with respect to analysis, design forces, ductilities and capacity design procedures, in accordance with 3.5.7 must also be considered.

C10.5.1.3 The detailed design requirements for structures with limited ductility, including walls, are specified in Section 14. Those requirements correspond with the general principles of 3.5.10. However, the major part of Section 10 is also applicable to walls of limited ductility. Accordingly Section 14 makes reference to the relevant clauses of this section[10.4].

C10.5.2 *Dimensional limitations*

C10.5.2.1 Because shear wall structures may be relatively thin, precautions must be taken to prevent possible instability in potential plastic hinge zones where repeated reversed load, causing yielding, may need to be sustained. In the absence of information on the compactness of ineelastic reinforced concrete wall sections, code rules relevant to short columns have been used. It is considered that instability should not govern the strength of an earthquake resistant structural wall. Therefore by similarity to 10.3.2.1 any part of a structural wall restrained only at floor levels should be considered as a column.

The limitations of 10.5.2 are only relevant to the potential plastic hinge zone of walls, which is normally located at the base of the wall. For the vertical extent of a plastic hinge region the assumptions of C10.5.4.2 should be used. Elsewhere in a shear wall structure, where yielding of the reinforcement and concrete compression strains in excess of 0.0015 are not expected, only the limitations of 10.3.2 apply.

A wall subjected to axial load and moment, due to in-plane earthquake forces, may exhibit compression strain patterns as shown in fig. C10.1. In establishing the strain pattern, across a critical wall section, the moment and axial force due to the design earthquake load, together with the appropriately factored gravity load and the relevant strength reduction factor ϕ must be used. The concrete strain in the extreme compression fibre of the section is in accordance with C6.2.1.3 assumed to be 0.003. With this common assumption the strain over the outer half of the compression

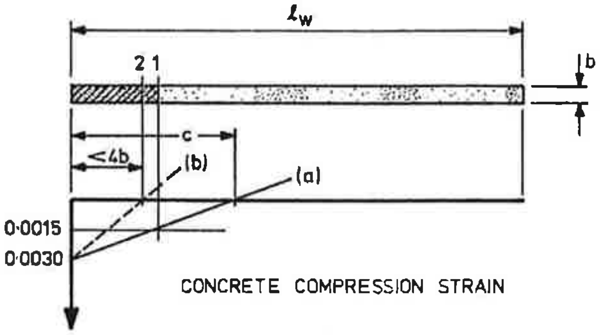


Fig. C10.1 STRAIN PROFILES AFFECTING THE LATERAL STABILITY OF THIN WALL SECTIONS

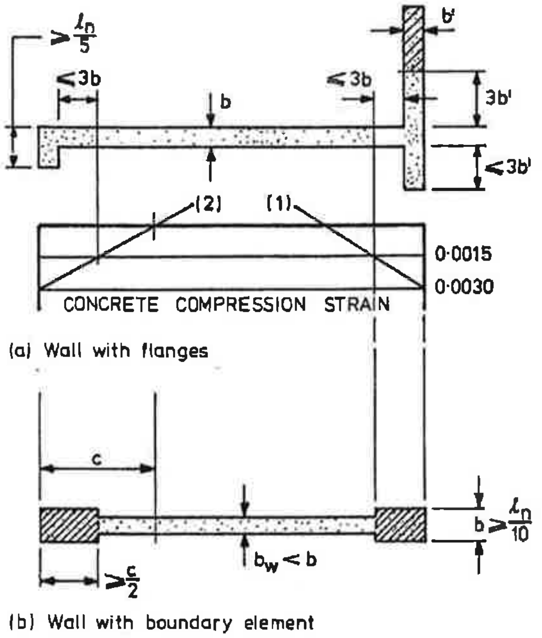


Fig. C10.2 PARTS OF WALL SECTIONS TO BE CONSIDERED FOR INSTABILITY

zone, to which the slenderness limit applies, will be more than 0.0015. For example the shaded area of the wall, extending over half of the neutral axis depth c, will need to satisfy the requirement of $b > \ell_n/10$ if under the design load a strain distribution along line *(a)* is obtained.

C10.5.2.2 In under-reinforced walls with small axial compression the neutral axis depth may be very small. In this case it is assumed that the low stressed adjacent parts of the wall will provide sufficient lateral restraint to the fully stressed narrow compression zone. This normally implies that the computed neutral axis depth for the wall section is the lesser of 4 b or $0.3\ell_w$. Line *(b)* and Section 2 in fig. C10 illustrate the limiting case when a wall is exempt from the requirements of 10.5.2.1.

C10.5.2.3 Certain components of walls, such as shown in fig. C10.2 provide continuous lateral support to adjacent components. Therefore any part of a wall, subjected to strains of 0.0015 or greater, which is within $3b$ of such a line of support is exempted from the slenderness limitation of $\ell_n/b < 10$. Figure C10.2 (a) shows a number of locations that are exempt. The shaded part of the flange is considered to be too remote to be effectively restrained by the web portion of the wall and hence it should comply with the slenderness requirement. In the absence of a flange, the width of which is at least $\ell_n/5$, a boundary element may be formed that satisfies the slenderness limitation of 10.5.2.1 as illustrated in fig. C10.2 (b).

C10.5.3 *Longitudinal reinforcement.* The requirements are intended for both the principal vertical flexural reinforcement, consisting usually of larger diameter bars located at the extremities or in boundary elements of a wall, and for the secondary, usually smaller, vertical bars, normally placed in the web portion of walls.

C10.5.3.1 In conformity with generally accepted practice the minimum steel content in any part of an earthquake resisting wall should not be less than $0.7/f_y$ of the area of that part. By a similarity to columns, the longitudinal steel should not exceed $16/f_y$ of the cross-sectional area of the wall in which it is placed. For example, at the left hand wall return shown in fig. C10.3 this requirement means that when f_y = 380 MPa

$\rho_\ell = 2A_b/bs_v \leqslant 16/380 = 0.042$

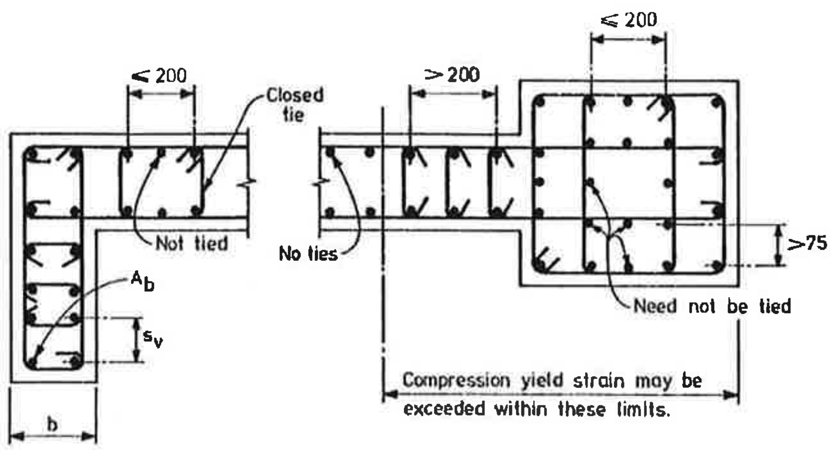


Fig. C10.3 EXAMPLES OF TRANSVERSE REINFORCEMENT IN ACCORDANCE WITH 10.5.4.

C10.5.3.2 A central single layer of reinforcement is permitted only for walls 200 mm or thinner and in which, under the maximum design load, the shear stresses do not exceed $0.3\sqrt{f_c'}$ MPa.

C10.5.3.3 The maximum diameter of bars is restricted to avoid the use of large bars in thin walls.

C10.5.4 *Transverse reinforcement.* Transverse reinforcement, also referred to as lateral reinforcement, is normally placed horizontally in vertical shear walls with the purpose of resisting horizontal shear forces, controlling shrinkage strains or giving restraint to vertical compression bars or confining the concrete in areas of large compression strains. The principles also apply to coupling beams of shear walls.

C10.5.4.1 The requirements with respect to spacing, quantity and bar sizes are the same as for the vertical reinforcement.

C10.5.4.2 The principles of the use for (horizontal) shear reinforcement in walls are the same as those for stirrups in flexural members in accordance with 7.3.14.9. In particular the same principles apply with respect to the allocation of shear resistance to concrete and steel. The region of a shear wall, where the formation of a plastic hinge may affect the shear strength, is normally at its base. This region should be assumed to extend a distance above the critical base section equal to at least the wall length, ℓ_w, or one-sixth of the height of the wall, h_w, whichever is the larger.

C10.5.4.3 This Clause intends to ensure that the principal longitudinal reinforcement, usually placed near the edges of walls, receives adequate lateral support, taking the Bauschinger effect into account, to enable it to be strained beyond compression yield. The requirements extend to areas, both horizontally and vertically, where according to the analysis for the maximum possible earthquake load, yielding of the longitudinal reinforcement could occur. Across a section these areas may be defined with the aid of fig. C10.1 and fig. C10.2 as those where the computed concrete compression strain exceeds 0.0015. The vertical extent of potential yielding is defined in 10.5.5.3. Walls with a single layer of reinforcement, or those containing in the critical flexural compression zone less than $2/f_y$ vertical reinforcement ratio are exempted from these requirements. Such walls are not expected to rely for strength or ductility on the compression strength of the longitudinal reinforcement.

The detailed requirements for tie shapes, tie leg area and spacing, as set out in 10.5.4.3 (a) (b) and (c),are similar to those for potential plastic hinge zone of flexural members as given in 6.5.3.3. The interpretation of these requirements is illustrated in fig. C10.3, which shows a small flange and a typical boundary element, containing the bulk of the longitudinal flexural reinforcement for a shear wall.

It should be noted that the low limit of $2/f_y$ for the longitudinal reinforcement ratio, computed from eq. 10-2, refers to two or more bars near the edges of a wall. For example in fig. C10.3 the local reinforcement ratio near the end of the flange will be $\rho_\ell = 2A_b/bs_v$.

C10.5.4.4 In areas of the wall where no compression yielding is expected but where the computed steel stresses exceed $0.5f_y$ when the wall develops its ideal strength, lateral reinforcement around longitudinal bars needs to satisfy at least the requirements of 6.4.4.2. When walls have a single layer of reinforcement, or the longitudinal bars spaced in two or more layers are such that ρ_ℓ, computed from eq. 10-2, does not exceed $2/f_y$, no lateral tie reinforcement is required to support longitudinal bars against buckling.

C10.5.4.5 This Section makes provision for the confinement of the outer half of the compression zone in the plastic hinge region of the wall, that is, where the compression strain under the governing loading condition may exceed 0.0015. These requirements apply only when the computed neutral axis depth of the section in the potential yield regions exceeds that given by equations 10-3 and 10-4. It is considered that in such a situation the normally assumed concrete compression strain at the extreme fibre of a section may not be sufficient to ensure adequate ductility of the section. Equation 10-3 is a conservative approximation that takes into account the probable plastic hinge length at the base of a cantilever, the height to length ratio of the wall, which influences the curvature ductility demand for a given displacement ductility, and the value of the structural type factors. The neutral axis depth given by eq. 10-3 will give adequate curvature ductility for a displacement ductility of 4 or more at an extreme fibre concrete compression strain of 0.004, for walls with a height to length ratio of 8 or less when $S = 1$, and $\phi_o = 1.4$ provided that the equal tension and compression steel content at each extremity of the section, as computed for normal flexural members, does not exceed 1%. The over-strength factor for cantilever shear walls is defined as

$$\phi_o = \frac{\text{Overstrength moment of resistance}}{\text{Moment resulting from code loading}} = \frac{M^o}{M_{code}}$$

where both moments refer to the base of a cantilever shear wall.

It is recognized that when flexural strength corresponding with structural type factor S larger than unity, and for overstrength factor ϕ_o larger than 1.4 (see C3.5.1.3) is provided, the ductility demand on the section will be less. Correspondingly the critical value or the neutral axis depth c_c increases with the product $\phi_o S^{10.3}$.

When the neutral axis depth is found to exceed that given by eq. 10-3 the more detailed eq. 10-4 may be used, as it will give larger values for c_c when $S > 0.8$ and particularly when the height to length ratio for a cantilever wall is less than 8. The principles of confinement to be used are those relevant to column sections. Equations 10-5 and 10-6 are similar to those given for columns in 6.5.4.3. As the neutral axis depth increases from c_c to $c = 0.8\ell_w$, the amount of confining steel increases to its maximum, as it does in columns when $P_e/f_c' A_g$ increases. This is discussed in C6.5.4.3. The part of the wall section to be confined is that which is subjected to computed strains larger than 0.0015, that is, the outer half of the compression zone, over which the assumed concrete compression stress is $0.85 f_c'$.

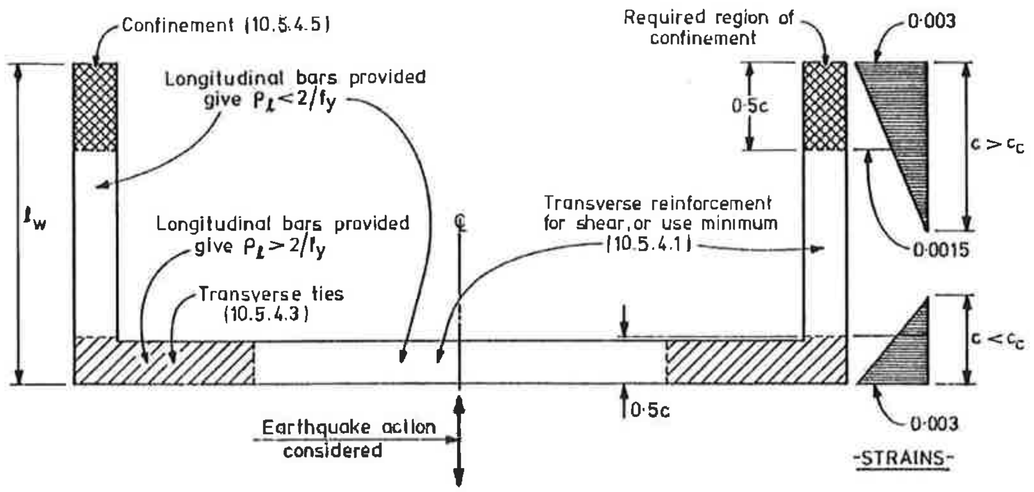


Fig. C10.4 REGIONS OF TRANSVERSE REINFORCEMENT IN ACCORDANCE WITH 10.5.4.

Figure C10.4 shows typical strain patterns that determine the regions in a section where confinement is required and others where transverse reinforcement is required only to provide lateral support to the longitudinal bars.

Walls with a single layer of reinforcement should never be used when the limit of eq. 10-3 is exceeded.

The potential yield region of a wall, behaving as a cantilever, is assumed to be at its base. Outside this region the special requirements for hoops need not be satisfied provided that the designer ensures that yielding of the flexural wall reinforcement will not occur. This may be achieved if the flexural reinforcement is curtailed in accordance with a linear bending moment envelope, such as shown in fig. C3.2, rather than the bending moment diagram derived for the lateral static loading.

C10.5.5 *Shear strength*

C10.5.5.1 Generally the same principles apply as for columns.

C10.5.5.2 The reasons for the shear stress limitations in the end regions of walls, where flexural yielding is expected, are discussed in C7.5.5.2.

C10.5.5.3 For the end regions, where the contribution of the concrete towards shear resistance is to be evaluated from eq. 7-43, modifications in respect of 7.5.3.2 are introduced to take into account the relative dimensions of cantilever or coupled shear walls which may be different from those of columns. The end region normally extends from the level at which the critical base moment can develop in a shear wall.

For reasons outlined for columns in C7.5.4.4, diagonal shear reinforcement in potential plastic hinge regions of shear walls will very seldom be required. Squat shear walls may be an exception, as discussed in C7.5.5.3. The maximum shear stress in shear walls is usually limited by eq. 7-44 rather than by eq. 7-42.

Outside the potential plastic hinge region the full value of v_c, as given in 7.3.14, may be used in determining the horizontal shear reinforcement, provided that no yielding of the flexural reinforcement is likely to occur. This is discussed at the end of C10.5.4.5.

C10.5.6 *Diaphragms*

C10.5.6.1 In the seismic design of buildings generally two types of diaphragm actions are encountered[10.7]. The first type of action occurs at every floor, where the floor slab, acting as a diaphragm transmits horizontal inertia forces to the different lateral load resisting components, such as frames or shear walls. Unless the shear strength of the floor slab is deliberately reduced, for example by joints or large openings, earthquake induced forces in such a situation will seldom be critical. The second type of action is encountered when, at a particular floor, large shear forces need to be transferred from one vertical lateral load resisting component, such as a shear core, to another, such as peripheral foundation walls. In such a situation diaphragm action in the affected floor is often critical.

As a general rule diaphragms should not be required to dissipate seismic energy. For this reason they should be designed to resist, without exceeding the dependable strength, forces which are required to develop the primary energy dissipating mechanisms in frames or shear walls. Capacity design procedures may be used in lieu of the provisions of NZS 4203 to estimate the maximum forces at each floor which might be required to develop the overstrength of the primary lateral load resisting systems.

Both flexure and shear should be considered when reinforcement is provided for diaphragm action.

C10.5.6.2 Diaphragms must be reinforced in both principal directions satisfying at least the requirements for minimum slab reinforcement.

C10.5.6.3 When capacity design procedures are used to estimate the maximum forces in diaphragms, or strength design corresponding with the requirements of NZS 4203 is used, no special detailing requirements apply. When yielding in a diaphragm cannot be avoided appropriate precautions in detailing must be taken to ensure that the intended forces can be transmitted, taking into account also the effects of reversed cyclic loading.

C10.5.6.4 When the primary lateral load resisting components are not expected to yield, or when capacity design procedures are inappropriate for the evaluation of critical diaphragm actions, the principles applicable to structures with limited ductility, as provided in 14.9, should be used.

C10.5.6.5 Joints, including construction joints, may reduce the ability of the diaphragm to transfer shear by shear friction. In evaluating shear stresses only the effective interface area at the weakest section should be considered.

C10.5.6.6 When the floor construction consists of precast elements, it is essential that continuity in shear transfer over the entire floor is assured. To this end a cast-in-place reinforced concrete topping slab, at least 50 mm thick, with at least the minimum required slab reinforcement, may be placed directly on precast elements. However, to ensure adequate thickness over elements with large camber, and adequate cover over reinforcement where lapped splices occur, 65 mm minimum thickness should be preferred. For composite action of the precast and cast-in-place parts of the finished slab, satisfactory bond between the two components is essential. This is expected to ensure the stability of the topping in transferring in-plane diaphragm forces.

When significant seismically induced shear forces are to be transferred by the cast-in-place topping alone, as measured by the shear stress in the topping in excess of $0.3\sqrt{f'_c}$, some doubt exists as to the effectiveness of bond in preventing the separation of the topping from the precast element in the region of the joints, where vertical movements during the inelastic seismic motions of the building are also possible. For this reason positive connection is required to prevent buckling of the topping slab under adverse seismic conditions. Embedment of connectors into mortared or grouted joints between precast elements, without engaging some transverse reinforcement projecting from such elements, should generally not be considered as effective anchorage. The recommendations are based on judgement rather than on experience of experiments.

C10.5.7 *Walls and diaphragms with openings*

C10.5.7.1 Cantilever shear walls and diaphragms shall be subject to rational analysis to ensure that no unintended weaknesses or brittle elements in potentially yielding regions can result from a particular arrangement of openings.

C10.5.7.2 When vertical rows of openings separate walls, coupled shear walls result. The connecting elements between openings need to be very ductile to ensure adequate overall ductility. To control shear displacements and to prevent sliding shear failures such coupling elements should be reinforced with diagonal reinforcement unless they are relatively shallow[10.3]. Beams with the same clear span to depth ratio may have to develop considerably larger ductilities in coupled shear walls than in ordinary frames. When the maximum shear stress is very small or the span to depth ratio of the beam becomes large, as governed by eq. 10-7, conventional beam flexural and shear reinforcement, in accordance with Sections 6 and 7, may be used. For example a beam with ℓ_n/h = 4 may be reinforced with stirrups if the earthquake induced reversible shear is less than $0.4\sqrt{f'_c}$. However, for such shear stresses some diagonal reinforcement will be required in the potential plastic hinge zones in accordance with C7.5.4.2, therefore it is likely to be more practical to use diagonal reinforcement for the entire earthquake load on the coupling beam also in the case when $\ell_n/h = 4$.

To enable diagonal bars, arranged in cages, to be fully effective in compression, the buckling of each bar, particularly at right angles to the beam or diaphragm, must be prevented.

Where coupling beams are monolithic with a floor slab it may be necessary to evaluate the contribution of the effective slab reinforcement, placed parallel with the coupled walls, to the overstrength that may be developed in such beams.

C10.5.8 *Special splice and anchorage requirements*

C10.5.8.1 Because a large quantity of flexural tension wall reinforcement may have to be carried up several storeys, some splicing in potential plastic hinge regions may be unavoidable. Such splices must be staggered so that not more than every third bar is spliced at the same level in a potential plastic hinge zone, defined in C10.5.4.2. In any other area, where, as a result of satisfying the curtailment recommendations of C3.5.7.3 and fig. C3.2, yielding at the splice is not expected, there are no splicing restrictions.

C10.5.8.2 Lateral ties should surround lapped spliced bars larger than 16 mm as shown in fig. C10.5. The area of such a tie, A_{tr}, must be computed in accordance with 5.5.1.2. The spacing of such ties must not exceed 10 d_b and the first tie should be as close to the end of the splice as possible.

C10.5.8.4 When diagonal or horizontal bars in a coupling beam are anchored in adjacent structural walls, the development length must be increased. This is in consideration of the likely adverse effect of reversed cyclic loading on the anchorage of a group of bars.

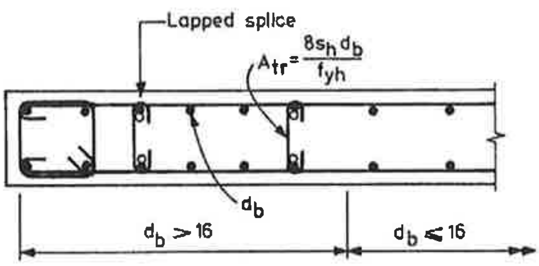


Fig. C10.5 TIES REQUIRED AT LAPPED BAR SPLICES

REFERENCES

10.1 Oberlander, G.D. and Everard, N.J., "Investigation of Reinforced Concrete Walls", ACI Journal, Proceedings V. 74, No. 6, June 1977, pp. 256-263.

10.2 Kriparanayanan, K.N., "Interesting Aspects of the Empirical Wall Design Equation", ACI Journal Proceedings, V. 74, No. 5, May 1977, pp. 204-207.

10.3 Paulay, T. and Williams, R.L., "The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, 1980, pp. 108-143.

10.4 Robinson, L., "Shear Walls of Limited Ductility", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, 1980, pp. 144-161.

10.5 Oesterle, R.G., Fiorato, A.E., Johal, L.S., Carpenter, J.E., Russell, H.G. and Corley, W.G., "Earthquake Resistant Structural Walls – Tests of Isolated Walls", Report to National Science Foundation, Portland Cement Association, Nov. 1976, 44 pp. (Appendix A, 38 pp.; Appendix B, 233 pp.).

10.6 Bertero, V.V., Popov, E.P., Wang, T.Y. and Vallenas, J., "Seismic Design Implications of Hysteretic Behaviour of Reinforced Concrete Structural Walls", 6th World Conference on Earthquake Engineering, New Delhi, 1977, Preprints 5, pp. 159-165.

10.7 Kolston, D. and Buchanan, B.W., "Diaphragms in Seismic Resistant Buildings", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, 1980, pp. 162-170.

COMMENTARY

C11 TWO-WAY SLAB SYSTEMS

C11.2 Scope

C11.2.1 to C11.2.4 The fundamental design principles contained in Section 11 are applicable to all planar structural systems subjected to transverse loads. However, some of the specific design rules, as well as historical precedents, limit the types of structures to which Section 11 is applicable. General characteristics of slab systems which may be designed according to Section 11 are described in this Clause. These systems include "flat slabs", "flat plates", "two-way slabs", and "waffle slabs". True "one-way slabs", slabs reinforced to resist flexural stresses in only one direction, are excluded. Also excluded are soil supported slabs, such as "slabs on grade" which do not transmit vertical loads from other parts of the structure to the soil.

For slabs supported on walls, the design procedures in this Section envisage the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel. Wall-like columns less than a full panel length can be treated as columns.

Rectangular or square panels supported by walls or relatively stiff beams on two opposite sides may be designed as one-way slabs, that is, as beam strips spanning in the direction perpendicular to the supports. When slabs are designed as one-way slabs, the designer must realize that true one-way action will exist only if the loads are uniformly distributed in the direction parallel to the supports and if the edges of the panel perpendicular to those supported on walls or stiff beams are themselves completely unsupported. If either of these conditions is not satisfied, transverse moments will exist and should be provided for in order to prevent the formation of large cracks and to provide adequate transverse distribution of non-uniform loads.

C11.2.6 The provisions of Section 11 apply only to reinforced concrete floor systems.

C11.2.7 The provisions of Section 11 do not apply to multi-storey flat plate or flat slab buildings which are used as seismic resistant structures, unless frames involving beams and columns or walls, or a combination of these components, are present to provide most of the strength and stiffness required to resist horizontal seismic forces. Without such additional strengthening and stiffening elements it is doubtful whether the structure would have sufficient ductility at the critical slab-column connections to withstand a major earthquake, and also considerable inter-storey deflections may occur due to the flexibility of the structure. It is emphasized that the equivalent frame method was not intended to be used in seismic design where lateral loads are significant.

C11.3 Design procedures

C11.3.1 *General.* This Clause permits a designer to base a design directly on fundamental principles of structural mechanics, provided he can demonstrate explicitly that all safety and serviceability criteria are satisfied.

C11.3.2 *Design methods.* This Clause lists methods of determination of the design moments and shears which have been proved to be acceptable.

C11.3.3 *Design for flexure.* Where the strength method of design is used to proportion sections, the factored load should be as given by the appropriate loadings code. Where the alternative design method is used to proportion sections the service load should be as given by the appropriate loadings code.

C11.3.4 *Effective area of concentrated loads.* The spread of the concentrated load is assumed to project at 45° from the contact area through any fill or surface material and at a reduced rate through the slab as shown in fig. C11.1. The area defined by equations 11-1 and 11-2, and shown in fig. C11.1, should be assumed to be uniformly loaded. Some comments on the elastic theory analysis of slabs with concentrated loads are made in C11.5.

C11.3.5 *Moment transfer between slab and column.* This Clause is concerned primarily with slab systems without beams. Unbalanced bending moments, to be transferred between slab and column, may arise from unequal slab spans, pattern loading and any lateral displacement the floor may need to sustain. A special analysis from first principles, such as a beam analogy, may be used to analyse the strength of the connection [11.1]. The alternative procedure recommended in lieu of a special analysis is conservative and is based on tests and experience. In this alternative procedure all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are 1.5 times the slab or drop panel thickness, 1.5 h, on each side of the column. The calculated shear stresses in the slab around the column must conform to the requirements of 7.3.16. See C7.3.16 for more details on application of this Clause.

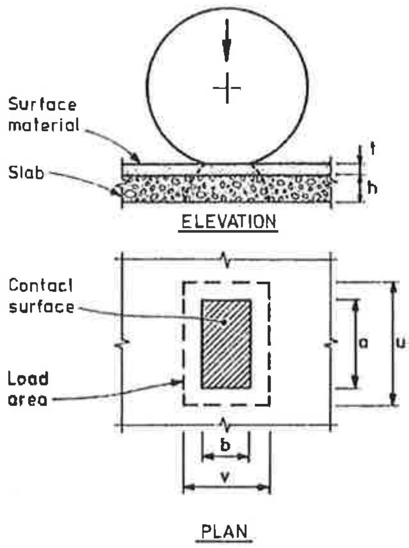


Fig. C11.1 SPREAD OF CONCENTRATED LOAD THROUGH FILL OR SURFACE MATERIAL AND SLAB

C11.3.7 *Openings in slabs.* C7.3.15.5 gives some details on openings in slab systems.

C11.4 Slab reinforcement

C11.4.2 The requirement that the centre-to-centre spacing of the principal reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to that in joists or waffle slabs. This limitation is intended to ensure slab action and reduce cracking and to provide for the possibility of loads concentrated on small areas of the slab.

C11.4.3, C11.4.5. Bending moments in slabs at spandrel beams can be subject to great variation. If spandrel beams are monolithic with walls, the slab approaches complete fixity. Without an integral wall, the slab could be largely simply supported dependent on the torsional rigidity of the spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

C11.4.6 Torsional moments are particularly high in slab corners and top and bottom reinforcement should be present there to control cracking of concrete. A special analysis from first principles may be used to determine the reinforcement necessary to resist torsional moments. One such method of analysis is described in C11.5. Also the special reinforcement in corners required in yield line theory design to prevent fan mechanisms developing in corners can be regarded as providing the necessary resistance to torsional moments. In lieu of such special analyses the special top and bottom reinforcement described in 11.4.6 (a), (b), (c) and (d) is recommended. Reinforcement which is already in the slab corners for other purposes may be considered to be part of this special top and bottom reinforcement.

C11.5 Design moments and shear forces from elastic thin plate theory

General comments. The distribution of moments and shears in slab systems may be calculated on the assumption that the slabs act as thin elastic plates. Such solutions are particularly useful for slabs of unusual shape or boundary conditions where standard solutions such as given by Timoshenko and Woinowsky-Krieger [11.2], Bares [11.3], Hahn [11.4], and others are available. Also the availability of digital computers makes the elastic theory solution of complex floor systems using finite element or finite difference methods possible. A Poisson's ratio of 0.15 is appropriate for concrete slabs, but in many cases with uniform loads sufficient accuracy is achieved if Poisson's ratio is assumed to be zero.

A convenient method for determining the moments induced in slabs from the action of concentrated loads is by the use of influence surfaces, such as those developed by Pucher [11.5]. Graphs derived from this approach for typical bridge slab shapes and for the specific case of standard vehicle wheel loads are presented in a publication of the Ministry of Works and Development [11.6]. The work done by Pucher has been extended substantially by recent work by Homberg [11.7, 11.8] who has derived influence surfaces for continuous or haunched slabs or both. For vehicular loading only the wheel load at the point being considered needs to be represented by a contact area; the more remote wheels may be treated as point loads. For slab and beam bridge construction full edge fixity of a panel should not be assumed when calculating the mid-span moments due to concentrated loads unelss the slab supports are sufficiently rigid to prevent rotation. In particular for slabs on longitudinal beams the transverse positive moment and the longitudinal moments used for design should be the average of those for the fully fixed and simply supported conditions at the beams. To allow for some support rotation and for the localized nature of the peak negative moment, the design transverse negative moment for interior spans may be taken as 0.8 of that for the condition of full fixity at the beams. In addition the moments induced by relative deflections and rotations of the beams should be investigated as these moments can significantly alter the local values calculated from elastic plate theory.

Reinforcement for a general moment field. In the general case of a slab with given loading and boundary conditions the elastic theory solution will provide the bending moments per unit width M_x and M_y in the x and y directions and the torsional moment per unit width M_{xy}. Generally, reinforcing bars are provided at right angles in the x and y directions for these moments because it is impracticable for the bars to follow the directions of the principal moments over the slab. Designers have tended to ignore the torsional moment M_{xy} because of a lack of a method to account for it, but clearly this is unsafe, particularly where twists are high, such as in the corner regions of slabs. The ultimate resisting moments per unit width required for reinforcing bars placed in the x and y directions for a general design moment field, M_x, M_y and M_{xy} (all per unit width) have been derived by Hillerborg [11.9] and Wood [11.10, 11.1]. The design rules for placing reinforcement based on this work can be stated as follows. At a point in a moment field where the moments per unit width are M_x, M_y and M_{xy} (the algebraic values of moments should be used), the reinforcement should be provided in the slab in the x and y directions so that the ultimate resisting moments per unit width in the x and y directions, M_{ux} and M_{uy}, are as follows:

Bottom reinforcement:

Generally $M_{ux} = M_x + |M_{xy}|$

and $M_{uy} = M_y + |M_{xy}|$

If either M_{ux} or M_{uy} is found to be negative, then the negative value of moment is changed to zero and the other moment is given as follows:

Either $M_{ux} = M_x + \left|\dfrac{M_{xy}^2}{M_y}\right|$ with $M_{uy} = 0$

or $M_{uy} = M_y + \left|\dfrac{M_{xy}^2}{M_x}\right|$ with $M_{ux} = 0$

If negative M_{ux} or M_{uy} still occurs, no bottom reinforcement is required. If both M_{ux} and M_{uy} are negative, no bottom reinforcement is required.

Top reinforcement:

Generally $M_{ux} = M_x - |M_{xy}|$

and $M_{uy} = M_y - |M_{xy}|$

If either M_{ux} or M_{uy} is found to be positive, then the positive value of moment is changed to zero and the other moment is given as follows:

$$\text{Either } M_{ux} = M_x - \left|\frac{M_{xy}^2}{M_y}\right| \quad \text{with } M_{uy} = 0$$

$$\text{or } M_{uy} = M_y - \left|\frac{M_{xy}^2}{M_x}\right| \quad \text{with } M_{ux} = 0$$

If positive M_{ux} or M_{uy} still occurs, no top reinforcement is required. If both M_{ux} and M_{uy} are positive, no top reinforcement is required.

C11.6 Design moments and shear forces from limit design theory

General comments. A limit design theory can be used in which regard is given to the redistribution of bending moments which can occur before failure of the slab system. Limit design of slabs has been allowed by the British code of practice for reinforced concrete since 1957. The commonly used limit design methods are yield line theory and the strip method. It should be noted that both of these methods give the flexural strength of the slab and the designer must also check the possibility of shear failure in the case of concentrated loads or reactions. In order to ensure that the sections of the slab are sufficiently ductile to develop the limit bending moment diagram, the tension steel reinforcement ratio ρ used should not exceed 0.4 ρ_b, where ρ_b is the ratio producing balanced conditions as defined by 6.4.1.2.

Yield line theory. The most widely used limit design method for slabs has been the yield line theory due to Johansen. In this method the ultimate load of the slab is determined when the ultimate moment has developed along a system of yield lines (lines of intense cracking across which the reinforcement has yielded) which convert the slab into a collapse mechanism. However, yield line theory is an upper bound limit design approach and therefore the designer should be careful to examine all possible yield line patterns to ensure that the one giving the lowest ultimate load is used, otherwise the strength of the slab may be overestimated. A comparison of test results from a wide range of slabs with predictions by yield line theory demonstrates that yield line theory gives a safe estimate of the ultimate load of slabs provided that the critical yield line pattern is used. In many cases there is a substantial reserve of strength not predicted by the theory which gives added safety. The critical yield line patterns and ultimate load formulae for slabs with various shapes, boundary conditions and loading are available in the literature. The English translation of one of Johansen's publications [11.11] covers a wide variety of slabs. Other references in English by Wood, Jones, Park and Gamble [11.12, 11.13, 11.14, 11.1] give a useful range of design information.

Cut-off points of negative moment steel may be calculated by examining the alternative yield line patterns which could form as a result of the curtailment.

Yield line theory shows a strength reduction due to the formation of fans of yield lines in slab corners which can be significant if top steel is absent. Both top and bottom steel should be present in the corner regions of all slabs. The reinforcement present in the top and bottom should be provided for a distance of 0.2 of the longer span in each direction from the corner and should provide an ultimate positive and negative resisting moment per unit width equal to the maximum positive ultimate moment per unit width in the slab.

The supporting beams of slabs designed by yield line theory may be designed on the basis of the loads transferred to the beams from the adjacent segments of the yield line pattern, except for slabs with one or more unsupported free edges [11.1, 11.14].

Strip method. An alternative limit design method is the strip method due to Hillerborg. This method follows from the lower bound principles for plates which may be stated as follows: "If a distribution of moments can be found which satisfies the slab equilibrium equation and boundary conditions for a given external load, and if the slab is at every point able to resist these moments, then the given external load will represent a lower limit of the carrying capacity of the slab". In the strip method the load carried by torsion in the slab is put equal to zero and the slab is considered as if composed of systems of strips, generally in two directions at right angles, which enables the design bending moments to be calculated by simple statistics involving the equilibrium of the strips. Recent publications in English by Hillerborg, Wood, Armer, Park and Gamble [11.15, 11.16, 11.17, 11.1] give treatments of the strip method. Hillerborg [11.15] also introduces the "advanced strip method" which uses triangular and rectangular elements rather than strips to determine the design moments. Wood, Armer, Park and Gamble [11.16, 11.17, 11.1] also describe the concept of "strong bands" which enables beamless slabs supported on columns, and slabs with re-entrant corners and openings, to be treated more easily than using the corner supported rectangular elements introduced by Hillerborg.

The supporting beams of slabs designed by the strip method may be designed on the basis of the loads transferred to the beams by the adjacent strips or segments.

The strip method is an attractive method since it involves simple concepts that can be relatively easily applied in the general case to obtain moment diagrams which can be used to determine lengths and quantities of reinforcement.

Arrangement of flexural reinforcement. Both upper and lower bound methods allow the designer freedom to choose arrangements of reinforcement which lead to simple detailing. However it cannot be over-emphasized that the arrangements of reinforcement chosen should be such that the resulting distribution of ultimate moments of resistance at the various sections throughout the slab does not differ widely from the distribution of moments given by the elastic theory of thin slabs. If large differences between the distribution of ultimate resisting moments and the elastic moments do exist it may mean that the cracking of concrete at the service load is excessive because low steel ratios at highly stressed sections may lead to high steel stresses and hence to large crack widths. Such regions of high steel stress at service load may also result in large deflections. Hence it is important that the designer should keep a feel for the elastic theory distribution of bending moments and use it to help decide the ratios of negative to positive ultimate resisting moments and ratios of the ultimate resisting moments to be used in the two directions. Just how far the reinforcement arrangement can differ from the elastic theory bending moments and still result in a serviceable slab has not been conclusively determined but the tests which have been carried out do indicate that sensible arrangements of steel result in serviceable slabs. It is recommended that ratios of negative to positive ultimate moments of resistance per unit width between 1 and 2 should be used with some account being taken of the degree of restraint at the edges. For example, if full restraint against rotation is anticipated, a value in the range 1.5 to 2.0 could be used, but if some rotation is expected a value in the range of 1.0 to 1.5 would be more appropriate. Ratios of the ultimate moments of resistance per unit width in the two directions should take account of the direction of maximum elastic bending moment. For example, in two-way slabs the greatest ultimate resisting moment per unit width should act in the direction of the short span. At edges which have been considered as simply supported, care should be taken to provide top reinforcement to control cracking due to fortuitous restraining moments. Such reinforcement should be for approximately 0.33 to 0.5 of the maximum positive ultimate moment of resistance per unit width.

Serviceability checks. Checks of deflections are discussed in detail in Section 4. Excessive cracking should not be a problem providing a reasonable arrangement of reinforcement is used as discussed previously. In cases of concern, maximum crack widths can be estimated by the equation given in 4.4.2.2 and checked against allowable values. The steel stress at service load is required for such a check. This stress can be estimated or, in cases where the distribution of ultimate resisting moments shows significant deviations from the elastic theory bending moments, elastic theory can be used to obtain a more accurate estimate of the steel stress at service load.

C11.7 Moment coefficients and loads on supporting beams for uniformly loaded two-way rectangular slabs supported on four sides

C11.7.1 *Moment coefficients.* The bending moment coefficients for uniformly loaded slabs given in table 11.1 are intended to apply to slabs built monolithically with walls or beams at all four sides. The moment coefficients are as given in CP 110:1972 *The structural use of concrete.* The moment coefficients are those derived using yield line theory for uniformly spread steel by Taylor, *et al* [11.18] multiplied by 1.33 to allow for steel distribution over the middle strip only as defined in fig. 11.1. Simple yield line theory was used in which the yield lines extended into the corners of the slab. The effect of top reinforcement at discontinuous edges was not taken into account in the strength calculations but top steel has been provided there for crack control. The ratios between the negative and positive resisting moments and the resisting moments in the two directions were kept to approximately the same values as in the long standing method due to Westergaard which was based on elastic theory supplemented by experimental data. However the magnitudes of the moment coefficients recommended in 11.7.1 lead to less fluctuation in the actual load factor available than previous methods.

C11.7.2 *Loads on supporting beams.* The bending moments and shear forces in the supporting beams due to uniform loading on the slabs may be determined using the approximate distribution of loading shown in fig. 11.2.

C11.8 Direct design method for uniformly loaded slab system with rectangular panels with or without supporting beams or walls

C11.8.1 *General.* The direct design method is based on analysis and on experimental results from an extensive series of tests on slab systems under gravity loading conducted in the United States [11.19] to [11.27].

The direct design method consists of a set of rules for the proportioning of slab and beam sections to resist flexural stresses. The rules have been developed to satisfy the safety requirements and most of the serviceability requirements simultaneously.

The direct design method involves three fundamental steps, as follows:

(1) Determination of the total factored static moment to design moment. (See 11.8.5.)

(2) Distribution of the total factored static moments to negative and positive moment sections. (See 11.8.6.)

(3) Distribution of the negative and positive factored moments to the column and middle strips and to the beams, if any. (See 11.8.7 to 11.8.10.)

C11.8.2 *Definitions*

C11.8.2.3 A panel, by definition, includes all flexural elements between column centre lines. Thus, the column strip includes the beam, if any.

C11.8.2.4 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule in this Clause are provided in fig. C11.2.

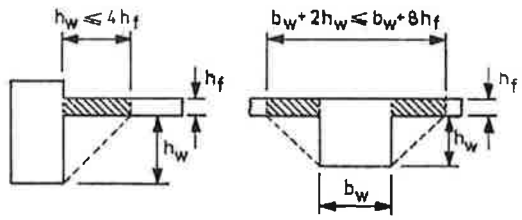


Fig. C11.2 EXAMPLES OF THE PORTION OF SLAB TO BE INCLUDED WITH THE BEAM UNDER 11.8.2.4

C11.8.3 *Limitations*

C11.8.3.1 The direct design method was developed from considerations of theoretical procedures for the determination of moments in slabs with and without beams, requirements for simple design and construction procedures, and precedents supplied by performance of slab systems. Consequently, the slab systems to be designed using the direct design method must conform to the limitations in this Clause.

C11.8.3.2 The primary reason for the limitation in this Clause is the magnitude of the negative moments at the interior support in a structure with only two continuous spans. The rules given for the direct design method assume tacitly that the slab system at the first interior negative moment section is neither fixed against rotation nor discontinuous.

C11.8.3.3 If the ratio of the two spans (long span/short span) of a panel exceeds two, the slab resists the moment in the shorter span essentially as a one-way slab.

C11.8.3.4 The limitation of this Clause is related to the possibility of developing negative moments beyond the point where negative moment reinforcement is terminated as prescribed in fig. 11.3.

C11.8.3.5 The designer is permitted to offset the columns within specified limits from a regular rectangular array. A cumulative total offset of 20% of the span is established as the upper limit.

C11.8.3.6 The direct design method is based on tests for uniform gravity loads and resulting column reactions determined by statics. Horizontal loads (such as wind and seismic) require a frame analysis. Inverted foundation mats designed as two-way slabs involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis is required.

C11.8.3.7 The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the given requirements for stiffness are satisfied.

C11.8.3.8 Moment redistribution as permitted by 3.3.3.4 is not intended where approximate values for bending moments are used. For the direct design method, 10% modification is allowed by 11.8.10.

C11.8.3.10 The designer is permitted to use the direct design method even if the structure does not fit the limitations in this Clause, provided it can be shown by analysis that the particular limitation does not apply to that structure. For example, in the case of a slab system carrying a non-movable load (such as a water reservoir in which the load on all panels is expected to be the same), the designer does not have to satisfy the live load limitation of 11.8.3.6.

C11.8.4 *Slab reinforcement.* The bend bar system is applicable for middle strips where $(d - d' + d_b) \leqslant 0.06\ell_n$ and for column strips where $(d - d' + d_b) \leqslant 0.04\ell_n$ which permits a bend angle of 45°.

Where two-way slabs act as primary members of a laterally unbraced frame resisting lateral loads, the resulting moments due to the combined gravity and lateral loads preclude use of the arbitrary minimum and maximum bends and minimum extension of bars shown in fig. 11.3. Note that the lateral loads applied should not be significant (see C11.8.3.6).

C11.8.5 *Total factored static moment for a span*

C11.8.5.2 Equation 11-7 follows directly from Nichol's derivation [11.28] with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, the designer will find it expedient to calculate static moments for two adjacent half panels, which include a column strip with a half middle strip along each side as shown in fig. C11.3.

C11.8.5.5 If the calculated value of ℓ_n is less than 0.65 ℓ_1, the span is considered as being 0.65 ℓ_1. If a supporting member does not have a rectangular cross-section, it is to be treated as a square support having the same area, as illustrated in fig. C11.4.

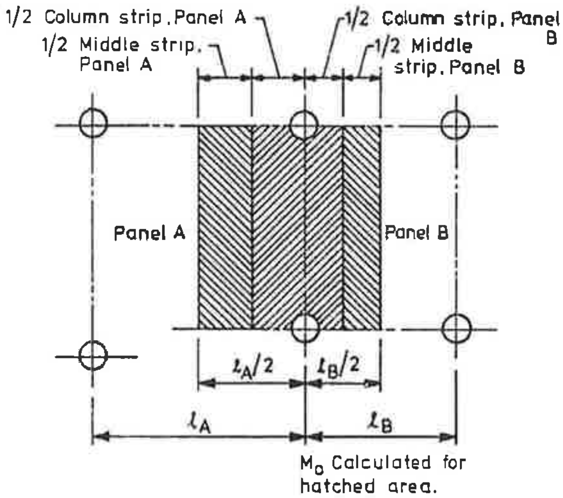


Fig. C11.3 SUGGESTED AREA TO BE CONSIDERED IN CALCULATING STATIC MOMENTS BY EQ. 11-7

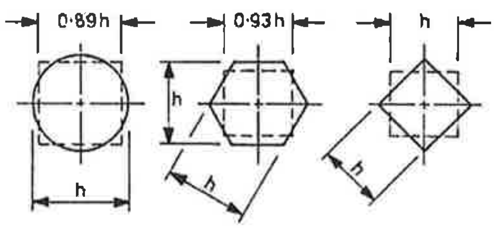


Fig. C11.4 EXAMPLES OF EQUIVALENT SQUARE SECTION FOR NON-RECTANGULAR SUPPORTING MEMBERS

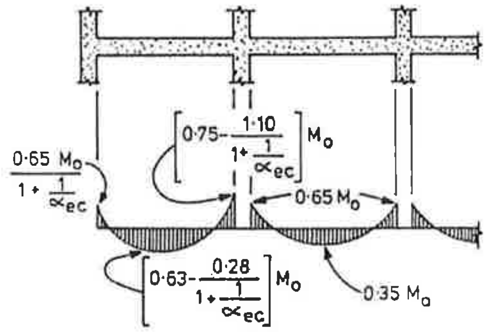


Fig. C11.5 SUMMARY OF RULES FOR DIVIDING THE TOTAL STATIC MOMENT INTO NEGATIVE AND POSITIVE MOMENTS

C11.8.6 *Negative and positive factored moments*

The rules for assigning the total factored static moment to the negative and positive factored moments are summarized in fig. C11.5. The proportions are based on three-dimensional analytical studies of elastic distribution of moments in various slab configurations tempered by the distributions of moments that have been in use.

C11.8.6.3 The term α_{ec} in the equations of this Clause is the flexural stiffness of an equivalent exterior column K_{ec} relative to the flexural stiffness of the slab and the beams, if any. The calculation of K_{ec} is described in 11.9.4 and 11.9.5 and rules are given for the calculation of K_c and K_s in connection with the equivalent frame method. Since the use of the direct design method is limited by the requirements of 11.8.3, it is permissible to make certain simplifications in the calculation of α_{ec} for use in 11.8.6.3 as follows:

(a) The stiffness of the slab-beam K_s may be calculated using a uniform cross-section between column centre lines, disregarding the requirement of 11.9.3.3, and the increase in column stiffness K_c provided by the capital may be neglected, disregarding the requirement of 11.9.4.4. Since both of these simplifications lead to less stiff elements, one should not be made without the other in order to minimize the change in relative stiffness

(b) The requirement in 11.9.5.4 may be waived in calculations of α_{ec} for use with the direct design method

(c) If a corner column is the same size as an adjacent exterior column, the value of α_{ec} computed for the adjacent exterior column may be used for the corner column. This approximation will usually lead to somewhat smaller exterior negative factored moments and somewhat larger positive and interior negative factored moments than the more rigorous analysis required for the equivalent frame method.

The detailing of the reinforcement transferring the moment from the slab to the exterior column is critical to both the performance and the safety of flat slabs without edge beams or equivalent cantilever slabs. This reinforcement must be placed in accordance with 11.3.5.

C11.8.6.4 The differences in slab moment on either side of a column or other type of support must be accounted for in the design of the support. If an analysis is made to distribute unbalanced moments, flexural stiffness may be obtained on the basis of the gross concrete section of the members involved.

C11.8.6.5 Moments perpendicular to, and at the edge of, the slab structure must be transmitted to the supporting columns or walls. Torsional stresses caused by the moment assigned to the slab must be investigated.

C11.8.7, C11.8.8 and C11.8.9 *Factored moments in column strips, beams, middle strips.* The rules given for assigning moments to the column strips, beams, and middle strips are based on studies of moments in linearly elastic slabs with different beam stiffness [11.29] tempered by the moment coefficients that have been used successfully in the past.

Where walls are used as supports along column lines, they can be regarded as very stiff beams with an $\alpha_1\,\ell_2/\ell_1$ value greater than one. Where the exterior support consists of a wall perpendicular to the direction in which moments are being determined, β_t may be taken as zero if the wall is of masonry without torsional resistance and β_t may be taken as 2.5 for a concrete wall with great torsional resistance which is monolithic with the slab.

For the purpose of establishing moments in the half column strip adjacent to an edge supported by a wall, ℓ_n in eq. 11-7 may be assumed equal to ℓ_n of the parallel adjacent column to column span, and the wall may be considered as a beam having a moment of inertia I_b equal to infinity.

C11.8.7.2 The effect of the torsional stiffness parameter β_t is to assign all of the exterior negative factored moment to the column strip, and none to the middle strip, unless the beam torsional stiffness is high relative to the flexural stiffness of the supported slab. In the definition of β_t the shear modulus has been taken as $E_{cb}/2$.

C11.8.11 *Factored shear in slab system with beams.* The tributary area for computing shear on an interior beam is shown shaded in fig. C11.6. If the stiffness for the beam $\alpha_1\,\ell_2/\ell_1$ is less than one, the shear on the beam may be obtained by linear interpolation. In such cases, the beams framing into the column will not account for all the shear force applied on the column. The remaining shear force will produce shear stresses in the slab around the column which must be checked in the same manner as for flat slabs, as required by 11.8.11.4. Clauses 11.8.11.1 to 11.8.11.3 do not apply to the calculation of torsional moments on the beams. These moments must be based on the calculated flexural moments acting on the sides of the beam.

C11.8.12 *Factored moments in columns and walls.* Equation 11-8 refers to two adjoining spans, with one span longer than the other, with full dead load plus one-half live load applied on the longer span and only dead load applied on the shorter span. The term α_{ec} refers to the flexural stiffness of the columns between the two spans.

In the calculation of α_{ec} it is permissible to make the simplifications given as items (a) and (b) in the commentary on 11.8.6.3. In addition, when applying eq. 11-8 to deter-

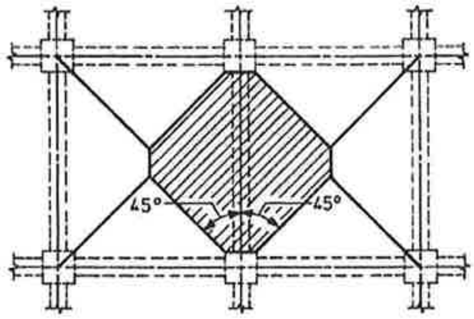


Fig. C11.6 TRIBUTARY AREA FOR SHEAR ON AN INTERIOR BEAM

mine the moment in an exterior column in a direction parallel to the edge of the panel, it is conservative, and therefore permissible, to use in eq. 11-8 the value of α_{ec} computed for the adjacent interior column, provided the columns are of the same size.

C11.8.13 *Provisions for effects of pattern loadings*. The requirements in this Clause limit the possible increases in moment as a result of pattern loadings at the service load level and are based on analytical and experimental studies summarized in reference [11.30]. Values of the column flexural stiffness $\alpha_{min.}$, required to limit the increase in bending moment caused by pattern loads to less than 33%, are listed in table 11.2 as a ratio of the flexural stiffness of the slab system. If the columns of a particular structure do not satisfy the required value of $\alpha_{min.}$, the positive moment in the slab must be increased in accordance with eq. 11-9.

When applying eq. 11-9 to moments in the half column strip parallel to an exterior panel edge, it is conservative, and therefore permissible, to use in eq. 11-9 the value of α_c computed for the adjacent interior column, provided the columns are of the same size.

C11.9 Equivalent frame method for uniformly loaded slab systems with rectangular panels with or without beams

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames which are then analysed for loads acting in the plane of the frames. The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with 11.8.7 (column strips), 11.8.8 (beams) and 11.8.9 (middle strips).

The equivalent frame method is comparable to the "elastic analysis" for flat slabs of early ACI Building Codes. On the basis of studies reported in references [11.31, 11.32] and [11.33], the equivalent frame method has been devised to provide a better representation in two dimensions of a three dimensional system through the strategem of defining flexural stiffnesses which reflect the torsional rotations possible in the three-dimensional system.

Application of the equivalent frame to a regular structure is illustrated in fig. C11.7. The three-dimensional building is divided into a series of two-dimensional frame bents (equivalent frames) centred on column or support centre lines with each frame extending the full height of the building. The width of each equivalent frame is bounded by the centre lines of the adjacent panels. The complete analysis of a slab system for a building consists of analysing a series of equivalent (interior and exterior) frames spanning longitudinally and transversely through the building.

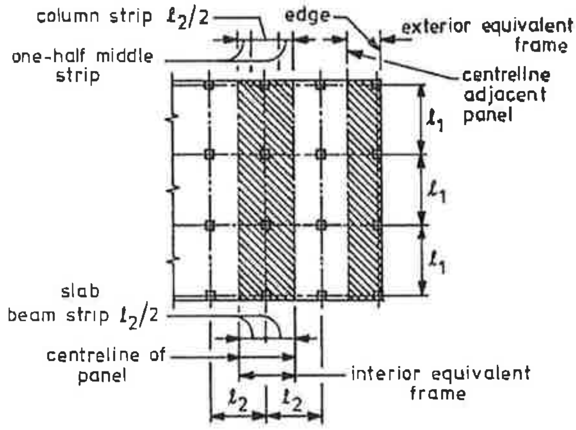


Fig. C11.7 DEFINITIONS OF EQUIVALENT FRAME

Applications of the frame definitions given in 11.9.2 are illustrated in fig. C11.7. The shaded areas represent one interior and one exterior frame taken transversely through the building. Note that the exterior frame extends from the edge of the building to the centre line of the adjacent panel. Similar frames are taken longitudinally through the building for design of the slab system in the longitudinal direction.

The equivalent frame comprises three parts: (1) the horizontal slab strip, including any beams spanning in the direction of the frame, (2) the columns or other vertical supporting members, extending above and below the slab and, (3) the elements of the structure that provide moment transfer between the horizontal and vertical members. If the vertical members are walls extending over the full width of the slab strip, as in fig. C11.8(a), the moment transfer connection is 100% effective and the equivalent frame can be treated as a conventional plane frame. At the other extreme, if the support is a column connected to a slab strip only at its edge, as in fig. C11.8(c), the efficiency of the moment transfer connection approaches zero. For intermediate cases, as in fig. C11.8(b), the "flexibility" of the moment-transfer connection is taken into account by utilizing the reduced equivalent column stiffness, K_{ec}, (see 11.9.4.2) in the conventional frame analysis for moments due to *gravity* loads. Corresponding procedures may be applied for moments generated in the columns due to lateral displacement of the structure.

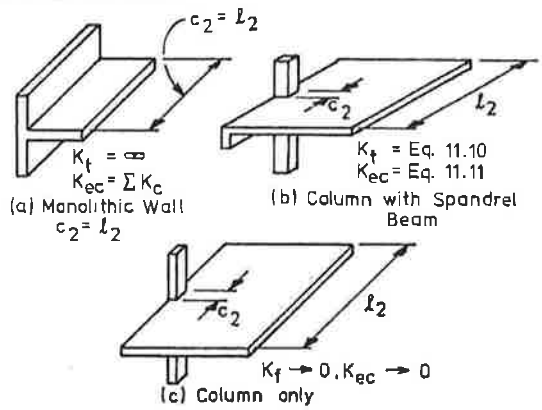
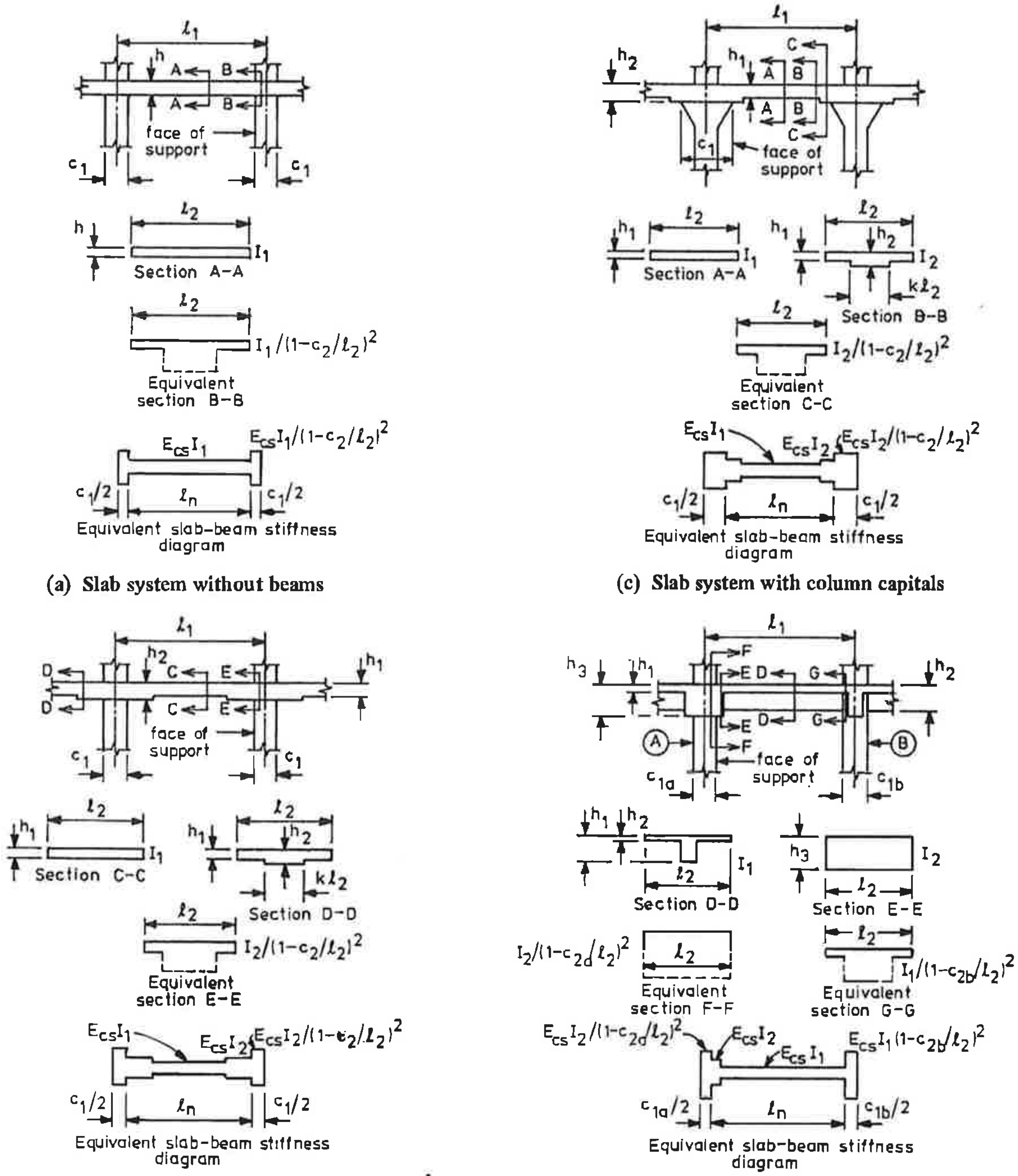


Fig. C11.8 SIMPLIFIED PHYSICAL MODELS ILLUSTRATING THE INTENT OF CLAUSE 11.9.2

C11.9.3 *Slab-beams.* Common types of slab systems with and without beams between supports are illustrated in fig. C11.9. Cross-sections for determining the stiffness of the slab-beam members, K_{sb}, between support centre lines are shown for each type. The equivalent slab-beam stiffness diagrams may be used to determine moment distribution constants and fixed-end moments for the equivalent frame analysis. Make-up of the various areas for stiffness calculation is based on the following considerations:

(a) The moment of inertia of the slab-beam between face of supports may be based on the gross cross-sectional area of the concrete. Variation in the moment of inertia along the axis of the slab-beam between supports is taken into account (see 11.9.3.1 and 11.9.3.2)

(b) A support is defined as a column, capital, bracket or wall. Note that a beam is not considered a support member for the equivalent frame (see 11.9.3.3)



(a) Slab system without beams

(c) Slab system with column capitals

(b) Slab system with drop panels

(d) Slab system with beams

Fig. C11.9 SECTIONS FOR CALCULATING SLAB-BEAM STIFFNESS, K_{sb}

(c) The moment of inertia of the slab-beam from the face of support to the centre line of support is assumed equal to the amount of inertia of the slab-beam at the face of support divided by the quantity $(1 - c_2/\ell_2)^2$ (see 11.9.3.3).

Clause 11.9.3.3 stipulates a finite moment of inertia for the slab-beam from the face of the column or capital to the centre line of the support to account for the flexibility of the slab on the "sides" of the column in that portion of the span. The magnification factor, $(1 - c_2/\ell_2)^2$, applied to the moment of inertia between support face and centre line, in effect, makes each slab-beam at least a haunched member within its length. Consequently, stiffness and carry-over factors and fixed-end moments based on the usual assumptions of uniform prismatic members cannot be applied to the slab-beam members. To aid in design, a list of stiffness and carry-over factors and fixed-end moment coefficients for four common types of slab systems are tabulated in tables C11.1 to C11.4 [11.34, 11.35].

C11.9.4 *Equivalent columns.* This Clause modifies the column flexural stiffness to account for the torsional flexibility of the slab-to-column connection which reduces its efficiency for transmission of moments. The intent of this Clause is illustrated by the simplified physical model in fig. C11.10 which represents a column *AB*, extending above and below the slab, with a portion of the slab *CD* attached thereto. A moment *M* applied along *CD* will cause a torsional rotation of the "cross beam" CD as well as a flexural rotation of the column. Thus, the rotational restraint on the slab-beam which spans in a direction perpendicular to *AB* and *CD*, depends on both the torsional rotation of *CD* and the flexural rotation of *AB*. The overall flexibility, $1/K_{ec}$, of the composite element shown in fig. C11.10 is assumed to be the sum of the flexibility of the columns, $1/\Sigma K_c$, and the torsional flexibility of the "beam", $1/K_t$.

An equivalent column as defined by 11.9.4.1 is illustrated in fig.C11.11. The equivalent column consists of the actual columns above and below the slab-beam plus attached torsional members on each side of the columns extending to the centre line of the adjacent panels. Note that for an exterior frame the attached torsional member is located to one side only. The presence of parallel beams will also influence the stiffness of the equivalent column.

For gravity loads only, the flexural stiffness of the equivalent column K_{ec} is given in terms of its inverse or flexibility by eq. 11-10. For purposes of computation, the designer may prefer that eq. 11-10 be given in terms of stiffness directly as follows:

$$K_{ec} = \Sigma K_c / \left(1 + \frac{\Sigma K_c}{K_t}\right)$$

The stiffness K_c is based on the length of the column from mid-depth of slab above to mid-depth of slab below and its moment of inertia, which is computed on the basis of its cross-section, taking into account the increase in stiffness provided by the capital, if any. The column is assumed to be infinitely stiff over the depth of the slab.

Common types of column and support conditions for slab systems are illustrated in fig. C11.12. The column stiffness diagrams may be used to determine column flexural

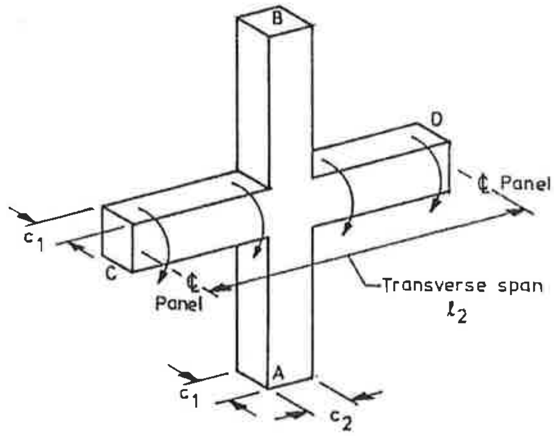


Fig. C11.10 SIMPLIFIED PHYSICAL MODEL ILLUSTRATING THE INTENT OF CLAUSE 11.9.4

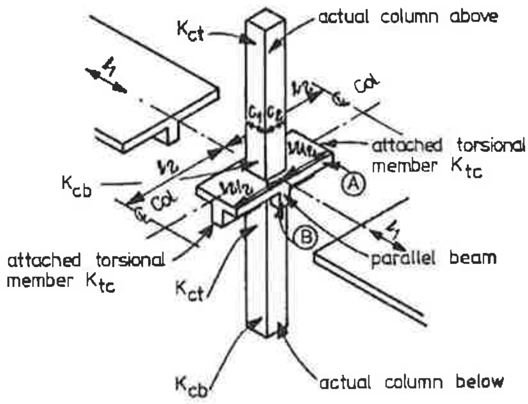


Fig. C11.11 EQUIVALENT COLUMN

stiffness, K_c. Make-up of the stiffness diagrams is based on the following considerations:

(a) The moment of inertia of the column outside the slab-beam joint is based on the gross cross-sectional area of the concrete. Variation in the moment of inertia along the axis of the column between slab-beam joints is taken into account. For columns with capitals, the moment of inertia is assumed to vary linearly from the base of the capital to the bottom of the slab-beam [11.33]. (See 11.9.4.3 and 11.9.4.4.)

(b) The moment of inertia is assumed infinite ($I = \infty$) from the top to the bottom of the slab-beam at the joint. As with the slab-beam members, the stiffness factor, K_c, for the columns cannot be based on the assumption of uniform prismatic members. (See 11.9.4.5.)

Increase in column stiffness provided by a capital may be disregarded when using the direct design method.

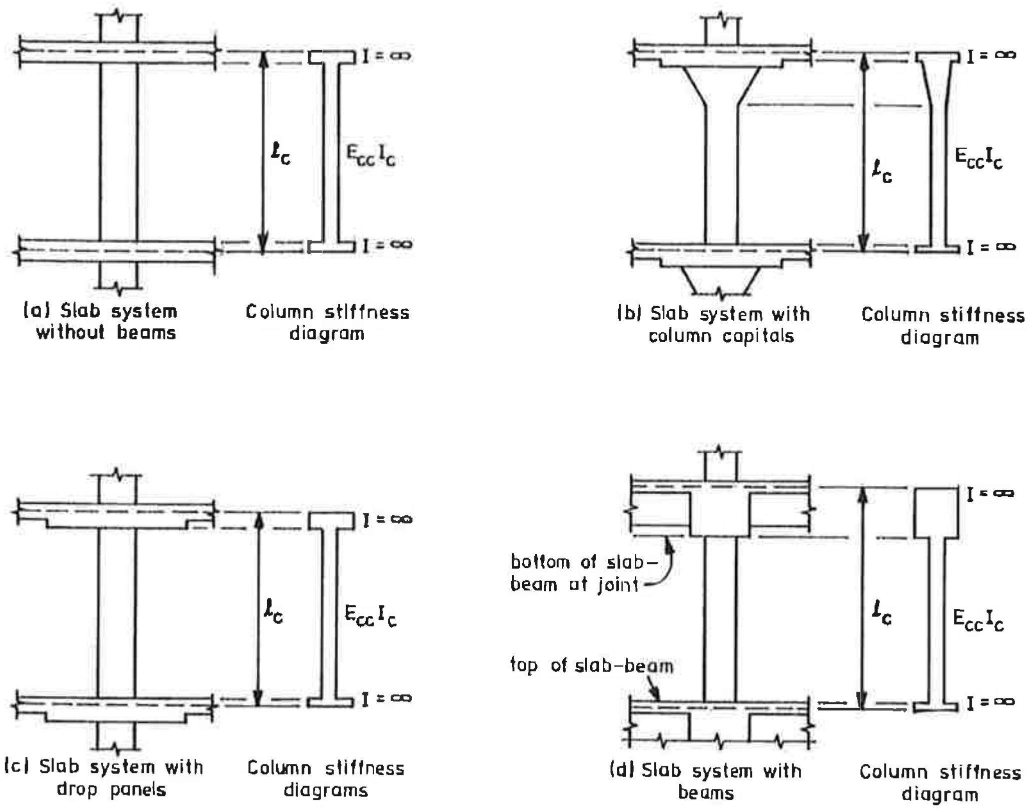


Fig. C11.12 SECTIONS FOR CALCULATING COLUMN STIFFNESS, K_c

Where the exterior edge of a slab system is supported on a concrete wall monolithic with the slab, the flexibility of the wall should replace the column flexibility $1/K_c$ in eq. 11-10 and the torsional flexibility $1/K_t$ of the wall may be assumed to be zero in the same equation. Where the exterior edge of the slab system is supported on an unreinforced masonry wall, K_{ec} may be taken as zero.

C11.9.5 *Attached torsional members.* Computation of the torsional stiffness K_t of the "attached torsional member" requires several simplifying assumptions. If no beam frames into the column, a portion of the slab equal to the width of the column or capital is assumed as the effective beam. If a beam frames into the column, T-beam or L-beam action is assumed, with the flanges extending on each side of the beam a distance equal to the projection of the beam above or below the slab but not greater than four times the thickness of the slab. Furthermore, it is assumed that no torsional rotation occurs in the beam over the width of the support.

Attached torsional members for common slab-beam joints are illustrated in fig. C11.13. The cross-section of a torsional member consists of the larger of the three conditions specified in 11.9.5.1. The governing condition (a), (b) or (c) is indicated below each illustration.

Studies of three-dimensional analyses of various slab configurations suggest that a reasonable value of the torsional stiffness can be obtained by assuming a moment distribution along the beam *CD* in fig. C11.10 that varies linearly from a maximum at the centre of the column to zero at the middle of the panel. The assumed distribution of unit twisting moment along the column centre line, the

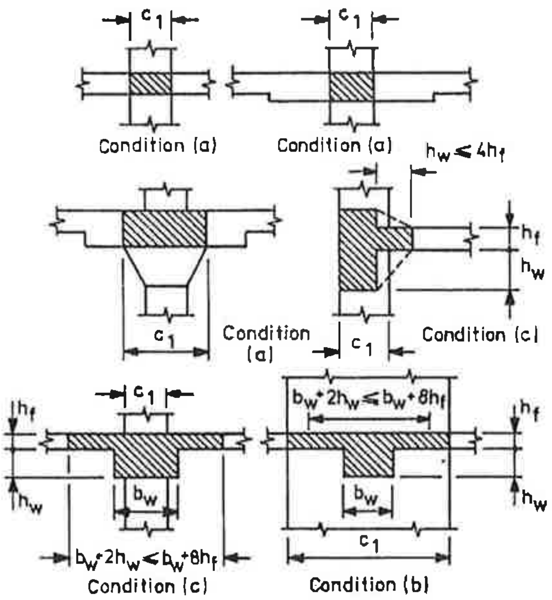


Fig. C11.13 ATTACHED TORSIONAL MEMBERS

twisting moment diagram, and the resulting unit rotation diagram are shown in fig. C11.14.

Equation 11-11 is an approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations. The development of this expression and the assumptions on which it is based, as well as the justification for its use, are discussed in references [11.31], [11.32], and [11.33].

The term C is a property of the cross-section having the same relationship to the torsional rigidity of a non-circular cross-section as does the polar moment of inertia for a circular cross-section. The term

$$(1 - 0.63\frac{x}{y})\,\frac{x^3y}{3}$$

in eq. 11-12 is a conservatively low approximation to the value of C for a rectangular section, assuming elastic behaviour.[11.34] The value of C is computed by dividing the cross-section of the torsional member into separate rectangular parts and summing the C values for each of the component rectangles. Since the value of C obtained by summing the values for each of the component rectangles making up a section will always be less than the theoretically correct value, it is appropriate to subdivide the cross-section in such a way as to result in the highest possible value of C. Application of the C expression is illustrated in fig. C11.15.

If a panel contains a beam parallel to the direction in which moments are being determined, the value of K_t obtained from eq. 11-11 may lead to equivalent column stiffnesses which are too low. In such cases the value of K_t given by eq. 11-11 should be increased as follows:

$$K_{ta} = K_t \frac{I_{sb}}{I_s}$$

where

K_{ta} = increased torsional stiffness due to the parallel beam (note Part B in fig. C11.11)

I_s = moment of inertia of a width of slab equal to the full width between panel centre lines, ℓ_2, excluding that portion of the beam stem extending above and below the slab (note Part A in fig. C11.11)

= $\ell_2 h^3/12$

I_{sb} = moment of inertia of the slab section specified for I_s including that portion of the beam stem extending above and below the slab. (For the parallel beam illustrated in fig. C11.11, I_{sb} is for the full tee section shown.)

After the values of K_c and K_t are determined, the equivalent column stiffness, K_{ec}, is computed. Using fig. C11.11 for illustration:

$$K_{ec} = \frac{K_{ct} + K_{cb}}{\left(1 + \dfrac{K_{ct} + K_{cb}}{K_{ta} + K_{ta}}\right)}$$

where

K_{ct} = flexural stiffness at top of lower column framing into joint

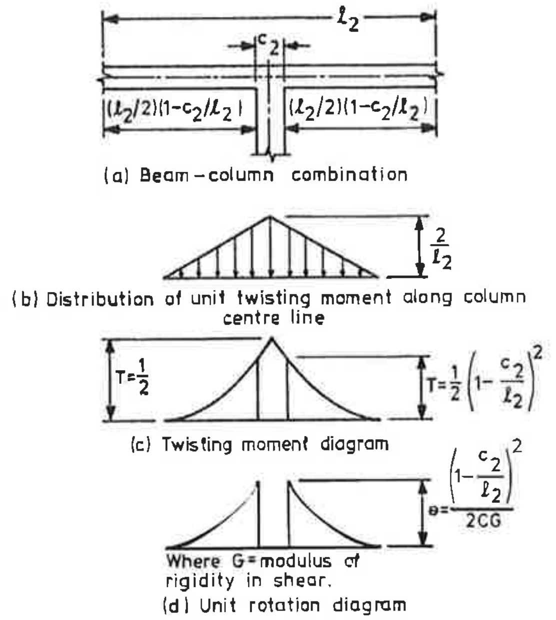


Fig. C11.14 ASSUMED DISTRIBUTION OF UNIT TWISTING MOMENT ALONG COLUMN CENTRE LINE, TWISTING MOMENT DIAGRAM, AND UNIT ROTATION DIAGRAM WITH θ THE MAXIMUM UNIT ROTATION ADJACENT TO COLUMN

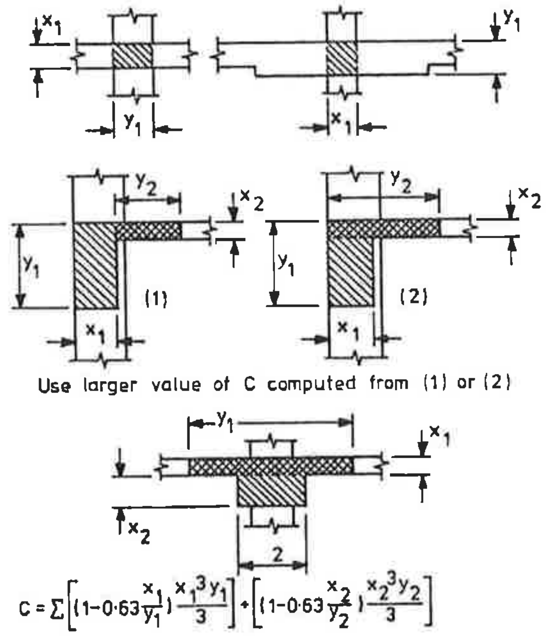


Fig. C11.15 CROSS-SECTIONAL CONSTANT, C, TO DEFINE TORSIONAL PROPERTIES OF ATTACHED TORSIONAL MEMBER

K_{cb} = flexural stiffness at bottom of upper column framing into joint

K_{ta} = torsional stiffness of each torsional member, one on each side of the column, increased due to the parallel beam.

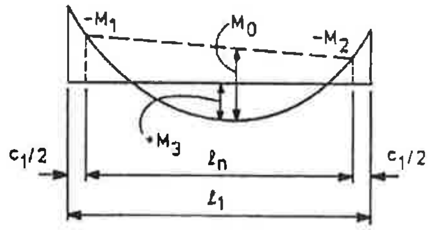
Note that all of the equations regarding the equivalent column stiffness concept are general in nature and are equally applicable to slab systems supported on precast concrete beams or structural steel members as long as there is frame action designed into the beam-column system (a rigid joint) and composite action between slab and beam is also provided for.

C11.9.6 *Arrangement of live load.* The use of only three-quarters of the full factored live load for maximum moment loading patterns is based on the fact that maximum negative and maximum positive live load moments cannot occur simultaneously and that redistribution of maximum moments is thus possible before failure occurs. This procedure, in effect, permits some local overstress under the full factored live load if it is distributed in the prescribed manner, but still insures that the ultimate capacity of the slab system after redistribution of moment is not less than that required to carry the full factored dead and live loads on all panels.

C11.9.7 *Factored moments* [11.35]

C11.9.7.1 to C11.9.7.3 These clauses correct the negative factored moments to the face of the supports. The correction is modified at an exterior support in order not to result in undue reductions in the exterior negative moment. Fig. C11.4 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with non-rectangular supports.

C11.9.7.4 This clause is a holdover from many previous ACI codes and is based on the principle that if two different methods are prescribed to obtain a particular answer,

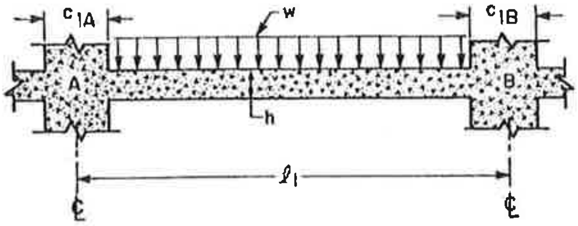


$M_0 = [(M_1 + M_2)/2] + M_3$ need not be greater than $w_u \ell_2 \ell_n^2/8$.
Permissible reduction for moments M_1, M_2 & $M_3 = [w_u \ell_2 \ell_n^2/8]/[(M_1 + M_2)/2 + M_3]$

Fig. C11.16 TOTAL FACTORED STATIC MOMENT FOR A SPAN

the code should not require a value greater than the least acceptable value. Due to the long satisfactory experience with designs having total factored static moments not exceeding those given by eq. 11-7 it is considered that these values are satisfactory for design when applicable limitations are met.

Should a designer choose to use the equivalent frame method to analyze a slab system which meets the limitations of the direct design method, then the factored moments may be reduced so that the total factored static moment (the sum of average negative and positive moments) need not exceed M_o computed by eq. 11-7. This permissible reduction is illustrated in fig. C11.16.

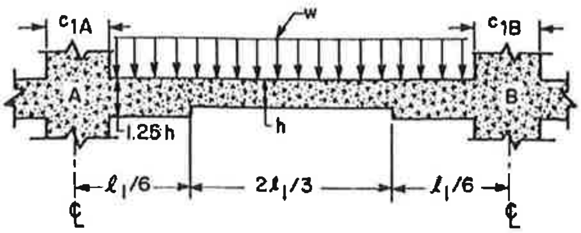


Column dimension		Uniform load FEM = Coef. $(w\ell_2\ell_1^2)$		Stiffness factor†		Carryover factor	
$\frac{c_{1A}}{\ell_1}$	$\frac{c_{1B}}{\ell_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
0.00	0.00	0.083	0.083	4.00	4.00	0.500	0.500
	0.05	0.083	0.084	4.01	4.04	0.504	0.500
	0.10	0.082	0.086	4.03	4.15	0.513	0.499
	0.15	0.081	0.089	4.07	4.32	0.528	0.498
	0.20	0.079	0.093	4.12	4.56	0.548	0.495
	0.25	0.077	0.097	4.18	4.88	0.573	0.491
	0.30	0.075	0.102	4.25	5.28	0.603	0.485
	0.35	0.073	0.107	4.33	5.78	0.638	0.478
0.05	0.05	0.084	0.084	4.05	4.05	0.503	0.503
	0.10	0.083	0.086	4.07	4.15	0.513	0.503
	0.15	0.081	0.089	4.11	4.33	0.528	0.501
	0.20	0.080	0.092	4.16	4.58	0.548	0.499
	0.25	0.078	0.096	4.22	4.89	0.573	0.494
	0.30	0.076	0.101	4.29	5.30	0.603	0.489
	0.35	0.074	0.107	4.37	5.80	0.638	0.481
0.10	0.10	0.085	0.085	4.18	4.18	0.513	0.513
	0.15	0.083	0.088	4.22	4.36	0.528	0.511
	0.20	0.082	0.091	4.27	4.61	0.548	0.508
	0.25	0.080	0.095	4.34	4.93	0.573	0.504
	0.30	0.078	0.100	4.41	5.34	0.602	0.498
	0.35	0.075	0.105	4.50	5.85	0.637	0.491
0.15	0.15	0.086	0.086	4.40	4.40	0.526	0.526
	0.20	0.084	0.090	4.46	4.65	0.546	0.523
	0.25	0.083	0.094	4.53	4.98	0.571	0.519
	0.30	0.080	0.099	4.61	5.40	0.601	0.513
	0.35	0.078	0.104	4.70	5.92	0.635	0.505
0.20	0.20	0.088	0.088	4.72	4.72	0.543	0.543
	0.25	0.086	0.092	4.79	5.05	0.568	0.539
	0.30	0.083	0.097	4.88	5.48	0.597	0.532
	0.35	0.081	0.102	4.99	6.01	0.632	0.524
0.25	0.25	0.090	0.090	5.14	5.14	0.563	0.563
	0.30	0.088	0.095	5.24	5.58	0.592	0.556
	0.35	0.085	0.100	5.36	6.12	0.626	0.548
0.30	0.30	0.092	0.092	5.69	5.69	0.585	0.585
	0.35	0.090	0.097	5.83	6.26	0.619	0.576
0.35	0.35	0.095	0.095	6.42	6.42	0.609	0.609

Table C11.1 MOMENT DISTRIBUTION CONSTANTS FOR SLABS WITHOUT DROP PANELS *

* Applicable when $c_1/\ell_1 = c_2/\ell_2$. For other relationships between these ratios, the constants will be slightly in error.

† Stiffness is $K_{AB} = k_{AB} E \frac{\ell_2 h^3}{12 \ell_1}$, and $K_{BA} = k_{BA} E \frac{\ell_2 h^3}{12 \ell_1}$.



Column dimension		Uniform load FEM = Coef. $(w\ell_2\ell_1^2)$		Stiffness factor†		Carryover factor	
$\frac{c_{1A}}{\ell_1}$	$\frac{c_{1B}}{\ell_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
0.00	0.00	0.088	0.088	4.78	4.78	0.541	0.541
	0.05	0.087	0.089	4.80	4.82	0.545	0.541
	0.10	0.087	0.090	4.83	4.94	0.553	0.541
	0.15	0.085	0.093	4.87	5.12	0.567	0.540
	0.20	0.084	0.096	4.93	5.36	0.585	0.537
	0.25	0.082	0.100	5.00	5.68	0.606	0.534
	0.30	0.080	0.105	5.09	6.07	0.631	0.529
0.05	0.05	0.088	0.088	4.84	4.84	0.545	0.545
	0.10	0.087	0.090	4.87	4.95	0.553	0.544
	0.15	0.085	0.093	4.91	5.13	0.567	0.543
	0.20	0.084	0.096	4.97	5.38	0.584	0.541
	0.25	0.082	0.100	5.05	5.70	0.606	0.537
	0.30	0.080	0.104	5.13	6.09	0.632	0.532
0.10	0.10	0.089	0.089	4.98	4.98	0.553	0.553
	0.15	0.088	0.092	5.03	5.16	0.566	0.551
	0.20	0.086	0.094	5.09	5.42	0.584	0.549
	0.25	0.084	0.099	5.17	5.74	0.606	0.546
	0.30	0.082	0.103	5.26	6.13	0.631	0.541
0.15	0.15	0.090	0.090	5.22	5.22	0.565	0.565
	0.20	0.089	0.094	5.28	5.47	0.583	0.563
	0.25	0.087	0.097	5.37	5.80	0.604	0.559
	0.30	0.085	0.102	5.46	6.21	0.630	0.554
0.20	0.20	0.092	0.092	5.55	5.55	0.580	0.580
	0.25	0.090	0.096	5.64	5.88	0.602	0.577
	0.30	0.088	0.100	5.74	6.30	0.627	0.571
0.25	0.25	0.094	0.094	5.98	5.98	0.598	0.598
	0.30	0.091	0.098	6.10	6.41	0.622	0.593
0.30	0.30	0.095	0.095	6.54	6.54	0.617	0.617

Table C11.2 MOMENT DISTRIBUTION CONSTANTS FOR SLABS WITH DROP PANELS *

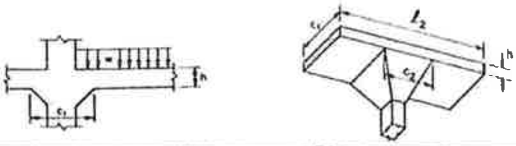
* Applicable when $c_1/\ell_1 = c_2/\ell_2$. For other relationships between these ratios, the constants will be slightly in error.

† Stiffness is $K_{AB} = k_{AB} E \frac{\ell_2 h^3}{12\ell_1}$, and $k_{BA} = k_{BA} E \frac{\ell_2 h^3}{12\ell_1}$.

FEM (uniform load w) $= Mw\ell_2(\ell_1)^2$

K(stiffness) $= kE\ell_2h^3/12\ell_1$

Carryover factor $= C$



c_1/ℓ_1	c_1/ℓ_2	M	k	C
	0.00	0.083	4.000	0.500
	0.05	0.083	4.000	0.500
	0.10	0.083	4.000	0.500
	0.15	0.083	4.000	0.500
	0.20	0.083	4.000	0.500
0.00	0.25	0.083	4.000	0.500
	0.30	0.083	4.000	0.500
	0.35	0.083	4.000	0.500
	0.40	0.083	4.000	0.500
	0.45	0.083	4.000	0.500
	0.50	0.083	4.000	0.500
	0.00	0.083	4.000	0.500
	0.05	0.084	4.047	0.503
	0.10	0.084	4.093	0.507
	0.15	0.084	4.138	0.510
	0.20	0.085	4.181	0.513
0.05	0.25	0.085	4.222	0.516
	0.30	0.085	4.261	0.518
	0.35	0.086	4.299	0.521
	0.40	0.086	4.334	0.523
	0.45	0.086	4.368	0.526
	0.50	0.086	4.398	0.528
	0.00	0.083	4.000	0.500
	0.05	0.034	4.091	0.506
	0.10	0.085	4.182	0.513
	0.15	0.085	4.272	0.519
	0.20	0.086	4.362	0.524
0.10	0.25	0.087	4.449	0.530
	0.30	0.087	4.535	0.535
	0.35	0.088	4.618	0.540
	0.40	0.088	4.698	0.545
	0.45	0.089	4.774	0.550
	0.50	0.089	4.846	0.554
	0.00	0.083	4.000	0.500
	0.05	0.084	4.132	0.509
	0.10	0.085	4.267	0.517
	0.15	0.086	4.403	0.526
	0.20	0.087	4.541	0.534
0.15	0.25	0.088	4.680	0.543
	0.30	0.089	4.818	0.550
	0.35	0.090	4.955	0.558
	0.40	0.090	5.090	0.565
	0.45	0.091	5.222	0.572
	0.50	0.092	5.349	0.579
	0.00	0.083	4.000	0.500
	0.05	0.085	4.170	0.511
	0.10	0.086	4.346	0.522
	0.15	0.087	4.529	0.532
	0.20	0.088	4.717	0.543
0.20	0.25	0.089	4.910	0.554
	0.30	0.090	5.108	0.564
	0.35	0.091	5.308	0.574
	0.40	0.092	5.509	0.584
	0.45	0.093	5.710	0.593
	0.50	0.094	5.908	0.602
	0.00	0.083	4.000	0.500
	0.05	0.085	4.204	0.512
0.25	0.10	0.086	4.420	0.525
	0.15	0.087	4.648	0.538
	0.20	0.089	4.887	0.550
	0.25	0.090	5.138	0.563

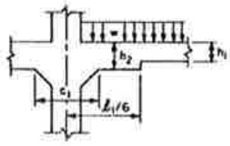
c_1/ℓ_1	c_2/ℓ_2	M	k	C
	0.30	0.091	5.401	0.576
	0.35	0.093	5.672	0.588
0.25	0.40	0.094	5.952	0.600
	0.45	0.095	6.238	0.612
	0.50	0.096	6.527	0.623
	0.00	0.083	4.000	0.500
	0.05	0.085	4.235	0.514
	0.10	0.086	4.488	0.527
	0.15	0.088	4.760	0.542
	0.20	0.089	5.050	0.556
0.30	0.25	0.091	5.361	0.571
	0.30	0.092	5.692	0.585
	0.35	0.094	6.044	0.600
	0.40	0.095	6.414	0.614
	0.45	0.096	6.802	0.628
	0.50	0.098	7.205	0.642
	0.00	0.083	4.000	0.500
	0.05	0.085	4.264	0.514
	0.10	0.087	4.551	0.529
	0.15	0.088	4.864	0.545
	0.20	0.090	5.204	0.560
0.35	0.25	0.091	5.575	0.576
	0.30	0.093	5.979	0.593
	0.35	0.095	6.416	0.609
	0.40	0.096	6.888	0.626
	0.45	0.098	7.395	0.642
	0.50	0.099	7.935	0.658
	0.00	0.083	4.000	0.500
	0.05	0.085	4.289	0.515
	0.10	0.087	4.607	0.530
	0.15	0.088	4.959	0.546
	0.20	0.090	5.348	0.563
0.40	0.25	0.092	5.778	0.580
	0.30	0.094	6.255	0.598
	0.35	0.095	6.782	0.617
	0.40	0.097	7.365	0.635
	0.45	0.099	8.007	0.654
	0.50	0.100	8.710	0.672
	0.00	0.083	4.000	0.500
	0.05	0.085	4.311	0.515
	0.10	0.087	4.658	0.530
	0.15	0.088	5.046	0.547
	0.20	0.090	5.480	0.564
0.45	0.25	0.092	5.967	0.583
	0.30	0.094	6.517	0.602
	0.35	0.096	7.136	0.621
	0.40	0.098	7.836	0.642
	0.45	0.100	8.625	0.662
	0.50	0.101	9.514	0.683
	0.00	0.083	4.000	0.500
	0.05	0.085	4.331	0.515
	0.10	0.087	4.703	0.530
	0.15	0.088	5.123	0.547
	0.20	0.090	5.599	0.564
0.50	0.25	0.092	6.141	0.583
	0.30	0.094	6.,60	0.603
	0.35	0.096	7.470	0.624
	0.40	0.098	8.289	0.645
	0.45	0.100	9.234	0.667
	0.50	0.102	10.329	0.690

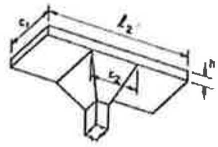
Table C11.3 MOMENT DISTRIBUTION CONSTANTS FOR SLAB-BEAM MEMBERS WITH COLUMN CAPITALS

FEM (uniform load w) $= Mw\ell_2\ell_1^2$

K (stiffness) $= kE\ell_2 h^3/12\ell_1$

Carryover factor $= C$





c_1/ℓ_1	c_2/ℓ_2*	Constants For $h_2 = 1.25h_1$			Constants For $h_2 = 1.5h_2$		
		M	k	C	M	k	C
0.00	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.795	0.542	0.093	5.837	0.589
	0.10	0.088	4.795	0.542	0.093	5.837	0.589
	0.15	0.088	4.795	0.542	0.093	5.837	0.589
	0.20	0.088	4.795	0.542	0.093	5.837	0.589
	0.25	0.088	4.795	0.542	0.093	5.837	0.589
	0.30	0.088	4.797	0.542	0.093	5.837	0.589
0.05	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.846	0.545	0.093	5.890	0.591
	0.10	0.089	4.896	0.548	0.093	5.942	0.594
	0.15	0.089	4.944	0.551	0.093	5.993	0.596
	0.20	0.089	4.990	0.553	0.094	6.041	0.598
	0.25	0.089	5.035	0.556	0.094	6.087	0.600
	0.30	0.090	5.077	0.558	0.094	6.131	0.602
0.10	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.894	0.548	0.093	5.940	0.593
	0.10	0.089	4.992	0.553	0.094	6.042	0.598
	0.15	0.090	5.039	0.559	0.094	6.142	0.602
	0.20	0.090	5.184	0.564	0.094	6.240	0.607
	0.25	0.091	5.278	0.569	0.095	6.335	0.611
	0.30	0.091	5.368	0.573	0.095	6.427	0.615
0.15	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	4.938	0.550	0.093	5.986	0.595
	0.10	0.090	5.082	0.558	0.094	6.135	0.602
	0.15	0.090	5.228	0.565	0.095	6.284	0.608
	0.20	0.091	5.374	0.573	0.095	6.432	0.614
	0.25	0.092	5.520	0.580	0.096	6.579	0.620
	0.30	0.092	5.665	0.587	0.096	6.723	0.626
0.20	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	4.978	0.552	0.093	6.027	0.597
	0.10	0.090	5.167	0.562	0.094	6.221	0.605
	0.15	0.091	5.361	0.571	0.095	6.418	0.613
	0.20	0.092	5.558	0.581	0.096	6.616	0.621
	0.25	0.093	5.760	0.590	0.096	6.816	0.628
	0.30	0.094	5.962	0.590	0.097	7.015	0.635
0.25	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	5.015	0.553	0.094	6.065	0.598
	0.10	0.090	5.245	0.565	0.094	6.300	0.608
	0.15	0.091	5.485	0.576	0.095	6.543	0.617
	0.20	0.092	5.735	0.587	0.096	6.790	0.626
	0.25	0.094	5.994	0.598	0.097	7.043	0.635
	0.30	0.095	6.261	0.600	0.098	7.298	0.644
0.30	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	5.048	0.554	0.094	6.099	0.599
	0.10	0.090	5.317	0.567	0.095	6.372	0.610
	0.15	0.092	5.601	0.580	0.096	6.657	0.620
	0.20	0.093	5.902	0.593	0.097	6.953	0.631
	0.25	0.094	6.219	0.605	0.098	7.258	0.641
	0.30	0.095	6.550	0.618	0.099	7.571	0.651

Table C11.4 MOMENT DISTRIBUTION CONSTANTS FOR SLAB-BEAM MEMBERS WITH COLUMN CAPITALS AND DROP PANELS

REFERENCES

11.1 Park, R. and Gamble, W.L., "Reinforced Concrete Slabs", John Wiley & Sons, New York, 1980.

11.2 Timoshenko, S. and Woinowsky-Krieger, S., "Theory of Plates and Shells", 2nd Edition, McGraw-Hill Book Co., New York, 1959.

11.3 Bares, R., "Table for the Analysis of Plates, Slabs and Diaphragms Based on the Elastic Theory", Bauverlag GmbH, Wiesbaden and Berlin, 1969.

11.4 Hahn, J., "Structural Analysis of Beams and Slabs", English translation by C.V. Amerongen, Pitman, London, 1966.

11.5 Pucher, A., "Influence Surfaces of Elastic Plates", Springer-Verlag, Vienna, 1964.

11.6 Ministry of Works and Development, "Highway Bridge Design Brief", Civil Division Publication CDP 701/D, Ministry of Works and Development, September 1978.

11.7 Homberg, H., "Double Webbed Slabs", Springer-Verlag, Berlin, 1973.

11.8 Homberg, H., "Dalles d'Epaisseur Variable", Danod, Paris, 1972.

11.9 Hillerborg, A., "Reinforcement of Slabs and Shells Designed According to the Theory of Elasticity", Beton, Vol. 38, No. 2, 1953, (Translated by Building Research Station, Watford, England, 1962. Library Communitation No. 1081).

11.10 Wood, R.H., "The Reinforcement of Slabs in Accordance With a Pre-determined Field of Moments", Concrete, Vol. 2, No. 2, February, 1968.

11.11 Johansen, K.W., "Yield-Line Formulae for Slabs", translated by Cement and Concrete Association, London, 1972.

11.12 Wood, R.H. and Jones, L.L., "Yield-Line Analysis of Slabs", Thames & Hudson and Chatto & Windus, London, 1967.

11.13 Jones, L.L., "Ultimate Load Analysis of Reinforced and Prestressed Concrete Structures", Chatto and Windus, London 1962.

11.14 Park, R., "Ultimate Strength Design of Reinforced Concrete Structures, Vol. 2, Limit Design of Reinforced Concrete Slabs", Department of Civil Engineering and Extension Studies, University of Canterbury, New Zealand.

11.15 Hillerborg, A., "Design of Reinforced Concrete Slabs According to the Strip Method", translated by Cement and Concrete Association, London, 1975.

11.16 Wood, R.H. and Armer, G.S.T., "The Theory of the Strip Method for the Design of Slabs", Proceedings of the Institution of Civil Engineers, Vol. 41, October 1968.

11.17 Armer, G.S.T., "The Strip Method: A New Approach to the Design of Slabs", Concrete, Vol. 2, No. 9, September 1968.

11.18 Taylor, R. and Mohemedbhai, G.T.G., "Coefficients for the Design of Slabs by the Yield-Line Theory", Concrete, Vol. 3, No. 5, May 1969.

11.19 Sozen, M.A. and Siess, C.P., "Investigation of Multiple-Panel Reinforced Concrete Floor Slabs: Design Methods – Their Evolution and Comaprison", ACI Journal, Proceedings V.60, No. 8, Aug. 1963, pp. 999–1028.

11.20 Hatcher, D.S., Sozen, M.A. and Siess, C.P., "Test of a Reinforced Concrete Flat Plate", Proceedings, ASCE, Vol. 91, ST5, Oct. 1965, pp. 205–231.

11.21 Guralnick, S.A. and La Fraugh, R.W., "Laboratory Study of a Forty-Five Foot Square Flat Plate Structure", ACI Journal, Proceedings, V.60, No. 9, Sept. 1963, pp. 1107–1185.

11.22 Hatcher, D.S., Sozen, M.A. and Siess, C.P., "Test of a Reinforced Concrete Flat Slab", Proceedings, ASCE, V.95, ST6, June 1969, pp. 1051–1072.

11.23 Jirsa, J.O., Sozen, M.A. and Siess, C.P., "Test of a Flat Slab Reinforced with Welded Wire Fabric", Proceedings, ASCE, V.92, ST3, June 1966, pp. 199–224.

11.24 Gamble, W.L., Sozen, M.A. and Siess, C.P., "Test of a Two-Way Reinforced Floor Slab", Proceedings, ASCE, V.95, ST6, June 1969, pp. 1073–1096.

11.25 Vanderbilt, M.D., Sozen, M.A. and Siess, C.P., Test of a Modified Reinforced Concrete Two-Way Slab", Proceedings, ASCE, V.95, ST6, June 1969, pp. 1907 –1116.

11.26 Sozen, M.A., "Investigation of Multiple-Panel Reinforced Concrete Floor Slabs: Design – Their Evaluation and Comparison", ACI Journal, Proceedings V.60, No. 8, Aug. 1963, pp. 999–1028.

11.27 Xanthakis, M. and Sozen, M.A., "An Experimental Study of Limit Design in Reinforced Concrete Flat Slabs", Civil Engineering Studies, Structural Research Series No. 277, University of Illinois, Dec. 1963, 159pp.

11.28 Nichols, J.R., "Statitical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors", Transactions, ASCE, Vol. 77, 1914, pp. 1670–1736.

11.29 Gamble, W.L., Sozen, M.A. and Siess, C.P., "Tests of a Two-Way Reinforced Concrete Floor Slab", Proceedings, ASCE, ST6, June 1969, pp. 1073–1096.

11.30 Jirsa, J.O., Sozen, M.A. and Siess, C.P., "Pattern Loadings on Reinforced Concrete Floor Slabs", Proceedings, ASCE, V.95, ST6, June 1969, pp. 1117 –1137.

11.31 Corley, W.G., Sozen, M.A. and Siess, C.P., "The Equivalent-Frame Analysis for Reinforced Concrete Slabs", Civil Engineering Studies, Structural Research Series No. 218, University of Illinois, June 1961, 166pp.

11.32 Jirsa, J.O., Sozen, M.A. and Seiss, C.P., "The Effects of Pattern Loadings on Reinforced Concrete Floor Slabs", Civil Engineering Studies, Structural Research Series No. 269, University of Illinois, July 1963.

11.33 Corley, W.G. and Jirsa, J.O., "Equivalent Frame Analysis for Slab Design", ACI Journal, Proceedings V.67, No. 11, Nov. 1970, pp. 875–884,

11.34 Seely, F.B.and Smith, J.A., "Advanced Mechanics of Materials, John Wiley and Sons, Inc., New York, 1956, 680pp.

11.35 Simmonds, S.H., and Misic, J., "Design Factors for the Equivalent Frame Method," ACI Journal, Proceedings V.68, No. 11, Nov. 1971, pp. 825–831.

COMMENTARY

C12 FOUNDATIONS

C12.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 12 of the Code.

b_o perimeter of critical section for slabs and foundations, mm

q_s soil bearing pressure as determined from the factored loads, kPa

C12.2 Scope

This Section documents provisions which apply to isolated foundations supporting a single column or wall. However, most of the provisions are generally applicable to combined foundation and mat systems supporting several columns or walls or a combination thereof. Reference to piles is generally limited towards establishing ductility requirements, since generally, ductile piles are the preferred method of providing post-elastic energy dissipation, should this be a design requirement. Basic pile design philosophy should be extracted from appropriate references [12.1, 12.2, 12.3, 12.4].

C12.3 General principles and requirements

C12.3.1 *Loads and reactions*

C12.3.1.1 to C12.3.1.3 These clauses require that foundations be proportioned to sustain the applied design loads and induced reactions which include axial loads, moments, and shears that have to be resisted at the base of the foundation or pile cap. Section 2 gives a formal definition of design load. Only the computed end moments that exist at the base of a column (or pedestal) need be transferred to the foundation; the minimum eccentricity requirement for slenderness considerations given in 6.4.11 need not be considered for transfer of forces and moments to footings.

Clause 12.3.1.2 is a precis of the foundation design section of NZS 4203 which is divided into two parts: design for factored loads and capacity design. The latter is applied to structures which are intended to yield in a fully ductile manner while the former (factored loads) applies to all structures.

In designing foundations for overturning moments either by the strength method or the alternative method, the soil stress may be assumed to be uniform throughout a rectangular stress block extending over a portion of the area instead of the commonly assumed triangular stress block.

In determining an allowable soil stress by the strength method a factor of safety of 1.8 should be applied to the average measured soil strength for the factored loads such as $1.4D + 1.7L_R$ while a partial factor of safety of 1.1 may be applied for capacity loads. For further information, reference should be made to the paper by Taylor[12.5].

For the alternative method, the traditional factors of safety of 3 for static loads and 2 for seismic loads apply.

C12.3.1.5 to C12.3.1.6 These clauses are included to remind Code users of other soil movement situations which should be considered in the overall foundation design process.

C12.3.3 *Moment in foundations*

C12.3.3.1 and C12.3.3.2 These clauses define the critical locations where maximum moments (and development of reinforcement) are to be computed for the various foundation support conditions.

C12.3.3.4 This clause is based upon the work described in ACI 318-77. Note, however, that β_c is the inverse ratio to that specified in the ACI Code and conforms with the definition of β_c in Section 7. The Clause states the requirement that the reinforcement in the short direction of rectangular foundations must be distributed so that an area of steel given by eq. 12-1 is provided in a band width equal to the length of the short side of the foundation. The band width is centred about the column centreline. The remaining reinforcement required in the short direction is to be distributed equally over the two segments outside the band width, one half to each segment.

C12.3.4 *Shear in foundations*

C12.3.4.1 and C12.3.4.2 The shear strength of foundations must be determined for the more severe condition of 7.3.15.1 (a) or 7.3.1.5.1 (b). The critical section for shear is "measured" from the face of supported member (column, pedestal or wall), except for supported members on steel base plates.

Clause 7.3.15.1 (a) considers the foundation essential as a wide beam with a critical section (potential crack) extending in a plane across the entire width. This case is analogous to a conventional beam, and the design proceeds accordingly.

Clause 7.3.15.1 (b) assumes two-way action, with a critical section (potential cracking) along the surface of a truncated cone or pyramid. The critical section of this case is taken at a distance $d/2$ from the perimeter of the column, pier, pile or other concentrated load.

Computation of shear requires that the soil bearing pressure q_s be obtained from the factored loads and the design be in accordance with the appropriate equation of Section 7.

Where necessary, shear around individual piles may be investigated in accordance with 7.3.15.1 (b). If shear perimeters overlap, the critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter which will actually resist the critical shear for the group under consideration. One such situation is illustrated in fig. C12.1.

C12.3.4.3 When piles are located inside the critical sections d or $d/2$ from face of column, analysis for shear in deep flexural members in accordance with 7.3.12 needs to be considered.

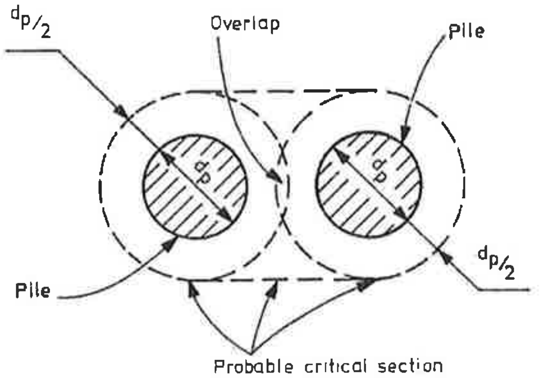


Fig. C12.1 MODIFIED CRITICAL SECTION FOR PERIMETER SHEAR WITH OVERLAPPING CRITICAL PERIMETERS

C12.3.5 *Development of reinforcement in foundations* Development lengths for reinforcement are calculated according to Section 5 regardless of whether the strength design method or the alternative design method of Appendix B is used.

C12.3.7 *Transfer of force at base of column or reinforced pedestal*

C12.3.7.1 This clause states in general that all forces and moments applied at the column base must be transmitted to the footing. All tensile forces, whether created by uplift, moment, or other reason, must be resisted entirely by reinforcement.

C12.3.7.2 The shear-friction method given in Section 7 may be used to check for transfer of lateral force from the base of a column to a footing. Shear keys or other appropriate means must be used where additional capacity for lateral force is needed.

C12.3.7.3 Compressive stress may be transmitted to the footing by bearing on concrete. For strength design, permissible bearing stresses on the actual loaded area will be equal to 0.85 $\phi f_c'$ (where $\phi = 0.7$), when the loaded area is equal to the area on which it is supported. Therefore, the permissible bearing stresses will be approximately 0.6 f_c' of the weaker concrete on the common loaded area.

In the common case of a column bearing on a footing larger than the column, bearing stress must be checked on the bottom of the column and the top of the footing. The permissible bearing stress on the column will normally be 0.6 f_c' of the column concrete. Strength in the lower part of the column must be checked since the column steel cannot be considered effective at the joint because the stress in it is not developed for some distance above the joint unless starters are provided or the steel is extended into the footing. The permissible bearing stress on the footing may be increased in accordance with 6.3.5 and will usually be twice 0.85 $\phi f_c'$ or approximately 1.2 f_c' of the footing concrete. The compressive force which exceeds that developed by the permissible concrete bearing stress on the bottom of the column or on the top of the footing must be carried by starters or extended reinforcement. Similar procedures apply where a column rests on a pedestal and where a pedestal rests on a footing.

For the alternative design method of Appendix B, permissible bearing stresses are limited to 50% of the values in 6.3.5.

C12.3.7.4 to C12.3.7.6 The Code does not require that all bars in a column be extended through and be anchored into the footing. However, steel at least equal to 0.005 A_g or an equal area of properly spliced starters must extend into the footing with proper anchorage, where A_g is the gross area of the supported column cross-section. At least four bars or starters must be used.

C12.3.9 *Combined foundations and mats*

C12.3.9.1 Any reasonable assumption with respect to the distribution of soil pressure or pile reactions can be used as long as it is consistent with the type of structure and properties of the soil, and conforms to established principles of soil mechanics (see 12.2).

Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mats, regardless of the soil pressure distribution.

Detailed recommendations for design of combined footings and mats are given in "Suggested Design Procedures for Combined Footings and Mats" reported by ACI Committee 336[12.6]. See also reference [12.7].

C12.4 Principles and requirements additional to 12.3 for members not designed for seismic loading

C12.4.1 *General*

C12.4.1.3 Piles deteriorate due to the action of mechanical, chemical and biological agencies and if an adequate service life is to be obtained from piles in aggressive conditions a correct choice of material and its treatment are necessary.

The corrosion of steel piles is an electro-chemical phenomenon caused by potential gradients between adjacent areas of the steel surface. The steel corrodes at surfaces that are anodic to the soil and water, but is probably protected by a layer of hydrogen that is released at the cathodic surfaces. Differences in potential are caused by differences in the surface conditions of the steel and by variations in the electrolyte and the amount of oxygen in solution in the water at different points in the length of a pile. The temperature and the time of exposure also determine the amount of corrosion.

The incidence of corrosion of a steel pile that is completely embedded in the ground is largely dependent on the ease with which aerated ground water can reach the pile. Thus, it is small where the permeability of the soil is low, as in a clay, but may be important in a porous soil, such as a sand, where air is present in the pores down to ground water level and dissolved oxygen may be available for some distance below. The rates of corrosion shown by experiments vary from practically nil to about 0.075 mm (0.003 in) per year, a commonly used (average) figure being 0.05 mm/year. It is a common practice to make an allowance for loss of thickness by corrosion when calculating the thickness of steel required in the wall of a tube pile or in the web and flanges of an H pile. In normal conditions that are not regarded as corrosive an increase of 1.5 mm in the thickness might be made.

For steel piles that are exposed to sea water and sea air as in the case of a jetty, the loss of steel would be least for that portion of the pile in the soil and greatest for the free standing portion. The corrosion of steel in sea water has been the subject of a number of experiments. In the tests by the Sea-Action Committee of the Institution of Civil Engineers (1920–38) the rate of loss at the surface of steel exposed to the sea water was found to vary from about 0.075 mm (0.003 in) per year in temperate waters to about 0.175 mm (0.007 in) per year in the tropics.

Provided due allowance has been made for corrosion with respect to the service life of the steel casing the remaining area of steel shell may be considered as providing a portion of the required longitudinal reinforcing for non-seismic forces. See also C12.5.2.1.

C12.5 Principles and requirements additional to 12.3 for members designed for seismic loading

C12.5.1 *Designing for ductility*

C12.5.1.1 to C12.5.1.3 The general philosophy of NZS 4203 as specified is that of capacity design which includes [12.5] estimation of and design for capacity loadings on the foundation elements. To quote from Clause 3.3.2.2 of NZS 4203: " . . . . energy-dissipating elements or mechanisms are chosen . . . . and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained . . . . ".

Others [12.8] have similarly stated this principle, to quote " . . . the criterion for the design of foundations of earthquake resisting structures is that the foundation system should be capable of supporting the design gravity loads while maintaining chosen seismic energy dissipating mechanisms of the structure. The foundation system in this context includes the foundation structure, consisting of reinforced concrete construction, piles, caissons and the supporting soil".

Thus these clauses, in conjunction with 3.5.12 on foundations, is intended to impress upon designers that where energy dissipation of large earthquake forces cannot be resisted elastically then it is essential that yielding occurs only at predictable locations and that such yielding can occur without serious damage. For further information and examples see reference [12.9].

C12.5.2 *Potential plastic hinge regions*

C12.5.2.1 Because of the generally high moments and shears induced at the tops of piles, it is essential to provide adequate confining steel to the concrete to ensure ductility.

For a cased pile the effect or contribution of the steel shell may be included with respect to confinement for the potential plastic hinge region. However, no such contribution from the shell shall be allowed for shear or moment computations because of the lack of compatibility of strains between concrete and steel.

C12.5.2.2 The length of the potential plastic hinge region specified for a pile is consistent with that documented in C6.5.4 for columns and piers. The effect of the axial load level as given by the computation $0.3\,\phi f'_c A_g$ for extending the plastic hinge region has not been included, however.

C12.5.3 *Reinforcement in piles*

C12.5.3.1 This clause is based on the equivalent requirements for columns and piers as specified in Section 6. However, it was felt that reduction of minimum reinforcement ratios was warranted for piles with large cross-sectional area. The committee considered that one-half of the minimum specified for columns with Grade 275 reinforcement was acceptable for piles exceeding 2 x 10^6 mm^2 in cross-sectional area, with increasing ratios for piles of smaller area. Thus, for Grade 275 reinforcement, the minimum reinforcement ratio for piles of cross-sectional area smaller than 0.5 x 10^6 is that specified for columns (2.2/275 = 0.008). In addition, concessions are made where Grade 380 reinforcement is used. Equation 12-2 provides for an interpolation for the required minimum reinforcement ratio when the area of the pile lies between 0.5 x 10^6 mm^2 ($\rho_{t,\ min.} = 2.2/f_y$) and 2 x 10^6 mm^2 ($\rho_{t,\ min.} = 1.1/f_y$).

C12.5.3.2 It was not considered appropriate to relax for piles the requirements of maximum reinforcement specified for columns. Thus the requirements of Section 6 remain applicable to piles.

C12.5.3.3 Requirements for transverse reinforcement are generally in accordance with Section 6 but again a relaxation is made for transverse reinforcement located in long length piles well away from potential plastic hinge regions.

REFERENCES

12.1 Woodward, R.J., Gardner, W.S. and Greer, D.M., "Drilled Pier Foundations", McGraw-Hill Book Company, 1972, 287p.

12.2 Whitaker, T., "The Design of Piled Foundations", Pergamon Press, 1970, 180p.

12.3 Department of the Navy, "Design Manual – Soil Mechanics, Foundations, and Earth Structures", NAVFAC DM-7, March 1971.

12.4 Teng, Wayne C., "Foundation Design", Prentice-Hall Inc., 1962, 466p.

12.5 Taylor, P.W., "Code Provisions Related to Soils and Foundations", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 9, No. 1, March 1976, pp. 68-73.

12.6 ACI Committee Report, "Suggested Design Procedures for Combined Footings and Mats", ACI Committee 336 (formerly 436), Journal American Concrete Institute, Proceedings, Vol. 63, No. 10, October 1966, pp. 1041-1057. Also ACI Manual of Concrete Practice, American Concrete Institute, Detroit.

12.7 Kramrisch, Fritz, and Rogers, Paul, "Simplified Design of Combined Footings", Proceedings, ASCE, Vol. 87, No. SM5, October 1961, page 19.

12.8 Binney, J.R. and Paulay, T., "Foundations for Shear Wall Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, June 1980, pp. 177–181.

12.9 Taylor, P.W. and Williams, R.L., "Foundations for Capacity Designed Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, June 1979, pp. 101–108.

COMMENTARY

C13 PRESTRESSED CONCRETE

C13.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 13 of the Code.

a depth of equivalent stress block, mm

A_{pf} and A_{pw} those portions of the prestressing steel required to develop the compressive strengths of the overhanging flanges and the web respectively. Refer C13.3.6 (b).

A_{ps} area of prestressed reinforcement in tension zone, mm^2

b width of compression face of member, mm

b_w web width, mm

d distance from extreme compression fibre to centroid of prestressing steel or to combined centroid when non-prestressing tension reinforcement is included

E_c modulus of elasticity of concrete, MPa

E_s modulus of elasticity of steel, MPa

f'_c specified compressive strength of concrete, MPa

f_{ps} calculated stress in prestressing steel at design load, MPa

f_{pu} ultimate strength of prestressing steel, MPa

f_{se} effective stress in prestressing steel after losses, MPa

Δf_s stress increase in prestressing steel above f_{se}, MPa

h_f flange thickness

K wobble coefficient, per m

M_u factored moment at section

P_e design axial load in compression with given eccentricity due to gravity and seismic loading, acting on the member simultaneously with shear stress v_i during an earthquake

μ coefficient of friction

ϕ strength reduction factor, refer 4.3.1.2

ρ ratio of non-prestressed reinforcement, $\dfrac{A_s}{bd}$

ρ_p ratio of non-prestressed reinforcement, $\dfrac{A_{ps}}{bd}$

ρ' ratio of non-prestressed compression reinforcement, $\dfrac{A'_s}{bd}$

ω $\dfrac{\rho f_y}{f'_c}$

ω' $\dfrac{\rho' f_y}{f'_c}$

ω_p $\rho_p f_{ps}/f'_c$

ω_w, ω_{pw}, ω'_w reinforcement indices for flanged sections computed as for ω, ω_p, ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only.

C13.2 Scope

C13.2.1 The provisions of Section 13 were developed primarily for structural members which are commonly used in buildings and bridges. However, many of the provisions may be applied to other types of construction so that application of the provisions to cases not specifically cited by the Code is a matter of judgement.

C13.2.2 The entire Code applies to prestressed concrete except where excluded or in direct conflict with Section 13. Some sections of the Code are excluded from use in the design of prestressed concrete for specific reasons. Explanations for such exclusions are provided.

Clause 3.3.3.4 of the Code is excluded since moment redistribution for prestressed concrete is covered in 13.3.10.

The empirical provisions of 3.3.6.2 (a), (b) and (c) for T-beams were developed for conventional reinforced concrete and if applied to prestressed concrete would exclude many standard prestressed products in satisfactory use today. Hence, proof by experience permits variations.

By excluding 3.3.6.2 (a), (b) and (c), no special requirements for prestressed concrete T-beams appear in the Code. Instead, the determination of an effective width of flange is left to the experience and judgement of the Engineer. Where possible, the flange widths in 3.3.6.2 (a), (b) and (c) should be used unless experience has proved that variations are safe and satisfactory.

It is not necessarily conservative in elastic analysis and design considerations to use the maximum flange width as permitted in 3.3.6.2 (a). Clauses 3.3.6.1 and 3.3.6.2 (d) provide general requirements for T-beams that are also applicable to prestressed concrete units. The spacing limitations for slab reinforcement are based on flange thickness, which for tapered flanges can be taken as the average thickness.

Clause 3.4.2 The empirical limits for concrete joist floors are justified for conventional reinforced concrete but not for prestressed concrete. Hence, they are excluded in prestressed concrete. Experience and judgement must be used.

For prestressed concrete the limitations on reinforcement given in 6.4.1.2, 6.4.2, 6.4.3 and 6.4.6 are replaced by those in 13.3.7, 13.3.8, and 13.4.3.

Section 11 The design of prestressed concrete slabs requires recognition of secondary moments induced by the prestressing tendons. Also volume changes due to the prestressing force can create additional loads on the structure that are not adequately covered in Section 11.

Because of these unique properties associated with prestressing, many of the design procedures of Section 11 are not appropriate for prestressed concrete structures for which more detailed study is required.

C13.3 General principles and requirements

C13.3.1 *General considerations.* As has been past practice in the design of prestressed concrete, the design investigation should include all load stages that may be significant. The three major stages are:

(a) Initial stage, or prestress transfer stage – when the tensile force in the prestressed steel is transferred to the concrete and stress levels may be high relative to concrete cylinder strength;

(b) Service load stage – after long-time volume changes have occurred; and

(c) The design load stage – when the capacity of the member is checked.

There may be other load stages that require investigation. For example, if the cracking load is significant, this load stage may require study, or the handling and transporting stage may be critical. In addition, for continuous members, time dependent creep deformations may modify the distribution of moments from those existing immediately after construction. This may, for example, be of importance in the design of long span structures erected in cantilever.

From the standpoint of satisfactory behaviour, the two stages of most importance are those for service load and design load.

Service load stage refers to the loads defined in the relevant loading code such as live load and dead load, while the design load stage refers to factored loads. When calculating the behaviour at the service load stage, the strength reduction factors given in 4.3.1 should not be included. It is necessary to investigate service load and design load stages to ensure member performance in regard to both serviceability and strength.

For example, a beam could be prestressed along its longitudinal axis in such a manner that it will support the specified loads without objectionable deflection but the strength could be below adequate safety requirements. Similarly, a design based on strength alone may provide unsatisfactory behaviour at service loads, such as excessive camber or deflection.

This means that should the actual design be performed for strength, using load factors and strength reduction factors, then an investigation at service load levels is necessary. For sections with axial load, a general analysis considering the stress-strain diagram for concrete is desirable. In this respect, a realistic approximation of the inelastic stress-strain diagram for prestressing steel is advisable. Clause 13.3.2 provides assumptions that are to be used for the investigation at service loads and after transfer of the prestressing force.

C13.3.1.4 This refers to the type of post-tensioning where the tendon makes contact with the prestressed concrete member intermittently. Precautions should be taken to prevent buckling of such members. In particular, if thin webs or flanges are under high pre-compression, buckling is possible between supports of slender members. If the tendon is in complete contact with the member being prestressed, or is an unbonded tendon in a duct not excessively larger than the tendon, it is not possible to buckle the member under the prestressing force being introduced.

C13.3.1.6 The deviation of cables causes radial forces which may cause damage if there is inadequate cover or restraint.

C13.3.1.7 In large prestressed concrete members, such as box girders, and where prestressing is remote from the faces of the member, supplementary reinforcement should be provided at the faces in the direction of the prestressing to control random cracking and aid distribution of stresses.

C13.3.2 *Basic assumptions*

C13.3.2.1 For values of E_s tests should be performed or data obtained from the manufacturer. For E_c refer to 3.3.4.1.

C13.3.2.2 Assumptions are provided for use in service load investigation and for review of sections at transfer of prestress forces. Note that this Clause does not apply to the design of compression members in general, but only to members that are prestressed by internal tendons.

C13.3.2.3 The axial load strength in pure compression is computed by the strength design methods of Section 6, including the effect of the prestressing force.

C13.3.3 *Unbonded tendons*

(a) For information on the advantages and disadvantages of unbonded tendons see reference [13.11]

(b) Unbonded prestressed slabs and rectangular beams having an average prestress less than the modulus of rupture become flexurally unstable at the cracking load. Such a low value of average prestress is not likely to be specified for beams, but can occur relatively often in slabs and flat plates. To avoid a sudden collapse of unbonded prestressed structures at excessive live loads and especially earthquake loads, the minimum value of average prestress in rectangular sections must always be substantially higher than the modulus of rupture which is the maximum calculated tensile stress at cracking in plain concrete beams subjected to flexure. The calculation of the tensile stress at cracking is based on the assumption that the stress distribution is linear. Although this assumption is not correct and introduces an error, the cracking moment capacity of a prestressed beam can be readily calculated by making the maximum tensile stress in the beam equal to the modulus of rupture

In the case of bonded tendons, cracking of the concrete causes only a gradual decrease in the slope of the load-deflection curve. Some members with unbonded tendons, however, exhibit a sudden decrease of the load capacity at cracking, and the load capacity does not recover even after a significant increase in deflection. In order to avoid such a sudden decrease of the load capacity, further prestress or additional bonded steel should be provided

(c) A prestressed concrete member with bonded tendons has a greater flexural strength than the equivalent member with unbonded tendons. Typically, the difference in flexural capacity between otherwise identical members may be 10 to 30%. The difference is because unbonded tendons can move relative to the concrete between the anchorages, and hence local concentrations of strain will tend to occur uniformly over the length of the tendon and result in a relatively small increase in steel stress. When necessary the flexural capacity of a member with unbonded tendons can be increased by addition of non-prestressed bonded reinforcement to the section.

In a structure with unbonded tendons, continuous over several spans, the failure of one span may result in the release of the prestressing force along the whole length of the tendons. Such an event could lead to the collapse of the whole structure. Consideration should be given to the consequence of such failure in any specific span to the overall stability of the structural system. One consideration would be to use reduced tendon lengths between anchorages or tendon couplers capable of acting as intermediate anchorages.

C13.3.5 *Loss of prestress*

C13.3.5.1 Prestress losses may be expected to vary substantially for different applications. Although the actual loss will have little effect on the design strength of the member, it will affect service load stresses and behaviour, such as deflection, camber and cracking load. These aspects can control the design.

This is generally critical for continuous structures only for which a detailed assessment of creep is desirable.

In such cases a satisfactory method of computing losses is given in reference [13.8] and [13.9].

It should be appreciated that there is, to date, little data available on the losses applicable for New Zealand materials. Research on this subject is proceeding. In the interim the above codes are considered appropriate provided they are interpreted conservatively. Figures for losses were given in the New Zealand Standard Recommendation (NZSR) 32, 1968. However, there is some evidence that these figures underestimated the losses for some situations.

The lump sum figures for losses after transfer of 250 MPa for pre-tensioned members and 180 MPa for post-tensioned members that appeared in the report of the ACI Committee 423 (ACI Journal Proceedings V.54, No. 7, Jan 1958 pp, 545–578) are reported to have generally given satisfactory results for many applications. They would therefore appear appropriate for simply supported beams where design is not very sensitive to calculated losses.

C13.3.5.2 Friction losses due to wobble and curvature can be computed by equations 13-1 and 13-2 of the Code. The coefficients tabulated in table C13.1 give a range which can be generally expected. Due to the many types of ducts, tendons and wrapping materials available, these values can only serve as a guide. Where rigid conduit is used for instance, the wobble coefficient K can be considered as zero. For large tendons in semi-rigid type conduit, the wobble factor can also be considered zero. Guidance on the friction that can be expected with particular type tendons and particular type ducts can be obtained from the specialist prestressing contractors. An unrealistically low evaluation of the friction loss can lead to improper camber of the structure and inadequate prestress. Overestimation of the friction may result in extra prestressing force if the estimated friction values are not attained in the field. This could lead to excessive camber and excessive shortening of a member. If the estimated friction factors are determined to be less than those assumed in the design, the stressing force should be adjusted to give only that theoretical prestressing force in the critical portions of the structure required by the design.

Lower values of friction than those given in table C13.1 are possible with good workmanship in the use of large diameter cables. On the other hand, structures made up from precast elements may have substantially greater friction values unless special precautions are taken.

C13.3.6 *Flexural strength.* The computation of strength (previously called ultimate flexural strength) may be carried out using the same equations as those provided in the ACI 318-63 Code.

Table C13.1 FRICTION COEFFICIENTS FOR POST-TENSIONED TENDONS FOR USE IN EQUATIONS 13-1 OR 13-2

		Wobble, coefficient, K (per m length) x 10^{-3}	*Curvature coefficient μ*
Tendons to be grouted in metal sheathing	Wire tendons High strength bars 7-wire strand	3 to 5 0.3 to 2 1.7 to 6.7	0.15 – 0.25 0.08 – 0.30 0.15 – 0.25
Unbonded tendons			
Mastic-coated	Wire tendons 7-wire strand	3.3 to 6.7 3.3 to 6.7	0.05 – 0.15 0.05 – 0.15
Pre-greased	Wire tendons 7-wire strand	1 to 6.7 1 to 6.7	0.05 – 0.15 0.05 – 0.15
	Tendons in preformed holes in concrete	5	0.55

(a) Rectangular sections, or flanged sections in which the neutral axis lies within the flange (usually where the flange thickness is more than $1.4\,d\rho_p f_{ps}/f_c'$):

$$M_u = \phi\,[A_{ps}f_{ps}d\,(1 - 0.59\omega_p)] = \phi\,[A_{ps}f_{ps}\,(d - \frac{a}{2})]$$

Where a is the depth of the equivalent rectangular stress block as defined by 6.3.1.7.

(b) Flanged sections in which the neutral axis falls outside the flange (usually where the flange thickness is less than $1.4\,d\rho_p f_{ps}/f_c'$):

$$M_u = \phi\,[A_{pw}f_{ps}\,(d - \frac{a}{2}) + 0.85 f_c'\,(b - b_w)\,h_f(d - \frac{h_f}{2})]$$

where $A_{pw} = A_{ps} - A_{pf}$

and $A_{pf} = 0.85 f_c'\,(b - b_w)\,h_f/f_{ps}$

A_{pf} and A_{pw} are those portions of the prestressing steel required to develop the compressive strengths of the overhanging flanges and the web respectively, where h_f is the flange thickness.

Development and full explanations of these equations are contained in a paper by Warwaruk, et al [13.10].

C13.3.6.1 (a). Equation 13-3 is a shortcut approximation to the more accurate calculation which involves a trial method based on reaching compatibility between stresses and strains. The approximate formula may underestimate the capacity of beams with high percentages of steel and for more accurate evaluations of their capacity, the stress-strain compatibility method should be used.

C13.3.6.1 (b). Equation 13-4 from ACI 318-63 has been adopted rather than the equation in ACI 318-71. Equation 18-4 of ACI 318-71 was derived from the results of tests on beams of which the shallowest had a span/depth ratio of 28. The increase in tendon stress above f_{se} at the flexural strength is directly related to the total change in length of the concrete at the level of the tendon. This change in length can be shown to be nonlinearly and inversely related to the span/depth ratio of the member. For a given deflection, or deflection taken as some fraction of the span, it is apparent that the strain increase in a slab with a span/depth ratio of, say, 44 is significantly less than that for a member with span/depth ratio of 28 or less. That the ACI 318–71 equation 18-4 is inadequate for thin slabs is demonstrated by tests conducted at the University of Texas by Burns, et al, as shown below:

Stress increase Δf_s above f_{se} in MPa in prestressed slabs with unbonded tendons and span/depth ratio of 44:

Slab mark	*Observed* Δf_s *MPa*	Δf_s *from ACI Code eq. 18-4* *MPa*	*Suggested* Δf_s *MPa*
Slab A *	131	274	100
Slab B *	145	370	100
Flat plate 1 *	172	161	100
Flat plate 2 *	97	325	100
Flat plate 3 *	34 – 159	166	100

* From results published in reports and theses during 1975–76 at the University of Texas at Austin.

Thus to obtain f_{ps} for slabs with unbonded tendons it would seem more appropriate to add 100 MPa to f_{se} rather than to add $(69 + \frac{f_c'}{100\rho_p})$ to f_{se}. In view of the fact that f_{se} may be, for example, 900 MPa, the ACI 318–71 equation can be significantly in error.

C13.3.7 *Size and number of prestressing tendons*

C13.3.7.1 The provisions of CP110[13.8] have been adopted. Alternatively the provisions of ACI 318–77 may be used, namely that the index

$$\omega_p,\ (\omega + \omega_p - \omega'),\ \text{or}\ (\omega_w + \omega_{pw} - \omega_w')$$

should be less than 0.30 for which case studies of strain compatibility as required by 13.3.7 are not needed. If however this index is greater than 0.30 the design moment may be computed from:

(a) For rectangular sections, or flanged sections in which the neutral axis lies within the flange:

$$M_u = \phi\,(0.25 f_c' b d^2)$$

(b) For flanged sections in which the neutral axis falls outside the flange:

$$M_u = \phi\,[0.25 f_c' b_w d^2 + 0.85 f_c'\,(b - b_w)\,h_f(d - 0.5h_f)]$$

The contribution of compression reinforcement may be added to the above values of M_u.

It should be noted that when the steel index exceeds 0.30, the strength as predicted by standard equations (with $\phi = 1$) does not correlate well with test results.

C13.3.7.2 This provision is a precaution against abrupt flexural failure resulting from rupture of the prestressing steel when failure occurs immediately after cracking. The usual member requires considerable additional load beyond cracking to reach design capacity. Thus, considerable deflection warns that the design capacity is being approached. However, if design capacity occurs shortly after cracking, the warning deflection may not occur.

C13.3.8 *Minimum bonded reinforcement where unbonded tendons are used* Some bonded reinforcement is required by the Code in members prestressed with unbonded tendons to ensure flexural performance at ultimate capacity, rather than behaviour as a tied arch, and to control cracking at service load when tensile stresses exceed the modulus of rupture of the concrete. Providing minimum bonded reinforcement, as specified in this Clause, helps to ensure adequate performance.

C13.3.8.1 The minimum amount of bonded reinforcement for members other than two-way flat plates is based on research comparing the behaviour of bonded and unbonded post-tensioned beams.[13.1] Although research is limited for members other than beams and flat plates, it is advisable to apply the provisions of 13.3.8 to these other members. The necessity for applying eq. 13-5 to two-way flat plates has not been substantiated by current test data and, therefore, the requirements originally in ACI 318–71 have been modified to reflect this new information and eliminate an unnecessary design complication.

C13.3.8.2 The minimum amount of bonded reinforcement in two-way flat plates is based on reports by ACI–ASCE Committee 423[13.2, 13.3]. Limited research available for two-way flat slabs with drop panels[13.4] or waffle slabs [13.5] indicates that behaviour of these particular systems is similar to the behaviour of flat plates. However, until more complete information is available, 13.3.8.2 should be applied only to two-way flat plates (solid slabs of uniform thickness) and 13.3.8.1 should be applied to all other two-way slab systems.

(a) For average loads and typical span arrangements, flat plate tests summarized in the Committee 423 Report [13.3] and experience under the 1963 ACI Building Code indicate satisfactory performance with no bonded reinforcement. For positive moment areas where service load tensile stresses do not exceed $0.17\sqrt{f_c'}$ no minimum bonded reinforcement is required

(b) In positive moment areas, where tensile stresses are between $0.17\sqrt{f_c'}$ and $0.50\sqrt{f_c'}$, a minimum bonded reinforcement area proportioned according to eq. 13-6 is required. The tensile force N_c is computed at service load on the basis of an uncracked, homogeneous section. Since flat plate tests have not clearly indicated the necessity for any positive moment bonded reinforcement, the 1.2 factor in the N_c term in ACI 318–71 was an unnecessary design complication and was removed

(c) Research evaluated by ACI–ASCE Committee 423 [13.3] shows that bonded reinforcement in negative moment regions of two-way flat plates, proportioned on the basis of 0.15% of the cross-sectional area of the column strip, provides adequate crack control and sufficient ductility. Equation 13-7 is modified to require the larger amount of bonded reinforcement to be placed in the direction of the larger span at supports with rectangular panels. Concentration of this reinforcement in the top of slabs directly over and immediately adjacent to the column is very important. Research also shows that where very low tensile stresses occur at service load, satisfactory behaviour has been achieved at design load without bonded reinforcement. However, current practice calls for the Code specified minimum bonded reinforcement regardless of service load stress levels to help insure flexural continuity and ductility, and to control cracking due to overload, temperature or shrinkage.

C13.3.8.3 Bonded reinforcement must be adequately anchored to develop design forces. The requirements of Section 5 will help ensure that bonded reinforcement required for flexural strength under design loads in accordance with 13.3.6.1 or for tensile stress conditions at service load in accordance with 13.2.8.2 (b) will be adequately anchored to develop tension or compression forces. For bonded reinforcement required by 13.2.8.1 or 13.2.8.2 (c), but not required for flexural strength in accordance with 13.3.6.1, the minimum lengths will apply. Research [13.3] on continuous spans shows that these minimum lengths provide adequate behaviour under service load and design load conditions.

C13.3.9 *End regions.* Many design methods for end regions have been developed and they can lead to widely differing answers. For detailed design techniques which are thought to be conservative, see references [13.8] and [13.15].

C13.3.10 *Redistribution of design moments.* As member capacity is approached, inelastic behaviour at some sections can result in a redistribution of moments in prestressed concrete beams. Recognition of this actual behaviour can be advantageous in design under certain circumstances. A rigorous design method for moment redistribution is quite complex. However, recognition of moment redistribution can be accomplished with the simple method of permitting a reasonable adjustment of the elastically calculated design load moments. The amount of adjustment must be kept within predetermined safe limits.

The amount of redistribution allowed depends on the ability of the critical sections to deform inelastically by a sufficient amount. Serviceability under service loads must still of course be checked in accordance with 13.4.1.

For commentary on the interpretation of the provisions for moment redistribution refer to C3.3.3.4.

Where special studies are made and confining steel is provided to increase the rotational capacity of the section, greater amounts of redistribution are permitted than implied by eq. 13-8; however 13.3.10.1 (b) and (c) still will apply.

C13.3.10.2 The moments due to reactions induced by prestressing forces, generally referred to as secondary moments, are significant in both the elastic and inelastic states. When plastic hinges and full redistribution of moments occur to create a statically determinate structure, secondary moments disappear. However, the elastic deformations caused by a nonconcordant tendon change the amount of inelastic rotation required to obtain a given amount of moment redistribution. Conversely, for a beam with a given inelastic rotational capacity, the amount by which the design moment at the support may be varied is changed by an amount equal to the secondary moment at the support due to prestressing. Thus, the Code requires that secondary moment be included in determining the design moments.

To determine the moments used in design, the order of calculation should be:

(a) Determine design moments due to dead and live load;
(b) Redistribute as permitted;
(c) Modify by algebraic addition of the secondary moments.

A positive secondary moment at the support caused by a tendon transformed downward from a concordant profile will, therefore, reduce the design negative moments near the supports and increase the design positive moments in the midspan regions. A tendon that is transformed upward will have the reverse effect.

For the moment redistribution principles of 13.3.10 to be applicable to beams with unbonded tendons, it is necessary they contain sufficient bonded reinforcement to ensure they will act as beams after cracking and not as a series of tied arches. The minimum bonded steel requirements of 13.3.8 will serve this purpose.

C13.4 Principles and requirements additional to 13.3 for members not designed for seismic loading

C13.4.1 *Serviceability.* Crack widths, deflections and performance under frequently repetitive loads must be within acceptable limits. Permissible stresses of 13.4.1.1 should control serviceability. They do not automatically guarantee adequate structural capacity which should be checked in conformance with other Code requirements. Tables 13.1 and 13.2 give four different load categories which should cover all cases. For highway bridges the appropriate category is nominated in reference [13.7]. In general load category III as described in tables 13.1 and 13.2 will be used for buildings except where live loads may be of long duration. A long duration load may be considered as one which is present for three months or more.

Lower stress limits are permitted for load category II in recognition of the fact that loads which are of a long duration or frequently repetitive may cause increased deflections or increased crack widths, or both. In general frequently repetitive loads may be considered as those which will occur more than 50 000 times in the life of the structure. Building loads will not normally be considered as frequently repetitive loads. (For a more complete discussion on fatigue loading see reference [13.13].)

C13.4.1.1 (b) This Clause gives simple rules for the design of cracked prestressed concrete also known as "partially prestressed" concrete. For most cases the crack width limits of 4.4.2 will be satisfied automatically if the permissible stress range in the bonded steel is satisfied. Deflections may need checking if the design is based on the cracked sections.

The load-deflection curves of prestressed concrete members may be idealized into an assumed bilinear curve. The first portion of the curve is a straight line from initial load up to the load that causes cracking of a magnitude sufficient to significantly reduce the member's stiffness. The second portion of the curve proceeds from this point of cracking at a flatter slope as load is increased. The change in slope is a function of the reduction in moment of inertia at cracking.

For most usual conditions, the effects of the change is negligible. In some cases the change is so gradual that the assumption of a bilinear curve is not necessary. However, where the reduction in moment of inertia can be large at cracking, the loss of stiffness, or increase in deflection is large. For this reason when cracking is permitted, the designer may need to compute the deflection using the cracked cross-section and the transformed areas of bonded steel to compute the moment of inertia.

For continuous structures if cracking is permitted, this bilinear behaviour will negate the elastic design assumptions and further verification will be required. Generally however the errors in an elastic analysis will not be serious.

The exclusion of two-way slab systems is based on the ACI–ASCE Committee 423 report [13.3] which recommends that the permissible tensile stress be not greater than $0.50 \sqrt{f'_c}$ for prestressed concrete flat plates analysed by the equivalent frame method or other approximate methods.

For flat plate designs based on more exact analyses, or for other two-way slab systems rigorously analysed and designed for strength and serviceability, the limiting stress may be exceeded.

In table 13.2 the stress range limits have been selected as follows:

ACI Committee 215 [13.13] recommends a stress range of $0.10\, f_{pu}$ for strands, $0.12\, f_{pu}$ for wire and 138 MPa for deformed reinforcing bars. As this stress range is to prevent fatigue failure a slightly lower figure for Load Category II of 100 MPa has been chosen to give adequate crack widths, as well as satisfactory fatigue behaviour.

For Load Categories III and IV a higher stress range of 200 MPa is considered acceptable.

It should be noted that these stress limits rely on good grouting so that the post-tensioned steel assists in crack control. High quality grouting is essential therefore. Even so if the cables are large and not placed close to the extreme tension fibre special studies may be called for.

C13.4.2 *Slab systems.* This Clause does not provide detailed Code provisions for design which will account for such aspects of behaviour which are unique to prestressed concrete. It is recommended that the analysis of prestressed slab systems be based on the equivalent frame method (refer 11.9) for determining both service and design moments. Tests [13.12, 13.3] have shown that the equivalent frame method accurately predicts moments in prestressed flat plate slab systems. Other methods of analysis should be used with caution until their correlation with that data is verified. Simplified methods using average coefficients do not apply for prestressed concrete.

Concerning the strength of prestressed slabs, tests indicate that strength is controlled primarily by the total amount of tendon capacity rather than by tendon distribution. Some tendons should be passed through the columns or around their edges. It is suggested that the maximum spacing of tendons in the column strips should not exceed four times the slab thickness and that the maximum spacing in the middle strips should not exceed six times the slab thickness.

For prestressed flat slabs continuous over two or more spans in each direction, it is suggested that the span-thickness ratio should generally not exceed 42 for floors and 48 for roofs and that these limits may be increased to 48 and 52, respectively, if calculations verify that both short and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

Short and long-term deflection and camber should be computed and checked for the requirements of serviceability of the particular usage of the structure.

The maximum length of a slab between construction joints is generally limited to 30 to 50 m to minimize the effect of slab shortening, and to avoid excessive loss of prestress due to friction.

C13.4.3 *Compression members – Combined axial load and bending*

C13.4.3.1 For compression members having less than 1.5 MPa prestress, the minimum vertical reinforcement required in Section 5 for columns or in Section 10 for walls must be provided.

C13.4.3.2 Prestressed concrete compression members will, in most cases, be precast and pre-tensioned. The high quality control associated with this type of construction may result in small column dimensions. Since the effects of accidental loads, load buckling, long-column action, shrinkage, creep, and non-uniform temperature distribution must be considered in the design, minimum column dimensions are not required for either reinforced or prestressed concrete.

C13.5 Principles and requirements additional to 13.3 for prestressed members designed for seismic loading

The design requirements for prestressed and partially prestressed members are similar in principle to those for non-prestressed members and many provisions in previous sections apply to prestressed as well as to reinforced members. Consequently 13.5 defines provisions peculiar to members with prestressing, and indicates which clauses of previous sections shall be adopted in design. The provisions are based mainly on the recommendations of the Seismic Committee of NZPCI [13.14].

It is of particular importance that the prestressing steel complies with the specified requirements for percentage elongation at rupture, to ensure adequate ductility. Where possible, flexural strength calculations should be based on the actual prestressing steel stress-strain curves.

C13.5.2 *Concrete*

(a) The slope of the falling branch of the concrete stress-strain curve increases, and the ultimate compressive strain reduces, with increasing concrete strength. Consequently, unless special transverse reinforcement in accordance with 6.5.4.3 is provided to increase the ultimate compressive strain, very high strength concrete should not be used in plastic hinge regions.

(b) The use of ungrouted post-tensioned tendons in moment resisting frames is undesirable for the following reasons:

(1) The tendons remain in the elastic range and therefore total reliance is placed on the concrete for energy dissipation and compressive strength

(2) Ductility is likely to be provided by inelastic flexural strains associated with a single wide crack at the critical section. The reduced equivalent plastic hinge length, in comparison with that for a bonded tendon, may significantly reduce the available ductility

(3) It is difficult to accurately predict the ultimate moment capacity of ungrouted sections under reversed loading. Consequently column moments induced by beam overstrength are equally difficult to predict

(4) Fluctuation of tendon forces could cause failure of the anchorage with the catastrophic result of release of prestressing force

These arguments are less valid when the prestress is used mainly to balance gravity loads, with non-prestressed steel reinforcement providing the bulk of the seismic resistance. Under these circumstances, ungrouted tendons are permitted, provided that the provisions of 13.5.5.3 are satisfied.

C13.5.3 *Design of beams*

C13.5.3.2 Design for ductile behaviour implies substantial capacity for moment redistribution. Therefore provided the upper limit on flexural steel content of 13.5.3.3 is satisfied, moment resulting from elastic analysis may be redistributed in accordance with 3.5.3.4 to gain a more advantageous moment envelope, and thus a more efficient design.

C13.5.3.3 The object is to ensure ductile behaviour in plastic hinge zones. When all the tendons are placed near the extreme tension fibre at an effective depth of 0.85 h, eq. 13-9 may be written as

$$\frac{A_{ps}\ f_{ps}}{bd\ f_c'} \leqslant 0.2$$

and its use will result in the same available curvature ductility as eq. 13-9. However for sections with tendons distributed at different positions down the section depth, or with additional non-prestressed steel reinforcement, the equivalent but more general eq. 13-9 should be used. If the higher flexural steel contents implied by eq. 13-10 are adopted, special transverse confining reinforcement in accordance with 6.5.4.3 must be provided to ensure adequate curvature ductility.

C13.5.3.4 The effect of slab steel in contributing to the ultimate beam moments may be considered in assessing beam overstrength, and must be considered when calculating the moments induced in columns when plastic hinges form in beams.

C13.5.3.5 Closed stirrups are required to be present in potential plastic hinge zones to provide confinement to the concrete and to prevent buckling of non-prestressed compression steel. The stirrups provided for this purpose can also be assumed to contribute towards the shear strength of the member.

C13.5.4 *Design of columns*

C13.5.4.1 General requirements for prestressed concrete columns are similar to those for reinforced concrete columns, provided allowance is made for the additional compression force imposed by the prestressing.

C13.5.4.4 The longitudinal steel should be distributed reasonably uniformly around the perimeter of the section in order to assist the confinement of concrete in potential plastic hinge regions.

C13.5.4.5 *Shear strength requirements.* The provisions of Section 7 are general, and therefore apply to prestressed as well as to reinforced concrete members. However, in calculating the shear carried by concrete in plastic hinge regions it is necessary to realise that the prestress provided by tendons close to the extreme fibres may not be fully effective after several cycles of inelastic hinge rotation. In such cases P_e in eq. 7-41 should include the effects of only those prestressing tendons close to the mid-depth of the section. At column sections where only limited yielding can occur, the full prestressing force may be included in eq. 7-41. In general any contribution to shear strength from inclination of prestressing tendons is considered to be negligible.

C13.5.5 *Joints in prestressed frames*

C13.5.5.1 Anchorages must be kept out of beam-column joint cores in order to avoid tensile bursting stresses in a region already subjected to severe diagonal tension from beam and column forces. At exterior joints, anchorages can be placed in stubs outside the joint core region.

C13.5.5.2 Such an arrangement of tendons results in more ductile plastic hinge behaviour of beams under inelastic cyclic loading than when the tendons are all concentrated at mid-depth in the beam. However, in addition to top and bottom tendons, it is very desirable to have at least one tendon located within the middle third of beam depth to help carry the joint core shear force (see 9.5.4.2 (b)).

C13.5.5.3 A possible design technique to satisfy this Section would involve prestressing steel designed to balance a portion of the service loads (for example, $D + 0.5L_R$), with the additional required seismic capacity and ductility provided by top and bottom layers of non-prestressed steel reinforcement. Under these circumstances the beam prestressing tendon or tendons at the column faces could be located in the central third of the beam depth to avoid loss of effective prestress force under reversed inelastic cycling, and to improve the shear resistance of the joint core.

C13.5.5.4 Corrugated ducts provide the best bond transfer between tendon and concrete and are thus preferred in regions of high bond stress, such as joint cores.

C13.5.5.5 Limited testing has indicated that precast joints at the faces of columns can function effectively with no other connection through the jointing material than the grouted tendons. Some form of mechanical interlock is required to hold the jointing material in place. Where possible, the plastic hinge zones should be forced to form away from the jointing faces, by the use of suitable reinforcing details, haunches, cruciform columns, or other means.

REFERENCES

13.1 Mattock, Alan H., Yamazaki, Jun., and Kattula, Basil T., "Comparative Study of Prestressed Concrete Beams, With and Without Bond", ACI Journal, Proceedings, V. 68, No. 2, Feb. 1971 pp. 116–125.

13.2 ACI–ASCE Committee 423, "Tentative Recommendations for Concrete Members Prestressed with Unbonded Tendons", ACI Journal, Proceedings, V. 66, No. 2, Feb. 1969, pp. 81–86.

13.3 ACI–ASCE Committee 423, "Tentative Recommendations for Prestressed Concrete Flat Plates", ACI Journal, Proceedings, V. 71, No. 2, Feb. 1974, pp. 61–71.

13.4 Odello, R.J., and Mehta, B.M., "Behaviour of a Continuous Prestressed Concrete Slab with Drop Panels", Report, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, 1967.

13.5 Muspratt, M.A., "Beheaviour of a Prestressed Concrete Waffle Slab with Unbonded Tendons", ACI Journal, Proceedings, V. 66, No. 12, Dec. 1969, pp. 1001–1004.

13.6 ACI Committee 443 "Prestressed Concrete Bridge Design", ACI Journal Proceedings V. 73 No. 11 Nov. 1976, pp. 605.

13.7 Ministry of Works and Development "Highway Bridge Design Brief".

13.8 CP110 "The structural use of Concrete", British Standards Institution, Part I November, 1972.

13.9 FIB–CEB "International Recommendations for the Design and Cosntruction of Concrete Structures", 1970.

13.10 Warwaruk, D.D., Sozen, M.A., and Siess, C.P., "Investigation of Prestressed Reinforced Concrete for Highway Bridges", part 3 – "Strength and behaviour in the use of Prestressed Concrete Beams", Bulletin No. 464, Engineering Experiment Station, University of Illinois, Urbana 1962, 105 pp.

13.11 R Park, "Post-tensioned tendons in Prestressed Concrete – to Grout or not to Grout?", New Zealand Engineering, Vol. 30 No. 11, November 1975.

13.12 "Post-tensioned Slabs for Commercial and Residential Buildings" Post-tensioning Institute, Gleaview, 1977.

13.13 ACI Committee 215 Report "Consideration for Design of Concrete Structures subject to Fatigue Loading", Journal ACI Proceedings Vol. 71, No. 3, March 1974, pp. 97–121.

13.14 Seismic Committee of the New Zealand Prestressed Concrete Institute, "Recommendations for the Design and Detailing of Ductile Prestresssed Concrete Frames for Seismic Loading", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 9, No. 2, June 1976, pp. 89–96.

13.15 The Australian Prestressed Concrete Code AS CA35, 1973.

COMMENTARY

C14 SEISMIC REQUIREMENTS FOR STRUCTURES OF LIMITED DUCTILITY

C14.1 Notation

The following symbols, which appear in this Section of the Commentary, are additional to those used in section 14 of the Code.

G factored gravity load – (dead load and/or live load) specified in NZS 4203 or other appropriate loadings code

M_g moment associated with G
$=M_e - M_{eq}$

V_g shear force associated with G
$=V_e - V_{eq}$

C14.2 Scope

C14.2.1 This Section deals with frames, walls and foundations in buildings and bridges of limited ductility. The provisions of other Sections of the Code cover the design of elements in structures required to be fully ductile to meet the seismic requirements for ductility specified in the appropriate loadings code. This Section modifies the requirements of the preceding Sections to set requirements for structures of limited ductility.

Typical building structures in which the application of Section 14 might be appropriate are:

(a) Frame structures in which the geometrical limitations of Section 6 cannot be met

(b) Low framed structures in which beams are unavoidably over-reinforced, because of gravity load or to meet minimum reinforcement requirements

(c) Structures which, in accordance with generally understood conditions compatible with ductile design, are irrational, such as deep membered frames arising from the random penetration of walls

(d) Long squat wall structures, such as occur as fire or party walls between adjacent residential units, and which possess a great excess of strength over requirements for lateral load resistance. Section 14 would also be applicable to frames lying parallel to such walls, where such frames are not treated as secondary elements in accordance with 3.5.14.

The provisions for bridges have been considered necessary [14.1] to accommodate forms of bridge for which application of the design provisions of 3.5.9 could result in an unjustifiably large financial investment. This is mainly the result of the large design forces necessary when a plastic mechanism cannot form in a particular direction in parts or all of the bridge at or close to the level of design loading specified for ductile structures; for example, in bridges with short piers, abutments giving transverse restraint to superstructures, or one or two span bridges where longitudinal restraint is acceptable in that direction, in the part of the bridge concerned. This should be achieved mainly in the form of lower protection against damage rather than by an appreciable increase in the risk of collapse. It would usually be desirable to improve the performance of such structures by using the alternative approach of incorporating mechanical energy dissipating devices in accordance with 3.5.13. It should be understood that a bridge may be designed as non-ductile in one principal direction and designed as a ductile bridge in the other principal direction. See also reference [14.5].

Other structures for which design to the requirements of Section 14 might be appropriate include minor structures such as tank stands, and vehicle parking decks built out from a bank to which they are anchored (half-structures).

C14.2.2 It is important, in applying the provisions of Section 14 to the design of members, to derive applied actions from loadings higher than those applicable to the analysis of ductile structures, as required by the design procedures specified. The higher strengths thus required provide the principal argument in justification of the relaxed design and detailing procedures permitted for members designed and detailed to the provisions of Section 14, and the consequently smaller ductility capacity furnished.

It is difficult to categorize all structures and to specify the appropriate design loadings and methods.It will occasionally occur that some unusual structures, not included in, but not specifically excluded by, the provisions of the loadings codes, will require design. Such structures may not be amendable to design for ductility, it may be inappropriate to use such design, or particular elements may not meet the geometrical limitations imposed for such design. It may then be considered by the designer appropriate to employ the provisions of Section 14 regarding which this clause notes the necessity to use higher loadings than those used in the design of ductile structures. These considerations dictate that these higher loads must also be applied to any of these special structures designed to the provisions of Section 14.

C14.2.3 The design loads specified in NZS 4203, for structures designed by the elastic response procedure, are such that the required ductility demand can be expected to be met by normally detailed reinforced concrete.

The design procedure will normally only be applied to some special forms of structures in which detailing for even limited ductility might not be economically justifiable. The application of this procedure to concrete structures is not expected to be widespread. Its use will tend to be limited to structures or structural elements possessing, by their nature and position, a high intrinsic strength, such as fire walls between residental units in low rise apartment buildings. Where the procedure is employed, no special detailing or analysis is required.

C14.3 Interpretation

C14.3.1 *General.* The greater strengths required of structures designed to the requirements of Section 14 reduce the ductility demand. Detailing as applied to structures designed for gravity and wind loads may therefore be applied, with appropriate modification, to structures of limited ductility covered by Section 14. It is to be appreciated that while the ductility demand is reduced, it is not eliminated. Consequently it is necessary to supplement the provisions of the non-seismic portions of the preceding Sections to accomplish a post-elastic behaviour significantly superior to that likely to be achieved through application of the principles and requirements for members not designed for seismic loading set in the preceding Sections of this Code.

The objective in specifying the provisions of Section 14 has been to present a simple design procedure. A modified strength design procedure, employing traditional concepts, therefore forms the basis for design. The additional principles and requirements for members designed for seismic loading specified in the preceding Sections need not therefore be complied with. The provisions of Section 3 should be read as statements of general principle applicable to all structures, and the appropriate clauses applied to structures designed to achieve limited ductility. The requirements of Section 3 relating to capacity design, design for concurrency, and the like, need not be complied with, but requirements which are quite general (3.5.1, 3.5.2, 3.5.3, 3.5.4, 3.5.5, 3.5.10, 3.5.14, and 3.5.15) should be met.

C14.3.2 *Bridge structures.* Section 14 specifies requirements applicable to all structures. Additional requirements to cover problems normally only encountered in the design of bridges are specified in 14.10.

C14.3.3 *Alternative design and detailing*

C14.3.3.1 The object of Section 14 is to ensure as far as is reasonably and simply practicable to do so, that structures with limited ductility designed under its provisions, are comparable in performance to those designed under the more strigent requirements of other Sections. It is to be appreciated however that detailing to produce greater ductility is likely to lead to more predictable structural performance under severe seismic motions and designers are encouraged to meet these fuller requirements whenever practicable.

The use of the preceding Sections for individual elements is also permitted. Such an approach might be employed for instance in determining lower shear force demand in flexural members. It is important however that the approach is consistent, so that in the above case, the use of capacity design would also be required for example in the determination of points for the curtailment of flexural reinforcement. The technique of designing a frame, for instance, to the provisions of the preceding Sections, and a parallel penetrated wall to the provisions of Section 14, is thus also permitted, but such an approach should be executed with caution, especially in view of the manner in which torsional response may be affected, due to the differing design strengths of the component systems.

C14.3.3.2 Where alternative design procedures are employed and it is subsequently established that the strength provided is at least that which would have been required had the structure been designed to the higher loads required of structures complying with the provisions of Section 14, relaxation of detailing is permitted in recognition of the smaller ductility demand resulting.

C14.4 General considerations

C14.4.1 *Structural type factors*

C14.4.1.1 and C14.4.1.2 Not all loadings codes specify the lateral seismic design load in the same manner. These clauses permit an equivalent structural type factor, S, to be determined, and, in accordance with 14.4.1.1, its value should not be less than 2.0.

C14.4.1.3 Ductile bridge structures are designed with an assumed displacement ductility demand of six. Design for the higher loads associated with an assumed displacement ductility demand of three or less may be assumed to qualify the structure for design to the requirements of Section 14.

C14.4.1.4 The provisions of this Section are intended to relate principally to low-rise structures for which a structural type factor $S = 2$ has been determined to be satisfactory. For buildings of more than four storeys (or, where a lightweight roof is employed, five storeys), frames exhibiting column sidesway mechanisms, for instance, are likely to behave poorly, and should therefore be made the subject of spcial study. Uniform cantilever shear walls designed to the provisions of this Section, however, are expected to perform satisfactorily even in high-rise buildings.

It is appreciated that there is a gradual transition from uniform walls through walls with openings and deep-membered frames, to walls of more usual proportions. Whether any openings are such as to produce overall frame-like action or to induce a wall to behave essentially as a wall with penetrations, depends on the relative sizes of the openings and their distribution. Furthermore, the mode of behaviour may be distinctly different in the inelastic range from that revealed by elastic analysis in the pre-yield range. It is accordingly difficult to introduce comprehensive guidelines to allow an exhaustive categorization to be presented.

Structures of limited ductility not proportioned according to the principles of capacity design present additional problems because of the difficulty of identifying the mode of collapse. Where design loadings are significant fractions of those derived from an elastic response analysis, application of capacity design becomes less important. Attendant on this consideration is the implication that structures which are more vulnerable to undesirable failure mechanisms, such as frames, should be designed to a higher equivalent static loading than those which are more tolerant, by using S-factors greater than 2.0.

C14.4.1.5 Treatment of some elements as secondary by excluding them from the primary lateral load resisting system is permissible, as provided in 3.5.14. Such a process would normally be restricted to beams excluded by the assumed removal of flexural continuity, but columns may be treated in a similar fashion. Where elements are excluded thus, and are therefore not assumed to contribute to the seismic resistance provided by the primary structure, a complex structural system can be reduced into much simpler sub-systems, and this clause permits the use of S-factors relevant to each sub-system to be used for design but not less than 2.0.

Elements excluded from the primary structure must meet the requirements specified for secondary elements in 3.5.14. It should be appreciated that the design actions for these secondary elements may be larger than those resulting had these elements not been so excluded and had therefore been designed as part of the primary structure. Thus the permitted procedure would normally only be employed when elements so treated would not be expected to contribute greatly to the lateral load resistance of the structure.

C14.4.2 *Required strengths*

C14.4.2.1 It is recognized that some dissipation of energy may occur due to processes other than ductile flexural yielding, such as inelastic shear deformation. Quantification of the dissipation occurring in such processes is difficult, however, and there appears to be no reliable method currently available which would allow assessment of the resulting degree of attenuation of seismic actions to be made with any precision. Further, the total seismic displacement ductility required of short period structures designed to the loadings specified in NZS 4203 may be very large, and therefore large inelastic strains are to be expected.

While such large inelastic shear displacements may be attainable for monotonic loading, achievement of them for cyclic loading is unlikely due to the presence of wide diagonal cracks and subsequent breakdown of the concrete. An inelastic mode involving yielding of shear reinforcement will provide energy dissipation only if the shear displacements are greater than in all previous inelastic cycles. Shear yielding is therefore an unsatisfactory mechanism if a significant amount of energy dissipation is required. It should also be noted that short period structures are more likely to be subjected to a greater number of yield excursions in a given earthquake than long period structures having yield strengths which are similar fractions of spectral values. Consequently the cumulative ductility demand, which has some relevance to damage potential, is high. A detailed discussion of these considerations is presented in references [14.2] and [14.3].

Because of the severe stiffness and strength degradation associated with inelastic shear deformation and the unreliability of the phenomenon in attenuating response, it should not be relied on. To reduce the likelihood of shear failure to an acceptable degree, the shear strength provided should have a suitable margin over that associated with unamplified actions derived from code loading used for flexural design. Amplification of the seismic component of total shear only is required. This is achieved through the additional term $(4/S - 1)\ V_{eq}$, which equals V_{eq} when $S = 2$. Note that V_{eq} is a component of V_e, so that an alternative formulation for eq. 14-1, in the common case where $S = 2$, is:

$$\phi V_i > V_g + 2\ V_{eq} \qquad \text{(Eq. C14-1)}$$

See also C14.4.2.3.

Equation 14-1 is based on recommendations in reference [14.2].

C14.4.2.2 Since capacity design is not required, and since flexural overstrength may induce larger than nominally required moments in the span between terminal hinges, yielding of reinforcement away from the end regions may occur.

In general, the flexural reinforcement may be terminated in accordance with the bending moments derived from the application of loads specified in the loadings code, and in accordance with the anchorage provisions of this Code, provided that the full length of the member is designed and detailed to the requirements specified for end regions.

To take advantage of concessions proposed for confinement and for concrete shear resistance beyond the end region defined in 14.5.2, it is recommended that the flexural reinforcement shall not be terminated unless the continuing bars provide a dependable moment of resistance greater than the moment derived from the loadings code. In many cases this requires reasonably small extensions of flexural reinforcement beyond cut-off points otherwise required. The anchorage provisions of this Code beyond the point so determined still need to be met, however.

In a similar fashion to the provisions for shear strength made in C14.4.2.1, the dependable flexural strength beyond the end region, where termination of flexural reinforcement is intended, should be enhanced, as recommended in reference [14.2]. In the common case where $S = 2$, eq. 14-2 is equivalent to:

$$\phi M_i > M_g + 1.5\ M_{eq} \qquad \text{(Eq. C14-2)}$$

This should reduce the likelihood of yielding, and, where it may yet occur, prevent yielding at loads below those levels of load at which ductility demand will be admissibly small. It is to be noted that this clause does not require an increase in reinforcement at the critical sections, but does generally require that the length to the point of termination of tensile flexural reinforcement be increased.

C14.4.2.3 Where moment redistribution is employed, it may be assumed that the actions requiring amplification are those associated with the redistributed moments.

Redistribution may be carried out to reduce shear demands for instance, as much as to reduce flexural demands, and this provision allows for this possibility. Two procedures are permitted in deducing the salient actions:

Clause 14.4.2.3 (a) provides for a simple procedure based on the actions derived from the elastic analysis, ignoring the effects of moment redistribution. Shear strength, for instance, provided in accordance with this procedure will not then be uniformly increased above those associated with yielding in individual elements, but in vertical elements total shear strength provided will nevertheless meet the requirements of 14.4.2.1, and significant shear displacement is not likely to occur until the entire shear strength is mobilized.

Clause 14.4.2.3 (b) provides for more complex procedures based on a general analysis. Seismic actions associated with redistribution for the entire loading combination are not immediately apparent. One procedure for determining the seismic actions, based on the assumption that redistribution, due to yielding, is due entirely to the seismic loads, is as follows:

(a) Adjust the actions corresponding to the elastic analysis of the structure for the particular loading combination of lateral seismic load and factored gravity load considered, to form a compatible set with the redistributed moments, in accordance with a rational analysis satisfying the requirements of statical equilibrium throughout the structure

(b) Deduct those actions corresponding to the factored gravity load considered in (a), acting alone, derived from an elastic analysis without redistribution, from the adjusted actions found in (a)

(c) Assign the resulting actions derived from (a) and (b) as those due to E, the lateral seismic load.

Alternative procedures are not excluded.

C14.4.3 *Capacity design and design for concurrency.* Capacity design is not required except where the requirements of other Sections are employed, in accordance with 14.3.3.1.

However, it may be desired to employ a capacity design approach to determine maximum likely member actions, especially shear forces or column moments, or as an ingredient of special study required by 14.4.1.4 (a).

C14.5 End regions

C14.5.1 Designation of end regions

C14.5.1.1 It is the responsibility of the designer to identify suitable locations for the regions of potential flexural hinging, and to employ suitable detailing in those regions. For members of prismatic form, the choice of the critical section is usually obviously evident. For non-prismatic members, such as haunched beams or walls in which there may be a sudden reduction in width a short distance above the base of the wall, the choice of the critical section should be made on the basis of ensuring a behaviour which is predictable and such that the end region can be clearly identified.

C14.5.1.2 The assurance that significant yielding will not occur to any great degree other than in the end regions and the length that the potential hinge region will extend beyond the critical section, depends as much on the design of sections beyond the designated end region as on the design of chosen critical sections in the designated end region. Where the provisions of 14.4.2.2 are met, it may be assumed that the risk of yielding beyond the end region is reduced to an acceptable level and that any inelastic deformation will be small enough to permit conventional non-seismic detailing to be employed.

C14.5.2 *Extent of end regions*

C14.5.2.1 Refer to C14.4.2.2.

C14.5.2.2 The designated end regions will normally be located at the face of intersecting members. For walls, the critical section will normally be located at the base. Where columns are designed as for walls, the end region will normally be located at the column base only, and in this case conventional detailing at higher levels may be employed, provided also that the flexural strength furnished beyond the base complies with 14.4.2.2. For beams, however, both ends of each span would normally be regarded as regions of potential flexural hinging and accordingly detailed as required for end regions.

The specified extent and location of the end regions does not absolve the designer from the responsibility of meeting the provisions of 14.5.1.1.

C14.6 Design for flexure and axial load

C14.6.1 *General*

C14.6.1.1 The theory for the design of cross-sections subjected to flexure, with or without axial load is well established, and may be applied directly to the design. In this respect there is no need to take into account the deep beam nature of squat walls; all vertical reinforcement including web reinforcement required for temperature and shrinkage control or for shear may be assumed to contribute fully to the required flexural strength. The strength reduction factor used in design should be that specified in 4.3.1.2, and therefore will lie in the range of $0.7 < \phi < 0.9$ depending on the level of axial stress.

C14.6.1.2 Because inelastic rotations will tend to concentrate at isolated localities, it is important to identify these critical locations and to detail them accordingly. While the use of $S = 2.0$ as a minimum will reduce flexural ductility demand, it will not eliminate it, the level of demand remaining high. Thus while detailing measures for confinement and for the prevention of buckling of principal flexural reinforcement can be somewhat relaxed below those required in elements designed to be fully ductile, certain minimum requirements remain.

C14.6.1.3 Away from regions of potential flexural yielding, the additional requirements, above those employed in non-seismic applications, need not be met. In order to reduce the likelihood of flexural hinging away from the identified hinging regions, flexural strength should be suitably increased as provided in 14.4.2.2.

It is to be noted that shear amplification is required in regions outside of the end regions.

C14.6.2 *Confinement within the end regions*

The procedure specified is based on the provisions of Sections 6 and 10. It will normally restrict the neutral axis depth to about 30% of the overall depth of the member.

The behaviour of members designed to the requirements specified in the appropriate loadings code for members with limited ductility, will normally be dominated by flexure. Therefore the procedure set out is similar to that provided in Section 10. However, since capacity design is not required, it is inappropriate to enforce calculation of overstrength flexural capacity. Without this, any rule relating to neutral axis depth as in Section 10 is also inappropriate.

It is doubtful that complex analytical procedures will shed much light on the true position, and, in any event, especially in view of the approximations used elsewhere, simple procedures are to be preferred. Furthermore, it is preferable for other reasons, such as in ensuring that lateral instability is not a critical design criterion, to restrict the neutral axis depth to the critical value or less, particularly in view of the relative ease with which this objective can be accomplished.

C14.6.2.1 The definition of M_e^* thus avoids algebraic complexity, but may require simple transformation because design moments will normally be related to the centroidal axis.

C14.6.2.2 The bracketed term in eq. 14-4 is a constant for the cross-section for a given spacing of the hoop sets, but the quantity of reinforcement given by the term is reduced by the reduction factor R_c.

Even when γ is greater than unity, thus indicating the possible need for confinement, it is likely that a small ratio of compression reinforcement will allow R_c to be put equal to zero, thus obviating confinement. In any event, some compression reinforcement may be required in order to attain R_c equal to or less than unity, reflecting the consideration that the most significant parameter in the enhancement of curvature ductility is the ratio of compression reinforcement.

C14.6.2.3 The reduction of the effective modulus of elasticity of the longitudinal reinforcement due to the Bauschinger effect is related to the steel strain in the previous cycle of inelastic deformation, and has therefore been assumed proportional to ductility demand. Thus the spacing of the confinement reinforcement is smaller than that employed in columns resisting gravity or wind load, but greater than that permitted in the end regions of ductile members.

C14.6.3 *Lateral tying of longitudinal reinforcement*

Tying will always be required when γ exceeds 1.0, so that when R_c is made zero by the provisions of compression reinforcement, some confinement will result. Even if γ is less than unity, tying is still required when the ratio of longitudinal reinforcement over any part of the cross-section employing two layers of reinforcement exceeds $3/f_y$

C14.6.4 *Longitudinal reinforcement in the end region of walls*

Considerations of possible congestion dictate that certain restrictions on bar size and total reinforcement be applied. In addition, in accordance with established practice, a minimum reinforcement ratio has been specified.

In computing the reinforcement ratio, the gross area of concrete may be taken as the square of the thickness of the wall at the bar locality, or as the product of the thickness times the bar spacing, whichever is more favourable.

C14.7 Design for shear

C14.7.1 *Design strength.* Refer to C14.4.2.1.

C14.7.2 *Shear stress provided by concrete*

C14.7.2.1 The most important phenomenon responsible for the deterioration of the shear strength of the concrete in the end region, as measured by v_c, is the degradation of the concrete as a result of flexural yielding during reversed cyclic seismic loading. The degradation is further accentuated by the number of occasions when such flexural inelastic excursions are encountered during an earthquake. It can therefore be expected that the value of v_c to be relied upon in the design, will diminish with small values of S. As the flexural capacity of the critical section increases, such as when larger values of S are specified, both the demand for flexural yielding and the number of inelastic displacement excursions will be reduced. Consequently the contribution of the concrete to the shear strength of the end region will increase.

With the value of S set at a minimum value of 2.0 it is to be expected that a significant fraction of v_c specified in non-seismic applications may be considered to be provided. The specified fraction of one-half is dependant also on the total strength furnished being at least that given by 14.4.2.1.

Equation 14-6 is based on considerations similar to those used in the derivation of eq. 7-43, but in addition reflects the less accurate determination of the design axial load.

C14.7.2.2 Outside of the designated end regions, significant inelastic deformation is not expected, and accordingly the normal values of v_c may be assumed to be realized.

C14.7.3 *Shear reinforcement in the end regions*

C14.7.3.1 The maximum spacing specified is to ensure that an adequate number of stirrups are intersected by any diagonal tension cracks. If the spacing selected also satisfies the requirements for special transverse reinforcement (10 longitudinal bar diameters) the shear reinforcement may be assumed to form a part of any required confinement or anti-buckling reinforcement.

C14.7.3.2 To qualify as shear reinforcement, any special transverse reinforcement must comply with the provisions specified for shear reinforcement. Otherwise its contribution to shear resistance is to be ignored.

C14.8 Special provisions for slabs and face loaded walls

C14.8.1 Provisions for slabs and face loaded walls have been considered appropriate in view of the relatively common use made of systems employing these elements in small buildings and in some minor bridges. Examples are rigid frames composed of slabs and walls, and flat slab structures providing the horizontal restraint to the structure. With regard to the latter form, the limitation noted for the equivalent column stiffness in C11.9.4. should be observed.

The restrictions specified are likely to prove less limiting than those associated with meeting drift limitations specified in the loadings code, or in complying with the requirements for joints specified in Section 9.

C14.8.2 and C14.8.3 The stress limitations preclude singly reinforced face loaded walls from being used if the maximum design axial load is greater than $0.1 f_c' A_g$, when $\phi = 0.7$ also applies in accordance with the provisions of 4.3.1.2, or if the design ideal shear stress is greater than $0.08\sqrt{f_c'}$ (commonly), in accordance with 14.7.2.1 and 7.3.2.2.

C14.8.4 Tests have indicated that sufficient ductility can be attained when lateral reinforcement is omitted, principally because S-factors specified for such systems are high and therefore ductility demand is low. Where shear stresses are high and shear reinforcement is consequently required, ductility will be enhanced if the shear reinforcement is placed as specified in 14.6.2.4. (See reference [14.7]).

C14.8.5 The considerations leading to the specified restrictions on longitudinal reinforcement applicable to walls loaded in their plane, are equally applicable to face loaded walls and slabs. Consequently the same restrictions apply.

C14.8.6 The concessions indicated in 14.8.4 are all that appear to be prudently applicable. Therefore in all other respects, such as shear amplification, the requirements of Section 14 apply.

C14.9 Special provisions for diaphragms

C14.9.1 *Type 1 diaphragms*

Conventional diaphragms, of Type 1, involved only in transfer of forces generated from their own mass and from mass tributary to them, or in redistributing small fractions of total storey shears and torsions, may be expected to yield. They should therefore be suitably designed and detailed as required for walls. Because of the often complex load distribution in diaphragms, particularly when torsional effects are significant, identification of end regions is seldom appropriate. Therefore the full length of the diaphragm should be detailed as a potential plastic hinge region. Where large forces corresponding to elastic response can be resisted without yielding of the diaphragm, the additional seismic design requirements of Section 14 need not be applied.

C14.9.2 *Type 2 diaphragms*

Transfer diaphragms, of Type 2, involved in redistribution of substantial shear forces, should not yield, as this may involve total reliance on the diaphragm for energy dissipation. Therefore these types of diaphragms should be designed for forces corresponding to elastic response of the structure. In this case the additional seismic detailing requirements of Section 14 need not be applied.

C14.9.3 *General requirements*

These provisions are similar to those specified in Section 10. The designer is therefore referred to the Commentary to that Section for an explanation of the requirements given in 14.9.3.

C14.10 Special provisions for bridges

C14.10.1 *Form*

It may be difficult to predict with certainty, for this type of structure, the relative maximum capacities of elements, such as of elastomeric bearings in shear or of piled foundations in shear or pull-out. Consequently, it is the designer's responsibility to choose a structural form with as predictable a behaviour as is feasible so that the likely distribution of forces through the structure will lead to the structure behaving as well as possible during earthquake shaking. It is understood that damage can occur at seismic loading corresponding to the coefficients specified in members such as back walls, shear keys, and holding-down bolts, and even foundations in some cases. However, the structural form and member detailing should be such that this will not lead to a collapse. Wherever possible, likely locations of damage caused by strong ground motion should be accessible for detection and repair.

C14.10.2 *Foundations*

Methods for the determination of suitable strength margins are given in reference [14.4].

C14.10.3 *Members resisting forces from bearings*

Suitable margins of strength of members resisting forces from bearings are given in reference [14.4].

Additional shears during concurrent seismic response along both major axes of a structure can arise from friction or shear stiffness of devices intended to prevent horizontal movement in the other principal axis to that being considered.

C14.10.4 *Displacements*

In order to allow for earthquake motions stronger than those corresponding to the design forces, a plane of weakness should be introduced to allow secondary damage to occur in a predetermined and limited manner, in order to permit early use of the bridge after an earthquake.

C14.10.5 *Structural integrity*

It is important to ensure that structural integrity is maintained at all times, Recommended design forces for linkages and holding-down devices are given in reference [14.6].

REFERENCES

14.1 "Highway Bridge Design Brief" Ministry of Works and Development, Wellington, New Zealand 701/D, September 1978 and Amendments November 1978.

14-2 Robinson, L.M., "Shear Walls of Limited Ductility", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2, June 1980.

14.3 Paulay, T., and Williams, R.L., "The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 2 June 1980.

14.4 Chapman, H.E., North, P.J. and Park, R., "Capacity Design – Principles and Practice" Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 3, September 1980.

14.5 Fisher, R.W., Lanigan, A.G., Stockwell, M.J., "Small Bridges" Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 3, September, 1980.

14.6 Lanigan, A.G., Preston, R.L., Fisher, R.W., Stockwell, M.J., "Structural and Non Structural Details", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 3, September, 1980.

14.7 Islam, S., and Park, R., "Tests on Slab-Column Connection with Shear and Unbalanced Flexure", Journal of the Structural Division ASCE, ST3, March 1976.

COMMENTARY

C15 STRENGTH EVALUATION OF EXISTING STRUCTURES

C15.2 Strength evaluation – general. Section 15 applies to existing structures where there is doubt concerning gravity load-carrying capacity. Typically, such doubt may arise if the materials supplied are considered to be deficient in quality, if the construction is suspect, or if the structure does not satisfy the Code in some respect. Similarly, doubt may occur about some existing bridge structures where the design loads and procedures are unknown and it is desired to fix a service load limit. In such cases, the Engineer may use Section 15 as a guide in an investigation regarding the safety of the structure.

Recognizing that, in some cases, load tests may not be feasible, or may not be the most appropriate method, evaluation by analytical methods is permitted as an alternative to load testing.

C15.3 Analytical investigations. In an analytical investigation, the analysis must be based on data gathered concerning the actual dimensions of the structure, the strength of the materials in place and all other pertinent details. The field examination should be thorough. For example, if coring, internal fracture, or pull-out testing (references [15.5, 15.6]) of the concrete is required, sufficient samples should be taken to obtain reliable average strength indications and to detect possible flaws at critical locations. (Typically, core tests provide about 85% of the strength of laboratory-cured cylinders for the same concrete.)

In some cases the Engineer may deem the analytical procedure to be preferable to load testing. In other cases, analytical evaluation may be the only practicable procedure. Certain members, such as columns and walls, may be difficult to load and the interpretation of the load test results equally as difficult unless severe damage or actual collapse occurs.

The Code states that the analysis shall demonstrate to the Engineer's satisfaction that the object of the Code has been satisfied. The object of the Code is to ensure public safety. The load factors and strength reduction factors ϕ (see 4.3.1 and 4.3.2) provide for possible loads in excess of the specific design loads, complexities involved in the analysis, workmanship variations, materials variations and similar factors which separately may be within tolerances but which cumulatively might adversely affect the strength of the structure or member. In general, it should be shown that the structure will have strength close to or in excess of that envisaged in the original design or as required by the Code. This is a matter of judgement involving consideration of relevant factors such as the possible consequences of collapse.

In the case of strength evaluation of existing bridges, the loading criteria should be based on current load limits laid down either by legislation or by regulations promulgated by the bridge controlling authority. It should be noted that the current design live loads of reference [15.1] may be somewhat greater than the current load limits, because of allowance for future increases in the limits. A load rating as a proportion of the current load limit may be given in accordance with 15.6. For evaluation of a newly built structure, the criterion for acceptance should be ability to carry the current design loads.

For bridges or other structures subjected to moving live loads, fatigue may be the governing condition, particularly for older structures. Working stress analysis may then be more appropriate than strength analysis. Criteria for such analyses are given in references [15.2] to [15.4] and clause 4.5.1.

C15.4 Load tests

C15.4.1 *General.* The selection of the portion of the structure to be tested, the test procedure, and the interpretation of the results should be done under the direction of a qualified engineer experienced in structural investigations and field tests and measurements.

C15.4.2 and C15.4.3 *Load tests of flexural members* Detailed procedures and criteria for load testing of flexural members are given in 15.4.2 for buildings and in 15.4.3 for bridges.

C15.4.2.3 and C15.4.3.3 The specified total test load for buildings is 85% of the dead and live load design strength required by the Code. For bridges, only the design live load has been factored up, in accordance with common test procedures for such structures. The factor of 1.4 on the design live load, including impact, includes allowance in New Zealand highway bridge requirements [15.1] for the design live load being 1/0.85 times greater than the legal limit.

C15.4.2.7 and C15.4.3.7 A general acceptance criterion for the behaviour of a structure under the test load is that it shall not show "visible evidence of failure". "Visible evidence of failure" will include cracking, spalling or deflection of such magnitude and extent that it is obviously excessive and incompatible with the safety requirements for the structure. No simple rules can be developed for application to all types of structure and conditions. If sufficient damage has occurred that the structure is considered to have failed the test, retesting is not permitted since it is considered that damaged members should not be put into service even at a lower load rating.

C15.4.2.8 and C15.4.3.8 If the structure shows no visible evidence of failure, "recovery of deflection" after removal of the test load is likely to be the most common criterion to determine whether or not the strength of the structure is satisfactory. In the case of bridges, the errors in deflection measurements are normally small compared with the actual deflections and recovery. This may not be so in the case of a very stiff building structure. To avoid penalizing a satisfactory structure in such a case, recovery requirements are waived if the maximum deflection is less than the limits of 15.4.2.8 (a).

C15.5 Members other than flexural members. Because the criteria for judging the results of load tests are not well established for Code purposes, except for members subjected to flexure only, an analytical method is preferred for the strength evaluation of other types of members. Load testing of any type of structure is not, however, excluded as an alternative procedure when feasible.

C15.6 Provision for a lower load rating. Except for load tested members that have visibly failed under a test (see 15.4.2.7 and 15.4.3.7) the Engineer may permit the use of a lower load rating that is judged to be safe and appropriate on the basis of the test results or analysis. Procedures for rating bridges are given in references [15.2] to [15.4].

REFERENCES

15.1 "Highway Bridge Design Brief", Civil Division Publication CDP 701/D, New Zealand Ministry of Works and Development, Sept 1978.

15.2 "Assessment of Posting Weight Limits for Highway Bridges", Civil Division Publication CDP 704/B, New Zealand Ministry of Works and Development, November 1974.

15.3 "Bridge Classification and Deck Grading for Overweight Permits", Civil Division Publication CDP 703/A, New Zealand Ministry of Works and Development, June 1973.

15.4 "Rules for Rating Existing Concrete Bridges": Manual of the American Railway Engineering Association, Chapter 8, Part 19, 1972.

15.5 "Internal fracture testing of *in situ* concrete: a method of assessing compressive strength." Information paper IP 22/80, Building Research Establishment, U.K. Oct, 1980.

15.6 "Tentative test method for pull-out strength of hardened concrete". ASTM C900–78T.

COMMENTARY

CB ALTERNATIVE DESIGN METHOD

CB2 SCOPE

As an alternative to the strength design method of the Code, the design provisions of Appendix B may be used to proportion reinforced concrete members not covered elsewhere by this Code. In the Alternative Method, a structural member (in flexure) is so designed that the stresses resulting from the action of "Alternative Method" load combintions in accordance with NZS 4203 or other appropriate loadings code, and computed by the straight-line theory in flexure, do not exceed allowable service load stresses. Service load is the load, such as dead, live and wind, which is assumed actually to occur when the structure is in service. The required load combinations to be used in design are as prescribed in an appropriate loadings code, for example, NZS 4203 for buildings. The stresses computed under the action of such load combinations are well within the elastic range so that the straight-line relationship between stress and strain is used (Clause B6).

The alternative method is similar to the "working stress design method" of previous ACI Codes (for example, ACI 318-63). For members subject to flexure without axial load the method is identical. Major differences in procedure occur in design of compression members with or without flexure (Clause B7) and, bond stress and development of reinforcement (Clause B5). For shear, the strength design permissible stresses are divided by a factor of safety and the resulting allowable service load stresses restated in B8.

In view of the simplifications permitted, the alternative design method of Appendix B is intended to give results slightly more conservative than the designs obtained using the strength design method of the Code. Load combinations are as set out in NZS 4203 and capacity reduction factors of 1.0 are used for both design and analysis. Also, design rules for proportioning by the straight-line theory of stress and strain in flexure have not been updated as thoroughly as the strength design method for proportioning reinforced concrete members.

CB2.1 Design by Appendix B does not apply to prestressed members except Section 13 permits linear stress–strain assumptions for computing service load stresses and prestress transfer stresses for investigation of behaviour at service conditions.

CB2.2 All other provisions of the Code, except those allowing moment redistribution, apply to the alternative design method. This includes control of deflection and control of cracking, as well as all of the provisions related to slender columns in Section 6.

CB2.3 The general serviceability requirements of the Code (Clause 4.4), such as the requirements for deflection and crack control, must be met regardless of whether the strength method or the alternative method is used for design.

CB4 ALLOWABLE SERVICE LOAD STRESSES

For convenience, allowable service load stresses are tabulated. Flexural compressive stress in the concrete (flexure without axial load) is limited to $0.45 f'_c$. Tensile stresses in the reinforcement are limited to 150 MPa for Grade 275 steel and to 200 MPa for Grade 380 and higher strength steel.

Allowable stresses for shear and bearing are percentages of the permissible stresses for strength design.

CB5 DEVELOPMENT AND SPLICES OF REINFORCEMENT

In computing development lengths and splice lengths, the provisions of Section 5 govern both methods of design equally since, in either case, the development lengths (and splice lengths as multiples of development lengths) are based on the yield strength of the reinforcement. Where M_i and V_u are referenced in Section 5 the service load resisting moment capacity and the applied service load shear force at the section are substituted for M_i and V_u respectively.

CB6 FLEXURE

The straight-line theory of stress and strain applies only to design of members in flexure without axial load. Stresses computed under the action of service loads are well within the elastic range so that the straight-line relationship between stress and strain is used with the maximum fibre stress in concrete limited to $0.45 f_c'$ and the tensile stress in reinforcement limited to 200 MPa for Grade 380 steel (see B4.2).

Straight-line theory may be used for all sectional shapes with or without compression reinforcement when axial load is not present. Since small axial compression loads tend to increase the moment capacity of a section, such axial loads may be disregarded in most cases concerning beams. When doubt exists as to whether or not the axial compression may be disregarded, the member should be investigated using B7.

Deep beams must be designed in accordance with 7.3.12.

In transforming compression reinforcement to equivalent concrete for flexural design, $2E_s/E_c$ must be used in locating the neutral axis and calculating moments of inertia. The lesser of twice the calculated stress in the compression reinforcement or the allowable tensile stress is then used to calculate the contribution of the compression reinforcement in computing the resisting moment at service loads.

CB7 COMPRESSION MEMBERS WITH OR WITHOUT FLEXURE

All compression members with or without flexure must be proportioned using the strength design method. Hence most working stress design aids for columns do not satisfy this Code.

The allowable service load is taken as 40% of the theoretical strength ($\phi = 1.0$) as computed by the provisions of Section 6, subject to possible further reduction due to effects of slenderness. Use of 40% of the theoretical strength is equivalent to an overall safety factor U/ϕ of 2.5.

The Commentary for Section 6 presents an alternative "Modified R-Factor Method" for slender columns. That procedure may also be used with the alternative method of design in lieu of the moment magnification method of 6.4.10 if desired.

CB8 SHEAR AND TORSION

For convenience of the user, a more complete set of design provisions for shear is provided in the new Appendix B than was provided in the ACI 318-71 Code.

The allowable concrete stresses and limiting maximum stresses for shear are 55% for beams, joists, walls and one-way slabs and 50% for two-way slabs and footings, respectively, of the permissible shear stresses given in the Code.

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